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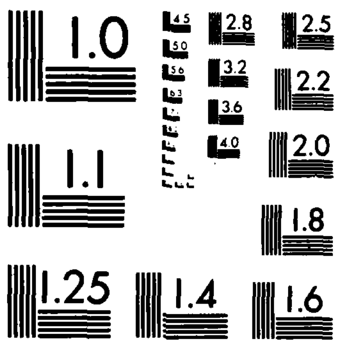

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PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED PROCEDURE FOR DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET)

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The need for a method of accurate and rapid preliminary design, and cost estimating for wastewater treatment plant construction projects has long been recognized. Various models have been developed which purport to preare planning or design level cost estimates. Few of these models are responsive to the requirements of the planner or engineer responsible for accurately projecting construction costs for the purpose of alternative evaluation. The CAPDET model was developed with the specific intent of assisting personnel responsible (cont)

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) for wastewater treatment planning in the evaluation of wastewater treatment alternatives based primarily on life cycle costs and degree of treatment provided. This cost estimating procedure uses both parametric and unit cost estimating techniques.

FORWARD

The concurrent increase in demand for adequate wastewater treatment and advances in wastewater treatment technology has confronted the engineering profession with a situation in which there is a high level of demand for a service and limited manpower and monetary resources for meeting such demand. Available expertise must therefore be efficiently and effectively utilized. Stated simply, there exists the need for a screening tool capable of providing a methodology whereby a large number of alternative wastewater treatment systems, each capable of meeting specified effluent criteria, can be simultaneously ranked on the basis of cost effectiveness.

Computer assisted techniques are particularly suited to the task of insuring maximum efficient use of available resources. Over the past several years numerous computer based cost estimating models have been developed. All of these models have various limitations resulting in difficulties in application or computation. The Computer Assisted Procedure for the Design and Evaluation of Wastewater Treatment Systems (CAPDET) is designed to overcome, where possible, many of the difficulties associated with computerized cost estimating. This, by no means however, is meant to imply that CAPDET overcomes all limitations. The overall goal of CAPDET is to provide accurate planning level cost estimates. CAPDET is not an attempt to replace the traditional engineering estimate or contractor's estimate based on a detailed set of plans and specifications.

This document provides technical information on the design, quantities, and cost algorithms contained within the CAPDET computer program.

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TABLE OF CONTENTS

Page

Page

LIST OF FIGURES

LIST OF TABLES

PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED PROCEDURE FOR DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET)

1.1 INTRODUCTION

1.1.1 Purpose. The purpose of this report is to present the design, quantities and cost estimating algorithms implemented within the Computer Assisted Procedure for Design and Evaluation of Wastewater Treatment Systems (CAPDET).

1.1.2 Applicability. The information contained within this report may be of interest to all persons concerned with the planning, design and cost estimating associated with wastewater treatment facilities which may be constructed under any of several wastewater management programs, including but not limited to 208 planning, 201 planning, urban studies planning, and planning for construction of facilities at military installations.

1.1.3 References. See Section 1.5 for a list of selected references which were consulted during preparation of this report. Additional references are also given at the end of each unit process design segment presented in Sections 2 and 3 of this report.

1.1.4 General. The need for a method of accurate and rapid preliminary design, and cost estimating for wastewater treatment plant construction projects has long been recognized. Various models have been developed which purport to prepare planning or design level cost estimates. Few of these models are responsive to the requirements of the planner or engineer responsible for accurately projecting construction costs for the purpose of alternative evaluation. The CAPDET model was developed by the Corps of Engineers with the specific intent of assisting personnel responsible for wastewater treatment planning in the evaluation of wastewater treatment alternatives based primarily on life cycle costs and degree of treatment provided.

The original version of CAPDET developed in 1973, utilized a system of process design based on well respected techniques, followed by development of costs based on cost curves. This parametric cost estimating approach limited the overall utility of the model in that it was difficult to update, did not adequately reflect regional differences in cost and could not accommodate site specific design requirements. In order to improve the accuracy and usefulness of CAPDET, a revised cost estimating procedure using both parametric and unit cost estimating techniques was developed. These techniques are presented in Section 1.2.

1.1.5 Model Limitations. Although the recent revisions to the CAPDET model have greatly expanded the capabilities and improved the accuracy of planning level design and cost estimating procedures, the user must be cautioned as to the limitations of generalized modeling approaches. Of necessity, the design and cost algorithms developed for any generalized estimating procedure must include many simplifying assumptions. Such items as engineering judgement, equipment design limitations, site limitations, and regulatory reliability and maintainability standards all enter into fomulation of the algorithms. The philosophy of CAPDET is to approach each of these problems in a sound engineering fashion rather than an academic exercise. During development of the CAPDET procedures, various Federal, State and local design standards were reviewed. An attempt was made, where possible, to develop generalized design standards based on these national requirements.

As discussed later in Section 1.2 the CAPDET methodology uses two separate cost estimating techniques. The cost of construction within unit process battery limits is computed using unit costing techniques. The problems associated with engineering judgement and equipment design limitations were addressed through use of a nationally recognized engineering fim for algorithm development and consultation with a variety of equipment manufacturers to detemine equipment limitations. Site specific costs are addressed through use of statistically generated cost curves based on national average costs. These curves are obtained from the U. S. Environmental Protection Agency publication entitled "Construction Costs for Municipal Wastewater Treatment Plants" (FRD11).

CAPDET is designed primarily as a cost estimating tool. Process design and effluent quality predictions were of secondary importance to the overall cost estimating accuracy desired of the model. Thus, any review of the assumptions and simplifications should be conducted primarily from a cost generation viewpoint. The generation of effluent quality predictions is limited and in many cases requires a user input percentage removal factor. CAPDET is not designed as a process simulation model.

CAPDET is not a mathematical optimization model. The CAPDET approach is to prepare cost estimates for user input alternatives. The model will then rank these alternatives by least annual cost but does not purport to provide the "mathematically optimal" solution to an infinite universe of alternatives. CAPDET approaches the facility planning process in the same fashion as would an engineer preparing alternative designs and cost estimates. The major emphasis of the CAPDET project has been the development of accurate planning level cost estimates for a large library of unit processes.

1.1.6 Organization and Use of the Report. This report is arranged in three sections so that the planner or engineer will have quick access to data for many wastewater treatment processes. To further aid planners and designers, reference is made to the CAPDET User's Guide which is available as a separate publication.

Section 1 of this report contains general information related to the planning and design of wastewater treatment facilities using the CAPDET model. Data is presented on CAPDET estimating methodology (Section 1.2), input source development (Section 1.3) and CAPDET cost evaluation techniques (Section 1.4).

Sections 2 and 3 of this report contain the design, quantities and cost equations for specific unit processes available within the CAPDET Program. Section 2 contains unit processes suitable for wastewater treatment of flows within the 0.3 to 300 mgd range. The library of available processes includes conventional, secondary, advanced, and land treatment processes. Design and simplifying assumptions used for each unit process are presented. Section 3 presents a library of processes developed for application to facilities with small flows, e.g. 0.01 to 0.5 mgd.

1.2 CAPDET COST ESTIMATING TECHNIQUE

1.2.1 General. The CAPDET model is primarily designed as a cost estimating model. To fully understand concepts utilized within the model, it is necessary to understand the available cost estimating techniques in general use in the wastewater facility planning process.

Four levels of cost estimating detail may be readily identified. These include the "horseback" estimate, also known as a good guess based on an engineer's past experience; the planning estimate, based on knowledge of the basic system formulations and the use of cost curves; the engineering estimate, based on review of plans and specifications; and of course the contractor's estimate, more commonly called the bid. The CAPDET model is designed to provide a refinement of the second level of detail, i.e., planning level cost estimating. Once this planning level concept is understood, the engineer can move into development and refinement of the cost estimating technique to be utilized for planning level estimating. For purposes of the CAPDET model, planning level estimating accuracy has been defined as ±15 percent for capital costs and ±20 percent for operation and maintenance costs. The goal of process algorithm development is to provide this accuracy level.

Two basic methods have been consistently utilized for planning level cost estimating. First, parametric cost estimating is based on a statistical approach, i.e., statistical analysis of the cost of facilities of similar size and characteristics at other locations. A modification of this statistical approach is the development of standard designs for various flows and formulation of a cost based on an engineering quantities takeoff. Second, unit cost estimating is based on identification of cost elements to which input unit prices are applied, i.e. cubic yards of concrete in a clarifier are quantified, to which an input cost value for reinforced concrete in place is applied to determine construction costs.

The basic advantage of the parametric approach is the limited number of user inputs required, generally limited to one distinguishing characteristic of the treatment process, such as flow, surface area, etc. Stated mathematically, the cost of a unit process is determined as a function of some characteristic of the process. The major disadvantages of the parametric approach which result from the statistical nature of the system are: updating is difficult requiring large data sets; the effects of local economic conditions are difficult to include in the cost analysis; and projecting the effects of inflation on project costs is complicated and often inaccurate, relying primarily on cost indices.

The unit costing approach has the advantage of more closely reflecting actual engineering concepts applicable to a specific facility. The unit costing approach to cost estimating has several basic advantages over parametric costing including: ease of updating to reflect increases in costs of various components of the facility; the effects of inflation are easily evaluated by adjusting unit price inputs;

local conditions may be assessed by input of local unit costs rather than national averages; and labor and materials cost may be separately evaluated. Unfortunately, unit costing models have characteristically suffered from one fatal flaw. In the quest for model accuracy, the proliferation of required unit cost inputs has tended to make such models cumbersome and difficult to use.

CAPDET successfully avoids the propensity for unit costing models to self destruct by utilizing a "modified cost element" approach. In the true cost element approach, construction details are well enough defined to adequately estimate the quantities of material, man-hours of labor, etc. necessary to construct and operate the facility. The modified cost element approach limits the detail required by selecting for in depth study only those cost elements which have a major impact on the cost of the treatment process.

CAPDET has combined both parametric and unit costing techniques to provide a technique for estimating total project costs. Costs associated with construction of a wastewater treatment facility are divided into three categories; unit process costs, other direct construction costs, and indirect project costs. Treatment process costs are those costs associated with a specific treatment process such as a clarifier. Battery limits are drawn such that the clarifier is an individual functioning unit. Cost element estimating is used to determine the costs of the unit process within these battery limits. Other direct construction costs are those construction items more site specific, such as yard piping, site electrical, etc., which are used to "tie" the treatment processes together to form a functional treatment facility. The costs of these items are derived parametrically from EPA developed cost curves based on bid data. Other direct construction costs included in the CAPDET model and their respective equations are presented in Table 1.2-1.

1.2.2 Model Methodology. The basic concept of the CAPDET cost estimating technique is the identification of all costs which are associated with wastewater treatment facility construction. These costs, once identified can be categorized into one of three categories: (1) unit process construction costs, (2) other direct construction costs, and (3) indirect project costs. Varying levels of cost estimating detail are applied to each category of costs. The use of the cost element approach for estimating treatment process costs is the basic key to the accuracy and flexibility of the CAPDET model. The remainder of this section describes the detailed cost estimating process used by the CAPDET model.

1.2.2.1 Unit Process Construction Costs. The unit process construction costs are developed using a unit price cost estimating technique. In order to apply this method, each unit process is defined within battery limits. In general, battery limits are established to be the physical dimensions of the process plus five feet. The modified cost element approach is utilized to provide an estimate of the construction costs associated with a specific unit process within the battery limits of that process.

More specifically, the cost of each process is developed using an equipment module concept which is defined as an item of mechanical equipment plus its supporting structural and housing related facilities. Major cost items are estimated in detail whereas minor cost items may be expressed as a percentage of the cost of the major items. For example, under this concept the cost of a clarifier consists of the cost of the clarifier mechanism and associated tankage, the cost of which are estimated in detail, plus the cost of minor items which are estimated as a percentage of the major items. Thus, total costs associated with a unit process are expressed as a sum of major component parts plus a percentage of the major costs representing minor associated items of construction. The cost estimating procedure for CAPDET was developed based on the premise that cost elements would identify at least 85 percent of capital costs and 75 percent of the operation, maintenance, and repair costs. The remainder of costs would be expressed as a percentage of the major cost elements. Thus, 100 percent of the construction cost is developed and displayed without the exceedingly burdensome task of identifying each individual item contributing to the cost of the facility.

CAPDET utilizes a three-step process for unit process design and cost estimating, entailing three distinct levels of effort. Figure 1.2-1 presents this process in schematic form. The process design calculations section (first-order design) is the basic sanitary process design of the proposed treatment process. This design level identifies such items as volume of tankage, air requirements, etc. The quantities calculations section (second-order design) identifies and quantifies the major cost items, i.e. volume of earthwork, volume of concrete, equipment size, etc. The cost calculations section (third-order design) calculates the unit process costs by applying input unit prices to the quantities and sizes calculated in the quantities calculations section. Each level of design is applied to both capital costs and operation, maintenance, and repair costs.

1.2.2.2 Process Design Calculations. The process design calculation section is the "basic sanitary process design" level of detail. At this stage, we are concerned with volumes, detention times, overflow rates, etc. A nationally recognized process design formulation was selected for each unit process. For the sake of convenience, the process design algorithms have been included in Sections 2 and 3 of this document.

1.2.2.3 Methodology of Quantities Calculations Section. The quantities calculations section identifies and quantifies the major cost items. The following is a discussion of the general methodology employed.

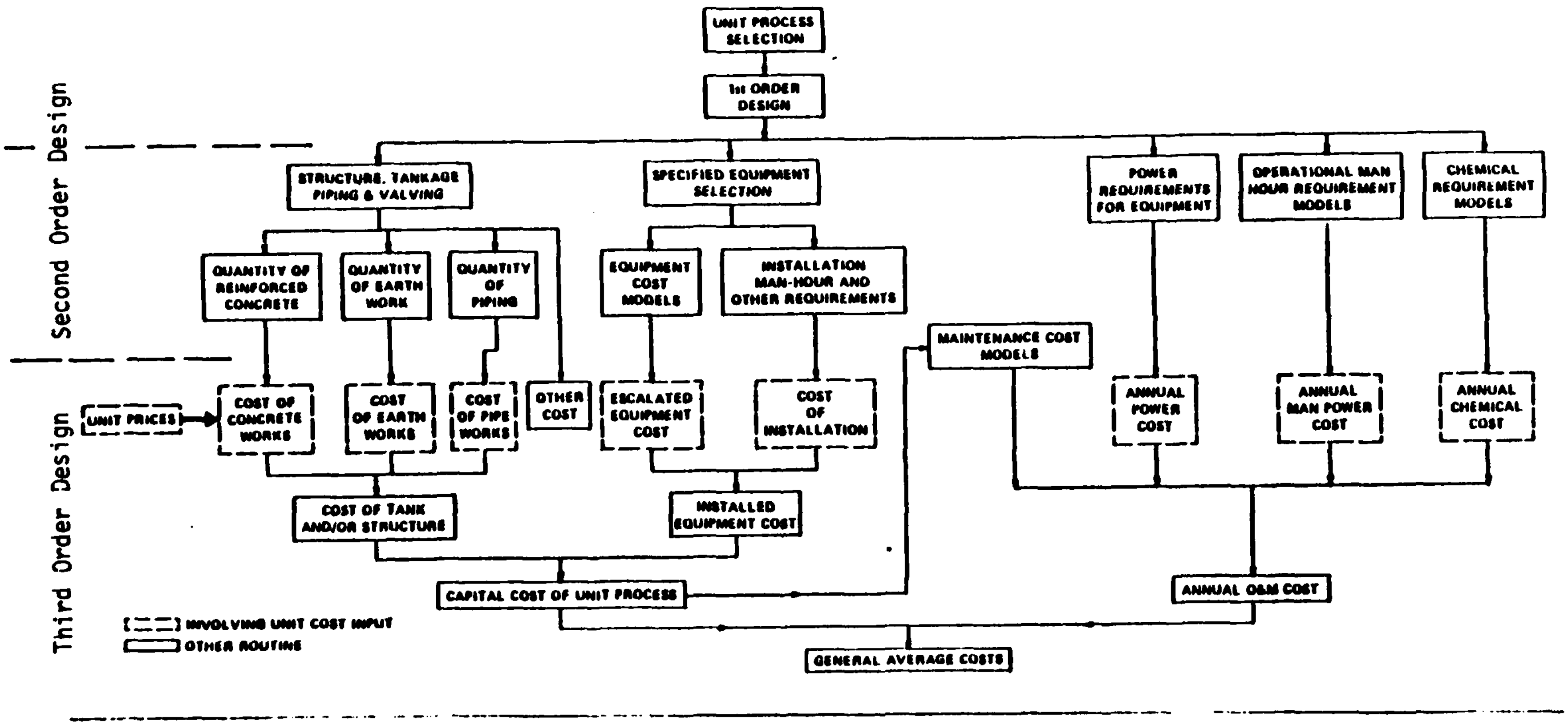

Figure 1.2 - 1   CAPDET Algorithm Development Logic Diagram

1.2.2.3.1 Structural Component. The major cost items for construction of any unit process can generally be categorized as follows:

Concrete or steel tanks and structures.
Installed equipment.
Building and housing.
Piping and insulation.
Electrical works, control systems, and other facilities.

If the first four items listed can be accurately assessed, more than 75 percent of the total capital costs are generally accounted for. The remainder of the cost items can be estimated as a percentage of the total expenditures. The quantities calculations as presented in this revision of CAPDET are keyed to the design of these major items.

For a generalized model to be applicable to a wide range of flows (from 0.5 to 300 mgd), design procedures for various flow ranges must of necessity be different. In addition, certain unit processes may not be applicable outside of specific flow ranges. For instance, it would be very unusual, based on current practice, to select an aerated lagoon system for the treatment of domestic sewage with flows in excess of 1 mgd. Furthermore, even within a specific project, the selected approximate flow ranges for the design of one treatment process may not be suitable for design of the others. For example, it is current practice to divide aeration tanks into batteries of equal number of smaller basins when the sewage flow is more than 100 mgd. The blower system, for economic reasons, would continue to be centralized into one building regardless of the air requirements or flow range. It is evident that no general model is possible for all possible ranges or treatment processes. Design concepts and problems are dealt with individually by using engineering judgement and design experience which are ascertainable for each unit process in Sections 2 and 3 of this report.

Selection of the number of units to be constructed for each unit process may be determined by economic considerations and/or regulations of government agencies. EPA policy documents, as well as the Ten State Standards, urge the design engineer to provide sufficient equipment and tankage in wastewater treatment plants to allow for effective operation of the plant when it is necessary to shut down an aerator, a clarifier, or a piece of equipment for cleaning and maintenance. Standby or reserve equipment, such as pumps, is usually provided. A recurring problem in estimating the cost of a unit process is identification of excess tankage required as a function of plant size. Optimization techniques have been developed by several researchers which address this problem. The number of tanks may also be governed by the numbers and capacity of installed mechanical equipment. Both the theoretically derived techniques from academic research and the practical aspects of design are given equal consideration in fomulating the model for each individual unit process.

The size and dimensions of concrete or steel structures can be calculated after the number of tanks, vessels, or pieces of equipment is determined. The dimensions of the basins are governed by the physical constraints of the individual mechanisms to be installed in these basins. For example, aeration basins using diffused areation devices are usually designed with a water depth of 15 feet and width of 30 feet for effective gas transfer. After the volume, depth, and width of tanks are fixed, the length can be easily calculated. It must be stressed that these characteristics may change during actual design but for planning level accuracy the above assumptions appear to be sufficient.

The thickness of walls and slabs associated with selected structures is a function of the water depth, soil conditions, groundwater elevation, and the percentage of the structure which would be constructed below grade. Certain assumptions must be established in order to formulate a generalized model. It is assumed that structures, except pump stations, will be built 3 feet below grade for concrete structures and at grade for steel structures. Having made these assumptions, the quantity of reinforced concrete, structural steel, and earthwork can be easily calculated by computer methods.

1.2.2.3.2 Equipment Component. The second most important cost item is installed equipment costs. The quantities calculations section (second-order design) includes equipment selection, space requirements for housing, electrical and control systems, and operational and maintenance requirements including labor, chemicals, and electrical power. The selection of equipment is interrelated to the selection of reaction vessel configuration. It is further complicated by the limited size of available off-the-shelf equipment items from manufacturers. Furthemore, different variations of equipment are available to carry out specific functions. Selection usually depends on the capacity required and, to a lessor degree, the preference of design engineers. In this study, and attempt is made to select the most economical and common equipment for appropriate capacity ranges.

After the number, type, and capacity of equipment are established for each unit process, the operation and maintenance requirements are estimated from the empirical fomulae established from this study. The analysis of operation and maintenance costs includes the factors listed below.

Operation and maintenance manpower.
Electrical energy required for operation.
Material required for repair.
Chemical and other requirements.
Replacement schedule.

For certain unit processes, piping systems contribute a major portion of total capital expenditures, thus warranting separate treatment. The piping systems studied are the process piping within each individual unit operation, exclusive of yard piping, which connect one unit process to another. The elements of the piping system include pipe, fittings, valves (with operators), insulation, and supports. Pipe materials considered are identified for each individual process. The design of the piping systems is governed by wastewater flow, number of reaction vessels, and complexity of the process requirements. For instance, the piping systems for a contact stabilization activated sludge are more complex than those for a complete mixed activated sludge. Due to the tremendous number of piping systems within a sewage treatment plant, only major piping systems are evaluated. Major piping systems are defined as those piping systems with pipe sizes larger than 4 inches in diameter. The cost of the minor piping is estimated as a percentage of the total piping costs. Specific basin arrangements are assumed. The basin and flow arrangements are determined for each unit process in the quantities calculations section. In many cases, piping costs are not individually identified and are included as a percentage of other construction costs.

Electrical work and process control equipment in a wastewater treatment plant are closely related to the mechanical equipment installed. The estimated costs of these items are incorporated as a percentage of the installed equipment costs.

The output data from the quantities calculations section is used as input to the cost calculations section to obtain cost estimates for the major cost items.

1.2.2.4 Methodology for Cost Calculations Section. The procedure used for capital cost estimating will be addressed according to the categories presented in the discussion of the methodology for the quantities calculations.

1.2.2.4.1 Structural Component. The cost of the structural component consists of the combined cost of reinforced concrete, earthwork, structures, and piping.

The construction of earthen basins (such as aerated lagoons or sludge lagoons) is generally attempted with equal cut-and-fill quantities. In other words, excavated material is utilized in embankments so that borrowing of dirt from outside is not necessary. The procedure is applicable only when soil and groundwater conditions are ideal, as is assumed in the CAPDET model to simplify costing procedures. The unit cost input will consist of dollars per cubic yard of earthwork assuming equal cut-and-fill. In the case of a specific application, actual soil considerations would be known so that the unit cost could be adjusted to obtain a more realistic estimate.

The costs of reinforced concrete structures will be estimated as the sum of costs of concrete slabs and concrete walls, due to the significant difference in costs between these two types of in-place structures. The unit cost inputs are the cost of in-place concrete slab in dollars per cubic yard and the cost of in-place concrete walls in dollars per cubic yard.

1.2.2.4.2 Equipment and Installation Costs. Equipment for the wastewater treatment system may constitute one of the largest items of identifiable fixable capital costs. It is desirable therefore to maintain up-to-date equipment cost data for CAPDET. With a limited number of unit cost input entries, it is very difficult to maintain a reliable cost data base. The following description outlines a procedure which produces an accurate estimate within these limitations. The installed equipment cost is considered in three components: the purchase cost of the equipment, installation labor cost, and other minor costs such as electrical works, minor piping, foundations, painting, etc.

The purchase cost of process equipment is a function of size or capacity. To minimize the number of cost inputs required, a standard size (or capacity) unit is selected and the purchase cost of all other size (or capacities) units of that type is expressed as a fraction or multiple of the standard unit purchase cost. This cost ratio versus size relationship has been developed for each major item of equipment required. These relationships assume the form shown:

$$\frac{(\mathrm{COST})_o}{(\mathrm{COST})_s} = F\left(\frac{A_o}{A_s}\right)$$

where

A = some characteristic size measurement such as volume, area, horsepower, or weight.

O and S = subscripts designating other and standard sizes, respectively.

F = a function of.

The exact form of the cost-versus-size relationship and the selection of the standard sizes for each major equipment item were determined from a review of manufacturer's information and available literature. In most cases these size-cost relationships are relatively unaffected by inflation and other cost changes.

Two options are available by which the purchase cost of equipment can be escalated to account for inflation. The first option is for the user to obtain from equipment manufacturers the current cost of the standard size equipment F.O.B. (Free-On-Board) at the treatment plant site. The purchase cost of any other size item of like equipment is then automatically escalated by the cost versus size relationships described above. The second option is to escalate the purchase costs by the use of cost indices. Only one input is required for this process; the Marshall and Swift Equipment Cost Index. The 1977 first quarter purchase prices of the standard size equipment are stored in the model and are updated automatically if the cost index is input into the program. The latter of the above methods is the least accurate; however, it requires fewer input values. If the model user inputs a cost for equipment, the index is not used to update the new cost.

Man-hour requirements for installation are dependent on the type and size of equipment. The relationships between man-hour requirements for installation and equipment size and type have been established and are presented in the designs for each unit process. These relationships resemble the following generalized form:

$$MH = F\ (A)$$

where

MH = man-hours required for installation.

A = size measurements as defined previously.

The cost of installation is estimated by multiplying the man-hour requirements by the input labor rates.

In many cases data concerning manpower requirements for equipment installation were found to be incomplete or non-existent. In such cases, the model uses a percentage of purchase price factor to calculate the cost of equipment installation. These factors, in general, were obtained from equipment manufacturers and published sources.

The other minor costs for each type of equipment may consist of piping, steel, instruments, electrical, insulation, painting, insurance, taxes, etc. These items are estimated as a percentage of purchase costs and these percentage values will vary with the type and size of equipment. These percentage values were established based on design experience, engineering judgement, manufacturers' input, and previously published literature.

The installed equipment costs are summarized by the following formula:

Purchase Cost of Equipment		\$ M
Man-Hours Required for Installation	= MH	
MH x Labor	=	\$ L
Other Minor Items	= X%	
\$M x X%	=	\$ XM
Total Installed Costs	=	(1 + X%) M + L

where

M = material costs (equipment costs).

L = labor costs.

X = percentage of material costs for minor items.

1.2.2.4.3 Cost of Building and Housing. Buildings are essential in certain unit processes for protection against weather or maintenance of a prerequisite environment. The building requirements are related to the equipment to be housed and are estimated in the quantities calculations section as square footage of floor space. Building costs are estimated by the following formula:

Cost of Building = (S.F. of floor space) x (Unit cost/sq ft)

1.2.2.4.4 Costs of Piping System. Piping costs may vary from 15 to 20 percent of wastewater treatment plant costs. They should be evaluated independently. Estimating process piping costs presents the greatest challenge for the cost engineer. Estimating from completely detailed drawings is an ardous, time-consuming task much beyond the scope of CAPDET. Evaluation on any other basis may produce widely varying results. To estimate the cost of the "major piping system", a combination of two well established estimating methods used by the chemical industries will be employed. The cost of material will be estimated by the use of the Dickson "N" method and the field erection cost will be estimated by the cost of "joints" method. The R.A. Dickson "N" method uses a technique to estimate purchase price of piping material similar to the one proposed to estimate equipment costs. Relationships are developed between the cost ratios, designated as N factors, and sizes of piping material, defined as follows:

$$N = \frac{\text{Cost of Pipe any size}}{\text{Cost of reference size pipe}} = F\text{ (pipe size)}$$

With these factors stored in CAPDET for cast iron pipe, steel pipe, fittings and valves, the user will have to input only a limited number of unit costs of the reference components. The field erection costs for the piping system can be estimated by use of the cost-per-joint method. The unit of work measurement is the joint (two for couplings and valves, three for tees, etc.) because joints require the bulk of piping labor for erection. The costs of handling, hanging pipe placement, and insulation are estimated as a fraction of the cost of makeup joints. The manhours of field erection per joint for various pipe sizes and materials, as well as the fraction for placing and insulating, are evaluated in the quantities calculations section. The field erection cost of the piping system will be estimated based on the labor requirements and unit labor price inputs.

The total piping system costs would be the sum of the following items:

Piping material costs.
Field erection costs.
Other minor costs as a percentage of total piping costs.

In many cases it is impractical, at the planning level, to identify piping quantities and sizes. In such cases, a percentage of other construction cost factor is used to estimate piping costs. The method used is specified for each process.

1.2.2.4.5 Operation and Maintenance. The operation and maintenance cost for a wastewater treatment unit process can be divided into several major categories: energy, operation labor, maintenance labor, chemical costs, and operation and maintenance material and supply costs. These costs and the techniques for estimating are presented below.

1.2.2.4.5.1 Energy. Energy costs are derived from the calculated use of fuel oil, natural gas or electric power. The quantities calculations section generates the quantities of energy use whereas the cost calculations section applies user input unit prices to calculate the unit process energy cost. The total energy cost of the treatment facility is simply the sum of the energy costs for the unit processes.

The cost of electric power is by far the predominant energy cost for most processes. Thus, electric power has been placed in a separate category on the program output formats. Since natural gas and fuel oil is consumed in relatively few processes, the cost of these fuels is tabulated as a materials costs. The procedure for calculating power cost is presented below. Similar techniques are also applied to the calculation of fuel oil and natural gas costs.

The electric power consumption has been determined for each unit process and is part of the output data from the quantities calculations section of each process. The power consumption for the treatment facility is simply the sum of the power consumptions for the unit processes. The power consumption is converted to costs by multiplying the power consumption in kilowatt hours per year by the unit price input for electric power costs in dollars per kilowatt hour. Electric power rates vary according to location, peak demand, and level of consumption. Because of the tremendous variability of power costs no general value can be used and the user must obtain this price input from the local utility company which would be supplying the power.

Power consumption is calculated as follows:

$$EPC = (PC)\ (UPIPC)$$

where

EPC = electric power costs, \$/yr.

PC = total power consumption, kwhr/yr.

UPIPC = unit price input for power costs, \$/kwhr.

1.2.2.4.5.2 Labor Costs. The cost of labor can be divided into four categories: administrative and general, operation, maintenance and laboratory. Operation and maintenance labor are applied to the unit processes specified in the treatment alternatives. Administrative and general labor as well as laboratory labor are computed for the treatment facility as a whole.

Recommended staffing for different levels of manpower required for each of the four labor groups was established by utilizing several publications concerning staffing of sewage treatment facilities Table 1.2-1 shows the four labor groups and the staff recommended for each group. However, based on the size of the facility, or man-hours required, the complete staff may not be required and more time may be required by one of the staff than by others. Further, each staff member demands a different salary. By utilizing staffing charts provided in the literature, weighted average salaries for each labor group were established for several different man-hour requirements for the groups. To reduce the number of inputs, the weighted average salaries were expressed as a percentage of the Operator II salary. Graphs were then made of percent Operator II salary versus man-hours required for the four labor groups. These curves were expressed in equation form and are presented in the discussion of each labor group which follows:

TABLE 1.2-1. ESTIMATED PLANT STAFFING COMPLEMENT

Administrative Group

- Superintendent
- Assistant Superintendent
- Clerk Typist
- Storekeeper

Operation Labor

- Operations Supervisor
- Shift Foreman
- Operator II
- Operator I
- Automotive Equipment Operator

Maintenance Labor

- Maintenance Supervisor
- Mechnical Maintenance Foreman
- Maintenance Mechanic II
- Maintenance Mechanic I
- Electrician II
- Electrician I
- Painter
- Maintenance Helper
- Laborer
- Custodian

Laboratory

- Chemist
- Laboratory Technician

End Table 1.2-1.

The administrative labor group consists of the management and office personnel as shown in Table 1.2-2. The total man-hour requirement for this group is related to the size of the plant and is calculated by the following equation:

$$AMH = 348.7\ (Q_{avg})^{0.7829}$$

where

AMH = administrative man-hours required, man-hours/yr.

Q_{avg} = average wastewater flow, mgd.

In order to convert the man-hours to costs, an average salary for this labor group is calculated based on the man-hours required for the labor group.

$$SALA = 20.92\ (AMH)^{-0.3210}\ (SALOP)$$

where

SALA = average salary for administrative labor group, \$/hr.

SALOP = salary for operator II, from input, \$/hr.

Finally, the cost per year for the administrative labor group is calculated by:

$$COSTA = (SALA)\ (AMH)$$

where

COSTA = annual cost for administrative labor group, \$/yr.

The laboratory group consists of the personnel required to run necessary tests to check the various parameters which must be monitored to assure effective treatment. The man-hour requirement for this group is calculated by the following equations.

For $0.01 < Q_{avg} \leq 20$ mgd

$$LMH = 2450\ (Q_{avg})^{0.1515}$$

$Q_{avg} > 20$ mgd

$$LMH = 1062\ (Q_{avg})^{0.4426}$$

where

LMH = laboratory man-hours required, man-hours/yr.

Q_{avg} = average wastewater flow, mgd.

To convert the man-hours to cost, an average salary for this labor group is calculated by the following equation:

$$SALL = 1.1\ (SALOP)$$

where

SALL = average salary for laboratory labor group, $/hr.

Finally, the cost per year for the laboratory labor group is calculated by:

$$COSTL = (SALL)\ (LMH)$$

where

COSTL = annual cost for laboratory labor group, $/yr.

The maintenance man-hours required have been calculated for each individual unit process. The total requirement is the sum of the requirement for each unit process to be used in the treatment facility.

$$MMHT = \Sigma MMH$$

where

MMHT = total maintenance man-hour required, man-hour/yr.

MMH = maintenance man-bours required for each unit proce man-hours/yr.

To convert the man-hours to cost, an average salary for this labor group is calculated by:

$$SALM = .388\ (MMHT)^{0.085}\ (SALOP)$$

where

SALM = average salary for maintenance labor group, $/hr.

The total annual cost for the maintenance labor group is calculated by:

$$COSTM = (SALM)\ (MMHT)$$

where

COSTM = annual cost for maintenance labor group, \$/yr.

The operation man-hours required have been calculated for each individual unit process. The total man-hour requirement is the sum of the requirements for each unit process to be used in the treatment facility.

$$OMHT = \sum OMH$$

where

OMHT = total operation man-hours required, man-hours/yr.

OMH = operation man-hours required for each unit process, man-hours/yr.

To convert the man-hours to cost, an average salary for this labor group is calculated by:

$$SALO = .97\ (SALOP)$$

where

SALO = average salary for operation labor group, \$/hr.

The total annual cost for the operation labor group is given by:

$$COSTO = (SALO)\ (OMHT)$$

where

COSTO = annual cost for operation labor, \$/yr.

The total annual operation and maintenance labor cost is the sum of the labor costs for each of the four labor groups.

$$TOMLC = COSTA + COSTL + COSTM + COSTO$$

where

TOMLC = total annual operation and maintenance labor costs, \$/yr.

1.2.2.4.5.3 Operation and Maintenance Material and Supply Costs. These costs have been calculated for each unit process and are an output from the cost calculations section for each unit process. The total operation and maintenance material and supply costs for the treatment facility are the sum of the costs for each unit process to be used in the treatment facility.

$$TOMLC = \Sigma\ OMMC$$

where

TOMMC = total annual operation and maintenance material and supply costs, \$/yr.

OMMC = operation and maintenance material and supply costs for unit process, \$/yr.

1.2.2.4.5.4 Chemical Costs. There are five different chemicals which are used most frequently in sewage treatment facilities. These chemicals are lime, alum, ferric chloride, polymers, and chlorine. Quantities of each chemical required by the treatment processes are calculated in the quantities calculation section. The chemical costs are then calculated by the following equations:

$$CLIME = (UPLIME)\ (LIME)\ (365)$$

$$CALUM = (UPALUM)\ (ALUM)\ (365)$$

$$CIRON = (UPIRON)\ (IRON)\ (365)$$

$$CPLMR = (UPLMR)\ (PLMER)\ (365)$$

$$CCLR = (UPCLR)\ (CR)\ (365)$$

where

CLIME = annual cost of lime, \$/yr.

UPLIME = unit price input lime, \$/lb.

LIME = lime dosage rate, lb/day.

CALUM = annual cost of ALUM, \$/yr.

UPALUM = unit price input for alum, \$/lb.

ALUM = alum dosage rate, lb/day.

CIRON = annual cost of ferric chloride, \$/yr.

UPIRON = unit price for ferric chloride, \$/lb.

IRON = iron dosage rate, lb/day.

CPLMR = annual cost of polymer, \$/yr.

UPLMR = unit price input for polymer, \$/lb.

PLMER = polymer dosage rate, lb/day.

CCLR = annual cost of chlorine, \$/yr.

UPCLR = unit price input for chlorine, \$/yr.

CR = chlorine dosage rate, lb/day.

The total annual chemical costs for the facility may be calculated by:

$$COSTCT = CLIME + CALUM + CIRON + CPLMR + CCLR$$

where

COSTCT = total annual chemical costs for the treatment facility, \$/yr.

1.2.2.4.5.5 Total Operation and Maintenance. The total annual operation and maintenance costs is the sum of the electric power costs, the operation and maintenance labor costs, the operation and maintenance material and supply costs, and the chemical costs.

$$AOMC = EPC + TOMLC + TOMMC + COSTCT$$

where

AOMC = total annual operation and maintenance costs, \$/yr.

1.2.2.5 Other Facility Construction Costs. The construction of wastewater treatment facilities involves not only the construction of unit processes, but also the supporting facilities and interconnecting piping which must also be completed in order to create a functional facility. These other facility construction costs have been found to represent from 35 to 50 percent of total facility construction cost. Other direct construction costs formulations available to the model user are presented in Table 1.2-2.

The other facility construction costs are generally those costs which are more site specific and do not lend themselves to the use of unit cost estimating techniques developed in a planning level model. The cost of these items may vary significantly depending on conditions at a particular site.

The CAPDET cost estimating methodology applies parametric cost estimating techniques to calculate approximate costs for these construction elements. The cost curves used in the model are derived from the U. S. Environmental Protection Agency Technical Report, "Construction Costs for Municipal Wastewater Treatment Plants: 1973-1978, "FRD11.

TABLE 1.2-2. Other Facility Construction Cost Formulations

Item Description	Formulation
Special Foundations	$55129\ Q^{0.57}$
Effluent Pumping	$55776\ Q^{0.61}$
Outfall Diffuser	$29988\ Q^{0.56}$
Mobilization	$52967\ Q^{0.69}$
Clearing and Grubbing and Site Preparation	$92734\ Q^{0.57}$
Site Electrical	$139519\ Q^{0.73}$
Yard Piping	$96076\ Q^{0.71}$
Laboratory, Maintenance and Administration Building	$161240\ Q^{0.58}$
Raw Waste Pumping	$109443\ Q^{0.63}$
Instrumentation and Control	$64997\ Q^{0.78}$
Non-Ocean Outfall	$50962\ Q^{0.77}$
Ocean Outfall	$251468\ Q^{1.06}$

The curves found in FRD11 were developed from a statistical analysis of over 737 wastewater treatment plant construction projects representing approximately 5.8 billion dollars of grant eligible treatment plant construction expenditures. Considering inflation and other factors it is estimated that the data used to develop these curves represent over one-half of the treatment projects which have gone to the construction stage (Step 3) since inception of the construction grants program. For further information on development of these curves, the reader is referred to FRD11.

Another significant facility construction cost is the allowance for contractor profit and overhead. Since this item should be calculated based on total facility construction cost, the bare costs of the unit processes plus the cost of other facility construction cost is multiplied by a percentage to determine a value for contractor profit and overhead. Typically this percentage ranges between 20 and 30 percent. USEPA data suggests use of 22 percent for a national average.

Total facility construction costs are calculated as the sum of unit process costs, other facility construction costs and contractor profit and overhead.

1.2.2.6 Nonconstruction Costs. The calculation of total project costs involves not only the cost of construction but also nonconstruction costs associated with the Step 1 and Step 2 planning and design effort as well as nonconstruction costs associated with the Step 3 construction effort. These costs are presented in Table 1.2-3.

The CAPDET cost estimating techniques again utilizes the data collected in the FRD11 study as the basis for calculating these costs. Nonconstruction costs are presented as a percentage of total construction costs. The reader is again referred to FRD11 for detailed formulation of these values.

Land requirements are generated using information derived from the USEPA report entitled "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", Report No. 17090, October 1971. Land area requirements are developed for each treatment alternative based on the assumption that traditional mechanical treatment processes are used. Should the alternative include lagoons or land treatment alternatives, the land requirements for these processes are added to overall land requirements. The land costs are based on the land area calculations times a user input cost per acre.

Table 1.2-3. Nonconstruction Cost Percentages

Item Description	Percentage of Total Construction Cost*
Contingencies	8
Administrative/Legal	2
201 Planning	3.5
Inspection	2.0
Technical	2.0
Miscellaneous Nonconstruction	5.0
Design	4.0-12.0

*May be changed by program user.

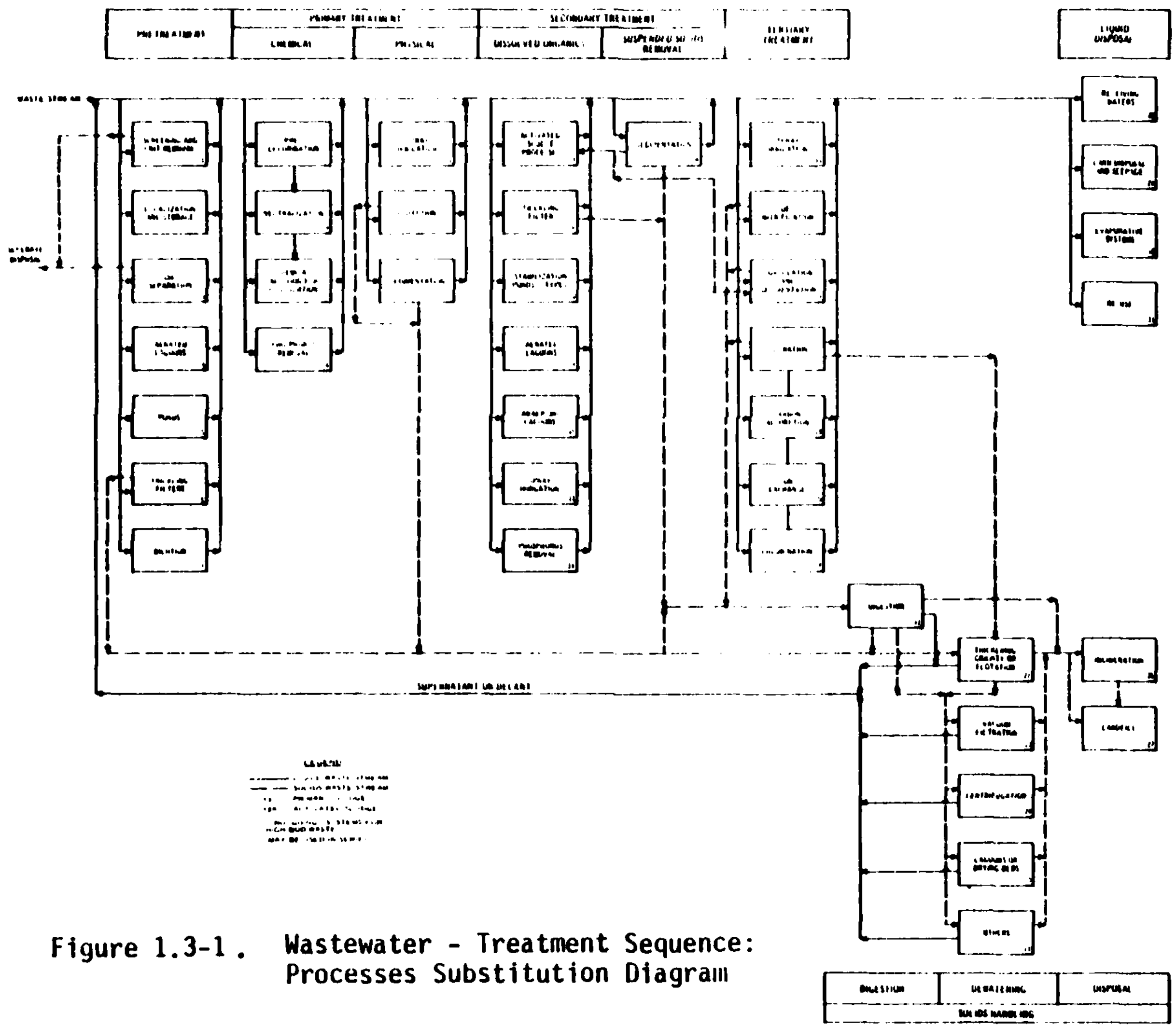

Figure 1.3-1. Wastewater - Treatment Sequence: Processes Substitution Diagram

1.3 INPUT SOURCE DEVELOPMENT

1.3.1 General. Use of the CAPDET model requires input of six categories of data. These include: unit process design data, scheme descriptions, wastewater characteristics, unit price data, other direct construction costs and indirect cost values. Default data are included for all input data requirements with the following exception: scheme descriptions and facility design flow. The model user is referred to the User's Guide for detailed information concerning input requirements and default data values. The remainder of this section presents a discussion of data sources and techniques for obtaining the required data inputs.

1.3.2 Unit Process Design Criteria. The CAPDET model allows the user to change various design parameters for each unit process. Chapter 3 and 4 of the CAPDET User's Guide contain a detailed discussion of the unit process design parameters for each process. This discussion, arranged by unit process in alphabetical order includes the following information: unit process design parameters, appropriate value ranges for the individual design parameters and values of the default data available within the program. The values presented in the User's Guide are based on an extensive review of available literature related to each unit process. Sections 2 and 3 of this document present additional design information for each unit process as well as a listing of publications consulted during model algorithm development.

1.3.3 Scheme Descriptions. The CAPDET model contains over 90 treatment processes in the program library. Each of these processes may be utilized by the model user to construct various wastewater treatment process trains (combinations of treatment processes). Selection of a process train will depend on numerous factors including:

- Stream standards and/or receiving water quality.
- Federal, state, and local effluent criteria.
- Wastewater characteristics.
- Initial and annual costs.
- Availability of fuel and/or electric power.
- Specific exclusions of certain processes.
- Possible requirements for future expansion and/or upgrading.

The process substitution diagram in Figure 1.3-1 is useful in system design and evaluation. In practice, the diagram is entered from the left and the designer may select processes for preliminary, primary, secondary, tertiary, and/or sludge treatment according to specific needs. For a given set of wastewater characteristics, there may be 30 or more viable treatment processes; a complete treatment system may employ from 1 to 20 processes. Thus, there are literally thousands of possible combinations and it is virtually impossible for an individual designer or design team to evaluate all viable processes and combinations of processes without the aid of the computer. To limit the amount of evaluation required, the designer must rely heavily on experiences and examples set by others to select processes or eliminate certain processes from consideration. This practice places a great burden on the designer and occasionally results in designs that are not cost effective.

Additional information on the applicability of a singular unit process for removal of specific pollutants is included in the discussion on each unit process in Sections 2 and 3 of this documentation.

1.3.4 Wastewater Characteristics. The CAPDET model allows input and tracking of 20 wastewater constituents. Default values are included for all values other than average, minimum and maximum flows. Information on wastewater characteristics is generally available in the literature. The following discussion presents a brief synopsis of available literature.

Water usage rates, wastewater production rates, and wastewater characterization data for domestic sewage have been documented for urban area design. These rates and qualities are representative of the "average" city and are subject to the engineer's knowledge of the area for their intelligent application. Needless to say, no table of statistical values is a good substitute for field data. However, the information presented in this chapter will allow the engineer to prepare a "first-look" design and obtain rudimentary cost data for the area. Subsequent iterations in the planning process demand that these average values be replaced with site-specific values.

One of the most important parameters required for the effective and realistic planning of any community sanitary facility is flow. Flow is the major determinant of size, location, and public acceptance of any plant. Unfortunately, flow may frequently be the most elusive parameter to forecast. Many factors dramatically affect the flow reading of a waste treatment facility; among these are (a) geographical location, (b) type of users (e.g., residential, industrial, agricultural, etc.), (c) sewer inflow, infiltration/exfiltration, (d) consumptive uses (e.g., lawn watering, car washing, etc.), (e) storm event history, and (f) precipitation and resulting runoff.

Metcalf and Eddy have found that between 60 and 80 percent of the per capita consumption of water will become sewage. Based on studies by the U. S. Public Health Service for the Select Committee on Natural Water Resources published in 1960, the average water consumption on a national basis was found to be approximately 147 gallons per capita per day (gpcd). This therefore represents an average wastewater flow from 88 to 118 gpcd or about the 100-gpcd figure frequently reported in the literature. It should again be emphasized that this value represents only an average value which must be used with caution.

In addition to the average flow, the designer must be cognizant of the maximum and minimum flows. The design of many unit processes is based on a knowledge of peak flows, while the design of others is based on minimum flows. Table 1.3-1 provides some comparisions found by Clark and Viessman in their research on domestic flow fluctuations.

The terms "inflow" and "infiltration" are used to describe flows into the sanitary sewer system from sources other than normal sewage connections. Inflow refers to connections of wastewater to the system which may not be septic in the biologic sense of the term. Such connections include roof drains, parking lot drains, misconnected storm water laterals, etc. Most inflows provide low pollutant levels with relatively high volume quantities. This water is excessively expensive to reclaim in the sanitary system and normally is better disposed of by other means. Frequently nonstructural solutions (e.g., sewer ordinances) are required to deal with areas of excessive inflows. On the other hand, infiltration refers to the inward seepage of groundwater into the collection system. The volume of such infiltration is a direct function of the length, size and condition of the collection system as well as the depth of the groundwater. Current EPA procedure is to determine if inflow/infiltration is excessive to a point where it is more cost-effective to replace and/or rehabilitate than to continue to transport and treat.

Like flow, the quality of raw sewage fluctuates from region to region. Table 1.3-2 gives a set of typical characteristics of domestic sewage representative of most urban areas. It must likewise be realized that heavy industrial or commercial loadings can have a dramatic effect on the values listed.

1.3.5 Unit Price Data. The accuracy of any cost estimate depends not only on the correct determination of sizes and quantities of material, equipment, and labor to be used on the project, but also on the unit price inputs used. The cost estimating technique used by the CAPDET model requires input of current unit prices if model accuracy is to be maintained. Default unit price data is available within the model and will be utilized in the calculation process if the user fails to input unit price data. While the CAPDET cost estimating methodology has reduced the number of unit price inputs required, there are some inputs still required.

Table 1.3-1 Residential Sewage Flows as Ratios to the Average

Flow	Ratio
Maximum daily	2.25 to 1
Maximum hourly	3.00 to 1
Minimum daily	0.67 to 1
Minimum hourly	0.33 to 1

Table 1.3-2 Typical Characteristics of Domestic Sewage (mg/l unless noted otherwise)

Parameter	Amount		
	High	Average	Low
BOD_5	350	200	100
COD	800	400	200
TOC	300	200	100
pH (units)	7.5	7.0	6.5
Total solids	1200	700	400
Suspended, total	350	200	100
Fixed	100	50	25
Volatile	250	150	75
Dissolved, total	850	500	300
Fixed	500	300	200
Volatile	350	200	100
Settleable solids (ml/l)	20	10	5
Total nitrogen (as N)	60	40	20
Free ammonia (as NH_3)	30	15	10
Total phosphorus (as P)	20	10	5
Chlorides (as Cl)	150	100	50
Sulfates (as SO_4)	40	20	10
Alkalinity (as $CaCO_3$)	350	225	150
Grease	150	100	50

Unit price inputs are divided into two categories. First, unit price inputs applying generally to all or several unit processes are input in the general data file. Second, those price inputs applying to a specific unit process are input on the ESTIMATE card contained in the unit process specification section of the input. A discussion of unit price input requirements is found in the CAPDET User's Guide. The following is a brief discussion of unit price information sources used for development of CAPDET.

1.3.5.1 Dodge Guide for Estimating Public Works Construction Costs. The Dodge Guide is a yearly publication from the McGraw-Hill Information Systems Company. This publication contains unit costs for labor, materials, and equipment for a wide range of public works construction projects. The costs are generated from sources throughout the United States and generally represent an average cost. The publication can be obtained from:

McGraw-Hill Infomation Systems Company
1221 Avenue of The Americas
New York, New York 10020

1.3.5.2 Means Building Construction Cost Data. The Means Cost Data is a yearly publication produced by the Robert Snow Means Company, Inc. This publication like the Dodge Guide presents average unit costs gathered from throughout the United States. The Means Building Construction Cost Data can be obtained from:

Robert Snow Means Company, Inc.
Construction Consultants & Publishers
100 Construction Plaza
Dixbury, Massachusetts 02332

1.3.5.3 "Chemical Engineering". "Chemical Engineering" is a magazine published biweekly by the McGraw-Hill Publication Company. The magazine is oriented toward the chemical process industry; however, since sewage treatment facilities use much of the same equipment, the cost index found in this publication should be more applicable to escalation of equipment costs than a general construction cost index. A subscription to this magazine may be obtained by writing:

Chemical Engineering
P. O. Box 507
Hightstown, New Jersey 08520

1.3.5.4 "Engineering News Record". "Engineering News Record" is a weekly magazine published by the McGraw-Hill Publications Company. The publication discusses a wide range of construction problems and techniques as well as construction management. This publication may be otained by writing:

Engineering New Record
P. O. Box 430
Hightstown, New Jersey 08520

1.3.5.5 "Journal Water Pollution Control Federation". The "Journal Water Pollution Construction Federation" is a monthly publication by the Water Pollution Control Federation. This publication may be obtained by writing:

Water Pollution Control Federation
3900 Wisconsin Avenue, N.W.
Washington, D.C. 20016

1.3.5.6 Local. The term "local" appearing in Table 1.3-3 indicates that this cost input is so variable that it must be obtained from sources in the local area where the facility is to be built.

1.3.5.7 Vendor. This term indicates that the cost input should be obtained from the equipment manufacturer or representative who distributes the particular type of equipment.

1.3.6 Optional Other Direct Construction Costs. The unit price cost estimating technique is applied to the calculation of costs associated with the construction of unit processes. In order to calculate total wastewater treatment facility construction costs, other cost components must be included. These cost components denoted as "other direct construction costs" are those components necessary to connect unit processes into a functioning treatment facility. Many of these componets are site related, i.e. will be required at some sites and not others, e.g. flood protection. Therefore, the CAPDET model allows specification of these cost components at the user's option.

As discussed in Section 1.2.3, the cost of these components is based on data collected by the EPA and published in FRD11. To improve the flexibility of the model, the user also has the option of specifying a dollar value for each cost component.

1.3.7 Indirect Project Costs. Total project costs also include various nonconstruction costs As in the case of the "other direct construction costs", the CAPDET model uses data collected by the EPA in preparation of FRD11. The user also has the option of overriding the EPA derived cost data by inputing a percentage which may be more appropriate for a particular project.

1.4 CAPDET COST EVALUATION TECHNIQUE

1.4.1 <u>General Cost Evaluation Procedures</u>. The purpose of cost evaluation in pollution control is the minimization of the costs required to achieve a certain water quality objective. This process is not to be confused with the concepts of economic optimization in which a system's output or production is maximized while minimizing its cost. In pollution control, the basic objective or output has been established by law or regulation. There remains the process of developing the least cost waste treatment management system to meet preestablished objectives rather than determining the economic tradeoffs resulting from improvements or degradation of water quality.

After the engineer or planner performs the initial cost calculations and determines estimates of the capital and operating and maintenance costs for each cost component in a waste treatment management system a cost analysis of alternative systems must be prepared. Before a true comparison of alternative systems can be made, a cost analysis recognizing the time value of money is required. The present worth or the equivalent annual cost method may be used in this analysis. The present worth method determines the sum which represents initial capital costs and the present monetary value of the time stream of operation and maintenance costs for a given interest rate. On the other hand, the equivalent annual cost method amortizes the capital expenditures and present value of operation and maintenance costs at a given interest rate over the planning period of the project. Both methods give an accurate indication of the relative monetary costs for alternative waste treatment management systems.

The CAPDET model generates both present worth and equivalent annual costs for each alternative waste treatment management system. The methodology used for these calculations is based as closely as possible on the cost-effectiveness analysis procedures promulgated by the Environmental Protection Agency. The remainder of this discussion is divided into two parts. Section 1.4.2 presents an overview of the relationship between CAPDET and the EPA cost-effectiveness guidelines whereas Section 1.4.3 et. seq. presents the detailed mathematics implemented within the model for calculating the monetary cost of an alternative in accordance with the guidelines.

1.4.2 <u>EPA Cost Effectiveness.</u>

1.4.2.1 General

Cost-effectiveness analyses guidelines have been promulgated by the Environmental Protection Agency under the authority of Sections 212(2)(c) of the Clean Water Act (PL 92-500 as amended by PL 95-217). Procedures for conducting a cost-effectiveness analysis are published as Appendix A to Subpart E of 40 CFR Part 35. The user of the CAPDET cost estimating model is referred to Appendix A for a detailed treatment of the requirements of the EPA cost-effectiveness analysis. This section is designed to briefly review the major monetary aspects of the analysis procedure and discuss the assumptions and rational implemented within CAPDET in order to comply as closely as possible with these regulations.

An Appendix A cost-effectiveness analysis is defined as "an analysis performed to determine which waste treatment management system or component part will result in the minimum total resources costs over time to meet Federal, State or local requirements." The total resource costs include both monetary and nonmonetary opportunity costs. The analysis of nonmonetary costs is based on a descriptive presentation of their significance and impacts. Nonmonetary factors include but may not be limited to such items as primary and secondary environmental effects, implementability, operability, performance reliability and flexibility. The analysis of the monetary aspects of a proposed alternative is based on use of time value of money concepts, i.e. present worth or equivalent annual cost. The most cost-effective alternative is that waste treatment management system which the analysis determines to have the lowest present worth or equivalent annual value unless the nonmonetary costs are overriding. In addition, the most cost-effective alternative must also meet the minimum requirements of applicable effluent limitations, groundwater protection, or other applicable standards established under the Clean Water Act.

The evaluation of monetary costs necessitates the preparation of various categories of data including: capital costs, operation and maintenance costs, revenues generated, inflation rate, discount rate, planning period, salvage values, useful life, and interest during construction.

Appendix A provides both specific and general guidance on formulation and use of each of these data inputs during the analysis process. The application of this guidance and its application to the cost estimating technique utilized in the CAPDET model are discussed below.

1.4.2.2 Capital Costs.

Capital construction costs are defined in Appendix A and essentially include all Step 3 project costs. The CAPDET model generates capital costs in four categories: unit process costs, other direct construction costs, land costs, and indirect project costs. Each of the costs developed in the CAPDET model must be included in the cost effectiveness analysis with the exception of the Step 2 design costs and the Step 1 facilities planning costs. The estimates generated by CAPDET for categories of costs are calculated on the basis of user specified unit prices for labor, materials and equipment. It is assumed that the model user will adjust these prices to reflect current market conditions at the site of the proposed project.

1.4.2.3 Operation and Maintenance.

The cost-effectiveness analysis must also include consideration of the annual costs for operation and maintenance of the wastewater treatment facilities. The CAPDET model categorizes annual operation and maintenance costs into five categories: operation labor, maintenance labor, chemicals, power, and materials.

Appendix A requires that these annual operation and maintenance costs be divided between fixed annual costs and costs which would depend on the annual quantity of waterwater collected and treated. The CAPDET model assumes that operation labor, maintenance labor and repair materials are fixed costs based on the design capacity of the treatment facility. Although technically incorrect, this assumption is made necessary by the lack of available data on the relationship of actual plant flow or age of equipment to these categories of costs. For practical purposes, the assumption that these costs are fixed is adequate considering overall model accuracy. The CAPDET model categorizes chemical costs and power costs as variable costs and calculates the cost of each as a function of flow.

The calculation of the variable costs is performed in a two step procedure. First, all operation and maintenance costs are calculated from quantities estimates generated for plant operation at design flow. The assumption is then made that design flow is reached in the final year of the planning period. The cost of the variable annual cost for the initial year is then calculated as a fraction of the design year variable cost based on the ratio of the initial years flow to the design year flow.

The present worth of the annual operation and maintenance costs is then calculated as the sum of the present worth of the fixed cost as determined from a uniform series and the present worth of the variable costs as determined from a uniform gradient series.

1.4.2.4 Revenues.

At the present stage of development, the CAPDET model does not calculate revenues which may result from implementation of any waste treatment management system. Projected revenues can, however, be directly input to the present worth analysis procedure for the overland flow and slow infiltration land treatment processes.

The present worth of revenues is calculated as the present worth of a uniform series.

1.4.2.5 Inflation.

In general, Appendix A does not sanction consideration of the effects of inflation on the cost of waste treatment management systems. The assumption is made that the general rate of inflation will affect all alternatives by approximately the same percentage. Two exceptions to the general rule are authorized. First, the cost of natural gas shall be escalated at a compound rate of 4 percent annually over the planning period unless the Regional Administrator approves a greater or lesser percentage. Second, land values may be escalated at a compound annual rate of 3 percent unless a greater or lesser percentage is approved by the regional administrator. The CAPDET program uses the 4 percent and 3 percent factors.

1.4.2.6 Discount Rate and Planning Period.

The evaluation interest (discount) rate is defined as the rate which is established annually by the Water Resources Council for evaluation of water resource projects planning period. The planning period defined by Appendix A as the period over which a waste treatment management system is evaluated for cost effectiveness is established as 20 years. The CAPDET user may input a variable planning period.

1.4.2.7 Useful Life.

Appendix A provides only general guidance for determining the useful life of various waste treatment management system alternatives. This general guidance is summarized as follows:

Land: permanent
Waste water conveyance structures: 50 years
Other structures: 30-50 years
Process equipment: 15-20 years
Auxiliary equipment: 10-15 years

Other useful life periods may be accepted upon sufficient justification. The CAPDET User's Guide provides information on anticipated equipment and structural service lives.

1.4.2.8 Salvage Value.

Appendix A provides only limited guidance on the treatment of salvage value. In general, the use of salvage value for those components other than land must be justified. It is doubtful that the salvage value of structures and process equipment of a 20-year old treatment plant will be significant. The relative significance of the salvage value will be further diluted when considered as a present worth value.

Depreciation is accomplished using a uniform depreciation over the useful life (straight line depreciation). Salvage values for unit processes are calculated for the structural component and equipment component. Items included as other direct construction costs and indirect costs are assumed to have a zero salvage value.

Calculation of a salvage value for land is somewhat unique. As stated above, Appendix A requires escalation of land values at the compound rate of 3 percent per year. Thus, the salvage value of land is somewhat greater than its initial cost.

The present worth of the salvage values is calculated using the single payment present worth factor.

1.4.2.9 Interest During Construction.

Appendix A allows the calculation of interest during construction by two alternative methods, depending on the length of the construction project. Section 1.4.3.2.1.5 includes detailed discussion of interest during construction.

1.4.2.10 Total Present Worth.

The total present worth is merely a summation of the present worth of the various cost components. The equivalent annual cost is calculated from the total present worth and the capital recovery factor for the appropriate interest rate. The detailed mathematics of the above discussion are presented in the following sections.

1.4.3 <u>Cost Effectiveness Calculations.</u>

1.4.3.1 <u>Unit Processes.</u>

1.4.3.1.1 Capital Costs.

Each unit process is made up of various construction elements such as equipment, structures, earthwork, etc., which have a capital cost associated with them. Also each element has a service life associated with it and as a result the capital cost is separated into initial cost and replacement cost.

1.4.3.1.1.1 Initial Cost.

The initial cost is simply the cost of the construction element at the time of construction. This is determined by adding the contractors overhead and profit to the calculated costs.

$$CEC = (CC)\ \frac{(1 + COAP)}{100}$$

where

CEC = construction element cost, \$.

CC = calculated construction cost, \$.

COAP = contractor's overhead and profit, %.

1.4.3.1.1.2 Replacement Cost.

This cost applies to those construction elements which have a service life less than the planning period and must therefore be replaced during the planning period. The guidelines for cost effective analyses do not allow for inflation of costs, except for land, and natural gas, so the replacement cost is the same as the initial cost.

1.4.3.1.2 Salvage Value.

The salvage value of an item is the value which can be assigned to it at the end of the planning period. A straight line depreciation is assumed for all construction elements except land which is permanent.

The unit processes can be broadly divided by construction elements into two components structural component and equipment component.

1.4.3.1.2.1 Structural Component. For purposes of calculating the salvage value of the structural componets a service life of 40 years is generally assumed, but may be changed by the model user.

$$SVSC = CEC\ \frac{RSST - PP}{RSST}$$

where

SVSC = salvage value of structural component, \$.

RSST = service life of structures, yrs.

CEC = construction element capital cost, \$.

PP = planning period.

1.4.3.1.2.2 Equipment Component. Each equipment component may have a different service life or replacement schedule. The calculation of salvage value is different for equipment with a service life less than the planning period and equipment with a service life greater than or equal to the planning period.

1.4.3.1.2.2.1 Replacement schedule less than the planning period.

$$SVEC = CEC \left(\frac{RS - [PP - (NR)(RS)]}{RS}\right)$$

$NR = \frac{PP}{RS}$ NR must be rounded down to an integer.

where

SVEC = salvage value of equipment component, $.

CEC = construction element capital cost, $.

RS = replacement schedule, or service life, yrs.

NR = number of replacements during the planning period.

PP = planning period, yrs.

1.4.3.1.2.2.2 Replacement schedule greater than or equal to the planning period.

$$SVEC = CEC \left(\frac{RS - PP}{RS}\right)$$

where

SVEC = salvage value of equipment component, $.

CEC = construction element capital cost, $.

RS = replacement schedule, or service life, yrs.

PP = planning period, yrs.

1.4.3.1.3 Operation and Maintenance Costs. There are two types of O&M costs, fixed and variable. The fixed costs are those costs which will remain the same throughout the planning period and will not change significantly as the flow increases during the planning period. The variable costs are those which vary directly with flow such as chemical costs and electrical costs. The CAPDET model assumes a straight line increase from the initial flow to the final flow during the planning period. The variable costs will be handled as an incremental cost which would be added each year.

1.4.3.1.3.1 Incremental O&M Costs.

$$IOMC = \frac{TOMCF - TOMCI}{PP}$$

where

IOMC = O&M incremental costs, $.

TOMCF = total O&M costs in the final year of the planning period, $, (based on design flow).

TOMCI = total O&M costs in the initial year of the planning period, $.

PP = planning period, yrs.

$$TOMCI = \frac{Q_{initial}}{Q_{design}} \; TOMCF$$

1.4.3.1.3.2 Fixed O&M costs.

$$FOMC = TOMCI - IOMC$$

where

FOMC = fixed O&M costs, $.

TOMCI = total O&M costs in the initial year of the planning period, $.

IOMC = incremental O&M costs, $.

1.4.3.1.4 Revenues. Some processes produce revenues from sale of products such as sludge or in the case of land treatment hay or corn. These revenues are considered in the cost effectiveness calculations as an annual decrease in operation costs.

1.4.3.1.5 Present Worth Calculations.

1.4.3.1.5.1 Capital Costs.

1.4.3.1.5.1.1 Initial Capital Costs. The initial capital cost does not have to be adjusted since it is a present cost.

1.4.3.1.5.1.2 Replacement Costs. The replacement cost must be adjusted by a present worth factor since the expenditure will be made sometime in the future.

$$PWRC = CEC\left[\frac{1}{(1+i)^{RS}} + \frac{1}{(1+i)^{2RS}} + \cdots + \frac{1}{(1+i)^{(NR)(RS)}}\right]$$

where

PWRC = present worth of replacement costs, \$.

CEC = construction element costs, \$.

i = interest, rate or discount rate set by the Water Resources Council, fraction.

RS = service life, or replacement schedule, yrs.

NR = number of replacements required during the planning period.

1.4.3.1.5.2 Salvage Value. The salvage value must be adjusted to detemine present worth since it is at the end of the planning period. The present worth factor is the same for all the salvage values which have been calculated.

1.4.3.1.5.2.1 Present worth of salvage of equipment component.

$$PWSEC = (SVEC)\left[\frac{1}{(1+i)^{PP}}\right]$$

where

PWSEC = present worth salvage value for equipment component, \$.

SVEC = salvage value of equipment component, \$.

i = interest or discount rate set by Water Resources Council, fraction.

PP = planning period, yrs.

1.4.3.1.5.2.2 Present worth of salvage of structural component.

$$PWSSC = (SVSC)\left[\frac{1}{(1+i)^{PP}}\right]$$

where

PWSSC = present worth salvage value for structural component, \$.

SVSC = salvage value of structural component, \$.

i = interest rate or discount rate set by Water Resources Council, fraction.

PP = planning period, yrs.

1.4.3.1.5.3 Operation and Maintenance Costs.

1.4.3.1.5.3.1 Present Worth of Fixed O&M Costs.

$$PWFOMC = \frac{(FOMC)\ [(1 + i)^{PP} - 1]}{i\ (1 + i)^{PP}}$$

where

PWFOMC = present worth of fixed O&M costs, \$.

FOMC = fixed O&M costs, \$/yr.

i = interest rate or discount rate set by the Water Resources Council, fraction.

PP = planning period, yrs.

1.4.3.1.5.3.2 Present worth of incremental O&M costs.

$$PWIOMC = (IOMC)\ [\frac{1}{i} - \frac{PP}{i}(\frac{i}{(1 + i)^{PP} - 1})]\ [\frac{(1 + i)^{PP} - 1}{i\ (1 + i)^{PP}}]$$

where

PWIOMC = present worth of incremental O&M costs, \$.

IOMC = incremental O&M costs, \$/yr.

i = interest rate or discount rate set by the Water Resource Council, fraction.

PP = planning period, yrs.

1.4.3.1.5.3.3 Present worth of total O&M costs.

$$PWTOMC = PWFOMC + PWIOMC$$

where

PWTOMC = present worth of unit process O&M costs, \$.

PWFOMC = present worth of fixed O&M costs, \$.

PWIOMC = present worth of incremental O&M costs, \$.

1.4.3.1.5.4 Revenues.

$$PWR = (AR)\ [\frac{(1 + i)^{PP} - 1}{i\ (1 + i)^{PP}}]$$

where

PWR = present worth of unit process annual revenues.

AR = unit process annual revenue, \$/yr.

i = interest rate or discount rate set by Water Resources Council, fraction.

PP = planning period, yrs.

1.4.3.1.5.5 Unit Process Present Worth. The unit process present worth is determined by adding the present worths of the costs involved in the unit process.

1.4.3.1.5.5.1 Initial Capital Costs.

$$UPCC = \sum CEC$$

where

$UPCC$ = unit process construction cost, \$.

CEC = construction element capital costs, \$.

1.4.3.1.5.5.2 Replacement costs.

$$UPWRC = \sum PWRC$$

where

$UPWRC$ = unit process present worth of replacement costs, \$.

$PWRC$ = present worth of replacement costs for construction elements, \$.

1.4.3.1.5.5.3 Salvage Value.

$$UPWSV = \sum PWSEC + \sum PWSSC$$

where

$UPWSV$ = unit process present worth of salvage value, \$.

$PWSEC$ = present worth of salvage for equipment components, \$.

$PWSSC$ = present worth of salvage value for structural components, \$.

1.4.3.1.5.5.4 Calculate Total Unit Process Present Worth.

$$UPPW = UPCC + UPWRC + PWTOMC - UPWSV - PWR$$

where

UPPW = total unit process present worth, $.

UPCC = unit process construction cost, $.

UPWRC = unit process present worth of replacement costs, $.

UPTOMC = present worth of unit process O&M costs, $.

UPWSV = unit process present worth of salvage value, $.

PWR = present worth of unit process annual revenue, $.

1.4.3.2 Treatment Facility.

1.4.3.2.1 Capital Cost.

Each facility is made up of various unit processes which have a capital cost associated with them. The total capital cost of a facility involves costs which have not been addressed in the unit processes, such as, other direct costs, indirect costs, land costs, and interest during construction.

1.4.3.2.1.1 Calculate Total of Unit Process Costs.

$$TUPC = \sum UPCC$$

where

TUPC = total of unit process costs, $.

UPCC = unit process construction costs, $.

1.4.3.2.1.2 Other direct costs. The other direct costs include items which are required to tie the unit processes together so that it will function as a complete facility. These costs include items such as yard piping, clearing and grubbing, instrumentation and administrative facilities. The other direct costs will be computed as described in Section 1.2.2.5

1.4.3.2.1.3 Land Costs. This is simply the cost of the land the facility is built on.

1.4.3.2.1.4 Indirect Costs. The indirect costs include items which are a part of the total costs for developing the facility. These costs include engineering fees, fees for technical services, legal and administrative fees, etc. The indirect costs will be computed as described in Section 1.2.2.6, however, Step 1 and Step 2 costs are not considered in the cost effectiveness analysis.

1.4.3.2.1.5 Interest During Construction. The interest during construction can be treated as capital cost. The interest is calculated based on total capital expenditures, the period of construction and the interest rate set by the Water Resources Council. The construction period is assumed to be 3 years if none is specified.

$$IDC = \frac{(TUPC + ODC + IC + LC)\ (P)\ (i)}{2}$$

where

IDC = interest during construction, \$.

TUPC = total of unit process costs, \$.

ODC = other direct costs, \$.

IC = indirect costs, \$.

P = construction period, yrs.

i = interest rate or discount rate set by the Water Resources Council, fraction.

LC = land costs, \$.

1.4.3.2.1.6 Total Project Costs.

$$TPC = TUPC + ODC + IC + LC + IDC$$

where

TPC = total project cost, \$.

TUPC = total of unit process costs, \$.

ODC = other direct costs, \$.

LC = land costs, \$.

IC = indirect costs, \$.

IDC = interest during construction.

1.4.3.2.2 Salvage Value.

1.4.3.2.2.1 Construction Elements. The salvage value of the construction elements for the total treatment facility is the sum of the salvage value calculated for each unit process.

$$TCESV = \sum SVEC + \sum SVSC$$

where

TCESV = total construction element salvage value, \$.

SVEC = salvage value of equipment component, \$.

SVSC = salvage value of structural component, \$.

1.4.3.2.2.2 Indirect Cost, Other Direct Costs, and Interest During Construction.

These items are assumed to have a life equal to the planning period so that they have no salvage value.

1.4.3.2.2.3 Land Costs. Land is treated differently from other items in that it may be escalated at a compound rate of 3% annually. A higher rate may be used if it can be justified by local conditions.

$$SVLC = LC \left(\frac{1 + ER}{100}\right)^{PP}$$

where

SVLC = salvage value of the land costs, \$.

LC = land costs, \$.

ER = escalation rate for land, percent.

PP = planning period, yrs.

1.4.3.2.3 Operation and Maintenance Costs. The operation and maintenance costs for the treatment facility is the sum of the O&M costs for the unit processes plus the costs for administration and laboratory labor and expenses.

1.4.3.2.4 Revenues. The revenues for the treatment facility is the sum of the revenues for each unit process.

1.4.3.2.5 Present Worth Calculations.

1.4.3.2.5.1 Initial Capital Costs. The initial capital cost does not have to be adjusted since it is a present cost.

1.4.3.2.5.2 Replacement Costs. The present worth of the replacement cost for the treatment facility is the sum of the present worth of the replacement costs for each unit process.

$$PWRCTF = \sum UPWRC$$

where

PWRCTF = present worth of replacement costs for treatment facility, $.

UPWRC = unit process present worth of replacement costs, $.

1.4.3.2.5.3 Salvage Value. The present worth of the salvage value for the treatment facility is equal to the sum of the present worth of the salvage value of each unit process plus the present worth of salvage value of the land costs.

$$PWTFS = UPWSV + SVLC \left[\frac{1}{(1 + i)^{PP}}\right]$$

where

PWTFS = present worth of treatment facility salvage value, $.

UPWSV = unit process present worth of salvage value, $.

SVLC = salvage value of land costs, $.

i = interest or discount rate as set by the Water Resources Council, fraction.

PP = planning period.

1.4.3.2.5.4 Operation and Maintenance Costs. The present worth of the operation and maintenance costs for the treatment facility is the sum of the present worth of the O&M costs for the unit processes plus the present worth of the administrative and laboratory labor and expenses.

$$OMPWTF = PWTOMC + \frac{ALC\ [(1 + i)^{PP} - 1]}{i\ (1 + i)^{PP}}$$

where

OMPWTF = O&M present worth for the treatment facility, $.

PWTOMC = present worth of unit process O&M costs, $.

ALC = administrative and laboratory labor and expenses, \$.

i = interest or discount rate as set by Water Resources Council, fraction.

PP = planning period, yrs.

1.4.3.2.5.5 Revenues. The present worth of the revenues for the treatment facility is the sum of the present worth of the revenues for each unit process.

$$PWRTF = \sum PWR$$

where

PWRTF = present worth of revenues for the treatment facility, \$.

PWR = present worth of the unit process annual revenue, \$.

1.4.3.2.5.6 Treatment Facility Total Present Worth.

$$TPWTF = TPC + OMPWTF + PWRCTF - PWTFS - PWRTF$$

where

TPWTF = total present worth of the treatment facility, \$.

TPC = total project costs, \$.

OMPWTF = O&M present worth for the treatment facility \$.

PWRCTF = present worth of replacement costs for the treatment facility, \$.

PWTFS = present worth of treatment facility salvage value, \$.

PWRTF = present worth of revenues for the treatment facility, \$.

1.5 WASTEWATER TREATMENT SYSTEM FUNDING

1.5.1 General. After the treatment processes have been selected and the wastewater treatment plant has been designed the next step is to build it. This requires large sums of money which are generally obtained from several sources; grants, loans, or bonds. The foremost concern of a municipality building a wastewater treatment plant is how to obtain the portion of the cost of the treatment facility which is not covered by a grant. This is nomally referred to as the "local share" of the cost. The local share is usually obtained by selling bonds or obtaining loans. The loans and bonds are then repaid over a period of time from the revenues generated from the wastewater treatment system. The CAPDET model develops the rate required to generate the revenues to operate the system and repay the bonds and loans. The methodology used is described below.

1.5.2 Funding Procedures.

1.5.2.1 Project Costs.

The project costs include, but are not limited to costs for planning, engineering, construction, administrative, and legal. Some items of cost are not eligible for EPA or state grants, such as land costs and right-of-way acquisition. For this reason project costs are segregated into grant eligible and grant ineligible costs.

1.5.2.2 Source of Funds.

The sources of funds available to finance the total project costs include federal grants and/or loans, state grants and/or loans, and locally issued bonds.

1.5.2.2.1 Grants.

The amount of grants by federal or state agencies are detemined as a percentage of the eligible project costs. Federal (EPA) grants are usually 75% unless the project is declared to be innovative or alternative in which case it is 85%. State grants vary from state to state. The CAPDET model allows the user to input variable rates.

1.5.2.2.2 Local Share.

The local share is detemined by subtracting the grant amount from the total project cost. The usual fom of fund raising by municipalities is from the sale of bonds. Bonds sold for construction of wastewater treatment systems are usually either general obligation bonds or revenue bonds. General obligation bonds are secured by the full faith and credit of the municipality and are amortized by tax levees. Revenue bonds are secured by a lien on the revenues of the system constructed from the proceeds of the bonds. CAPDET is based on the use of only revenue bonds.

When revenue bonds are sold to provide funds for the local share of a project, there are other costs included in the total amount of bonds sold.

The first cost is that of marketing the bonds and includes bond council fees, engineering fees, fiscal consultant fees, the costs of advertising the bonds for sale, and costs of printing the bonds.

The second cost is the amount of bonds sold to escrow interest during construction and funding of reserve funds; such as principle and interest reserves and repair and replacement reserves; if they are required by the bond resolution. The funding of interest during construction and reserve accounts are generally required only for wastewater treatment systems that do not have the capability of collecting user service charges until after the new system has been placed in service.

For this reason these costs are omitted from these calculations. Table 1.5-1 illustrates how the amount of revenue bonds to be sold is determined.

1.5.3 Development of User Charge.

After bonds have been sold the user charge must be adjusted to cover the operation and maintenance costs, principle and interest reserve accounts, contingency reserve accounts, as well as, debt service on the bonds.

1.5.3.1 Annual Principle and Interest Payments.

After bonds have been sold or loans have been procured, a table which shows the principle amortized each year together with the interest required is supplied. This is commonly referred to as the "Debt Service Table". There is no need to generate this table for the purposes of these calculations, therefore the capital recovery factor will be used to generate the combined annual payment of principle and interest.

TABLE 1.5-1

LOCAL SHARE CALCULATION EXAMPLE

A. ESTIMATED PROJECT COST		
EPA Step 1		$ 9,000
EPA Step 2		180,000
EPA Step 3		8,217,000
Sub-Total EPA Eligible Items		$8,406,000
Right-of-Way		450,000
Estimated Project Cost		$8,856,000
B. SOURCE OF FUNDS		
EPA Steps 1, 2, and 3 (75%)		$6,304,000
State Interest Free Loan (Eligible Items)		
Old	$ 23,000	
New	299,000	
		332,000
Local Share		2,230,000
Total Funds		$8,856,000
C. FUNDING LOCAL SHARE OF PROJECT COST		
Local Share for Construction		$2,230,000
Other Costs		
Allowance for Financing	$120,000	
Sub-Total - Other Costs		120,000
Revenue Bonds Required (1)		$2,350,000

(1) 30-year revenue bonds.

1.5.3.2 Debt Service Coverage.

Revenue bonds ordinances usually require that user charges be maintained during the life of the bond issue to insure adequate revenues to repay principle and pay current interest outstanding plus an additional percentage to create adequate surplus and fund reserve accounts.

"Debt Service Coverage" is the term usually applied to this surplus and is defined as the ratio of the annual net revenue of the wastewater treatment system divided by the annual principle and interest payment. The term net revenue is defined as the annual revenues derived from use service charges less the operation and maintenance cost of the system. Debt service coverage varies from 1.3 to 1.7 and depends primarily on the credit rating of the wastewater treatment system.

1.5.3.3 User Charges.

User Charges are calculated by dividing the total annual required revenues by the annual billing units. Billing units are normally in thousand gallons, so that dividing the annual required revenues by the thousands of gallons of water sold produces a rate in dollars per thousand gallons.

1.5.4 User Charge Calculations.

1.5.4.1 Input Data.

1.5.4.1.1 EPA grant participation, EPAG, %.

1.5.4.1.2 Other grant participation, OG, %.

1.5.4.1.3 Revenue Bonds.

1.5.4.1.3.1 Interest rate on Bonds, IB, %.

1.5.4.1.3.2 Number of years bonds financed, NB, yrs.

1.5.4.1.4 Loans.

1.5.4.1.4.1 Amount of Loan 1, L1, %.

1.5.4.1.4.2 Interest rate on Loan 1, I1, %.

1.5.4.1.4.3 Number of years Loan 1 financed, N1, yrs.

1.5.4.1.4.4 Amount of Loan 2, L2, %.

1.5.4.1.4.5 Interest rate on Loan 2, L2, %.

1.5.4.1.4.6 Number of years Loan 2 financed, N2, yrs.

1.5.4.1.5 Allowance for financing, AFF, %.

1.5.4.1.6 Number of billing units, BU, 1000 gal.

1.5.4.1.7 Persons per household, PPH.

1.5.4.1.8 Per capita water use, GPCD, gal/day.

1.5.4.1.9 Existing sewer rate, EBUR, $/BU.

1.5.4.2 Calculate eligible project cost.

1.5.4.2.1 All projects except land treatment.

$$TEC = TCC + COST1 + COST2$$

where

TEC = total eligible project cost, $.

TCC = total construction cost, $.

COST1 = Step 1 costs, $.

COST2 = Step 2 costs, $.

1.5.4.2.2 Land treatment projects.

$$TEC = TCC + COST1 + COST2 + LCOST$$

where

TEC = total eligible project costs, $.

TCC = total construction costs, $.

COST1 = Step 1 costs, $.

COST2 = Step 2 costs, $.

LCOST = land costs, $.

1.5.4.3 Calculate local share to be financed.

1.5.4.3.1 For all projects except land treatment.

$$LS = [(1.0 - \frac{EPAG + OG}{100}) (TEC) + LCOST] [1 + \frac{AFF}{100}]$$

where

LS = local share to be financed, \$.

EPAG = EPA grant participation, %.

OG = other grant participation, %.

TEC = total eligible project costs, \$.

AFF = allowance for financing, %.

LCOST = land cost, \$.

1.5.4.3.2 Land Treatment Projects.

$$LS = \left(1.0 - \frac{EPAG + OG}{100}\right) (TEC) \left(1 + \frac{AFF}{100}\right)$$

where

LS = local share to be financed, \$.

EPAG = EPA grant participation, %.

OG = other grant participation, %.

TEC = total eligible project costs, \$.

AFF = allowance for financing, %.

1.5.4.4 Calculate the annual debt service.

$$ADS = (LS - L1 - L2) \left[\frac{IB/100\ (1 + IB/100)^{NB}}{(1 + IB/100)^{NB} - 1}\right] + L1 \left[\frac{I1/100\ (1 + I1/100)^{N1}}{(1 + I1/100)^{N1} - 1}\right] + L2 \left[\frac{I2/100\ (1 + I2/100)^{N2}}{(1 + I2/100^{N2} - 1}\right]$$

where

ADS = annual debt service, \$.

LS = local share to be financed, \$.

IB = interest rate on bonds, %.

NB = number of years bonds financed, yrs.

L1 = amount of loan 1, L1, \$.

N1 = number of years Loan 1 financed, yrs.

L2 = amount of loan 2, \$.

I2 = interest on loan 2, %.

N2 = number of years loan 2 financed, yrs.

1.5.4.5 Calculate annual reserve accounts.

1.5.4.5.1 Principle and interest reserve.

$$PIR = \frac{ADS}{7}$$

where

PIR = annual principle and interest reserve, \$.

ADS = annual debt service, \$.

1.5.4.5.2 Contingency reserve.

$$CR = \frac{ADS}{7}$$

where

CR = annual contingency reserve, \$.

ADS = annual debt service, \$.

1.5.4.6 Calculate total annual operating cost of new system.

$$TAOCN = TOAMN + ADS + PIR + CR$$

where

TAOCN = total annual operating cost for new system, \$.

TOAMN = total annual O&M cost for new system, \$.

ADS = annual debt service, \$.

PIR = annual principle and interest reserve, \$.

CR = annual contingency reserve, \$.

1.5.4.6 Calculate cost per 1000 gallons treated for new system.

$$CPTG = \frac{TAOCN \times 1000}{Qave}$$

where

CPTG = annual cost per 1000 gallons treated new system. $.

TAOCN = total annual operating cost for new system, $.

Qave = design flow, mgd.

1.5.4.7 Calculate cost per 1000 gallons treated for total system.

$$CPTT = \frac{(TAOCN + EOMR)\ 1000}{Qave}$$

where

CPTT = cost per 1000 gallons treated total system, $.

TAOCN = total annual operating cots for new system, $.

EOMR = annual operating cost for existing system, $.

Qave = design flow, mgd.

1.5.4.8 Calculate cost per 1000 gallons for new system.

$$CPTN = \frac{TAOCN}{BU}$$

where

CTTN = annual cost per 1000 gallons for new system, $.

1.5.4.9 Calculate cost per 1000 gallons of water billed for new system.

$$CPTN = \frac{TAOCN \times 1000}{Qave}$$

where

CPTN = annual cost per 1000 gallons treated for new system, $.

TAOCN = total annual operating cost for new sytem, $.

BU = billing units.

1.5.4.10 Calculate cost per 1000 gallons water sold for total system.

$$CPTT = CPTN + EBUR$$

where

CPTT = cost per 1000 gal. for total system.

CPTN = cost per 1000 gal. for new portion of system.

EBUR = existing sewer rate.

1.5.4.11 Calculate cost per household for new system.

$$CPHN = \frac{CPTN(PPH)(GPCD)}{1000}$$

where

CPHN = cost per household for new system.

CPTN = cost per 1000 gal. for new portions of system.

PPH = persons per household.

GPCD = gallons per capita per day.

1.5.4.12 Calculate cost per household for total system.

$$CPHT = CPHN + \frac{(EBUR)(PPH)(GPCD)}{1000}$$

where

CPHT = cost per household for total system.

CPTN = cost per 1000 gal. for new portion of system.

PPH = persons per household.

GPCD = gallonsper capita per day.

2.0 TREATMENT PROCESS DESIGNS (MAJOR SYSTEM)

2.01 Contents of Section.

This section includes the equations and algorithms which are used to generate sizes and quantities of equipment and materials, cost of construction, and effluent quality for the unit processes shown in the preceding Table of Contents. Also this section gives the assumptions which have been made to simplify the calculations and minimize required input. The information presented in this Section applies to major systems which is defined as systems which have a wastewater flow greater than 0.5 mgd.

2.02 Organization of Section.

Each unit process is presented in a separate subsection and arranged in alphabetical order. Those unit processes which have a number of variations, have each variation presented as a separate, complete subsection. This causes some repetition but is convenient in that all the information concerning a unit process is found in that subsection. Each unit process subsection is introduced with a "Background" section which gives a description of the process and general design information. Each unit process subsection is further divided into three calculation sections: Process Design Calculations, Quantities Calculations, and Cost Calculations. Each of these is concluded with an output data subsection which enumerates the data which was generated.

Each unit process subsection is concluded with a bibliography with the references arranged in alphabetical order.

2.1 ACTIVATED SLUDGE

2.1.1 Background.

2.1.1.1 Activated sludge is defined as "sludge floc produced in raw or settled wastewater by the growth of zoogleal bacteria and other organisms in the presence of dissolved oxygen and accumulated in sufficient concentration by returning floc previously found." Similarly, the activated-sludge process is defined as "a biological wastewater treatment process in which a mixture of wastewater and activated sludge is agitated and aerated." The activated sludge is subsequently separated from the treated wastewater (mixed liquor) by sedimentation and wasted or returned to the process as needed.

2.1.1.2 In the past few decades, many modifications of this process have been developed, although only two process variations are significant: the conventional system, which achieves absorption, flocculation, and synthesis in a single step; and contact stabilization during which oxidation and synthesis of removed organics occur in a separate aeration tank.

2.1.1.3 In the following paragraphs, seven activated sludge process modifications and variations will be considered: conventional plug flow, complete mix, step aeration, modified-aeration or high-rate, contact stabilization, extended aeration, and pure oxygen system. While other modifications of the activated sludge process exist (namely tapered aeration and the Kraus process) they will not be included in this chapter.

2.1.2 General Description Complete Mix Activated Sludge.

2.1.2.1 There has been much discussion and some confusion concerning the definition of complete mix activated sludge. Nevertheless, any definition will be arbitrary and many differences of opinion will be aired. In this manual it will be assumed that a complete mix activated sludge is achieved when the oxygen uptake rate is uniform throughout all parts of the aeration tank and when sufficient mixing is provided to maintain the solids in the aeration tank in suspension. In complete mixing, the influent primary clarified wastewater and returned sludge flow are distributed at various points in the aeration tank. The tank serves to equalize or stabilize variations in flow and waste strength; it also acts as a diluent for toxic materials.

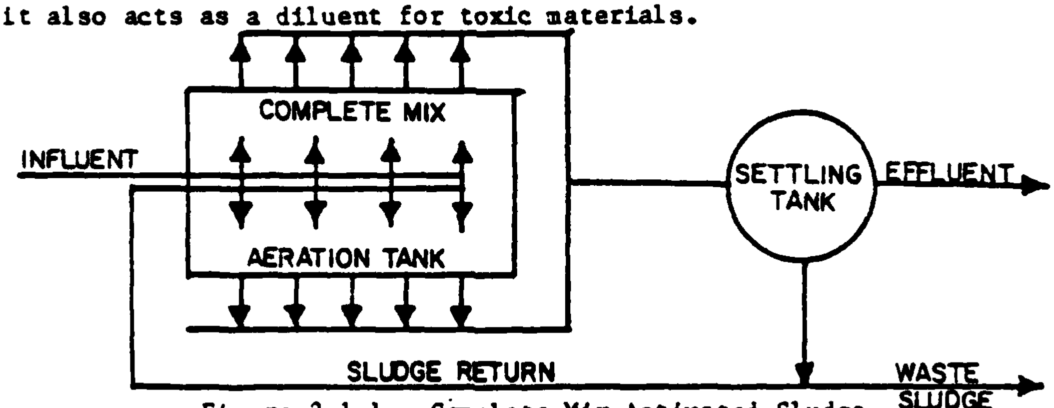

Figure 2.1-1. Complete Mix Activated Sludge

2.1.2.2 Organic loading and oxygen demand are uniform throughout the aeration tank, and mechanical or diffused aeration is used to completely mix the mixed liquor.

2.1.3 General Description Contact Stabilization Activated Sludge

2.1.3.1 The contact stabilization process is defined as "a modification of the activated sludge process in which raw wastewater is aerated with a high concentration of activated sludge for a short period, usually less than 60 min to obtain BOD removal by adsorption. The solids are subsequently removed by sedimentation and transferred to a stabilization tank where aeration is continued further to oxidize and condition them before their reintroduction to the raw wastewater flow".

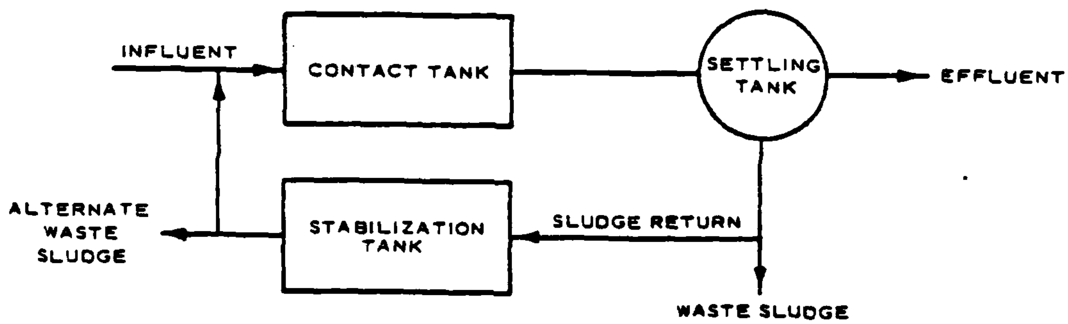

Figure 2.1-2. Contact Stabilization Activated Sludge

2.1.3.2 Contact stabilization was developed to take advantage of the absorptive properties of the sludge floc. Contact stabilization achieves adsorption in the contact tank, and oxidation and synthesis of removed organics occur in a separate aeration tank.

2.1.3.3 The volume requirement for aeration is approximately one-half of that of a conventional plug-flow unit. Therefore, it is often possible to double the capacity of an existing plug-flow plant by simply repiping or making minor changes in aeration equipment.

2.1.4 General Description Extended Aeration Activated Sludge

2.1.4.1 The extended aeration process is defined as "a modification of the activated sludge process which provides for aerobic sludge digestion within the aeration system. The concept envisages the stabilization of organic matter under aerobic conditions and disposal of the end products into the air as gases and with the plant effluent as finely divided suspended matter and soluble matter". The process operates in the endogenous respiration phase which requires a relatively small F/M ratio and long detention time. In the process, the aeration detention time is determined by the time required to oxidize the solids produced by synthesis from the BOD removed. The accumulation of volatile solids is very low and approaches the theoretical minimum; however, since some of the biological solids are inert, an accumulation of solids occurs in the system. Sludge storage facilities should be provided; most states make it a requirement. The flow chart for the extended aeration process is identical to that for plug flow.

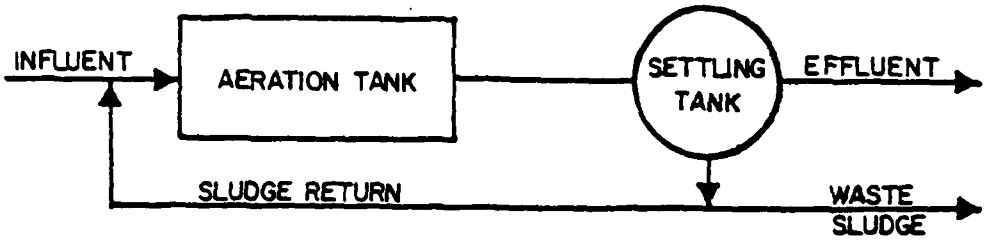

Figure 2.1-3. Extended Aeration Activated Sludge

2.1.5 General Description High-Rate Activated Sludge

2.1.5.1 Modified or high-rate aeration activated sludge is defined as "a modification of the activated sludge process in which a shortened period of aeration is used with a reduced quantity of suspended solids in the mixed liquor". The flow diagram is the same as that for the plug-flow process; the difference in the process design parameters (shorter detention time and lower mixed liquid suspended solids) results in less air requirements and, hence, less power consumption. Modified aeration is also characterized by a poor settling sludge and low BOD removal efficiencies.

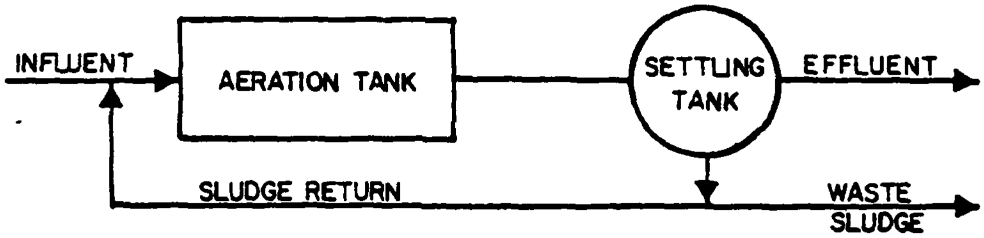

Figure 2.1-4 High-rate Activated Sludge

2.1.6 General Description Plug Flow Activated Sludge

2.1.6.1 The plug flow activated sludge process uses an aeration tank, a settling tank, and a sludge return line to treat wastewater.

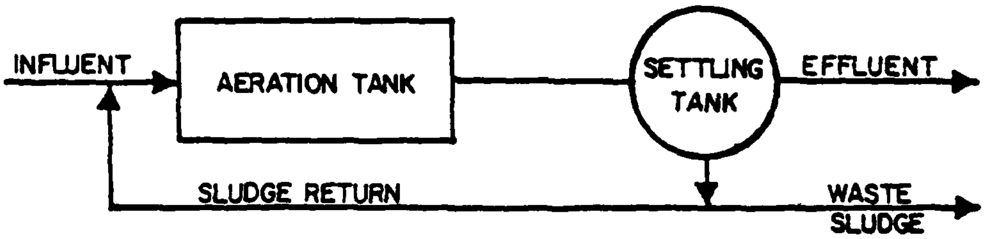

Figure 2.1-5. Plug Flow Activated Sludge

2.1.6.2 Wastewater and returned sludge from the secondary clarifier enter the head of the aeration tank to undergo a specified period of aeration. Diffused or mechanical aeration is used to provide the necessary oxygen and adequate mixing of the influent waste and recycled sludge, the concentration of which is high at the head of the tank and decreases with aeration time. Absorption, flocculation, and synthesis of the organic matter take place during aeration. The mixed liquor (sludge floc plus liquid in the aeration tank) is settled in the secondary clarifier, and sludge is returned at a rate sufficient to maintain the desired mixed liquor suspended solids in the aeration tank.

2.1.7 General Description Pure Oxygen Activated Sludge

2.1.7.1 As early as 1949, Okun reported the results of tests using pure oxygen as a substitute for air in the activated sludge process.

2.1.7.2 The pure oxygen system may be used for aeration in activated sludge systems that operate in either the plug flow or complete mix hydraulic regimes. It is readily adaptable to new or existing complete mix systems and can be used to upgrade and extend the life of overloaded plug-flow systems. To use the pure oxygen system, the aeration tanks must be covered and the oxygen introduced should be recirculated. The amount of oxygen that can be injected into the liquid (for a specific set of conditions) is approximately four times the amount that could be injected with an air system. Adjustment of pH may be necessary to maintain a proper balance between the CO_2 removed and buffer capacity of the wastewater.

2.1.7.3 Several advantages, such as increased bacterial activity, reduced aeration tank volume, decreased sludge volume, and better settling sludge have been cited for pure oxygen aeration. To substantiate these findings, however, further testing of this process in a number of varying applications will be necessary.

2.1.8 General Description Step Aeration Activated Sludge

2.1.8.1 Step aeration is defined as "a procedure for adding increments of settled wastewater along the line of flow in the aeration tanks of an activated sludge plant". It is a modification of the activated sludge process in which an attempt is made to equalize the food-to-microorganisms ratio (F/M). The first application of this process was at the Tallmans Island plant in New York City in 1939.

2.1.8.2 Baffles are used to split the aeration tank into four (or more) parallel channels. Each of these channels is a separate step and all channels are linked together in series.

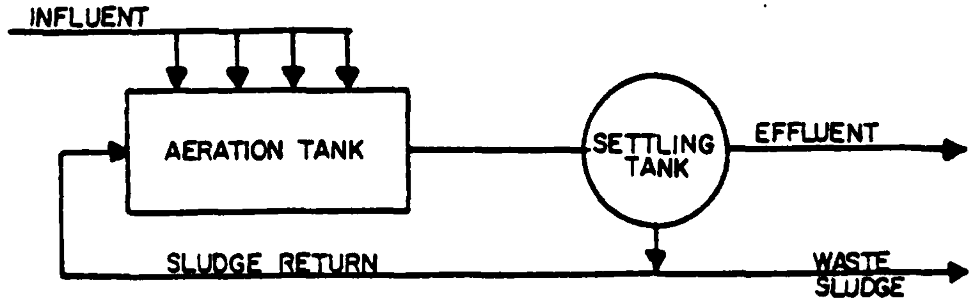

Figure 2.1-6. Step Aeration Activated Sludge

2.1.8.3 Flexibility of operation is a prime factor to consider in the design of the process. The oxygen demand is essentially uniform over the length of the basin, resulting in more efficient utilization of the oxygen supplied. Introduction of influent waste at multiple locations and return of a highly absorptive sludge floc permit this process to remove soluble organics within a relatively short contact time; therefore, it is characterized by higher volumetric loadings than the conventional plug flow activated sludge process.

2.1.9 Complete Mix Activated Sludge (Diffused Aeration).

2.1.9.1 Input Data.

2.1.9.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.9.1.2 Wastewater Strength.

2.1.9.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.9.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.9.2.2.3 Suspended solids, mg/l.

2.1.9.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.9.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.9.1.3 Other Characterization.

2.1.9.1.3.1 pH.

2.1.9.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.9.1.3.3 Nitrogen,[1] mg/l.

2.1.9.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.9.1.3.5 Oils and greases, mg/l.

2.1.9.1.3.6 Heavy metals, mg/l.

2.1.9.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.9.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.9.1.4 Effluent Quality Requirements.

2.1.9.1.4.1 BOD_5, mg/l.

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[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.9.1.4.2 SS, mg/l.

2.1.9.1.4.3 TKN, mg/l.

2.1.9.1.4.4 P, mg/l.

2.1.9.1.4.5 Total nitrogen (TKN + NO_3 - N), mg/l.

2.1.9.1.4.6 Settleable solids, mg/l/hr.

2.1.9.2 Design Parameters.

2.1.9.2.1 Reaction rate constants and coefficients.

Constants	Range
Eckenfelder	
k	0.0007-0.002 l/mg/hr
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
f	0.40
f'	0.53

2.1.9.2.2 F/M = (0.3-0.6).

2.1.9.2.3 Volumetric loading = 50-120.

2.1.9.2.4 t = (3-6) hr.

2.1.9.2.5 t_s = (3-7) days.

2.1.9.2.6 MLSS = (3000 - 6000) mg/l.

2.1.9.2.7 MLVSS = 0.7 MLSS = (2100 - 4200) mg/l.

2.1.9.2.8 Q_r/Q = (0.25 - 1.0).

2.1.9.2.9 lb 0_2/lb $BOD_r \geq 1.25$.

2.1.9.2.10 lb solids/lb BOD_r = (0.5 - 0.7).

2.1.9.2.11 0 = (1.0 - 1.04).

2.1.9.2.12 Efficiency = (>90 percent).

2.1.9.3 Process Design Calculations.

2.1.9.3.1 Assume the following design parameters from above when unknown.

2.1.9.3.1.1 BOD removal rate constant (k).

2.1.9.3.1.2 Fraction of BOD synthesized (a).

2.1.9.3.1.3 Fraction of BOD oxidized for energy (a').

2.1.9.3.1.4 Endogenous respiration rate (b and b').

2.1.9.3.1.5 Mixed liquid suspended solids (MLSS).

2.1.9.3.1.6 Mixed liquid volatile suspended solids (MLVSS).

2.1.9.3.1.7 Food-to-microorganism ratio (F/M).

2.1.9.3.1.8 Nonbiodegradable fraction of VSS in influent (f).

2.1.9.3.1.9 Degradable fraction of the MLVSS (f').

2.1.9.3.1.10 Temperature correction coefficient (0).

2.1.9.3.2 Adjust rate constant for temperature.

$$K_T = K_{20}0^{(T-20)}$$

where

K_T = rate constant at desired temperature, ^{o}C.

K_{20} = rate constant at $20^{o}C$.

0 = temperature correction coefficient.

T = temperature, ^{o}C.

2.1.9.3.3 Determine the size of the aeration tank by first determining the detention time t ·

$$t = \frac{24S_o}{(X_V)\ (F/M)}$$

where

t = hydraulic time, hr

S_o = influent BOD_5, mg/l.

X_V = MLVSS, mg/l

F/M = food-to-microorganism ratio

2.1.9.3.4 Check detention time for treatability

$$\frac{S_e}{S_o} = \frac{1}{1 + kX_V t}$$

where

S_e = BOD_5 (soluble) in effluent, mg/l.

S_o = BOD_5 in influent, mg/l.

k = BOD removal rate constant, l/mg/hr

X_V = MLVSS, mg/l.

t = detention time, hr.

Solve for t and compare with t above and select the larger.

2.1.9.3.5 Calculate the volume of aeration tank.

$$V = Q_{avg} \times \frac{t}{24}$$

where

V = volume, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.9.3.6 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'S_r}{t} + b'X_V$$

or

$$O_2 = a' (S_r) (Q_{avg}) (8.34) + b' (X_V) (V) (8.34)$$

where

dO/dt = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed (S_o - S_e), mg/l.

t = detention time, hr.

b' = endogenous respiration, 1/hr.

X_V = MLVSS.

O_2 = oxygen requirement, lb/day.

Q_{avg} = average flow rate, mgd.

V = volume of aeration tank, million gal.

and check the oxygen supplied against $\geq$ 1.25.

$$\text{lb } O_2/\text{lb } BOD_r = \frac{O_2}{Q(S_r) \times 8.34}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.9.3.7 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing 0.1 hp/1000 gal.

2.1.9.3.7.1 Standard transfer efficiency, percent, from manufacturer (5-8 percent).

2.1.9.3.7.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.9.3.7.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.9.3.7.4 Correction factor for pressure $\approx$ 1.0.

2.1.9.3.8 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.9.3.9 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{[(C_s)_T(\beta)(p) - C_L]}{9.17} \alpha \, (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, ^oC, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

T = temperature, ^oC.

2.1.9.3.10 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{O_2(10^5)\ (7.43)}{(OTE)\quad 0.0176\ \frac{lb\ O_2}{ft^3 air}\quad 1440\ \frac{min}{day}\quad V}$$

where

R_a = required air flow, cfm/1000 ft^3.

O_2 = required oxygen, lb/day.

OTE = operating transfer efficiency, percent.

V = volume of basin, gal.

2.1.9.3.11 Calculate sludge production.

$$\Delta X_v = [aS_rQ_{avg} - bX_VV + fQ\ (VSS) + Q(SS - VSS)]\ 8.34$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

b = endogenous respiration rate/day.

X_V = MLVSS, mg/l.

V = volume of basin, gal.

f = nonbiodegradable fraction of influent VSS.

Q = flow, mgd.

VSS = volatile suspended solids in effluent, mg/l.

SS = suspended solids in influent, mg/l.

2.1.9.3.12 Calculate solids produced per pound of BOD_5 removed and check X_V against value given.

$$\frac{\text{lb solids}}{(\text{lb } BOD_r)} = \frac{\Delta X_V}{S_r\ (Q)\ (8.34)}$$

where

ΔX_V = sludge produced, lb/day.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.1.9.3.13 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q} = \frac{X_a}{X_u - X_a}$$

where

Q_r/Q = sludge recycle ratio.

Q_r = volume of recycled sludge, mgd.

Q = flow, mgd.

X_a = MLSS, mg/l.

X_u = solids concentration in return sludge, mg/l.

2.1.9.3.14 Calculate solids retention time.

$$SRT = \frac{(V)X_a(8.34)}{\Delta X_a}$$

where

SRT = solids retention time, days.

$$\Delta X_a = \frac{\Delta X_V}{\%\ \text{volatile}}$$

2.1.9.3.15 Effluent Characteristics.

2.1.9.3.15.1 BOD_5.

$$BODE = Se + 0.84\ (X_V)_{eff}\ f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.9.3.15.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.9.3.15.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.9.3.15.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.9.3.15.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.9.3.15.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.9.3.16 Determine nutrient requirements, lb/day.

for nitrogen

$$N = 0.123\ \Delta M_T (\text{or } \Delta X_V)$$

and phosphorus

$$P = 0.026\ \Delta M_T (\text{or } \Delta X_V)$$

where

M_T = sludge produced, lb/day.

ΔX_V = sludge produced, lb/day.

and check against BOD:N:P = 100:5:1.

2.1.9.4 Process Design Output Data

2.1.9.4.1 Aeration Tank.

2.1.9.4.1.1 Reaction rate constant, l/mg/hr.

2.1.9.4.1.2 Sludge produced per BOD removed.

2.1.9.4.1.3 Endogenous respiration rate (b, b').

2.1.9.4.1.4 O_2 utilized per BOD removed.

2.1.9.4.1.5 Influent nonbiodegradable VSS (f).

2.1.9.4.1.6 Effluent degradable VSS (f').

2.1.9.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.9.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.9.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.9.4.1.10 Aeration time, hr.

2.1.9.4.1.11 Volume of aeration tank, million gal.

2.1.9.4.1.12 Oxygen required, lb/day.

2.1.9.4.1.13 Sludge produced, lb/day.

2.1.9.4.1.14 Nitrogen requirement, lb/day.

2.1.9.4.1.15 Phosphorus requirement, lb/day.

2.1.9.4.1.16 Sludge recycle ratio, percent.

2.1.9.4.1.17 Solids retention time, days.

2.1.9.4.2 Aeration System.

2.1.9.4.2.1 Standard transfer efficiency, percent.

2.1.9.4.2.2 Operating transfer efficiency, percent.

2.1.9.4.2.3 Required air flow, cfm/1000 ft^3.

2.1.9.4.3 Effluent BOD_5 concentration, BODE, mg/l.

2.1.9.4.4 Effluent soluble BOD_5 concentration, Se, mg/l.

2.1.9.4.5 Effluent COD concentration, CODE, mg/l.

2.1.9.4.6 Effluent soluble COD concentration, CODSE, mg/l.

2.1.9.4.7 Effluent total Kjeldahl nitrogen concentration, TKNE, mg/l.

2.1.9.4.8 Effluent ammonia nitrogen concentration, NH3E, mg/l.

2.1.9.4.9 Effluent phosphorus concentration, PO4E, mg/l.

2.1.9.4.10 Effluent oil and grease concentration, OAGE, mg/l.

2.1.9.5 <u>Quantities Calculations</u>.

2.1.9.5.1 Design values for activated sludge system.

$$V_d = V \frac{10^6}{7.48}$$

$$CFM_d = (Ra)\ (V)\ (133.7)$$

where

V = volume of aeration tanks, million gallons.

2.1.9.5.2 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

where

NT = number of aeration tanks per battery.

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.9.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.9.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.9.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.9.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.9.5.3.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.9.5.4 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

NT = number of aeration tanks per battery.

2.1.9.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.1.9.5.6 Design of aeration tanks.

2.1.9.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.9.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.1.9.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.1.9.5.7 Aeration tank arrangements.

2.1.9.5.7.1 Figure 2.1-7 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\ (\frac{Q_{avg}}{NB})$$

where

PGW = pipe gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.9.5.8 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.1.9.5.8.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ [\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.1.9.5.8.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ [\frac{(15.75(NT)+15.5)\ (2L+PGW+20)\ +\ (15.75(NT)+2.5)\ (2L+PGW+28)}{2}]$$

2.1.9.5.9 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.1.9.5.9.1 For NT less than 4:

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

2.1.9.5.9.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)\ (2L + PGW + 200)]$$

2.1.9.5.10 Reinforced concrete wall quantities.

2.1.9.5.10.1 The wall constructions are different for complete mix and plug flow systems. In order to achieve complete mix, the inflows to the aeration tanks must be distributed uniformly along one side of the aeration tank, flowing across the width of the tank and being discharged along the other.

2.1.9.5.10.2 When NT is less than 4:

$$V_{cw} = [(L(30.3) + NT\ [(L)(29.4) + 1372.5)] + 22.5]\ (NB)$$

where

V_{cw} = R.C. wall quantity required, cu ft.

L = length of aeration tanks, ft.

2.1.9.5.10.3 When NT is equal to or greater than 4:

$$V_{cw} = [30.3(NT)\ (L) + 57(L) + 1417.5\ (NT) + 45(PGW) + 135]\ NB$$

2.1.9.5.11 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

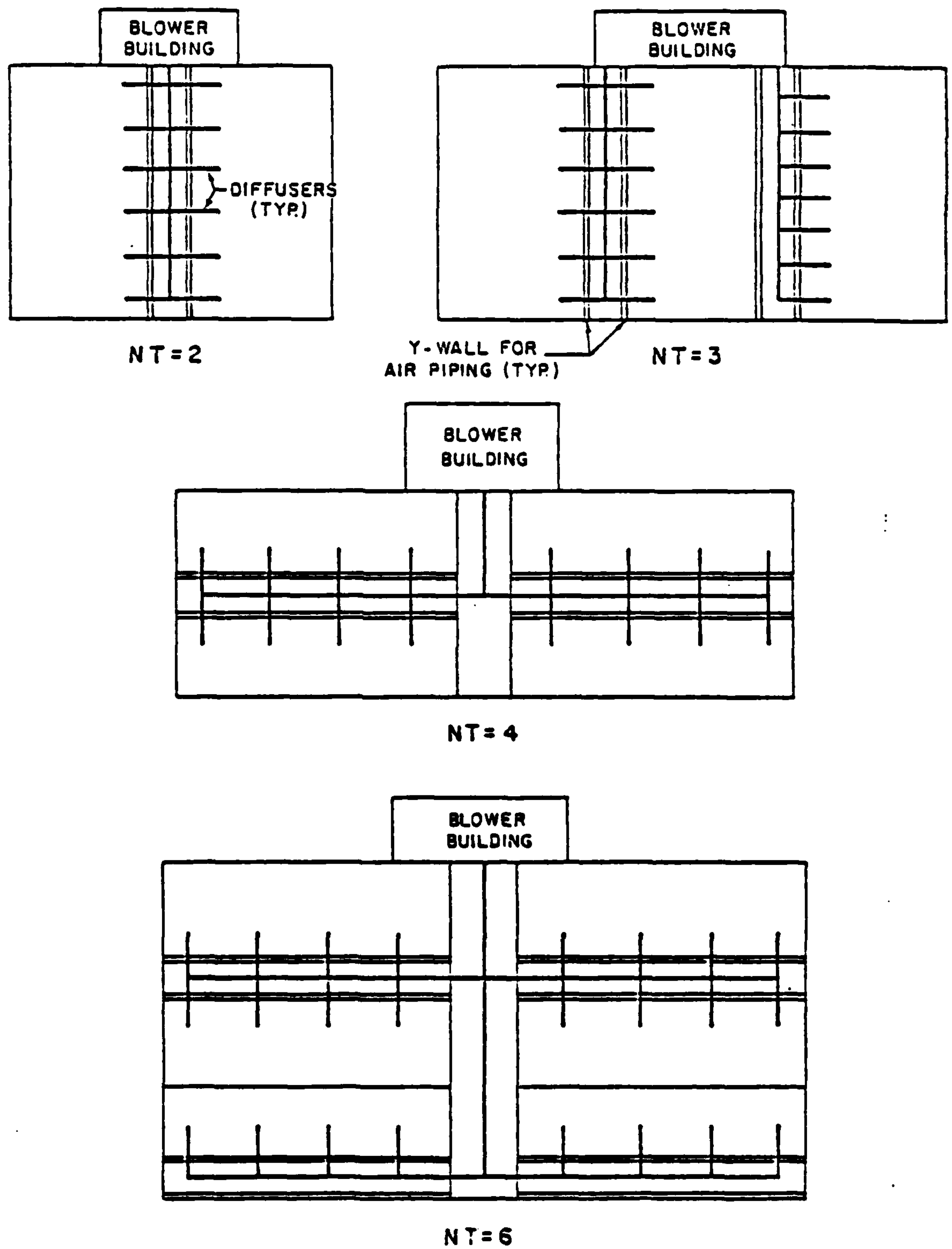

FIGURE 2.1.-7 AERATION TANK ARRANGEMENT

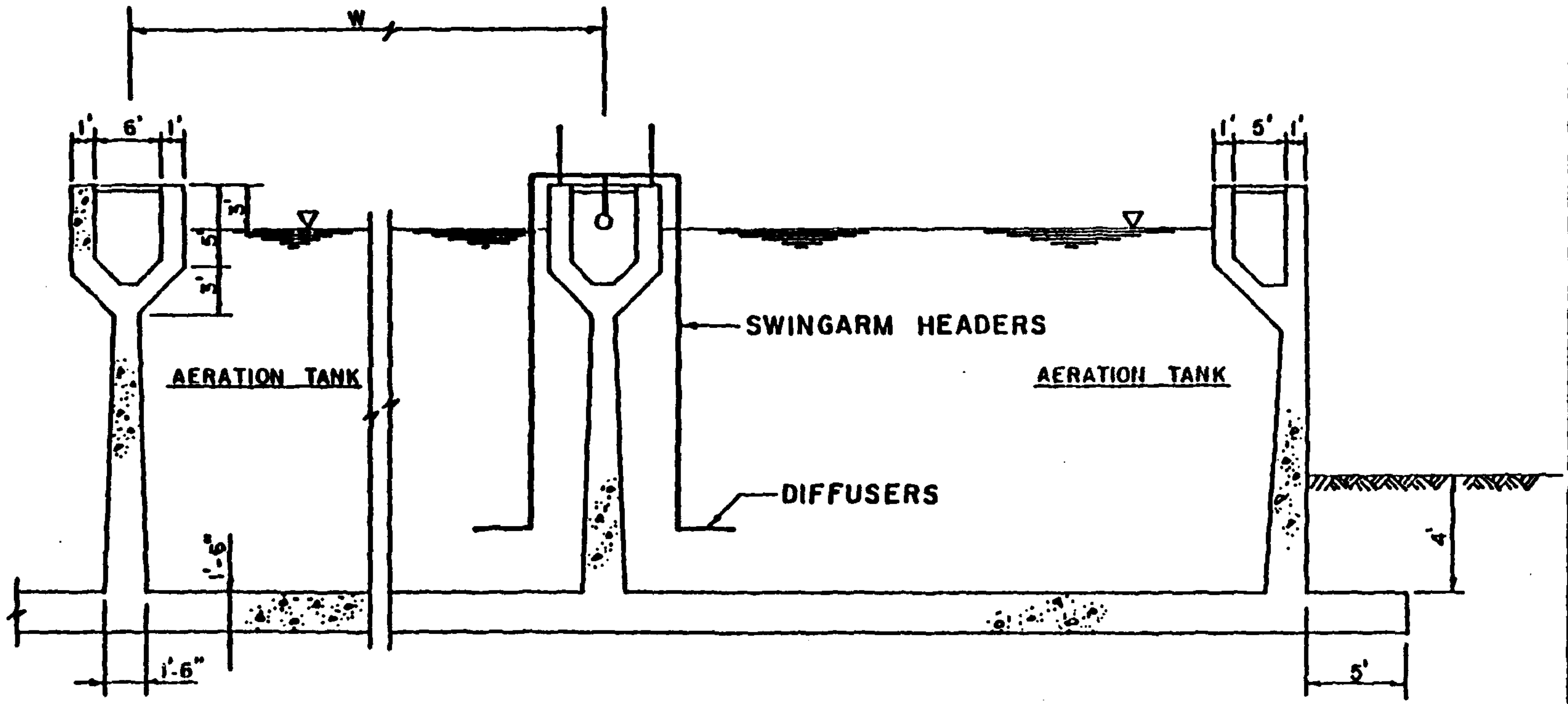

FIGURE 2.1.-8 - TYPICAL SECTION OF COMPLETE MIX DIFFUSED AERATION TANKS

2.1.9.5.11.1 If NT is less than 4:

$$LHR = [2(NT)\ (L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.1.9.5.11.2 If NT is greater than or equal to 4:

$$LHR = [2(NT)\ (L) + (4L) + 36.5(NT) + 2\ PGW + 13]\ NB$$

where

LHR = handrail length, ft.

2.1.9.5.12 Calculate operation manpower requirements.

2.1.9.5.12.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.1.9.5.12.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.1.9.5.13 Calculate maintenance manpower requirements.

2.1.9.5.13.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.1.9.5.13.2 If CFM_d > 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.1.9.5.14 Energy requirement for operation. The electrical energy required for operation is related to the air requirement by the following equation:

$$KWH = (CFM_d)\ (241.6)$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.1.9.5.15 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.1.9.5.16 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.9.6 Quantities Calculation Output Data.

2.1.9.6.1 Number of aeration tanks, NT.

2.1.9.6.2 Number of diffusers per tank, ND_t.

2.1.9.6.3 Number of process batteries, NB.

2.1.9.6.4 Number of swing arm headers per tank, NSA_t.

2.1.9.6.5 Length of aeration tanks, L, ft.

2.1.9.6.6 Width of pipe gallery, PGW, ft.

2.1.9.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.1.9.6.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.1.9.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.1.9.6.10 Quantity of handrail, LHR, ft.

2.1.9.6.11 Operation manpower requirement, OMH, MH/yr.

2.1.9.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.1.9.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.1.9.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.1.9.6.15 Correction factor for minor construction costs, CF.

2.1.9.7 <u>Unit Price Input Required</u>.

2.1.9.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.9.7.2 Unit price ~~input~~ R.C. wall in-place, UPICW, \$/cu yd.

2.1.9.7.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.1.9.7.4 Unit price input for handrails in-place, UPIHR, \$/ft.

2.1.9.7.5 Cost per diffuser, COSTPD, \$, (optional).

2.1.9.7.6 Cost per swing arm header, COSTPH, \$, (optional).

2.1.9.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.9.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.1.9.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.1.9.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.1.9.8 <u>Cost Calculations</u>.

2.1.9.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.9.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.1.9.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price R.C. slab in-place, \$/cu yd.

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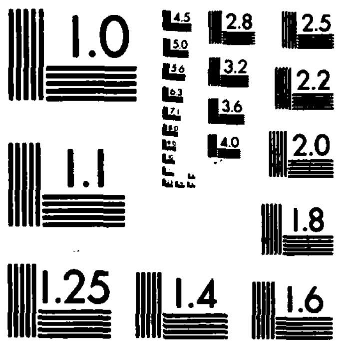


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

2.1.9.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.1.9.8.5 Cost of diffusers.

2.1.9.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.1.9.8.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, \$.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.1.9.8.6 Cost of swing arm diffuser headers.

2.1.9.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5,000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5,000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.9.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, $.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.1.9.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.1.9.8.8 Crane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.1.9.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.1.9.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.1.9.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.1.9.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.1.9.8.10 Other costs associated with the installed equipment. This category includes the cost for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.9.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\left(1 + \frac{PMINC}{100}\right) + (IMH)(LABRI) + (CH)(UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.1.9.8.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.1.9.8.13 Operation and maintenance material costs.

$$OMCC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/yr.

OMMP = operation and maintenance material supply costs, as percent of total bare construction cost, percent.

2.1.9.9 <u>Cost Calculations Output Data.</u>

2.1.9.9.1 Total bare construction cost of diffused aeration activated sludge system, TBCC, dollars.

2.1.9.9.2 Operation and maintenance material and supply costs, OMMC, dollars.

2.1.10 <u>Complete Mix Activated Sludge (Mechanical Aeration)</u>.

2.1.10.1 Input Data.

2.1.10.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.10.1.2 Wastewater Strength.

2.1.10.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.10.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.10.1.2.3 Suspended solids, mg/l.

2.1.10.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.10.1.2.5 Nonbiodegrable fraction of VSS, mg/l.

2.1.10.1.3 Other Characterization.

2.1.10.1.3.1 pH.

2.1.10.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.10.1.3.3 Nitrogen,[1] mg/l.

2.1.10.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.10.1.3.5 Oils and greases, mg/l.

2.1.10.1.3.6 Heavy metals, mg/l.

2.1.10.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.10.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.10.1.4 Effluent Quality Requirements.

2.1.10.1.4.1 BOD_5, mg/l.

2.1.10.1.4.2 SS, mg/l.

2.1.10.1.4.3 TKN, mg/l.

2.1.10.1.4.4 P, mg/l.

2.1.10.1.4.5 Total nitrogen (TKN + NO_3 - N), mg/l.

2.1.10.1.4.6 Settleable solids, mg/l/hr.

2.1.10.2 Design Parameters.

2.1.10.2.1 Reaction rate constants and coefficients.

Constants	Range
Eckenfelder	
k	0.0007-0.002 l/mg/hr
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
f	0.40
f	0.53

2.1.10.2.2 F/M = (0.3-0.6).

2.1.10.2.3 Volumetric loading = 50-120.

2.1.10.2.4 t = (3-6) hr.

2.1.10.2.5 t_s = (3-7) days.

2.1.10.2.6 MLSS = (3000-6000) mg/l.

2.1.10.2.7 MLVSS = 0.7 MLSS = (2100-4200) mg/l.

2.1.10.2.8 Q_r/Q = (0.25-1.0).

2.1.10.2.9 lb O_2/lb $BOD_r \geq 1.25$.

2.1.10.2.10 lb solids/lb BOD_r = (0.5-0.7).

2.1.10.2.11 0 = (1.0-1.04).

2.1.10.2.12 Efficiency = (>90 percent).

2.1.10.3 Process Design Calculations.

2.1.10.3.1 Assume the following design parameters from above when unknown.

2.1.10.3.1.1 BOD removal rate constant (k).

2.1.10.3.1.2 Fraction of BOD synthesized (a).

2.1.10.3.1.3 Fraction of BOD oxidized for energy (a').

2.1.10.3.1.4 Endogenous respiration rate (b and b').

---

[1] The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.10.3.1.5 Mixed liquid suspended solids (MLSS).

2.1.10.3.1.6 Mixed liquid volatile suspended solids (MLVSS).

2.1.10.3.1.7 Food-to-microorganism ratio (F/M).

2.1.10.3.1.8 Nonbiodegradable fraction of VSS in influent (f).

2.1.10.3.1.9 Degradable fraction of the MLVSS (f').

2.1.10.3.1.10 Temperature correction coefficient (0).

2.1.10.3.2 Adjust rate constant for temperature.

$$K_T = K_{20} 0^{(T-20)}$$

where

K_T = rate constant at desired temperature, ^oC.

K_{20} = rate constant at 20^oC.

0 = temperature correction coefficient.

T = temperature, ^oC.

2.1.10.3.3 Determine the size of the aeration tank by first determining the detention time t.

$$t = \frac{24S_o}{(X_V)(F/M)}$$

where

t = hydraulic time, hr.

S_o = influent BOD_5, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.10.3.4 Check detention time for treatability.

$$\frac{S_e}{S_o} = \frac{1}{1 + kX_V t}$$

where

S_e = BOD_5 (soluble) in effluent, mg/l.

S_o = BOD_5 in influent, mg/l.

k = BOD removal rate constant, 1/mg/hr.

X_V = MLVSS, mg/l.

t = detention time, hr.

Solve for t and compare with t above and select the larger.

2.1.10.3.5 Calculate the volume of aeration tank.

$$V = Q_{avg} \times \frac{t}{24}$$

where

V = volume, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.10.3.6 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'S_r}{t} + b'X_V$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34) + b'(X_V)(V)(8.34)$$

where

dO/dt = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed $(S_o - S_e)$, mg/l.

t = detention time, hr.

b' = endogenous respiration, 1/hr.

X_V = MLVSS

O_2 = oxygen requirement, lb/day.

Q_{avg} = average flow rate, mgd.

V = volume of aeration tank, million gal.

and check the oxygen supplied against 1.25.

$$\text{lb } O_2/\text{lb } BOD_r = \frac{O_2}{Q(S_r) \times 8.34}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S = BOD removed, mg/l.

2.1.10.3.7 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing 0.1 hp/1000 gal.

2.1.10.3.7.1 Standard transfer efficiency, lb/hp-hr (0 dissolved oxygen, 20°C, and tap water) (3-5 lb/hp-hr).

2.1.10.3.7.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.10.3.7.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.10.3.7.4 Correction factor for pressure $\approx$ 1.0.

2.1.10.3.8 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.10.3.9 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE\ \frac{[(C_s)_T(\beta)(p) - C_L]}{9.17}\ \alpha\ (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, ^{o}C.

2.1.10.3.10 Calculate horsepower requirement.

$$hp = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)(V)} \times 1000$$

where

hp = horsepower required/1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of basin, gal.

2.1.10.3.11 Calculate sludge production.

$$\Delta X_V = [aS_rQ_{avg} - bX_VV + fQ(VSS) + Q(SS - VSS)]\ 8.34$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

b = endogenous respiration rate/day.

X_V = MLVSS, mg/l.

V = volume of basin, gal.

f = nonbiodegradable fraction of influent VSS

Q = flow, mgd.

VSS = volatile suspended solids in effluent, mg/l.

SS = suspended solids in influent, mg/l.

2.1.10.3.12 Calculate solids produced per pound of BOD_5 removed and check ΔX_V against value given.

$$\frac{lb\ solids}{(lb\ BOD_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)}$$

where

ΔX_V = sludge produced, lb/day.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.1.10.3.13 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q} = \frac{X_a}{X_u - X_a}$$

where

Q_r/Q = sludge recycle ratio.

Q_r = volume of recycled sludge, mgd.

Q = flow, mgd.

X_a = MLSS, mg/l.

X_u = solids concentration in return sludge, mg/l.

2.1.10.3.14 Calculate solids retention time.

$$SRT = \frac{(V)\ X_a(8.34)}{X_a}$$

where

SRT = solids retention time, days.

$$\Delta X_a = \frac{\Delta X_V}{\%\ \text{volatile}}$$

2.1.10.3.15 Effluent Characteristics.

2.1.10.3.15.1 BOD_5.

$$BODE = Se + 0.84\ (X_V)_{eff}\ f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.10.3.15.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.10.3.15.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.10.3.15.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.10.3.15.5 Oil and grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.10.3.15.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.10.3.16 Determine nutrient requirements, lb/day.

for nitrogen

$$N = 0.123\ \Delta M_T (\text{or } \Delta X_V)$$

and phosphorus

$$P = 0.026\ \Delta M_T (\text{or } \Delta X_V)$$

where

ΔM_T = sludge produced, lb/day.

ΔX_V = sludge produced, lb/day.

and check against BOD:N:P = 100:5:1

2.1.10.4 <u>Process Design Output Data</u>.

2.1.10.4.1 Aeration Tank.

2.1.10.4.1.1 Reaction rate constant, l/mg/hr.

2.1.10.4.1.2 Sludge produced per BOD removed.

2.1.10.4.1.3 Endogenous respiration rate (b, b').

2.1.10.4.1.4 O_2 utilized per BOD removed.

2.1.10.4.1.5 Influent nonbiodegradable VSS (f).

2.1.10.4.1.6 Effluent degradable VSS (f').

2.1.10.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.10.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.10.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.10.4.1.10 Aeration time, hr.

2.1.10.4.1.11 Volume of aeration tank, million gal.

2.1.10.4.1.12 Oxygen required, lb/day.

2.1.10.4.1.13 Sludge produced, lb/day.

2.1.10.4.1.14 Nitrogen requirement, lb/day.

2.1.10.4.1.15 Phosphorus requirement, lb/day.

2.1.10.4.1.16 Sludge recycle ratio, percent.

2.1.10.4.1.17 Solids retention time, days.

2.1.10.4.2 Aeration System.

2.1.10.4.2.1 Standard transfer efficiency, lb O_2/hp-hr.

2.1.10.4.2.2 Operating transfer efficiency, lb O_2/hp-hr.

2.1.10.4.2.3 Horsepower required.

2.1.10.4.3 Effluent BOD_5 concentration, BODE, mg/l.

2.1.10.4.4 Effluent soluble BOD_5 concentration, Se, mg/l.

2.1.10.4.5 Effluent COD concentration, CODE, mg/l.

2.1.10.4.6 Effluent soluble COD concentration, CODSE, mg/l.

2.1.10.4.7 Effluent total Kjeldahl nitrogen concentration, TKNE, mg/l.

2.1.10.4.8 Effluent ammonia nitrogen concentration, NH3E, mg/l.

2.1.10.4.9 Effluent phosphorus concentration, NH3E, mg/l.

2.1.10.4.10 Effluent oil and grease concentration, OAGE, mg/l.

2.1.10.5 Quantities Calculations.

2.1.10.5.1 The design values for activated sludge system would be:

$$V_d = V \cdot \frac{10^6}{7.48}$$

$$HP_d = (HP)\ (V)\ (133.7)$$

where

V_d = design volume of aeration basin, cu ft.

V = volume of aeration basin million gallons.

2.1.10.5.2 Selection of number of aeration tanks and mechanical aerators per tank. The following rule will be utilized in the selection of number of aeration tanks and mechanical aerators per tank.

Q_{avg} (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NT
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.10.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.10.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.10.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.10.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.10.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.10.5.4 Mechanical aeration equipment design.

2.1.10.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.1.10.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)(NT)(NA)}$$

If HPN > 150 HP and NT = 2 or 3, then repeat the calculation with NT = NT + 1.

If HPN > 150 HP and NT $\geq$ 4, then repeat the calculation with NT = NT + 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.1.10.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.1.10.5.5 Design of aeration tanks.

2.1.10.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.10.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When $HPSN \leq 100$ HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

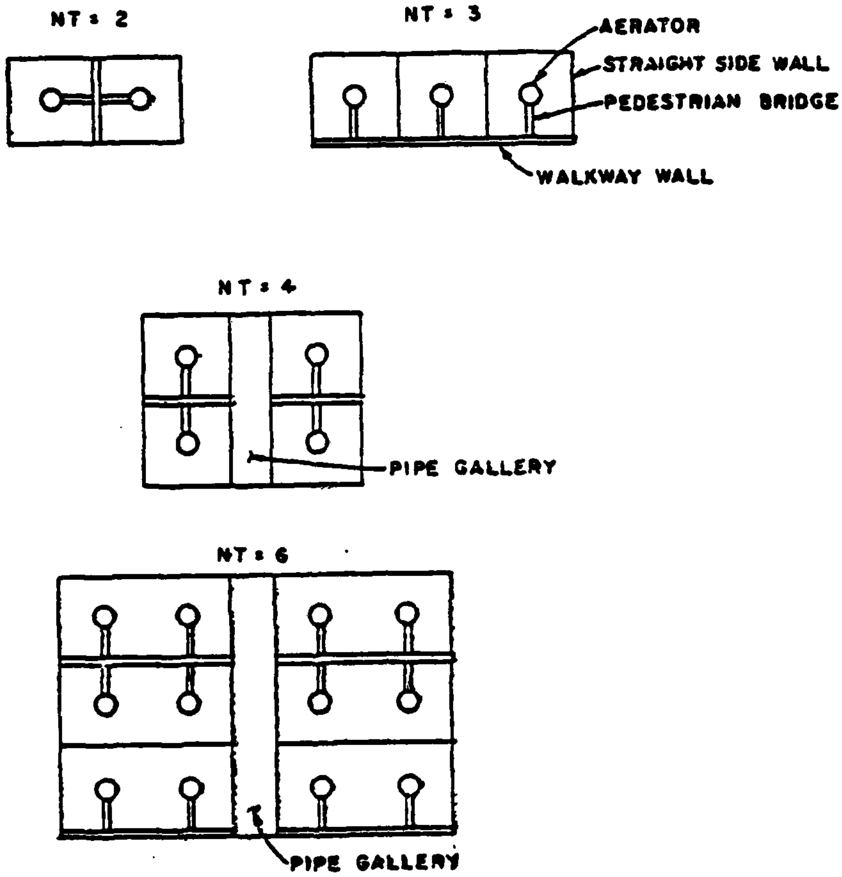
When $HPSN > 100$ HP

$$DW = 15 \text{ ft}$$

where

DW = water depth of the aeration tanks, ft.

$HPSN$ = capacity of the aerator, HP.



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.1.-9 - EXAMPLES OF TANK ARRANGEMENTS
ACTIVATED SLUDGE PROCESSES

2.1.10.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.1.10.5.6 Aeration tank arrangements.

2.1.10.5.6.1 Figure 2.1-9 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.1.10.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\ \left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.10.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.1.10.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\ [(2\ W + 18.5)\ (W + 17) + (2\ W + 26.5)\ (W + 25)]$$

where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.1.10.5.7.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$

2.1.10.5.7.3 When NT > 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB)\ [(L_s + 4)\ (W_s + 4) + (L_s + 12)\ (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.1.10.5.8 Reinforced concrete slab quantity.

2.1.10.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.1.10.5.8.2 For NT = 2,

$$V_{cs} = 1.5\ (2\ W + 14.5)\ (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.1.10.5.8.3 NT = 3,

$$V_{cs} = 1.5\ (3\ W + 16)\ (W + 13)$$

2.1.10.5.8.4 When NT > 4,

$$V_{cs} = 1.5\ (L_s)\ (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.1.10.5.9 Reinforced Concrete Wall Quantity.

2.1.10.5.9.1 The wall constructions are different for complete mix and plug flow systems. In order to achieve complete mix, the inflows to the aeration tanks would be distributed uniformly along one side of the aeration tank, flowing across the width of the tank and being discharged along the other side wall. Thus a Y-wall construction will be used so that the top section of the wall can be an open channel for influent and/or effluent discharges. Figure 2.1-10 shows a typical section of the complete mix aeration tank.

2.1.10.5.9.2 When NT = 2,

$$V_{cw} = W\ [8.75\ DW + 88]$$

where

V_{cw} = R.C. wall quantity, cu ft.

W = width of individual aeration tank, ft.

DW = water depth of aeration tank, ft.

2.1.10.5.9.3 When NT = 3,

$$V_{cw} = 6\ (W + 2)\ (1.25\ DW + 13.45) + 5\ W\ [DW + 3]$$

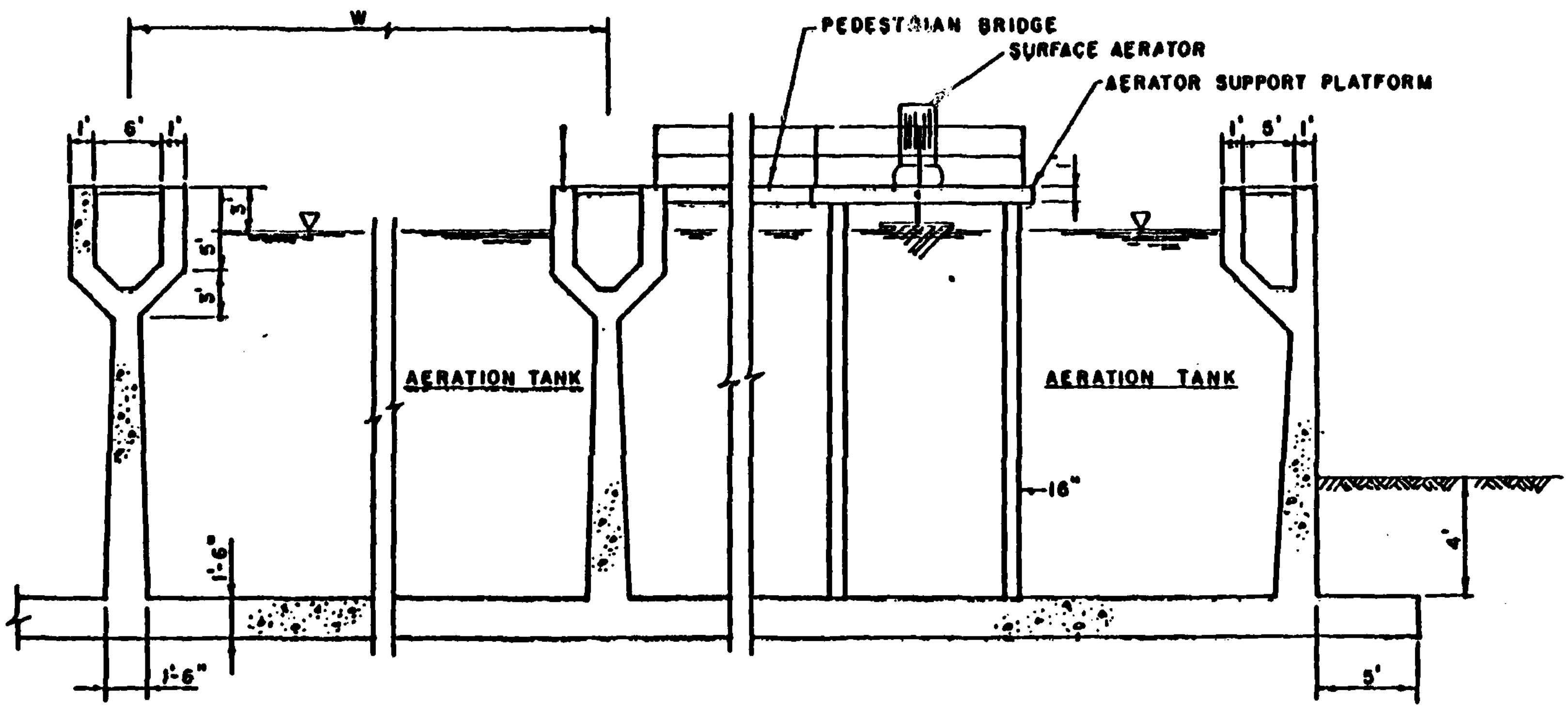


FIGURE 2.1.-10 - TYPICAL SECTION OF COMPLETE MIX AERATION TANKS

When NT > 3,

$$V_{cw} = (NB) \quad 4 \cdot L \cdot [1.25\ (DW + 3) + 9.7] + (NT - 2) \cdot L \cdot (1.25\ DW + 31) + 2.5\ (NT)\ (W)\ (DW + 3)$$

2.1.10.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.1.10.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.1.10.5.10.2 When NT ≥ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5)\ (PGW)\ [\ \frac{(NT)\ (W)}{2} + 0.75\ (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2\ (PGW)\ (NB)\ (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.1.10.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.1.10.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.1.10.5.11.2 Figure 2.1-11 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078\ (HPSN)$$

where

X = width of the platform, ft.

$HPSN$ = horsepower of the mechanical aerator, HP.

2.1.10.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6\ (DW + 2)]\ (NT)\ (NA)\ (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.1.10.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2\ (W - X)]\ (NB)\ (NT)\ (NA)$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.1.10.5.12 Summary of reinforced concrete structures.

2.1.10.5.12.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.1.10.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

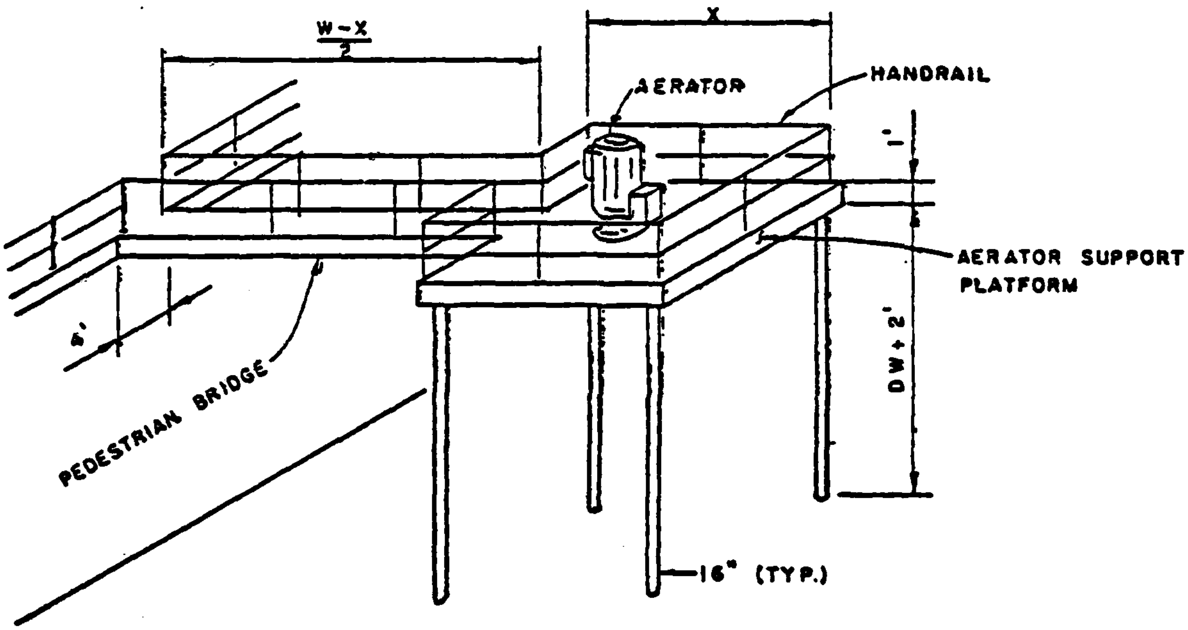


FIGURE 2.1.-11 - AERATOR SUPPORT PLATFORM

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.1.10.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.1.10.5.13.1 When NT = 2,

$$LHR = 4\,W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.1.10.5.13.2 When NT = 3,

$$LHR = 6\,W + 10 + 3 \cdot (3X + W - 4)$$

2.1.10.5.13.3 When NT ≥ 4,

If $\frac{NT}{2}$ is an even number,

$$LHR = \left\{PGW + (NT)\,(W) + [L + 3 - 4\,(NA)]\,(NT) + (NA) \cdot (NT) \cdot (3X + W - 4)\right\} \cdot (NB)$$

If $\frac{NT}{2}$ is an odd number,

$$LHR = \left\{PGW + (NT)\,(W) + [L + 3 - 4\,(NA)]\,(NT + 2) + (NA)\,(NT)\,(3X + w - 4)\right\} \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.1.10.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.1.10.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)\ (NT)\ (NA)\ (HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.1.10.5.14.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\ (TICA)^{0.3731}$$

When TICA $\geq$ 200 hp

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.1.10.5.14.3 The maintenance manpower requirement can be estimated as follows:

When TICA $\leq$ 100 hp

$$MMH = 106.3\ (TICA)^{0.4031}$$

When TICA > 100 hp

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.1.10.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.10.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.1.10.5.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.10.6 <u>Quantities Calculations Output Data.</u>

2.1.10.6.1 Number of aeration tanks, NT.

2.1.10.6.2 Number of aerators per tank, NA.

2.1.10.6.3 Number of process batteries, NB.

2.1.10.6.4 Capacity of each individual aerator, HPSN, hp.

2.1.10.6.5 Depth of aeration tanks, DW, ft.

2.1.10.6.6 Length of aeration tanks, L, ft.

2.1.10.6.7 Width of aeration tanks, W, ft.

2.1.10.6.8 Width of pipe gallery, PGW, ft.

2.1.10.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.1.10.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.1.10.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.1.10.6.12 Quantity of handrail, LHR, ft.

2.1.10.6.13 Operation manpower requirement, OMH, MH/yr.

2.1.10.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.1.10.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.1.10.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.1.10.6.17 Correction factor for minor capital cost items, CF.

2.1.10.7 <u>Unit Price Input Required</u>.

2.1.10.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.1.10.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.1.10.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.1.10.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, \$, optional.

2.1.10.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.1.10.7.6 Equipment installation labor rate, \$/MH.

2.1.10.7.7 Crane rental rate, UPICR, \$/hr.

2.1.10.7.8 Unit price of handrail, UPIHR, \$/L.F.

2.1.10.8 <u>Cost Calculations</u>.

2.1.10.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.10.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, \$.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.1.10.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.1.10.8.4 Cost of installed aeration equipment.

2.1.10.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, $.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.1.10.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.1.10.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16{,}300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16{,}300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.10.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When $HPSN \leq 60$ hp

$$IMH = 39 + 0.55\ (HPSN)$$

When HPSN > 60 hp

$$IMH = 61.3 + 0.18\ (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.10.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.10.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.10.8.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, dollars.

LABRI = labor rate, dollars/man-hour.

UPICR = crane rental rate, dollars/hr.

2.1.10.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, $ per lineal foot. A value of $25.20 per foot for the first quarter of 1977 is suggested.

2.1.10.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.1.10.8.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, dollars.

CF = correction factor for minor cost items, from second-order design output.

2.1.10.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, $/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.1.10.9 Cost Calculations Output Data.

2.1.10.9.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, dollars.

2.1.10.9.2 Operation and maintenance supply and material costs, OMMC, dollars.

2.1.11 CONTACT STABILIZATION (Diffused Aeration)

2.1.11.1 Input Data.

2.1.11.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.11.1.2 Wastewater Strength.

2.1.11.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.11.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.11.1.2.3 Suspended solids, mg/l.

2.1.11.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.11.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.11.1.3 Other Characterization.

2.1.11.1.3.1 pH.

2.1.11.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.11.1.3.3 Nitrogen,[1] mg/l.

2.1.11.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.11.1.3.5 Oils and greases, mg/l.

2.1.11.1.3.6 Heavy metals, mg/l.

2.1.11.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.11.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.11.1.4 Effluent Quality Requirements.

2.1.11.1.4.1 BOD_5, mg/l.

2.1.11.1.4.2 SS, mg/l.

2.1.11.1.4.3 TKN, mg/l.

2.1.11.1.4.4 P, mg/l.

2.1.11.1.4.5 Total nitrogen (TKN + No_3 - N), mg/l.

---

[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.11.1.4.6 Settleable solids, mg/l.

2.1.11.2 Design Parameters.

2.1.11.2.1 Contact tank detention time, t_1.

$$t_1 = (0.5\text{-}1.0) \text{ hr}$$

2.1.11.2.2 System organic loading (F/M).

$$F/M = (0.2\text{-}0.6)$$

2.1.11.2.3 Volumetric loading = 60-75.

2.1.11.2.4 Stabilization tank detention time, t_2.

$$t_2 = (2\text{-}4) \text{ hr}$$

2.1.11.2.5 Contact tank MLSS, X_{ac}.

$$X_{ac} = (2500\text{-}3500) \text{ mg/l}$$

2.1.11.2.6 Contact tank MLVSS, X_{vc}.

$$X_{vc} = 0.7X_{ac} = (1750\text{-}2450) \text{ mg/l}$$

2.1.11.2.7 Stabilization tank MLSS, X_{as}.

$$X_{as} = (4000\text{-}8000) \text{ mg/l}$$

2.1.11.2.8 Stabilization tank MLVSS, X_{vs}.

$$X_{vs} = (2800\text{-}5600) \text{ mg/l}$$

2.1.11.2.9 Air requirement (lb O_2/lb BOD_r).

$$\text{lb } O_2/\text{lb } BOD_r = 1.25\text{-}1.5$$

2.1.11.2.10 lb solids/lb BOD_r = (0.2-0.4).

2.1.11.2.11 Recycle ratio, Q_r/Q.

$$Q_r/Q = (0.25\text{-}1.00)$$

2.1.11.2.12 Efficiency = (> 90 percent).

2.1.11.3 Process Design Calculations.

2.1.11.3.1 Assume the following design parameters.

2.1.11.3.1.1 Aeration time in contact tank, hr.

2.1.11.3.1.2 Aeration time in stabilization tank, hr.

2.1.11.3.1.3 MLSS in contact tank, mg/l.

2.1.11.3.1.4 MLSS in stabilization tank, mg/l.

2.1.11.3.1.5 MLVSS in contact tank, mg/l.

2.1.11.3.1.6 MLVSS in stabilization tank, mg/l.

2.1.11.3.1.7 O_2 requirements, lb/day.

2.1.11.3.1.8 Sludge produced, lb/day.

2.1.11.3.2 Determine contact tank volume.

$$V_c = Q_{avg} \frac{t_1}{24}$$

where

V_c = volume of contact tank, million gal.

Q_{avg} = average flow, mgd.

t_1 = detention time in contact tank, hr.

2.1.11.3.3 Determine stabilization tank volume.

$$V_s = Q_{avg} \frac{t_2}{24}$$

where

V_s = volume of stabilization tank, million gal.

Q_{avg} = average flow, mgd.

t_2 = detention time in stabilization tank, hr.

2.1.11.3.4 Calculate system organic loading and check against desired loading.

$$(F/M)_{system} = \frac{Q_{avg}(S_o)}{V_c(X_{vc}) + V_s(X_{vs})}$$

where

F/M = food-to-microorganism ratio.

Q_{avg} = average flow, mgd.

S_o = influent BOD_5, mg/l.

V_c = volume of contact tank, million gal.

X_{vc} = MLVSS in contact tank, mg/l.

V_s = volume of stabilization tank, million gal.

X_{vs} = MLVSS in stabilization tank, mg/l.

2.1.11.3.5 Calculate volumetric loading and check against desired loading.

$$\text{lb}/1000\ \text{ft}^3 = \frac{Q_{avg}(S_o)(62.4)}{(V_c + V_s) \times 10^3}$$

where

Q_{avg} = average flow, mgd.

S_o = influent BOD_5, mg/l.

V_c = contact tank volume, million gal.

V_s = stabilization tank volume, million gal.

2.1.11.3.6 Calculate the system oxygen required and select 1.25-1.5 lb O_2/lb BOD_r.

$$O_2 = (1.25\text{-}1.5)Q_{avg} \times S_r(8.34)$$

where

O_2 = required oxygen, lb/day.

Q_{avg} = average flow, mgd.

S_r = BOD removed $(S_o - S_e)$, mg/l.

Supply approximately one-half of O_2 for contact tank and one-half for stabilization tank.

2.1.11.3.7 Design aeration system.

2.1.11.3.7.1 Assume the following design parameters.

2.1.11.3.7.1.1 Standard transfer efficiency, percent, from manufacturer (5-8 percent).

2.1.11.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.11.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.11.3.7.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.11.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.11.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{(C_s)_T (\beta)(p) - C_L}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

T = temperature, °C.

2.1.11.3.7.4 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{O_2(10^5)(7.48)}{(OTE)\ \ 0.0176\,\frac{lb\ O_2}{ft^3 air}\ \ 1440\,\frac{min}{day}\ \ V}$$

where

R_a = required air flow, cfm/1000 ft^3.

O_2 = required oxygen, lb/day.

OTE = operating transfer efficiency, percent.

V = volume of basin ($V_s + V_c$), gal.

2.1.11.3.8 Determine sludge production. Select 0.2-0.4 lb solids/lb BOD_r.

$$\Delta X_V = (0.2\text{-}0.4)Q(S_r)8.34$$

where

ΔX_V = sludge produced, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.11.3.9 Effluent Characteristics.

2.1.11.3.9.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.11.3.9.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.11.3.9.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.11.3.9.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.11.3.9.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.11.3.9.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.11.3.10 Determine nutrient requirements for nitrogen.

$$N = 0.123\ \Delta X_V$$

and phosphorus

$$P = 0.026\ \Delta X_V$$

where

ΔX_V = sludge produced, lb/day.

and check against rule of thumb BOD:N:P = 100:5:1.

2.1.11.3.11 Determine recycle ratio required and check against 0.25-1.0.

$$\frac{Q_r}{Q} = \frac{X_{ac}}{X_{as} - X_{ac}}$$

where

Q_r = volume of recycled sludge, mgd.

Q = total flow, mgd.

X_{ac} = MLSS in contact tank, mg/l.

X_{as} = MLSS in stabilization tank, mg/l.

2.1.11.4 Process Design Output Data.

2.1.11.4.1 Aeration Tank.

2.1.11.4.1.1 lb BOD/lb MLSS-day.

2.1.11.4.1.2 Mixed liquor SS contact tank, mg/l (MLSS).

2.1.11.4.1.3 Mixed liquor SS stabilization tank, mg/l $(MLSS)_s$.

2.1.11.4.1.4 Aeration time contact tank, hr (t_c).

2.1.11.4.1.5 Aeration time stabilization tank, hr (t_s).

2.1.11.4.1.6 Volume of contact tank, million gal (V_c).

2.1.11.4.1.7 Volume of stabilization tank, million gal (V_s).

2.1.11.4.1.8 Oxygen required, lb/day.

2.1.11.4.1.9 Sludge produced, lb/day.

2.1.11.4.1.10 Nitrogen requirement, lb/day.

2.1.11.4.1.11 Phosphorus requirement, lb/day.

2.1.11.4.1.12 Sludge recycle ratio, percent.

2.1.11.4.1.13 Volumetric loading, lb BOD/million ft^3.

2.1.11.4.2 Aeration System.

2.1.11.4.2.1 Standard transfer efficiency, percent.

2.1.11.4.2.2 Operating efficiency, percent.

2.1.11.4.2.3 Required air flow, $cfm/1000\ ft^3$.

2.1.11.5 Quantities Calculations.

2.1.11.5.1 Design values for activated sludge system.

$$V_d = (V_c + V_s) \cdot \frac{10^6}{7.48}$$

$$CFM_d = (CFM) \cdot (V_c + V_s)\ (133.7)$$

where

V_d = design volume of aeration tanks, cu ft.

V_c = volume of contact tanks, million gallons.

V_s = volume of stabilization tanks, million gallons.

CFM_d = design capacity of blowers, scfm.

CFM = blower capacity, scfm/1000 cu ft tank volume.

2.1.11.5.2 Selection of numbers of aeration tanks for contact-stabilization modification. Due to the fact that the ratio between the volumes of the stabilization tank and contact tank generally ranges from 3:1 to 5:1, and also that it is general practice to keep the dimensions and sizes identical among aeration tanks within a sewage treatment plant, the following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (MGD)	Total Number of Tanks NT	Possible Tank Number Selections for Contact or Stabilization Process
0.5 - 10	4	*c: 1 s: 3
10 - 20	6	c: 1 or 2 s: 4 or 5
20 - 30	8	c: 1, 2 or 3 s: 5, 6 or 7
30 - 40	10	c: 1, 2, 3 or 4 s: 6, 7, 8 or 9
40 - 50	12	c: 1, 2, 3 or 4 s: 8, 9, 10 or 11
50 - 70	14	c: 1, 2, 3 or 4 s: 10, 11, 12 or 13
70 - 100	16	c: 2, 3, 4, 5 or 6 s: 10, 11, 12, 13 or 14

* c: Possible number of tanks to be used as contact basin.
  s: Possible number of tanks to be used as stabilization basin.

Where Q_{avg} is larger than 100 mgd, several batteries of units will be used. See subsection 2.1.11.5.3 for details.

2.1.11.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.11.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.11.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsections 2.1.11.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.11.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.11.5.4 Number of diffusers. The oxygen transfer rates used dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer

where

ND_t = number of diffusers per tank.

2.1.11.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer

where

NSA_t = number of swing arm headers per tank.

2.1.11.5.6 Design of aeration tanks.

2.1.11.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.11.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.1.11.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.1.11.5.7 Aeration tank arrangements.

2.1.11.5.7.1 Figure 2.1-12 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\ \left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = pipe gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.11.5.8 Typical wall constructions.

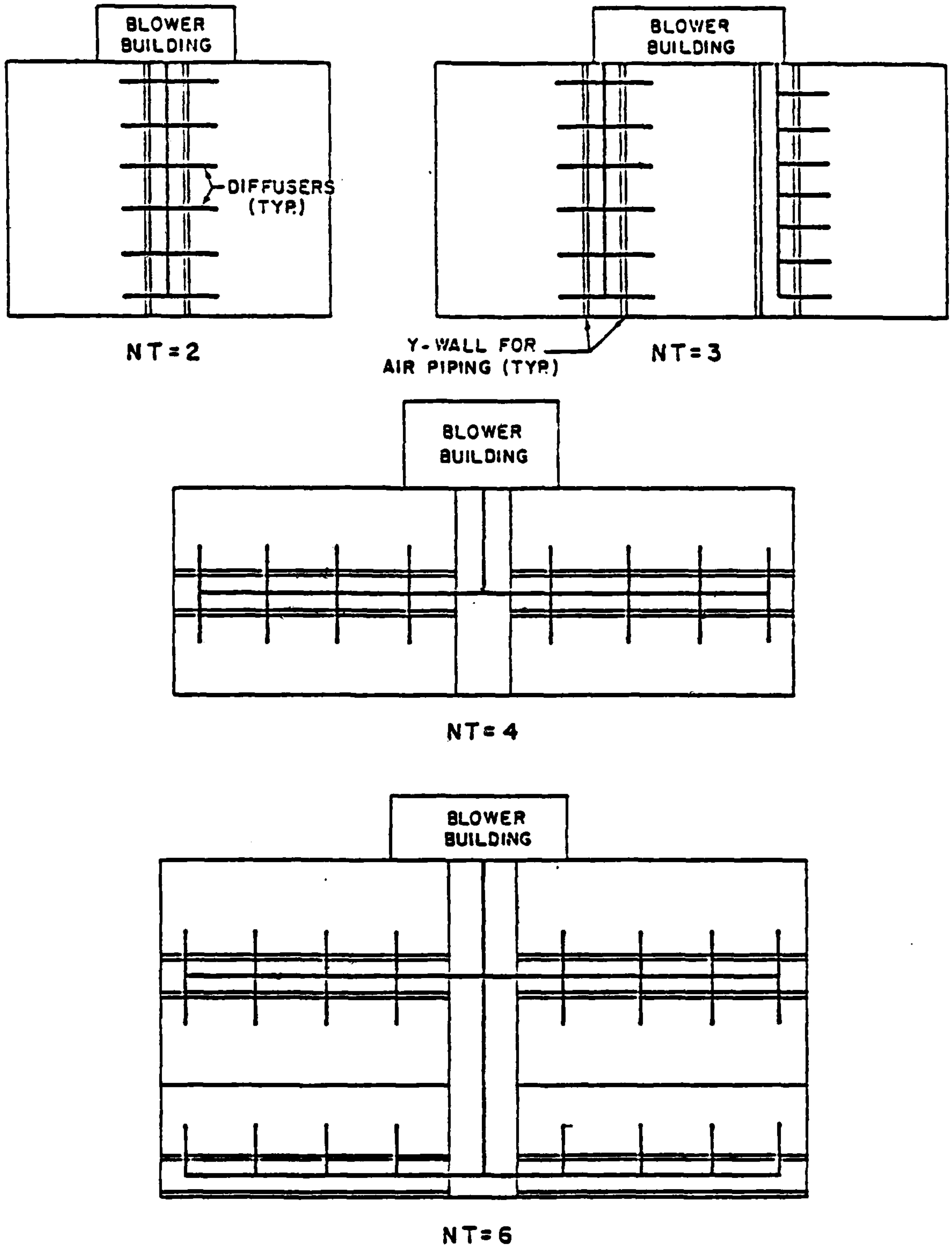


FIGURE 2.1-12 AERATION TANK ARRANGEMENT

2.1.11.5.8.1 In the plug flow system the influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required as shown in Figure 2.1-13. One would be a simple straight side wall and the other would be a Y wall as in the complete mix flow. This Y wall would be required to carry the air piping and headers.

2.1.11.5.9 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations.

2.1.11.5.9.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ \left[\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}\right]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.1.11.5.9.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\left[\frac{(15.75(NT)+15.5)\,(2L+PGW+20)+(15.75(NT)+23.5)\,(2L+PGW+28)}{2}\right]$$

2.1.11.5.10 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.1.11.5.10.1 For NT less than 4:

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

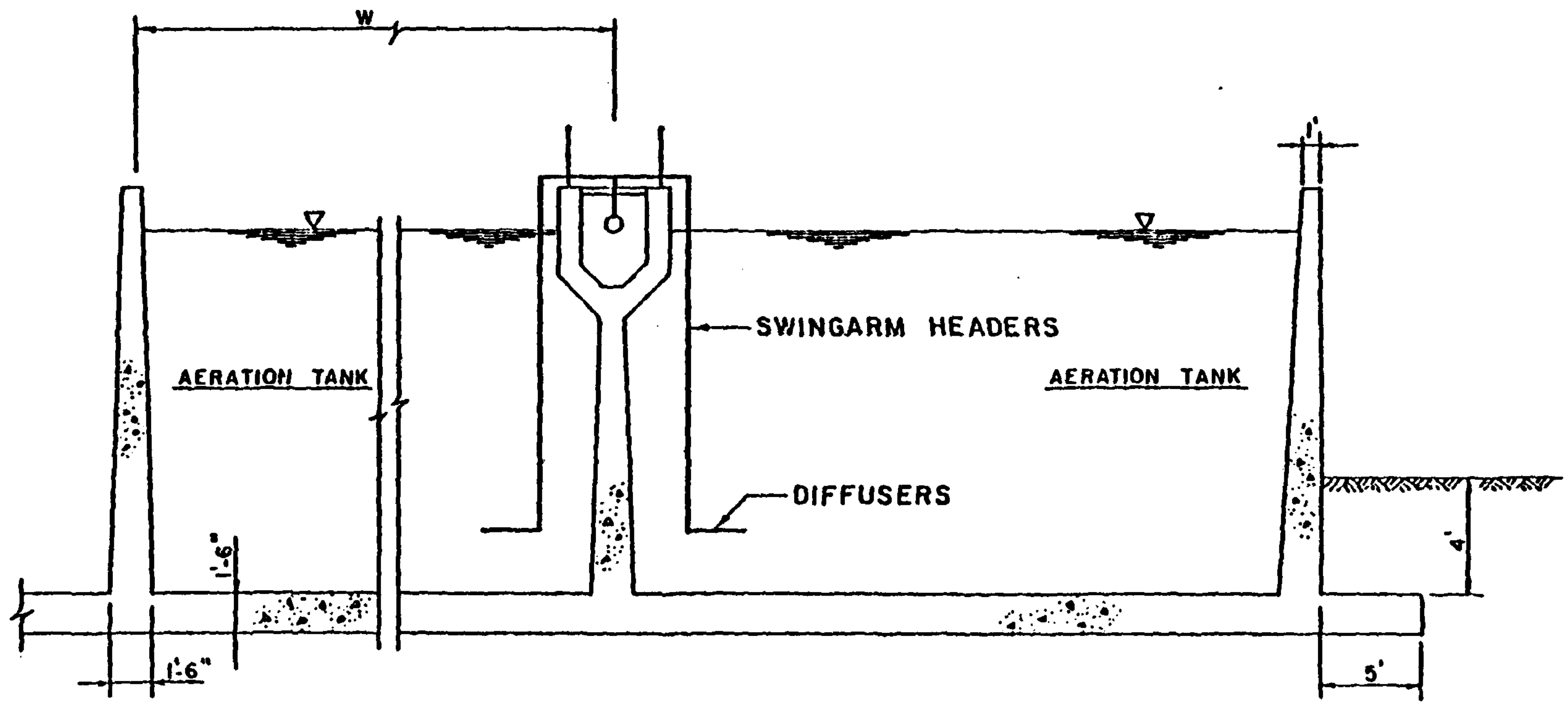
2.1.11.5.10.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)(2L + PGW + 200)]$$

2.1.11.5.11 Reinforced concrete wall quantity.

2.1.11.5.11.1 When NT is less than 4

$$V_{cw} = (NB)[75.3(L) + [29.4(NT) - 58.8]\ (L) + 1383.75(NT) + 33.75]$$



2.1-13 TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

2.1.11.5.11.2 When NT = 4, 8, 12, 16, 20, etc.

$$V_{cw} = (NB)[29.35(NT)(L) + 56.8\ (L) + 1350\ (NT) + 45\ (PGW) + 185.65]$$

2.1.11.5.11.3 When NT = 6, 10, 14, 18, 22, etc.

$$V_{cw} = (NB)[15.15(NT)(L) + 28.5\ (L) + 1367\ (NT) + 45\ (PGW) + 168.75]$$

2.1.11.5.12 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls, and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.1.11.5.12.1 For NT = 2

$$LHR = [2(NT)(L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.1.11.5.12.2 For NT = 3

$$LHR = [2(NT)(L) + (4L) + (36.5)\ (NT) + 2\ PGW + 13]\ NB$$

2.1.11.5.12.3 For NT greater than or equal to 4

$$LHR = (NT + 4)\ (L) + 34(NT) + 2PGW + 3$$

2.1.11.5.13 Calculate operation manpower requirements.

2.1.11.5.13.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.1.11.5.13.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.1.11.5.14 Calculate maintenance manpower requirements.

2.1.11.5.14.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.1.11.5.14.2 If CFM_d > 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.1.11.5.15 Energy requirement for operation. The electrical energy required for operation is related to the average wastewater flow by the following equation:

$$KWH = 248{,}950.8\ (Q_{avg})^{0.9809}$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.1.11.5.16 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

$OMMP$ = operation and maintenance material costs as percent of total bare construction cost, percent.

2.1.11.5.17 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.11.6 Quantities Calculations Output Data.

2.1.11.6.1 Number of aeration tanks, NT.

2.1.11.6.2 Number of diffusers per tank, ND_t.

2.1.11.6.3 Number of process batteries, NB.

2.1.11.6.4 Number of swing arm headers per tank, NSA_t.

2.1.11.6.5 Length of aeration tanks, L, ft.

2.1.11.6.6 Width of pipe gallery, PGW, ft.

2.1.11.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.1.11.6.8 Quantity of R.C. Salb required, V_{cs}, cu ft.

2.1.11.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.1.11.6.10 Quantity of handrail, LHR, ft.

2.1.11.6.11 Operation manpower requirement, OMH, MH/yr.

2.1.11.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.1.11.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.1.11.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.1.11.6.15 Correction factor for minor construction costs, CF.

2.1.11.7 Unit Price Input Required.

2.1.11.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.11.7.2 Unit price input R.C. wall in-place, UPICW, \$/cu yd.

2.1.11.7.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.1.11.7.4 Unit price input for handrails in-place, UPIHR, \$/cu ft.

2.1.11.7.5 Cost per diffuser, COSTPD, \$, (optional).

2.1.11.7.6 Cost per swing arm header, COSTPH, \$, (optional).

2.1.11.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.11.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.1.11.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.1.11.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.1.11.8 <u>Cost Calculations</u>.

2.1.11.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.11.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.1.11.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \; UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.1.11.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.1.11.8.5 Cost of diffusers.

2.1.11.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \; \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, dollars.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.1.11.8.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, $.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.1.11.8.6 Cost of swing arm diffuser headers.

2.1.11.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.11.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, $.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.1.11.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.1.11.8.8 Crane requirement for installation.

$$CH = (.1)\ (IMH)$$

where

CH = crane time requirement for installation, hr.

2.1.11.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.1.11.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.1.11.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.1.11.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.1.11.8.10 Other costs associated with the installed equipment. This category includes the costs for weir installation, painting, inspection, etc., and can be added as a percentage of the purchase equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.11.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\left(1 + \frac{PMINC}{100}\right) + (IMH)(LABRI) + (CH)(UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.1.11.8.11.1 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.1.11.8.12 Operation and maintenance material costs.

$$OMMC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/yr.

OMMP = operation and maintenance material and supply costs as percent of total bare construction cost, percent.

2.1.11.9 Cost Calculations Output Data.

2.1.11.9.1 Total bare construction cost, TBCC, \$.

2.1.11.9.2 Operation and maintenance material and supply costs, OMMC, \$.

2.1.12 CONTACT STABILIZATION (Mechanical Aeration)

2.1.12.1 Input Data.

2.1.12.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.12.1.2 Wastewater Strength.

2.1.12.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.12.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.12.1.2.3 Suspended solids, mg/l.

2.1.12.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.12.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.12.1.3 Other Characterization.

2.1.12.1.3.1 pH.

2.1.12.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.12.1.3.3 Nitrogen,[1] mg/l.

2.1.12.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.12.1.3.5 Oils and greases, mg/l.

2.1.12.1.3.6 Heavy metals, mg/l.

2.1.12.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.12.1.3.8 Temperature, ^oF or ^oC.

2.1.12.1.4 Effluent Quality Requirements.

2.1.12.1.4.1 BOD_5, mg/l.

2.1.12.1.4.2 SS, mg/l.

2.1.12.1.4.3 TKN, mg/l.

2.1.12.1.4.4 P, mg/l.

2.1.12.1.4.5 Total nitrogen (TKN + No_3 - N), mg/l.

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[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.12.1.4.6 Settleable solids, mg/l.

2.1.12.2 <u>Design Parameters.</u>

2.1.11.2.1 Contact tank detention time, t_1.

$$t_1 = (0.5\text{-}1.0)\ \text{hr}$$

2.1.12.2.2 System organic loading (F/M).

$$F/M = (0.2\text{-}0.6)$$

2.1.12.2.3 Volumetric loading = 60-75.

2.1.12.2.4 Stabilization tank detention time, t_2.

$$t_2 = (2\text{-}4)\ \text{hr}$$

2.1.12.2.5 Contact tank MLSS, X_{ac}.

$$X_{ac} = (2500\text{-}3500)\ \text{mg/l}$$

2.1.12.2.6 Contact tank MLVSS, X_{vc}.

$$X_{vc} = 0.7X_{ac} = (1750\text{-}2450)\ \text{mg/l}$$

2.1.12.2.7 Stabilization tank MLSS, X_{as}.

$$X_{as} = (4000\text{-}8000)\ \text{mg/l}$$

2.1.12.2.8 Stabilization tank MLVSS, X_{vs}.

$$X_{vs} = (2800\text{-}5600)\ \text{mg/l}$$

2.1.12.2.9 Air requirement (lb O_2/lb BOD_r).

$$\text{lb } O_2/\text{lb } BOD_r = 1.25\text{-}1.5$$

2.1.12.2.10 lb solids/lb BOD_r = (0.2-0.4).

2.1.12.2.11 Recycle ratio, Q_r/Q.

$$Q_r/Q = (0.25\text{-}1.00)$$

2.1.12.2.12 Efficiency = (> 90 percent).

2.1.12.3 Process Design Calculations.

2.1.12.3.1 Assume the following design parameters.

2.1.12.3.1.1 Aeration time in contact tank, hr.

2.1.12.3.1.2 Aeration time in stabilization tank, hr.

2.1.12.3.1.3 MLSS in contact tank, mg/l.

2.1.12.3.1.4 MLSS in stabilization tank, mg/l.

2.1.12.3.1.5 MLVSS in contact tank, mg/l.

2.1.12.3.1.6 MLVSS in stabilization tank, mg/l.

2.1.12.3.1.7 O_2 requirements, lb/day.

2.1.12.3.1.8 Sludge produced, lb/day.

2.1.12.3.2 Determine contact tank volume.

$$V_c = Q_{avg} \frac{t_1}{24}$$

where

V_c = volume of contact tank, million gal.

Q_{avg} = average flow, mgd.

t_1 = detention time in contact tank, hr.

2.1.12.3.3 Determine stabilization tank volume.

$$V_s = Q_{avg} \frac{t_2}{24}$$

where

V_s = volume of stabilization tank, million gal.

Q_{avg} = average flow, mgd.

t_2 = detention time in stabilization tank, hr.

2.1.12.3.4 Calculate system organic loading and check against desired loading.

$$(F/M)_{system} = \frac{Q_{avg}\ (S_o)}{V_c(X_{vc}) + V_s(X_{vs})}$$

where

F/M = food-to-microorganism ratio.

Q_{avg} = average flow, mgd.

S_o = influent BOD_5, mg/l.

V_c = volume of contact tank, million gal.

X_{vc} = MLVSS in contact tank, mg/l.

V_s = volume of stabilization tank, million gal.

X_{vs} = MLVSS in stabilization tank, mg/l.

2.1.12.3.5 Calculate volumetric loading and check against desired loading.

$$\text{lb}/1000\ \text{ft}^3 = \frac{Q_{avg}(S_o)(62.4)}{(V_c + V_s) \times 10^3}$$

where

Q_{avg} = average flow, mgd.

S_o = influent BOD_5, mg/l.

V_c = contact tank volume, million gal.

V_s = stabilization tank volume, million gal.

2.1.12.3.6 Calculate the system oxygen required and select 1.25-1.5 lb O_2/lb BOD_r.

$$O_2 = (1.25\text{-}1.5)Q_{avg} \times S_r(8.34)$$

where

O_2 = required oxygen, lb/day.

Q_{avg} = average flow, mgd.

S_r = BOD removed (S_o - S_e), mg/l.

Supply approximately one-half of O_2 for contact tank and one-half for stabilization tank.

2.1.12.3.7 Design aeration system.

2.1.12.3.7.1 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing $\geq$ 0.1 hp/1000 gal.

2.1.12.3.7.1.1 Standard transfer efficiency, lb/hp-hr (0 dissolved oxygen, 20°C, and tap water) (3-5 lb/hp-hr).

2.1.12.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.12.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.12.3.7.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.12.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.12.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{(C_s)_T (\beta) (p) - C_L}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

T = temperature, °C.

2.1.12.3.7.4 Calculate horsepower requirement.

$$hp = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr}(24)(V)} \times 1000$$

where

hp = horsepower required/1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of basin, gal.

2.1.12.3.8 Detemine sludge production. Select 0.2-0.4 lb solids/lb BOD_r.

$$\Delta X_V = (0.2\text{-}0.4)Q(S_r)8.34$$

where

ΔX_V = sludge produced, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.12.3.9 Effluent Characteristics.

2.1.12.3.9.1 BOD_5.

$$BODE = Se + 0.84\ (X_V)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.12.3.9.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.12.3.9.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/1.

TKN = influent total Kjeldahl nitrogen concentration, mg/1.

NH3E = effluent ammonia nitrogen concentration, mg/1.

2.1.12.3.9.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/1.

PO4 = influent phosphorus concentration, mg/1.

2.1.12.3.9.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/1.

2.1.12.3.9.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/1.

2.1.12.3.10 Determine nutrient requirements for nitrogen.

$$N = 0.123\ \Delta X_V$$

and phosphorus

$$P = 0.026\ \Delta X_V$$

where

X_V = sludge produced, lb/day.

2.1.12.4.2.2 Operating efficiency, lb O_2/hp-hr.

2.1.12.4.2.3 Horsepower required, hp.

2.1.12.5 <u>Quantities Calculations</u>.

2.1.12.5.1 Design values for activated sludge system.

$$V_d = (V_c + V_s) \cdot \frac{10^6}{7.48}$$

$$HP_d = (hp)\ (V_c + V_s)\ (133.7)$$

where

V_d = design volume of aeration tanks, cu ft.

V_c = volume of contact tanks, million gallons.

V_s = volume of stabilization tanks, million gallons.

HP_d = design capacity of aeration equipment, hp.

HP = calculated aerator horsepower required, hp.

2.1.12.5.2 Selection of numbers of aeration tanks for contact-stabilization modification. Due to the fact that the ratio between the volumes of the stabilization tank and contact tank generally ranges from 3:1 to 5:1, and also that it is general practice to keep the dimensions and sizes identical among aeration tanks within a sewage treatment plant, the following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (MGD)	Total Number of Tanks NT	Possible Tank Number Selections for Contact or Stabilization Process
0.5 - 10	4	*c: 1 s: 3
10 - 20	6	c: 1 or 2 s: 4 or 5
20 - 30	8	c: 1, 2 or 3 s: 5, 6 or 7
30 - 40	10	c: 1, 2, 3 or 4 s: 6, 7, 8 or 9
40 - 50	12	c: 1, 2, 3 or 4 s: 8, 9, 10 or 11
50 - 70	14	c: 1, 2, 3 or 4 s: 10, 11, 12 or 13
70 - 100	16	c: 2, 3, 4, 5 or 6 s: 10, 11, 12, 13 or 14

* c: Possible number of tanks to be used as contact basin.
  s: Possible number of tanks to be used as stabilization basin.

Where Q_{avg} is larger than 100 mgd, several batteries of units will be used. See subsection 2.1.12.5.3 for details.

2.1.12.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.12.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.12.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsections 2.1.12.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.12.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.12.5.4 Mechanical aeration equipment design.

2.1.12.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.1.12.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)(NT)(NA)}$$

If HPN > 150 HP and NT = 2 or 3, then repeat the calculation with NT = NT + 1.

If HPN > 150 HP and NT ≥ 4, then repeat the calculation with NT = NT + 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.1.12.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.1.12.5.5 Design of aeration tanks.

2.1.12.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.12.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When HPSN ≤ 100 HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

When HPSN > 100 HP

$$DW = 15 \text{ ft}$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.1.12.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.1.12.5.6 Aeration tank arrangements.

2.1.12.5.6.1 Figure 2.1-14 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.1.12.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\ \frac{(Q_{avg})}{NB}$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.12.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.1.12.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\ [(2\ W + 18.5)\ (W + 17) + (2\ W + 26.5)\ (W + 25)]$$

where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.1.12.5.7.2 When NT = 3, earthwork required would be:

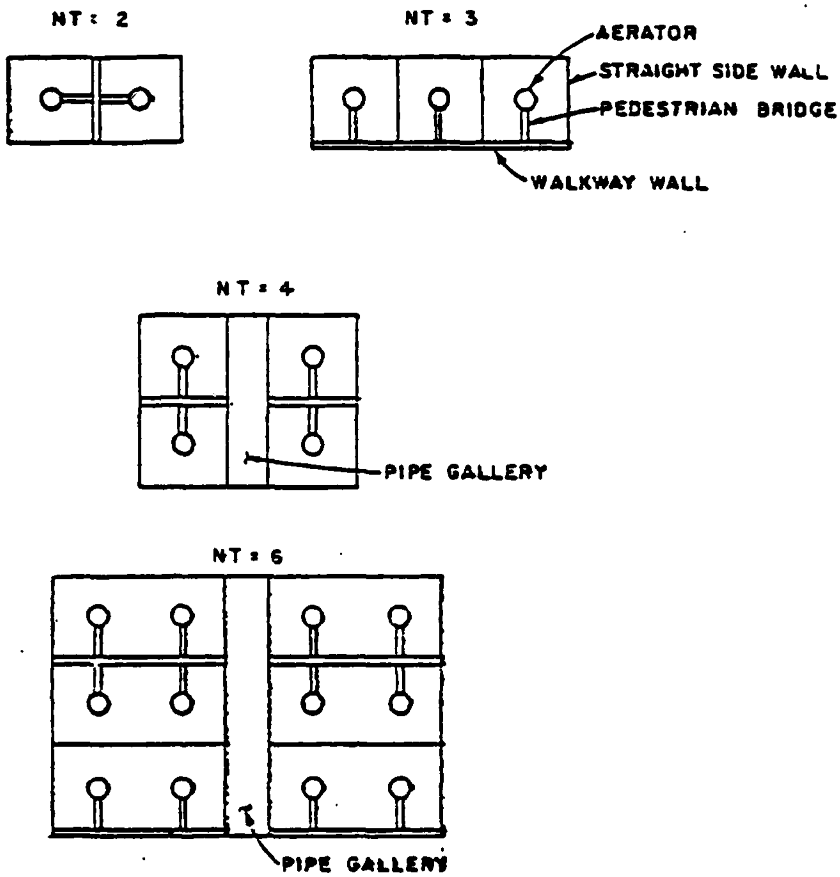
$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$

2.1.12.5.7.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.1-14 EXAMPLES OF TANK ARRANGEMENTS
ACTIVATED SLUDGE PROCESSES

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB) [(L_s + 4) (W_s + 4) + (L_s + 12) (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.1.12.5.8 Reinforced concrete slab quantity.

2.1.12.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.1.12.5.8.2 For NT = 2,

$$V_{cs} = 1.5 (2 W + 14.5) (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.1.12.5.8.3 NT = 3,

$$V_{cs} = 1.5 (3 W + 16) (W + 13)$$

2.1.12.5.8.4 When NT $\geq$ 4,

$$V_{cs} = 1.5 (L_s) (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.1.12.5.9 Reinforced Concrete Wall Quantity.

2.1.12.5.9.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-15. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

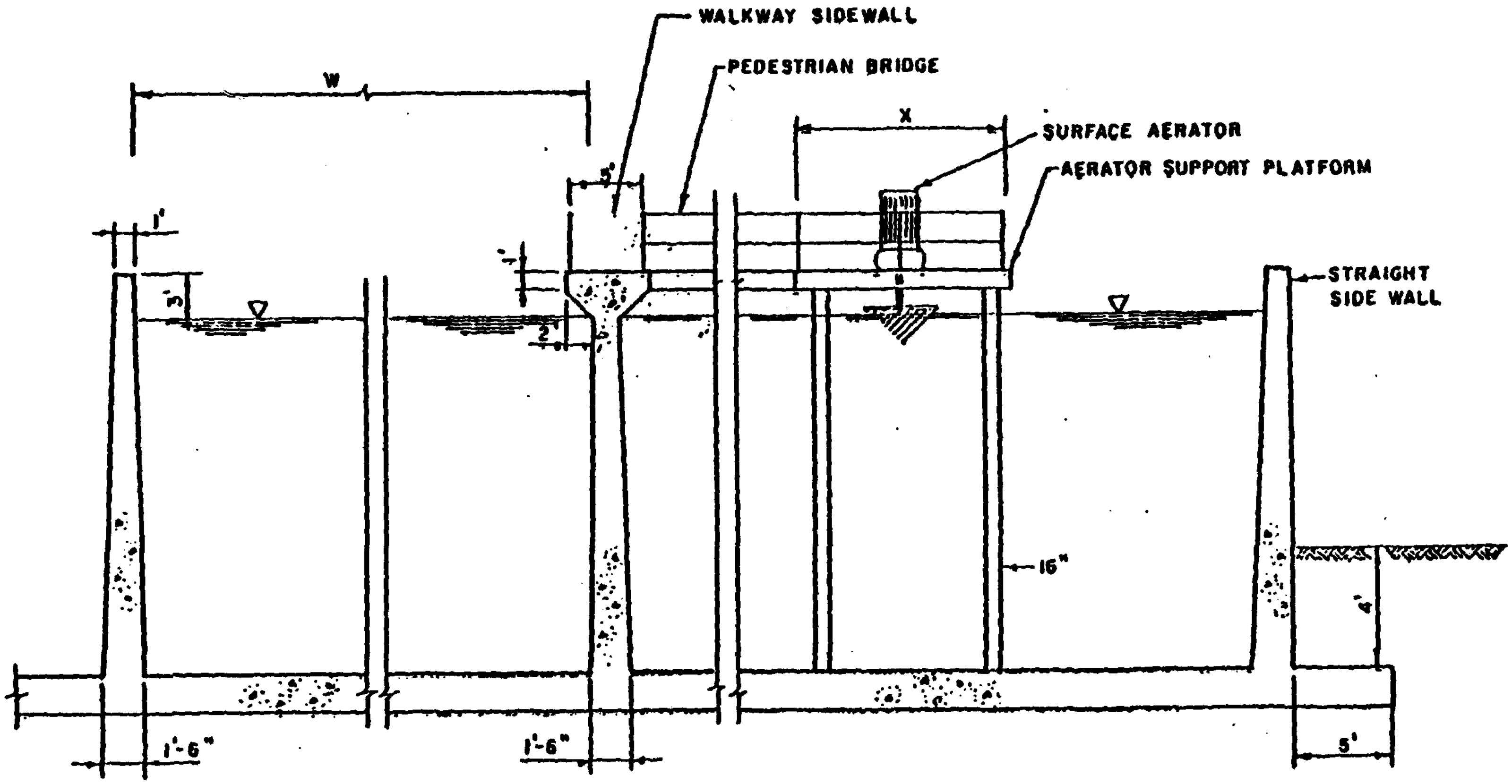


FIGURE 2.1-15 TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

2.1.12.5.9.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.12.5.9.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.12.5.9.4 When NT ≥ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

2.1.12.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.1.12.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.1.12.5.10.2 When NT ≥ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5)\ (PGW)\ [\frac{(NT)\ (W)}{2} + 0.75\ (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2\ (PGW)\ (NB)\ (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.1.12.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.1.12.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.1.12.5.11.2 Figure 2.1-16 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078\ (HPSN)$$

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.1.12.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6\ (DW + 2)]\ (NT)\ (NA)\ (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.1.12.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2\ (W - X)]\ (NB)\ (NT)\ (NA)$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.1.12.5.12 Summary of reinforced concrete structures.

2.1.12.5.12.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

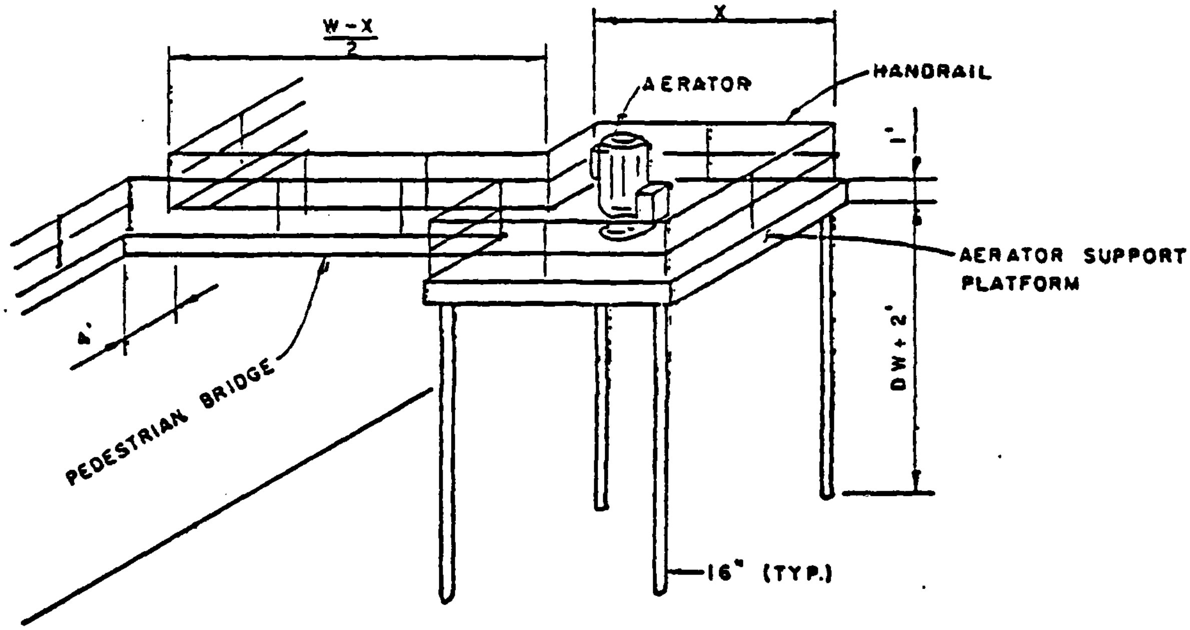


FIGURE 2.1-16 AERATOR SUPPORT PLATFORM

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.1.12.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.1.12.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.1.12.5.13.1 When NT = 2,

$$LHR = 4\,W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.1.12.5.13.2 When NT = 3,

$$LHR = 6\,W + 10 + 3 \cdot (3X + W - 4)$$

2.1.12.5.13.3 When NT $\geq$ 4,

If $\frac{NT}{2}$ is an even number,

$$LHR = PGW + (NT)\,(W) + [L + 3 - 4\,(NA)]\,(NT) + (NA) \cdot (NT) \cdot (3X + W - 4) \cdot (NB)$$

$$\text{If } \frac{NT}{2} \text{ is an odd number,}$$

$$LHR = PGW + (NT)(W) + [L + 3 - 4(NA)](NT + 2) + (NA)(NT)(3X + w - 4) \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.1.12.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.1.12.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)(NT)(NA)(HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.1.12.5.14.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\ (TICA)^{0.3731}$$

When TICA ≥ 200 hp

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.1.12.5.14.3 The maintenance manpower requirement can be estimated as follows:

When TICA ≤ 100 hp

$$MMH = 106.3\ (TICA)^{0.4031}$$

When TICA > 100 hp

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.1.12.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.12.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.1.12.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.1$$

where

CF = correction factor to account for the minor cost items.

2.1.12.6 Quantities Calculations Output Data.

2.1.12.6.1 Number of aeration tanks, NT.

2.1.12.6.2 Number of aerators per tank, NA.

2.1.12.6.3 Number of process batteries, NB.

2.1.12.6.4 Capacity of each individual aerator, HPSN, hp.

2.1.12.6.5 Depth of aeration tanks, DW, ft.

2.1.12.6.6 Length of aeration tanks, L, ft.

2.1.12.6.7 Width of aeration tanks, W, ft.

2.1.12.6.8 Width of pipe gallery, PGW, ft.

2.1.12.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.1.12.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.1.12.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.1.12.6.12 Quantity of handrail, LHR, ft.

2.1.12.6.13 Operation manpower requirement, OMH, MH/yr.

2.1.12.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.1.12.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.1.12.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.1.12.6.17 Correction factor for minor capital cost items, CF.

2.1.12.7 <u>Unit Price Input Required</u>.

2.1.12.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.12.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.1.12.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.1.12.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, \$, optional.

2.1.12.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.1.12.7.6 Equipment installation labor rate, \$/MH.

2.1.12.7.7 Crane rental rate, UPICR, \$/hr.

2.1.12.7.8 Unit price of handrail, UPIHR, \$/L.F.

2.1.12.8 <u>Cost Calculations</u>.

2.1.12.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.12.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, \$.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.1.12.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, $.

V_{cst} = quantity of concrete slab, $/cu yd.

UPICS = unit price input of R.C. slab in-place, $/cu yd.

2.1.12.8.4 Cost of installed aeration equipment.

2.1.12.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, $.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.1.12.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.1.12.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16,300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16{,}300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.12.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When HPSN $\leq$ 60 hp

$$IMH = 39 + 0.55\ (HPSN)$$

When HPSN > 60 hp

$$IMH = 61.3 + 0.18\ (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.12.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.12.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.12.8.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, dollars.

$LABRI$ = labor rate, dollars/man-hour.

$UPICR$ = crane rental rate, dollars/hr.

2.1.12.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

$UPIHR$ = unit price input for handrail cost, \$ per lineal foot. A value of \$25.20 per foot for the first quarter of 1977 is suggested.

2.1.12.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.1.12.8.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

$TBCC$ = total bare construction costs, dollars.

CF = correction factor for minor cost items, from second-order design output.

2.1.12.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

$OMMC$ = operation and maintenance material and supply costs, \$/yr.

$OMMP$ = percent of the installed aerator cost as O&M material and supply expenses.

2.1.12.9 Cost Calculations Output Data.

2.1.12.9.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, dollars.

2.1.12.9.2 Operation and maintenance supply and material costs, OMMC, dollars.

2.1.13 EXTENDED AERATION ACTIVATED SLUDGE (DIFFUSED AERATION).

2.1.13.1 Input Data.

2.1.13.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.13.1.2 Wastewater Strength.

2.1.13.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.13.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.13.1.2.3 Suspended solids, mg/l.

2.1.13.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.13.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.13.1.3 Other Characterization.

2.1.13.1.3.1 pH.

2.1.13.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.13.1.3.3 Nitrogen,[1] mg/l.

2.1.13.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.13.1.3.5 Oils and greases, mg/l.

2.1.13.1.3.6 Heavy metals, mg/l.

2.1.13.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.13.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.13.1.4 Effluent Quality Requirements.

2.1.13.1.4.1 BOD_5, mg/l.

2.1.13.1.4.2 SS, mg/l.

2.1.13.1.4.3 TKN, mg/l.

2.1.13.1.4.4 P, mg/l.

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[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.13.2 <u>Design Parameters</u>.

2.1.13.2.1 Reaction rate constants.

Constants	Range
Eckenfelder	
k	0.0007-0.002 1/mg/hr
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
a_O	0.77a = 0.56
f^O	0.40
f'	0.53

2.1.13.2.2 F/M = (0.05-0.15).

2.1.13.2.3 Volumetric loading = 10-25.

2.1.13.2.4 t = (18-36) hr.

2.1.13.2.5 t_s = (20-30) days.

2.1.13.2.6 MLSS = (3000-6000) mg/l.

2.1.13.2.7 MLVSS = (2100-4200) mg/l.

2.1.13.2.8 Q_r/Q = (0.75-1.5).

2.1.13.2.9 lb O_2/lb $BOD_r \geq 1.5$.

2.1.13.2.10 lb solids/lb $BOD_r \geq 0.2$.

2.1.13.2.11 0 = (1.0-1.03).

2.1.13.2.12 Efficiency = (> 90 percent).

2.1.13.3 <u>Process Design Calculations</u>.

2.1.13.3.1 Assume the following parameters.

2.1.13.3.1.1 Fraction of BOD synthesized (a).

2.1.13.3.1.2 Fraction of BOD oxidized for energy (a').

2.1.13.3.1.3 Endogenous respiration rate (b and b').

2.1.13.3.1.4 Fraction of BOD_5 synthesized to degradable solids (a_o).

2.1.13.3.1.5 Nonbiodegradable fraction to VSS in influent (f).

2.1.13.3.1.6 Mixed liquor suspended solids (MLSS).

2.1.13.3.1.7 Volatile solids in mixed liquor suspended solids (MLVSS).

2.1.13.3.1.8 Temperature correction coefficient (θ).

2.1.13.3.1.9 Degradable fraction of the MLVSS (f').

2.1.13.3.1.10 Food-to-microorganism ratio (F/M).

2.1.13.3.1.11 Effluent soluble BOD_5 (S_e).

2.1.13.3.2 Adjust the BOD removal rate constant for temperature.

$$K_T = K_{20}\theta^{(T-20)}$$

where

K_T = rate constant for desired temperature.

K_{20} = rate constant at 20°C.

θ = temperature correction coefficient.

T = temperature, °C.

2.1.13.3.3 Determine the size of the aeration tank.

$$V = \frac{a_o(S_o - S_e)Q_{avg}}{(X_V)(f')(b)}$$

where

V = aeration tank volume, million gal.

a_o = fraction of BOD_5 synthesized to degradable solids.

S_o = influent BOD_5, mg/l.

S_e = effluent soluble BOD_5, mg/l.

Q_{avg} = waste flow, mgd.

X_V = MLVSS, mg/l.

f' = degradable fraction of the MLVSS.

b = endogenous respiration rate, l/day.

2.1.13.3.4 Calculate the detention time.

$$t = \frac{V}{Q} 24$$

where

t = detention time, hr.

V = volume, million gal.

Q = flow, mgd.

2.1.13.3.5 Assume the organic loading and calculate detention time.

$$t = \frac{24 S_o}{X_V \frac{F}{M}}$$

where

t = detention time, days.

S_o = influent BOD_5, mg/l.

X_V = volatile solids in raw sludge, mg/l.

F/M = organic loading (food-to-microorganism ratio).

Select the larger of the two detention times from 2.1.13.3.4 or 2.1.13.3.5.

2.1.13.3.6 Determine the oxygen requirement allowing 60 percent for nitrification during summer.

$$O_2 = a'S_rQ_{avg}(8.34) + b'X_VV(8.34) + 0.6(4.57)(TKN)(Q_{avg})8.34$$

where

O_2 = oxygen required, lb/day.

a' = fraction of BOD oxidized for energy.

S_r = BOD_5 removed, mg/l.

Q_{avg} = average waste flow, mgd.

b' = endogenous respiration rate, l/day.

X_V = MLVSS, mg/l.

V = aeration tank volume, million gal.

TKN = total Kjeldahl nitrogen, mg/l.

and calculate oxygen requirement (≥ 1.5).

$$\frac{\text{lb } O_2}{\text{lb BOD}_r} = \frac{O_2}{(Q)(S_r)(8.34)}$$

where

O_2 = oxygen required, lb/day.

Q = waste flow, mgd.

S_r = BOD_5 removed, mg/l.

2.1.13.3.7 Design aeration system and check horsepower supply for complete mixing against horsepower required for complete mixing ≥ 0.1 hp/1000 gal.

2.1.13.3.7.1 Assume the following design parameters.

2.1.13.3.7.1.1 Standard transfer efficiency, percent, from manufacturer (5-8 percent).

2.1.13.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.13.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.13.3.7.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.13.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.13.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{(C_s)_T\ (\beta)\ (p) - C_L}{9.17} \propto (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, ^oC, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

T = temperature, ^oC.

2.1.13.3.7.4 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{O_2\ (10^5)\ (7.48)}{(OTE)\ \ 0.0176\ \frac{lb\ O_2}{ft^3\ air}\ \ 1440\ \frac{min}{day}\ \ V}$$

where

R_a = required air flow, cfm/1000 ft^3.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, percent.

V = volume of basin, gal.

2.1.13.3.7.5 Calculate sludge production.

$$\Delta X_V = 8.34\ [a(S_r)(Q) - (b)(X_V)(V) - Q(SS_{eff}) + Q(VSS)f' + Q(SS - VSS)]$$

where

ΔX_V = volatile sludge produced, lb/day.

a = fraction of BOD synthesized.

S_r = BOD_5 removed, mg/l.

Q = waste flow, mgd.

b = endogenous respiration rate, 1/day.

X_V = volatile solids in raw sludge, mg/l.

V = aeration tank volume, million gal.

SS_{eff} = effluent suspended solids, mg/l.

VSS = volatile suspended solids in influent, mg/l.

f' = degradable fraction of the MLVSS.

2.1.13.3.7.6 Calculate solids produced per pound of BOD removed.

$$\frac{\text{lb solids}}{\text{lb BOD}_r} = \frac{\Delta X_V}{Q(S_o - S_e)8.34}$$

where

ΔX_V = volatile sludge produced, lb/day.

Q = waste flow, mgd.

S_o = influent BOD_5, mg/l.

S_e = effluent soluble BOD_5, mg/l.

2.1.13.3.7.7 Calculate the solids retention time.

$$t_s = \frac{X_a(V)(8.34)}{\Delta X_V}$$

where

t_s = solids retention time, days.

X_a = MLSS, mg/l.

V = volume of aeration tank, million gal.

ΔX_V = volatile sludge produced, lb/day.

2.1.13.3.7.8 Effluent Characteristics.

2.1.13.3.7.8.1 BOD_5.

$$BODE = Se + 0.84\,(X_v)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.13.3.7.8.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.13.3.7.8.3 Nitrogen.

$$TKNE = (0.4)\ TKN$$

$$NH3E = TKNE$$

$$NO3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

NO3E = effluent nitrate concentration, mg/l.

2.1.13.3.7.8.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.13.3.7.8.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.13.3.7.8.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.13.3.7.9 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q_{avg}} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q_{avg} = average flow, mgd.

X_a = MLSS, mg/l.

X_u = suspended solids concentration in returned sludge, mg/l.

2.1.13.3.7.10 Calculate the nutrient requirements for nitrogen.

$$N = 0.123\,\Delta X_V$$

and phosphorus

$$P = 0.026\,\Delta X_V$$

where

ΔX_V = sludge produced, lb/day.

2.1.13.4 <u>Process Design Output Data.</u>

2.1.13.4.1 Aeration Tank.

2.1.13.4.1.1 Reaction rate constant, l/mg/hr.

2.1.13.4.1.2 Sludge produced per BOD removed.

2.1.13.4.1.3 Endogenous respiration rate (b, b').

2.1.13.4.1.4 O_2 utilized per BOD removed.

2.1.13.4.1.5 Influent nonbiodegradable VSS, mg/l.

2.1.13.4.1.6 Effluent degradable VSS, mg/l.

2.1.13.4.1.7 lb BOD/lb MLSS-day (F/M).

2.1.13.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.13.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.13.4.1.10 Aeration time, hr.

2.1.13.4.1.11 Volume of aeration tank, million gal.

2.1.13.4.1.12 Oxygen required, lb/day.

2.1.13.4.1.13 Sludge produced, lb/day.

2.1.13.4.1.14 Nitrogen requirement, lb/day.

2.1.13.4.1.15 Phosphorus requirement, lb/day.

2.1.13.4.1.16 Sludge recycle ratio, percent.

2.1.13.4.1.17 Solids retention time, days.

2.1.13.4.2 Aeration System.

2.1.13.4.2.1 Standard transfer efficiency, percent.

2.1.13.4.2.2 Operating transfer efficiency, percent.

2.1.13.4.2.3 Required air flow, cfm/1000 ft^3.

2.1.13.5 Quantities Calculations.

2.1.13.5.1 Design values for activated sludge system.

$$V_d = V \frac{10^6}{7.48}$$

$$CFM_d = (CFM)\ (V)\ (133.7)$$

where

V = volume of aeration tanks, million gallons.

2.1.13.5.2 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.13.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.13.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.13.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.13.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.13.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.13.5.4 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

2.1.13.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.1.13.5.6 Design of aeration tanks.

2.1.13.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.13.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.1.13.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.1.13.5.7 Aeration tank arrangements.

2.1.13.5.7.1 Figure 2.1-17 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = pipe gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.13.5.8 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.1.13.5.8.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\left[\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}\right]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.1.13.5.8.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\left[\frac{(15.75(NT)+15.5)\ (2L+PGW+20) + (15.75(NT)+2.5)\ (2L+PGW+28)}{2}\right]$$

2.1.13.5.9 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.1.13.5.9.1 For NT less than 4:

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

2.1.13.5.9.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)\ (2L + PGW + 200)]$$

2.1.13.5.10 Reinforced concrete wall quantities.

2.1.13.5.10.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-18. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.1.13.5.10.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.13.5.10.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.13.5.10.4 When NT $\geq$ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) +$$

$$2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

V_{cw} = R.C. wall quantity required, cu ft.

L = length of aeration tanks, ft.

2.1.13.5.11 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.1.13.5.11.1 If NT is less than 4:

$$LHR = [2(NT)\ (L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.1.13.5.11.2 If NT is greater than or equal to 4:

$$LHR = [2(NT)\ (L) + (4L) + 36.5(NT) + 2\ PGW + 13]\ NB$$

where

LHR = handrail length, ft.

2.1.13.5.12 Calculate operation manpower requirements.

2.1.13.5.12.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.1.13.5.12.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.1.13.5.13 Calculate maintenance manpower requirements.

2.1.13.5.13.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.1.13.5.13.2 If CFM_d > 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.1.13.5.14 Energy requirement for operation. The electrical energy required for operation is related to the air requirement by the following equation:

$$KWH = (CFM_d)\ (241.6)$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.1.13.5.15 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

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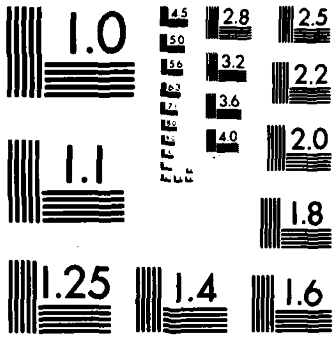


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

$$OMMP = 3.57\,(Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.1.13.5.16 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.13.6 Quantities Calculation Output Data.

2.1.13.6.1 Number of aeration tanks, NT.

2.1.13.6.2 Number of diffusers per tank, ND_t.

2.1.13.6.3 Number of process batteries, NB.

2.1.13.6.4 Number of swing arm headers per tank, NSA_t.

2.1.13.6.5 Length of aeration tanks, L, ft.

2.1.13.6.6 Width of pipe gallery, PGW, ft.

2.1.13.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.1.13.6.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.1.13.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.1.13.6.10 Quantity of handrail, LHR, ft.

2.1.13.6.11 Operation manpower requirement, OMH, MH/yr.

2.1.13.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.1.13.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.1.13.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.1.13.6.15 Correction factor for minor construction costs, CF.

2.1.13.7 Unit Price Input Required.

2.1.13.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.1.13.7.2 Unit price input R.C. wall in-place, UPICW, $/cu yd.

2.1.13.7.3 Unit price input R.C. slab in-place, UPICS, $/cu yd.

2.1.13.7.4 Unit price input for handrails in-place, UPIHR, $/ft.

2.1.13.7.5 Cost per diffuser, COSTPD, $, (optional).

2.1.13.7.6 Cost per swing arm header, COSTPH, $, (optional).

2.1.13.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.13.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.1.13.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.1.13.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.1.13.8 Cost Calculations.

2.1.13.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.13.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.1.13.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price R.C. slab in-place, \$/cu yd.

2.1.13.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, $.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, $/ft.

2.1.13.8.5 Cost of diffusers.

2.1.13.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.1.13.8.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, $.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.1.13.8.6 Cost of swing arm diffuser headers.

2.1.13.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5,000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5,000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.13.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, $.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.1.13.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.1.13.8.8 Crane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.1.13.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.1.13.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.1.13.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.1.13.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.1.13.8.10 Other costs associated with the installed equipment. This category includes the cost for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.13.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)$$

where

IEC = installed equipment cost, \$.

$LABRI$ = labor rate, \$/MH.

$UPICR$ = crane rental rate, \$/hr.

2.1.13.8.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

$TBCC$ = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.1.13.8.13 Operation and maintenance material costs.

$$OMCC = TBCC\ \frac{OMMP}{100}$$

where

$OMMC$ = operation and maintenance material supply costs, \$/yr.

$OMMP$ = operation and maintenance material supply costs, as percent of total bare construction cost, percent.

2.1.13.9 <u>Cost Calculations Output Data</u>.

2.1.13.9.1 Total bare construction cost of diffused aeration activated sludge system, TBCC, dollars.

2.1.13.9.2 Operation and maintenance material and supply costs, OMMC, dollars.

2.1.14 EXTENDED AERATION ACTIVATED SLUDGE (MECHANICAL AERATION).

2.1.14.1 Input Data.

2.1.14.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.14.1.2 Wastewater Strength.

2.1.14.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.14.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.14.1.2.3 Suspended solids, mg/l.

2.1.14.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.14.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.14.1.3 Other Characterization.

2.1.14.1.3.1 pH.

2.1.14.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.14.1.3.3 Nitrogen,[1] mg/l.

2.1.14.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.14.1.3.5 Oils and greases, mg/l.

2.1.14.1.3.6 Heavy metals, mg/l.

2.1.14.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.14.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.14.1.4 Effluent Quallity Requirements.

2.1.14.1.4.1 BOD_5, mg/l.

2.1.14.1.4.2 SS, mg/l.

2.1.14.1.4.3 TKN, mg/l.

2.1.14.1.4.4 P, mg/l.

---

[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.14.2 **Design Parameters.**

2.1.14.2.1 Reaction rate constants.

Constants	Range
Eckenfelder	
k	0.0007-0.002 1/mg/hr
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
a_0	0.77a = 0.56
f	0.40
f'	0.53

2.1.14.2.2 F/M = (0.05-0.15).

2.1.14.2.3 Volumetric loading = 10-25.

2.1.14.2.4 t = (18-36) hr.

2.1.14.2.5 t_s = (20-30) days.

2.1.14.2.6 MLSS = (3000-6000) mg/l.

2.1.14.2.7 MLVSS = (2100-4200) mg/l.

2.1.14.2.8 Q_r/Q = (0.75-1.5).

2.1.14.2.9 lb O_2/lb $BOD_r \geq 1.5$.

2.1.14.2.10 lb solids/lb $BOD_r \geq 0.2$.

2.1.14.2.11 0 = (1.0-1.03).

2.1.14.2.12 Efficiency = (> 90 percent).

2.1.14.3 **Process Design Calculations.**

2.1.14.3.1 Assume the following parameters.

2.1.14.3.1.1 Fraction of BOD synthesized (a).

2.1.14.3.1.2 Fraction of BOD oxidized for energy (a').

2.1.14.3.1.3 Endogenous respiration rate (b and b').

2.1.14.3.1.4 Fraction of BOD_5 synthesized to degradable solids (a_o).

2.1.14.3.1.5 Nonbiodegradable fraction to VSS in influent (f).

2.1.14.3.1.6 Mixed liquor suspended solids (MLSS).

2.1.14.3.1.7 Volatile solids in mixed liquor suspended solids (MLVSS).

2.1.14.3.1.8 Temperature correction coefficient (θ).

2.1.14.3.1.9 Degradable fraction of the MLVSS (f').

2.1.14.3.1.10 Food-to-microorganism ratio (F/M).

2.1.14.3.1.11 Effluent soluble BOD_5 (S_e).

2.1.14.3.2 Adjust the BOD removal rate constant for temperature.

$$K_T = K_{20}\theta^{(T-20)}$$

where

K_T = rate constant for desired temperature.

K_{20} = rate constant at 20°C.

θ = temperature correction coefficient.

T = temperature, °C.

2.1.14.3.3 Determine the size of the aeration tank.

$$V = \frac{a_o(S_o - S_e)Q_{avg}}{(X_V)(f')(b)}$$

where

V = aeration tank volume, million gal.

a_o = fraction of BOD_5 synthesized to degradable solids.

S_o = influent BOD_5, mg/l.

S_e = effluent soluble BOD_5, mg/l.

Q_{avg} = waste flow, mgd.

X_V = MLVSS, mg/l.

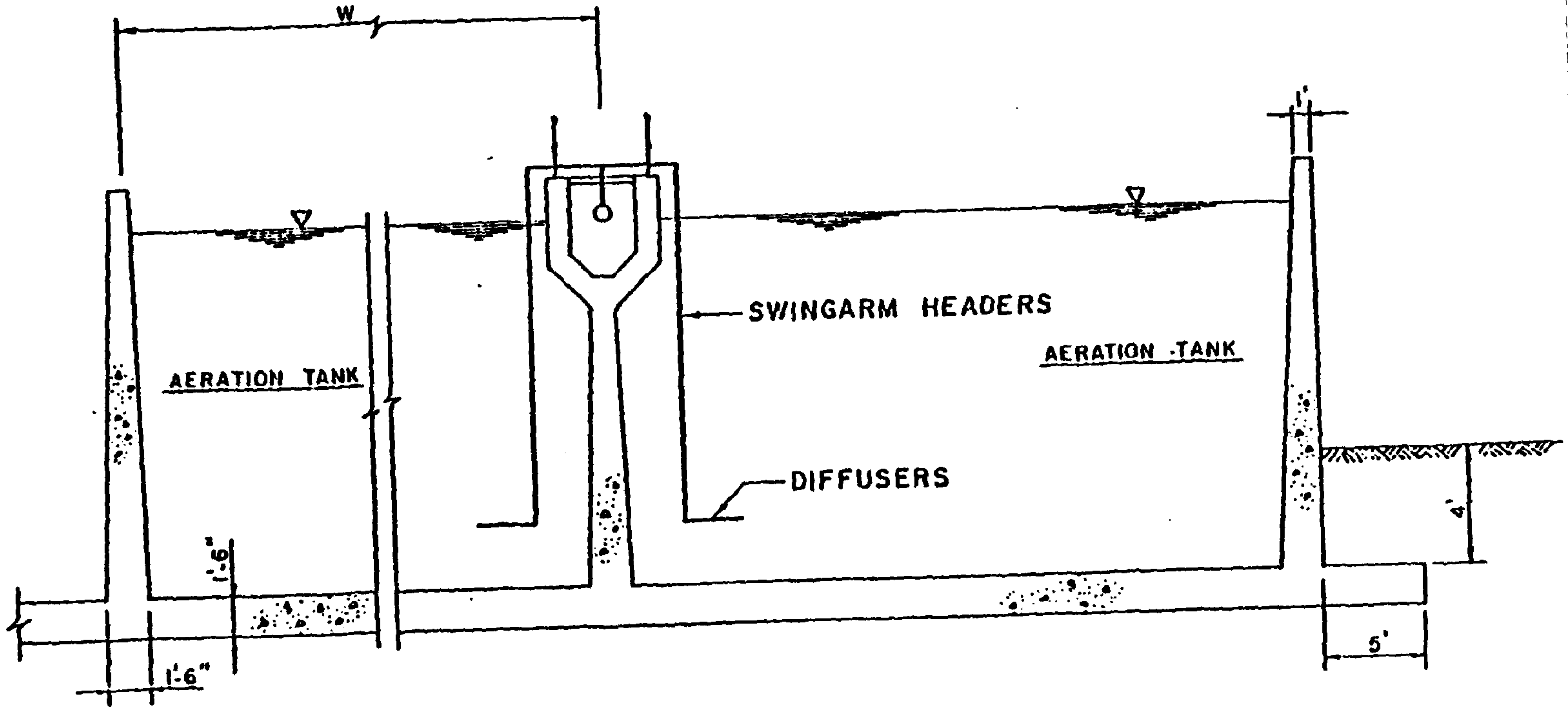


FIGURE 2.1-18 TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

f' = degradable fraction of the MLVSS.

b = endogenous respiration rate, 1/day.

2.1.14.3.4 Calculate the detention time.

$$t = \frac{V}{Q} 24$$

where

t = detention time, hr.

V = volume, million gal.

Q = flow, mgd.

2.1.14.3.5 Assume the organic loading and calculate detention time.

$$t = \frac{24S_o}{X_V \frac{F}{M}}$$

where

t = detention time, days.

S_o = influent BOD_5, mg/l.

X_V = volatile solids in raw sludge, mg/l.

F/M = organic loading (food-to-microorganism ratio).

Select the larger of the two detention times from 2.1.14.3.4 or 2.1.14.3.5.

2.1.14.3.6 Determine the oxygen requirement allowing 60 percent for nitrification during summer.

$$O_2 = a'S_rQ_{avg}(8.34) + b'X_VV(8.34) + 0.6(4.57)(TKN)(Q_{avg})8.34$$

where

O_2 = oxygen required, lb/day.

a' = fraction of BOD oxidized for energy.

S_r = BOD_5 removed, mg/l.

Q_{avg} = average waste flow, mgd.

b' = endogenous respiration rate, 1/day.

X_V = MLVSS, mg/l.

V = aeration tank volume, million gal.

TKN = total Kjeldahl nitrogen, mg/l.

and calculate oxygen requirement (1.5).

$$\frac{\text{lb } O_2}{\text{lb } BOD_r} = \frac{O_2}{(Q)(S_r)(8.34)}$$

where

O_2 = oxygen required, lb/day.

Q = waste flow, mgd.

S_r = BOD_5 removed, mg/l.

2.1.14.3.7 Design Aeration System.

2.1.14.3.7.1 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing 0.1 hp/1000 gal.

2.1.14.3.7.1.1 Standard transfer efficiency, lb/hp-hr (0 dissolved oxygen, 20°C, and tap water) (3-5 lb/hr-hr).

2.1.14.3.7.1.2 O_2 transfer in waste/O_2 transfer in water ≈ 0.9.

2.1.14.3.7.1.3 O_2 saturation in waste/O_2 saturation in water ≈ 0.9.

2.1.14.3.7.1.4 Correction factor for pressure ≈ 1.0.

2.1.14.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.14.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{[(C_s)_T(\beta)(p) - C_L]}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$1.0.

C_L = minimum dissolved oxygen to be maintained in the basin $\approx$ 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, °C.

2.1.14.3.7.4 Calculate horsepower requirement.

$$hp = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)(V)} \times 1000$$

where

hp = horsepower required/1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of basin, gal.

2.1.14.3.8 Calculate sludge production.

$$\Delta X_V = 8.34\ [a(S_r)(Q) - (b)(X_V)(V) - Q(SS_{eff}) + Q(VSS)f' + Q(SS - VSS)]$$

where

ΔX_V = volatile sludge produced, lb/day.

a = fraction of BOD synthesized.

S_r = BOD_5 removed, mg/l.

Q = waste flow, mgd.

b = endogenous respiration rate, l/day.

X_V = volatile solids in raw sludge, mg/l.

V = aeration tank volume, million gal.

SS_{eff} = effluent suspended solids, mg/l.

VSS = volatile suspended solids in influent, mg/l.

f' = degradable fraction of the MLVSS.

2.1.14.3.9 Calculate solids produced per pound of BOD removed.

$$\frac{\text{lb solids}}{\text{lb BOD}_r} = \frac{\Delta X_V}{Q(S_o - S_e)8.34}$$

where

ΔX_V = volatile sludge produced, lb/day.

Q = waste flow, mgd.

S_o = influent BOD_5, mg/l.

S_e = effluent soluble BOD_5, mg/l.

2.1.14.3.10 Calculate the solids retention time.

$$t_s = \frac{X_a(V)(8.34)}{\Delta X_V}$$

where

t_s = solids retention time, days.

X_a = MLSS, mg/l.

V = volume of aeration tank, million gal.

ΔX_V = volatile sludge produced, lb/day.

2.1.14.3.11 Effluent Characteristics.

2.1.14.3.11.1 BOD_5.

$$\text{BODE} = \text{Se} + 0.84\ (X_v)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.14.3.11.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.14.3.11.3 Nitrogen.

$$TKNE = (0.4)\ TKN$$

$$NH3E = TKNE$$

$$NO3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

NO3E = effluent nitrate concentration, mg/l.

2.1.14.3.11.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.14.3.11.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.14.3.11.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.14.3.12 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q_{avg}} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q_{avg} = average flow, mgd.

X_a = MLSS, mg/l.

X_u = suspended solids concentration in returned sludge, mg/l.

2.1.14.3.13 Calculate the nutrient requirements for nitrogen.

$$N = 0.123\,\Delta X_V$$

and phosphorus

$$P = 0.026\,\Delta X_V$$

where

ΔX_V = sludge produced, lb/day.

2.1.14.4 <u>Process Design Output Data.</u>

2.1.14.4.1 Aeration Tank.

2.1.14.4.1.1 Reaction rate constant, l/mg/hr.

2.1.14.4.1.2 Sludge produced per BOD removed.

2.1.14.4.1.3 Endogenous respiration rate (b, b').

2.1.14.4.1.4 O_2 utilized per BOD removed.

2.1.14.4.1.5 Influent nonbiodegradable VSS, mg/l.

2.1.14.4.1.6 Effluent degradable VSS, mg/l.

2.1.14.4.1.7 lb BOD/lb MLSS-day (F/M).

2.1.14.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.14.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.14.4.1.10 Aeration time, hr.

2.1.14.4.1.11 Volume of aeration tank, million gal.

2.1.14.4.1.12 Oxygen required, lb/day.

2.1.14.4.1.13 Sludge produced, lb/day.

2.1.14.4.1.14 Nitrogen requirement, lb/day.

2.1.14.4.1.15 Phosphorus requirement, lb/day.

2.1.14.4.1.16 Sludge recycle ratio, percent.

2.1.14.4.1.17 Solids retention time, days.

2.1.14.4.2 Aeration System.

2.1.14.4.2.1 Standard transfer efficiency, lb O_2/hp-hr.

2.1.14.4.2.2 Operating transfer efficiency, lb O_2/hp-hr.

2.1.14.4.2.3 Horsepower required, hr.

2.1.14.5 <u>Quantities Calculations</u>.

2.1.14.5.1 The design values for activated sludge system would be:

$$V_d = V \cdot \frac{10^6}{7.48}$$

$$HP_d \text{ (hp)} \cdot (V)\ (133.7)$$

where

V = volume of aeration basin million gallons.

2.1.14.5.2 Selection of number of aeration tanks and mechanical aerators per tank. The following rule will be utilized in the selection of number of aeration tanks and mechanical aerators per tank.

Q_{avg} (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NT
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.14.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.14.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.14.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.14.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.14.5.3.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.14.5.4 Mechanical aeration equipment design.

2.1.14.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.1.14.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)(NT)(NA)}$$

If HPN > 150 HP and NT = 2 or 3, then repeat the calculation with NT = NT + 1.

If HPN > 150 HP and NT $\geq$ 4, then repeat the calculation with NT = NT + 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.1.14.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.1.14.5.5 Design of aeration tanks.

2.1.14.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.14.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When HPSN $\leq$ 100 HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

When HPSN $>$ 100 HP

$$DW = 15\ ft$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.1.14.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.1.14.5.6 Aeration tank arrangements.

2.1.14.5.6.1 Figure 2.1-19 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.1.14.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\ \frac{(Q_{avg})}{NB}$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.14.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.1.14.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\ [(2\ W + 18.5)\ (W + 17) + (2\ W + 26.5)\ (W + 25)]$$

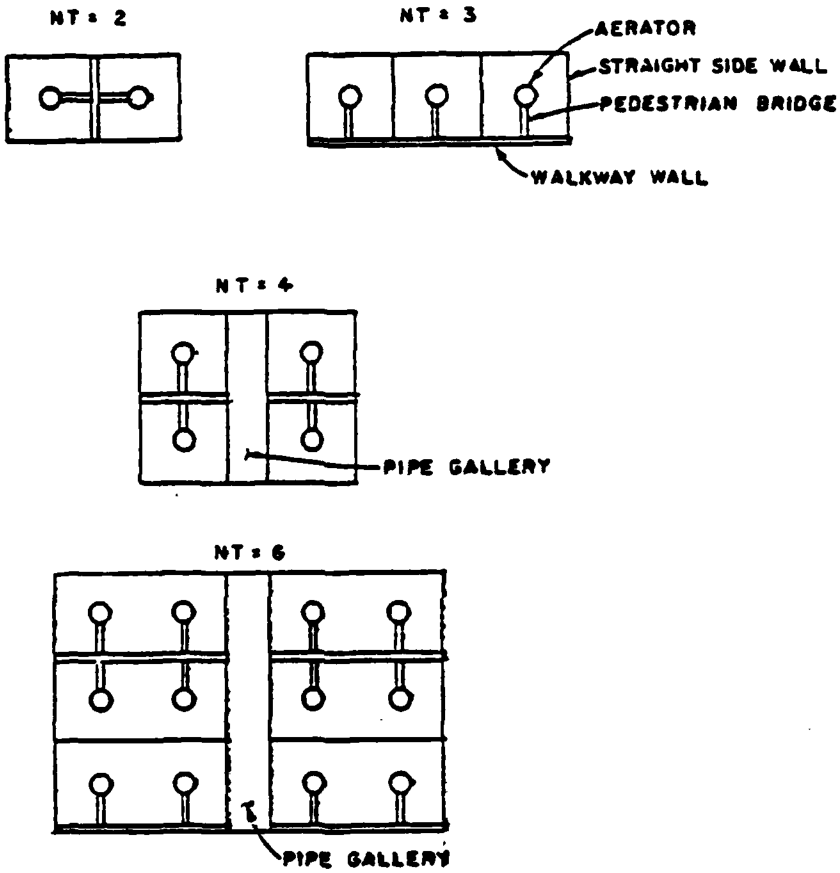
where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.1.14.5.7.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.1-19 EXAMPLES OF TANK ARRANGEMENTS ACTIVATED SLUDGE PROCESSES

2.1.14.5.7.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB)\ [(L_s + 4)\ (W_s + 4) + (L_s + 12)\ (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.1.14.5.8 Reinforced concrete slab quantity.

2.1.14.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.1.14.5.8.2 For NT = 2,

$$V_{cs} = 1.5\ (2\ W + 14.5)\ (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.1.14.5.8.3 NT = 3,

$$V_{cs} = 1.5\ (3\ W + 16)\ (W + 13)$$

2.1.14.5.8.4 When NT $\geq$ 4,

$$V_{cs} = 1.5\ (L_s)\ (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.1.14.5.9 Reinforced Concrete Wall Quantity.

2.1.10.5.9.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-20. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.1.14.5.9.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.14.5.9.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.14.5.9.4 When NT ≥ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

2.1.14.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.1.14.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.1.14.5.10.2 When NT ≥ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5)\ (PGW)\ [\frac{(NT)\ (W)}{2} + 0.75\ (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

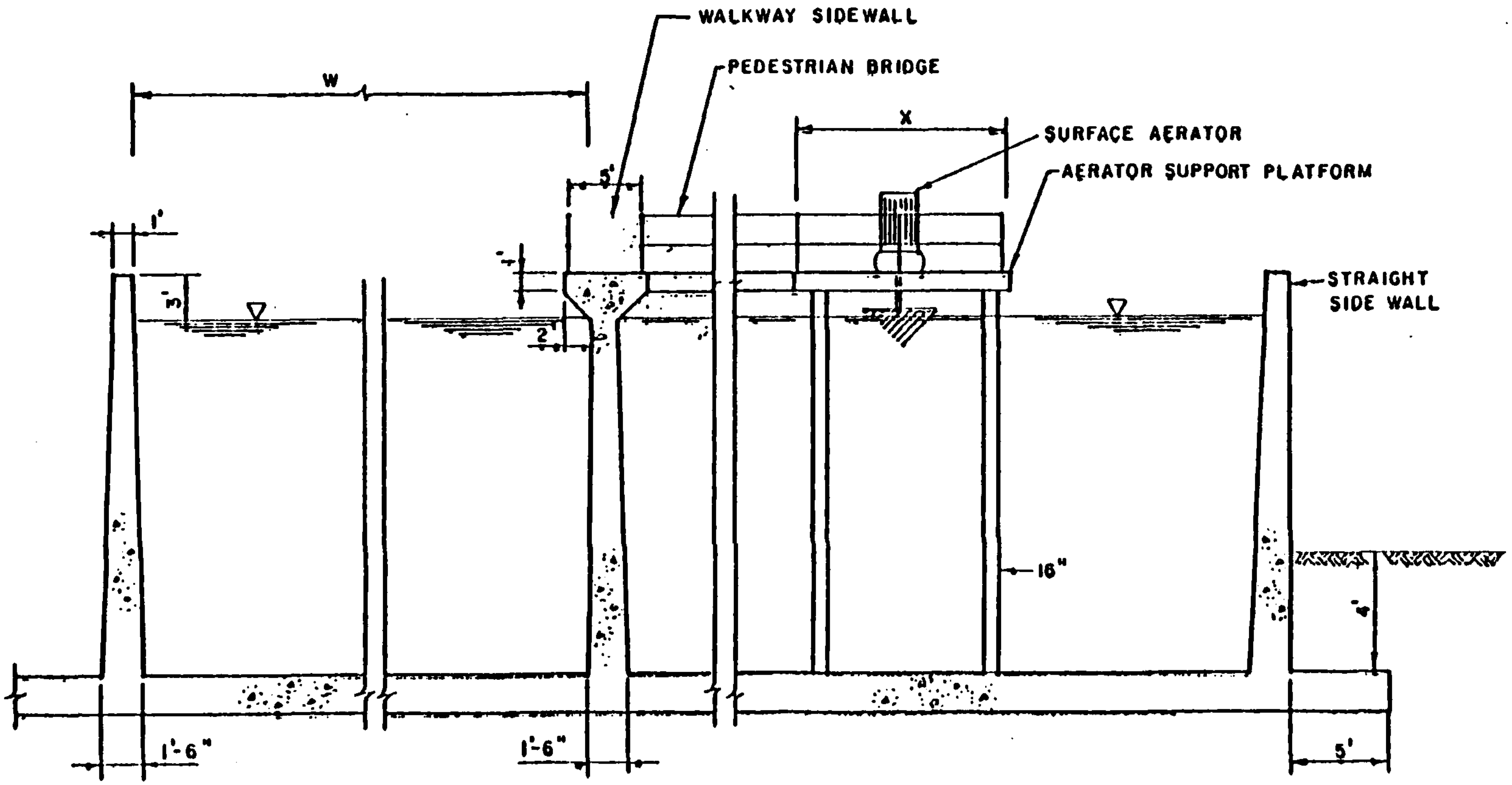


FIGURE 2.1-20 TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

$$V_{cgw} = 2 \text{ (PGW) (NB) (DW + 3)}$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.1.14.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.1.14.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.1.14.5.11.2 Figure 2.1-21 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078 \text{ (HPSN)}$$

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.1.14.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6 \text{ (DW + 2)] (NT) (NA) (NB)}$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.1.14.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2 \text{ (W - X)] (NB) (NT) (NA)}$$

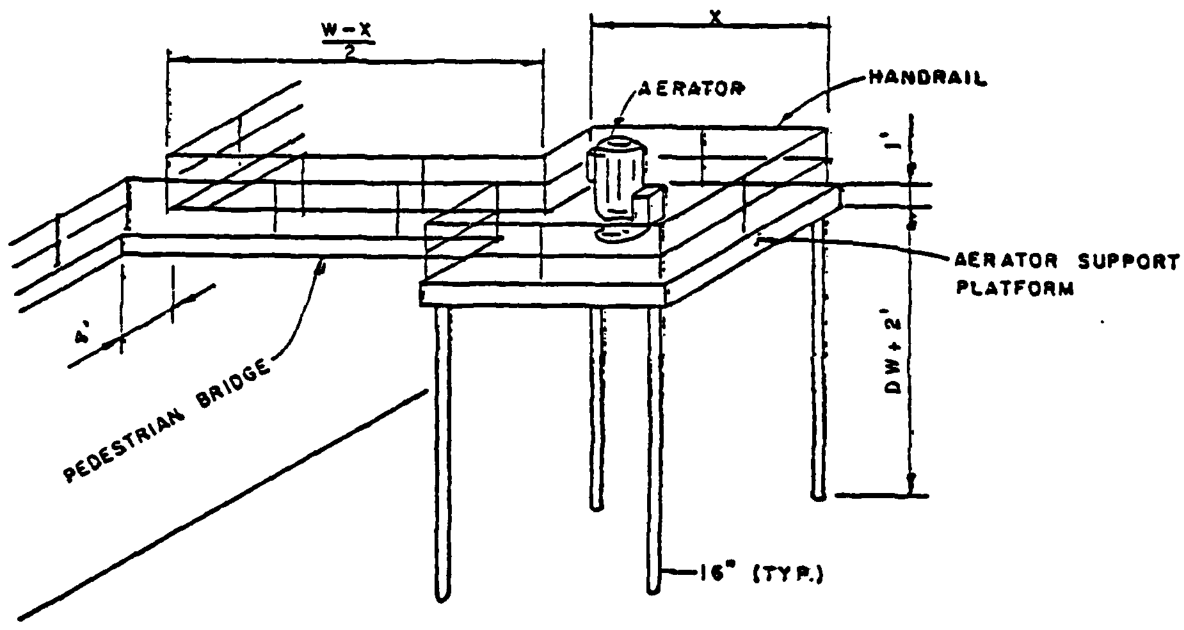


FIGURE 2.1-21 AERATION SUPPORT PLATFORM

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.1.14.5.12 Summary of reinforced concrete structures.

2.1.14.5.12.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.1.14.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.1.14.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.1.14.5.13.1 When NT = 2,

$$LHR = 4\ W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.1.14.5.13.2 When NT = 3,

$$LHR = 6\ W + 10 + 3 \cdot (3X + W - 4)$$

2.1.14.5.13.3 When NT $\geq$ 4,

If $\frac{NT}{2}$ is an even number,

$$LHR = PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT) + (NA) \cdot (NT) \cdot (3X + W - 4) \cdot (NB)$$

If $\frac{NT}{2}$ is an odd number,

$$LHR = PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT + 2) + (NA)\ (NT)\ (3X + w - 4) \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.1.14.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.1.14.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)\ (NT)\ (NA)\ (HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.1.14.5.14.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\ (TICA)^{0.3731}$$

$$\text{When TICA} \geq 200 \text{ hp}$$

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.1.14.5.14.3 The maintenance manpower requirement can be estimated as follows:

$$\text{When TICA} \leq 100 \text{ hp}$$

$$MMH = 106.3\ (TICA)^{0.4031}$$

$$\text{When TICA} > 100 \text{ hp}$$

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.1.14.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.14.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.1.14.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.14.6 Quantities Calculations Output Data.

2.1.14.6.1 Number of aeration tanks, NT.

2.1.14.6.2 Number of aerators per tank, NA.

2.1.14.6.3 Number of process batteries, NB.

2.1.14.6.4 Capacity of each individual aerator, HPSN, hp.

2.1.14.6.5 Depth of aeration tanks, DW, ft.

2.1.14.6.6 Length of aeration tanks, L, ft.

2.1.14.6.7 Width of aeration tanks, W, ft.

2.1.14.6.8 Width of pipe gallery, PGW, ft.

2.1.14.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.1.14.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.1.14.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.1.14.6.12 Quantity of handrail, LHR, ft.

2.1.14.6.13 Operation manpower requirement, OMH, MH/yr.

2.1.14.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.1.14.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.1.14.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.1.14.6.17 Correction factor for minor capital cost items, CF.

2.1.14.7 Unit Price Input Required.

2.1.14.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.1.14.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.1.14.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.1.14.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, $, optional.

2.1.14.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.1.14.7.6 Equipment installation labor rate, $/MH.

2.1.14.7.7 Crane rental rate, UPICR, $/hr.

2.1.14.7.8 Unit price of handrail, UPIHR, $/L.F.

2.1.14.8 Cost Calculations.

2.1.14.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.14.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, \$.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/ cu yd.

2.1.14.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.1.14.8.4 Cost of installed aeration equipment.

2.1.14.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.1.14.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.1.14.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16,300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16,300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.14.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When HPSN $\leq$ 60 hp

$$IMH = 39 + 0.55\ (HPSN)$$

When HPSN $>$ 60 hp

$$IMH = 61.3 + 0.18\ (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.14.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.14.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, dollars.

LABRI = labor rate, dollars/man-hour.

UPICR = crane rental rate, dollars/hr.

2.1.14.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, \$ per lineal foot. A value of \$25.20 per foot for the first quarter of 1977 is suggested.

2.1.14.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.1.14.8.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, dollars.

CF = correction factor for minor cost items, from second-order design output.

2.1.14.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, $/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.1.14.9 Cost Calculations Output Data.

2.1.14.9.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, dollars.

2.1.14.9.2 Operation and maintenance supply and material costs, OMMC, dollars.

2.1.15 HIGH RATE ACTIVATED SLUDGE (DIFFUSED AERATION).

2.1.15.1 Input Data.

2.1.15.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.15.1.2 Wastewater Strength.

2.1.15.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.15.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.15.1.2.3 Suspended solids, mg/l.

2.1.15.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.15.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.15.1.3 Other Characterization.

2.1.15.1.3.1 pH.

2.1.15.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.15.1.3.3 Nitrogen,[1] mg/l.

2.1.15.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.15.1.3.5 Oils and greases, mg/l.

2.1.15.1.3.6 Heavy metals, mg/l.

2.1.15.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.15.1.3.8 Temperature, ^oF or ^oC.

2.1.15.1.4 Effluent Quality Requirements.

2.1.15.1.4.1 BOD_5, mg/l.

2.1.15.1.4.2 SS, mg/l.

2.1.15.1.4.3 TKN, mg/l.

2.1.15.1.4.4 P, mg/l.

2.1.15.1.4.5 Total nitrogen ($TKN + NO_3 - N$), mg/l.

---

[1] The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.15.1.4.6 Settleable solids, mg/l.

2.1.15.2 <u>Design Parameters</u>.

2.1.15.2.1 Reaction rate constants and coefficients.

Eckenfelder	
k	0.0007-0.002 l/mg/hr.
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
f	0.40
f'	0.53

2.1.15.2.2 F/M = (1.5-5.0).

2.1.15.2.3 Volumetric loading = 100-250.

2.1.15.2.4 t = (1.5-3.0) hr.

2.1.15.2.5 t_s = (0.2-0.5) days.

2.1.15.2.6 MLSS = (200-1000) mg/l.

2.1.15.2.7 MLVSS = (140-700) mg/l.

2.1.15.2.8 Q_r/Q = (0.05-0.15).

2.1.15.2.9 lb O_2/lb BOD_r = (0.5-0.75).

2.1.15.2.10 lb solids/lb BOD_r = (0.65-0.85).

2.1.15.2.11 Efficiency = (50-60 percent).

2.1.15.3 <u>Process Design Calculations</u>.

2.1.15.3.1 Assume the following design parameters when unknown.

2.1.15.3.1.1 Fraction of BOD synthesized (a).

2.1.15.3.1.2 Fraction of BOD oxidized for energy (a').

2.1.15.3.1.3 Endogenous respiration rate (b and b').

2.1.15.3.1.4 Mixed liquor suspended solids (MLSS).

2.1.15.3.1.5 Mixed liquor volatile suspended solids (MLVSS).

2.1.15.3.1.6 Food-to-microorganism ratio (F/M).

2.1.15.3.1.7 Temperature correction coefficient (θ).

2.1.15.3.1.8 Nonbiodegradable fraction of VSS in influent (f).

2.1.15.3.1.9 Degradable fraction of the MLVSS (f').

2.1.15.3.2 Determine the size of the aeration tank by first determining the detention time.

$$t = \frac{24S_o}{(X_V)(F/M)}$$

where

t = detention time, hr.

S_o = influent BOD, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.15.3.3 Calculate the volume of aeration tank.

$$V = Q_{avg} \frac{t}{24}$$

where

V = volume of aeration tank, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.15.3.4 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'(S_r)}{t}$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34)$$

where

dO/dt = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed (S_o - S_e), mg/l.

t = detention time, hr.

O_2 = oxygen uptake rate, lb/day.

Q_{avg} = average flow rate, mgd.

2.1.15.3.5 Check the oxygen supplied against pounds of BOD removed (0.5-0.7).

$$1b\ O_2/1b\ BOD_r = \frac{O_2}{Q(S_r)(8.34)}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.15.3.6 Design Aeration System.

2.1.15.3.6.1 Assume the following design parameters.

2.1.15.3.6.1.1 Standard transfer efficiency, percent, from manufacturer (5-8 percent).

2.1.15.3.6.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.15.3.6.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.15.3.6.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.15.3.6.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.15.3.6.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{(C_s)_T(\beta)(p) - C_L}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water ≈ 0.9.

p = correction factor for pressure ≈ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water ≈ 0.9.

T = temperature, °C.

2.1.15.3.6.4 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{O_2(10^5)(7.48)}{(OTE)\ \ 0.0176\ \frac{lb\ O_2}{ft^3 air}\ \ 1440\ \frac{min}{day}\ \ V}$$

where

R_a = required air flow, cfm/1000 ft^3.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, percent.

V = volume of basin, gal.

2.1.15.3.7 Calculate sludge production.

$$\Delta X_V = [aS_rQ_{avg} + fQ(VSS) + Q(SS - VSS]\ 8.34$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

f = nonbiodegradable fraction of VSS in influent.

Q = flow, mgd.

VSS = volatile suspended solids, mg/l.

SS = suspended solids in influent, mg/l.

2.1.15.3.8 Check ΔX_V against 0.65-0.85 lb solids/lb BOD_r.

$$\frac{\text{lb solids}}{(\text{lb } BOD_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)}$$

where

ΔX_V = sludge produced, lb/day.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.1.15.3.9 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q = flow, mgd.

X_a = MLSS

X_u = suspended solids concentration in returned sludge, mg/l.

2.1.15.3.10 Calculate solids retention time.

$$SRT = \frac{V(X_a)8.34}{\Delta X_a}\ (\%\ \text{volatile})$$

where

SRT = solids retention time, days.

V = volume of basin, million gal.

X_a = MLSS.

ΔX_a = sludge produced, lb/day.

2.1.15.3.11 Effluent Characteristics.

2.1.15.3.11.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.15.3.11.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.15.3.11.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.15.3.11.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.15.3.11.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.15.3.11.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.15.3.12 Determine nutrient requirements for nitrogen.

$$N = 0.123 \Delta X_V$$

and phosphorus

$$P = 0.026 \quad X_V$$

where

ΔX_V = sludge produced, lb/day.

and check against BOD:N:P = 100:5:1.

2.1.15.4 <u>Process Design Output Data.</u>

2.1.15.4.1 Aeration Tank.

2.1.15.4.1.1 Reaction rate constant, l/mg/hr.

2.1.15.4.1.2 Sludge produced per BOD removed.

2.1.15.4.1.3 Endogenous respiration rate (b, b').

2.1.15.4.1.4 O_2 utilized per BOD removed.

2.1.15.4.1.5 Influent nonbiodegradable VSS, mg/l.

2.1.15.4.1.6 Effluent degradable VSS, mg/l.

2.1.15.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.15.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.15.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.15.4.1.10 Aeration time, hr.

2.1.15.4.1.11 Volume of aeration tank, million gal.

2.1.15.4.1.12 Oxygen required, lb/day.

2.1.15.4.1.13 Sludge produced, lb/day.

2.1.15.4.1.14 Nitrogen requirement, lb/day.

2.1.15.4.1.15 Phosphorus requirement, lb/day.

2.1.15.4.1.16 Sludge recycle ratio, percent.

2.1.15.4.1.17 Solids retention time, days.

2.1.15.4.1.18 Volumetric loading, lb BOD/1000 ft^3.

2.1.15.4.2 Aeration System.

2.1.15.4.2.1 Standard transfer efficiency, percent.

2.1.15.4.2.2 Operating transfer efficiency, percent.

2.1.15.4.2.3 Required air flow, cfm/1000 ft^3.

2.1.15.5 Quantities Calculations.

2.1.15.5.1 Design values for activated sludge system.

$$V_d = V \frac{10^6}{7.48}$$

$$CFM_d = (CFM)\ (V)\ (133.7)$$

where

V = volume of aeration tanks, million gallons.

2.1.15.5.2 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.15.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.15.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.15.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.15.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.15.5.3.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.15.5.4 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

2.1.15.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.1.15.5.6 Design of aeration tanks.

2.1.15.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.15.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.1.15.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.1.15.5.7 Aeration tank arrangements.

2.1.15.5.7.1 Figure 2.1-22 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\ (\frac{Q_{avg}}{NB})$$

where

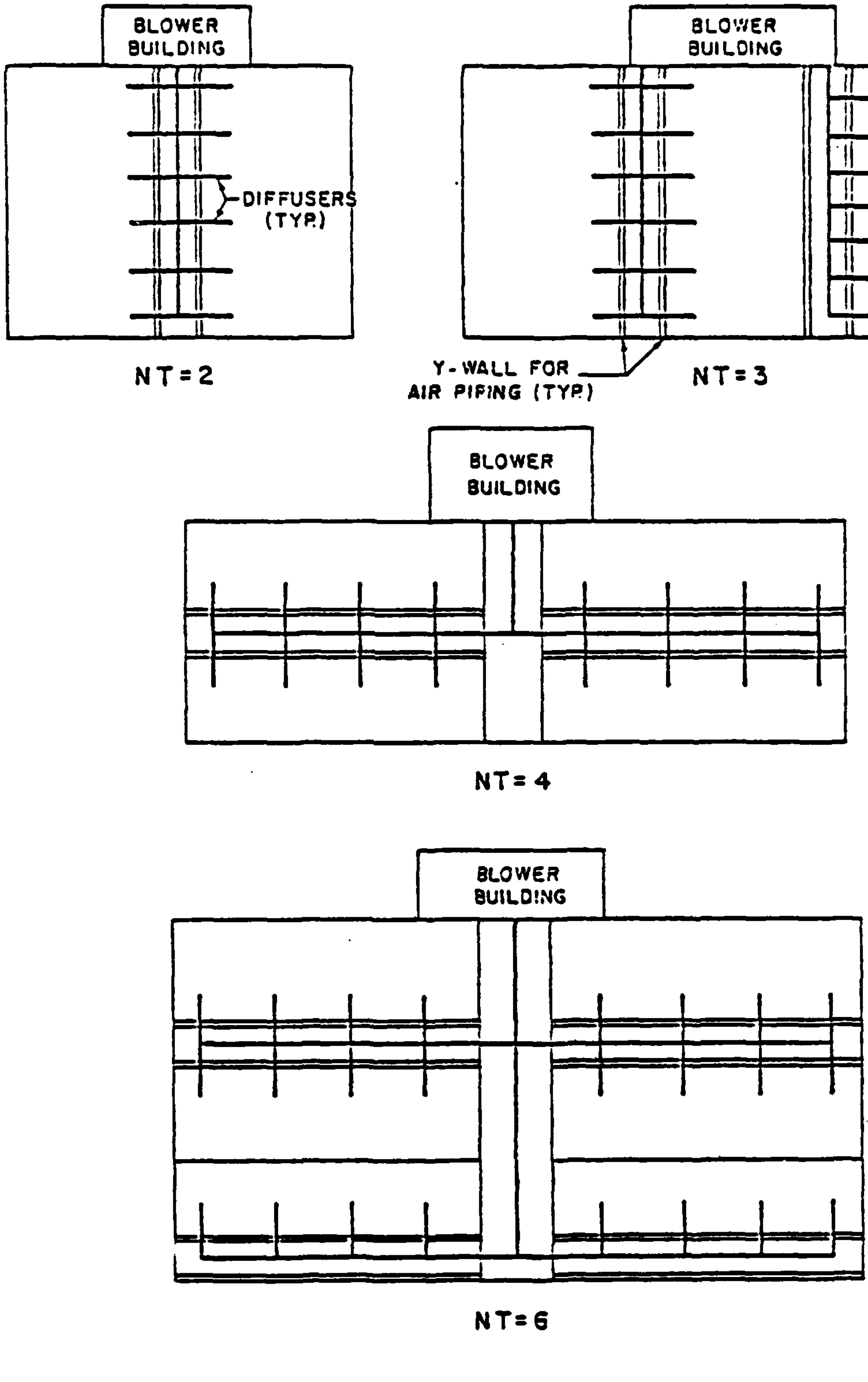


FIGURE 2.1-22 AERATION TANK ARRANGEMENT

PGW = pipe gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.15.5.8 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.1.15.5.8.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ \left[\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}\right]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.1.15.5.8.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ \left[\frac{(15.75(NT) + 15.5)\ (2L + PGW + 20) + (15.75(NT) + 2.5)\ (2L + PGW + 28)}{2}\right]$$

2.1.15.5.9 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.1.15.5.9.1 For NT less than 4:

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

2.1.15.5.9.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)\ (2L + PGW + 200)]$$

2.1.15.5.10 Reinforced concrete wall quantities.

2.1.15.5.10.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-23. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

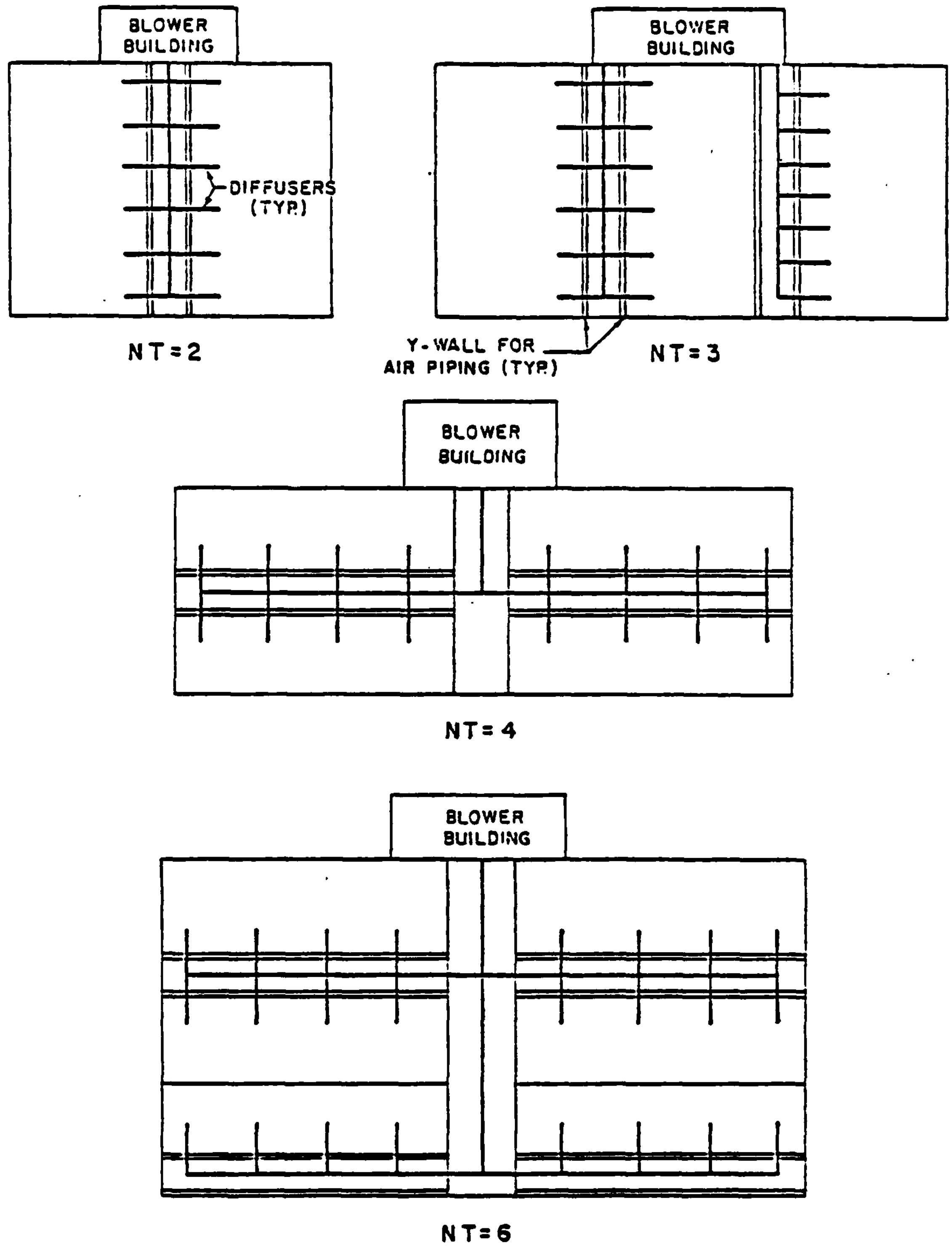


FIGURE 2.1-17 AERATION TANK ARRANGEMENT

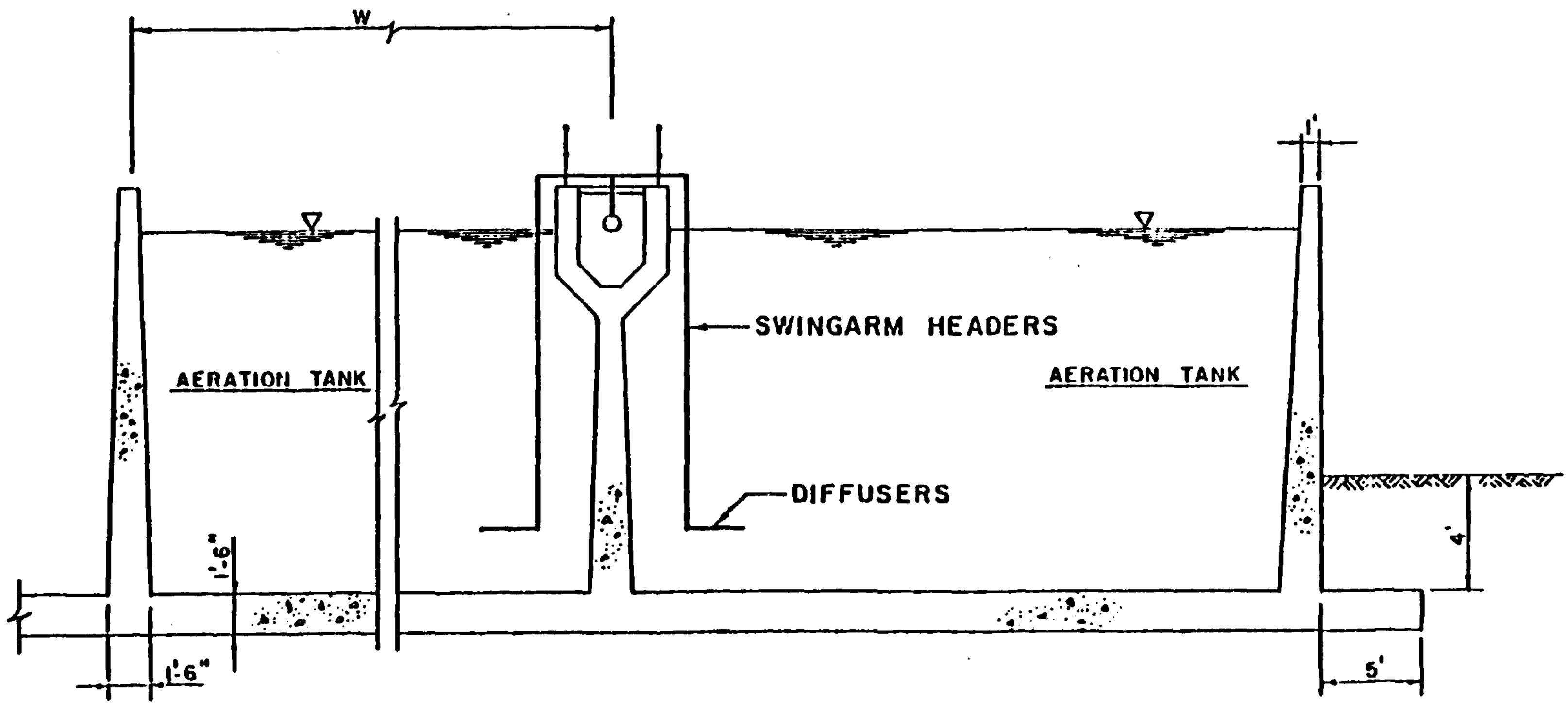


FIGURE 2.1-23 TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

2.1.15.5.10.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.15.5.10.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.15.5.10.4 When NT ≥ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

where

V_{cw} = R.C. wall quantity required, cu ft.

L = length of aeration tanks, ft.

2.1.15.5.11 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.1.15.5.11.1 If NT is less than 4:

$$LHR = [2(NT)\ (L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.1.15.5.11.2 If NT is greater than or equal to 4:

$$LHR = [2(NT)\ (L) + (4L) + 36.5(NT) + 2\ PGW + 13]\ NB$$

where

LHR = handrail length, ft.

2.1.15.5.12 Calculate operation manpower requirements.

2.1.15.5.12.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.1.15.5.12.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.1.15.5.13 Calculate maintenance manpower requirements.

2.1.15.5.13.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.1.15.5.13.2 If CFM_d 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.1.15.5.14 Energy requirement for operation. The electrical energy required for operation is related to the air requirement by the following equation:

$$KWH = (CFM_d)\ (241.6)$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.1.15.5.15 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.1.15.5.16 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.15.6 Quantities Calculation Output Data.

2.1.15.6.1 Number of aeration tanks, NT.

2.1.15.6.2 Number of diffusers per tank, ND_t.

2.1.15.6.3 Number of process batteries, NB.

2.1.15.6.4 Number of swing arm headers per tank, NSA_t.

2.1.15.6.5 Length of aeration tanks, L, ft.

2.1.15.6.6 Width of pipe gallery, PGW, ft.

2.1.15.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.1.15.6.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.1.15.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.1.15.6.10 Quantity of handrail, LHR, ft.

2.1.15.6.11 Operation manpower requirement, OMH, MH/yr.

2.1.15.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.1.15.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.1.15.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.1.15.6.15 Correction factor for minor construction costs, CF.

2.1.15.7 Unit Price Input Required.

2.1.15.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.15.7.2 Unit price input R.C. wall in-place, UPICW, \$/cu yd.

2.1.15.7.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.1.15.7.4 Unit price input for handrails in-place, UPIHR, \$/ft.

2.1.15.7.5 Cost per diffuser, COSTPD, \$, (optional).

2.1.15.7.6 Cost per swing arm header, COSTPH, \$, (optional).

2.1.15.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.15.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.1.15.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.1.15.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.1.15.8 Cost Calculations.

2.1.15.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.15.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.1.15.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price R.C. slab in-place, \$/cu yd.

2.1.15.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.1.15.8.5 Cost of diffusers.

2.1.15.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.1.15.8.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, \$.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.1.15.8.6 Cost of swing arm diffuser headers.

2.1.15.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5{,}000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5{,}000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.15.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, \$.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.1.15.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.1.15.8.8 Crane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.1.15.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.1.15.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.1.15.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.1.15.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.1.15.8.10 Other costs associated with the installed equipment. This category includes the cost for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.15.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.1.15.8.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.1.15.8.13 Operation and maintenance material costs.

$$OMCC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/yr.

OMMP = operation and maintenance material supply costs, as percent of total bare construction cost, percent.

2.1.15.9 Cost Calculations Output Data.

2.1.15.9.1 Total bare construction cost of diffused aeration activated sludge system, TBCC, dollars.

2.1.15.9.2 Operation and maintenance material and supply costs, OMMC, dollars.

2.1.16 HIGH RATE ACTIVATED SLUDGE (MECHANICAL AERATION).

2.1.16.1 Input Data.

2.1.16.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.16.1.2 Wastewater Strength.

2.1.16.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.16.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.16.1.2.3 Suspended solids, mg/l.

2.1.16.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.16.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.16.1.3 Other Characterization.

2.1.16.1.3.1 pH.

2.1.16.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.16.1.3.3 Nitrogen,[1] mg/l.

2.1.16.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.16.1.3.5 Oils and greases, mg/l.

2.1.16.1.3.6 Heavy metals, mg/l.

2.1.16.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.16.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.16.1.4 Effluent Quality Requirements.

2.1.16.1.4.1 BOD_5, mg/l.

2.1.16.1.4.2 SS, mg/l.

2.1.16.1.4.3 TKN, mg/l.

2.1.16.1.4.4 P, mg/l.

2.1.16.1.4.5 Total nitrogen ($TKN + NO_3 - N$), mg/l.

---

[1] The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.16.1.4.6 Settleable solids, mg/l.

2.1.16.2 <u>Design Parameters</u>.

2.1.16.2.1 Reaction rate constants and coefficients.

Eckenfelder	
k	0.0007-0.002 l/mg/hr.
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
f	0.40
f'	0.53

2.1.16.2.2 F/M = (1.5-5.0).

2.1.16.2.3 Volumetric loading = 100-250.

2.1.16.2.4 t = (1.5-3.0) hr.

2.1.16.2.5 t_s = (0.2-0.5) days.

2.1.16.2.6 MLSS = (200-1000) mg/l.

2.1.16.2.7 MLVSS = (140-700) mg/l.

2.1.16.2.8 Q_r/Q = (0.05-0.15).

2.1.16.2.9 lb O_2/lb BOD_r = (0.5-0.75).

2.1.16.2.10 lb solids/lb BOD_r = (0.65-0.85).

2.1.16.2.11 Efficiency = (50-60 percent).

2.1.16.3 <u>Process Design Calculations</u>.

2.1.16.3.1 Assume the following design parameters when unknown.

2.1.16.3.1.1 Fraction of BOD synthesized (a).

2.1.16.3.1.2 Fraction of BOD oxidized for energy (a').

2.1.16.3.1.3 Endogenous respiration rate (b and b').

2.1.16.3.1.4 Mixed liquor suspended solids (MLSS).

2.1.16.3.1.5 Mixed liquor volatile suspended solids (MLVSS).

2.1.16.3.1.6 Food-to-microorganism ratio (F/M).

2.1.16.3.1.7 Temperature correction coefficient (θ).

2.1.16.3.1.8 Nonbiodegradable fraction of VSS in influent (f).

2.1.16.3.1.9 Degradable fraction of the MLVSS (f').

2.1.16.3.2 Determine the size of the aeration tank by first determining the detention time.

$$t = \frac{24S_o}{(X_V)(F/M)}$$

where

t = detention time, hr.

S_o = influent BOD, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.16.3.3 Calculate the volume of aeration tank.

$$V = Q_{avg} \frac{t}{24}$$

where

V = volume of aeration tank, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.16.3.4 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'(S_r)}{t}$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34)$$

where

dO/dt = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed (S_o - S_e), mg/l.

t = detention time, hr.

O_2 = oxygen uptake rate, lb/day.

Q_{avg} = average flow rate, mgd.

2.1.16.3.5 Check the oxygen supplied against pounds of BOD removed (0.5-0.7).

$$\text{lb } O_2/\text{lb } BOD_r = \frac{O_2}{Q(S_r)(8.34)}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.16.3.6 Design Aeration System.

2.1.16.3.6.1 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing $\geq$ 0.1 hp/1000 gal.

2.1.16.3.6.1.1 Standard transfer efficiency, lb/hp-hr (0 dissolved oxygen, 20°C, and tap water) (3-5 lb/hp-hr).

2.1.16.3.6.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.16.3.6.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.16.3.6.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.16.3.6.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.16.3.6.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{[(C_s)_T(\beta)(p) - C_L]}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, ^oC, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, ^oC.

2.1.16.3.6.4 Calculate horsepower requirement.

$$hp = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)(V)} \times 1000$$

where

hp = horsepower required/1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of basin, gal.

2.1.16.3.7 Calculate sludge production.

$$\Delta X_V = [aS_rQ_{avg} + fQ(VSS) + Q(SS - VSS)]\ 8.34$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

f = nonbiodegradable fraction of VSS in influent.

Q = flow, mgd.

VSS = volatile suspended solids, mg/l.

SS = suspended solids in influent, mg/l.

2.1.16.3.8 Check ΔX_V against 0.65-0.85 lb solids/lb BOD_r.

$$\frac{\text{lb solids}}{(\text{lb } BOD_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)}$$

where

ΔX_V = sludge produced, lb/day.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.1.16.3.9 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q = flow, mgd.

X_a = MLSS

X_u = suspended solids concentration in returned sludge, mg/l.

2.1.16.3.10 Calculate solids retention time.

$$SRT = \frac{V(X_a)8.34}{\Delta X_a} (\%\ \text{volatile})$$

where

SRT = solids retention time, days.

V = volume of basin, million gal.

X_a = MLSS.

ΔX_a = sludge produced, lb/day.

2.1.16.3.11 Effluent Characteristics.

2.1.16.3.11.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.16.3.11.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.16.3.11.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.16.3.11.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.16.3.11.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.16.3.11.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.16.3.12 Determine nutrient requirements for nitrogen.

$$N = 0.123 \Delta X_V$$

and phosphorus

$$P = 0.026 \Delta X_V$$

where

ΔX_V = sludge produced, lb/day.

and check against BOD:N:P = 100:5:1.

2.1.16.4 Process Design Output Data.

2.1.16.4.1 Aeration Tank.

2.1.16.4.1.1 Reaction rate constant, 1/mg/hr.

2.1.16.4.1.2 Sludge produced per BOD removed.

2.1.16.4.1.3 Endogenous respiration rate (b, b').

2.1.16.4.1.4 O_2 utilized per BOD removed.

2.1.16.4.1.5 Influent nonbiodegradable VSS, mg/l.

2.1.16.4.1.6 Effluent degradable VSS, mg/l.

2.1.16.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.16.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.16.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.16.4.1.10 Aeration time, hr.

2.1.16.4.1.11 Volume of aeration tank, million gal.

2.1.16.4.1.12 Oxygen required, lb/day.

2.1.16.4.1.13 Sludge produced, lb/day.

2.1.16.4.1.14 Nitrogen requirement, lb/day.

2.1.16.4.1.15 Phosphorus requirement, lb/day.

2.1.16.4.1.16 Sludge recycle ratio, percent.

2.1.16.4.1.17 Solids retention time, days.

2.1.16.4.1.18 Volumetric loading, lb BOD/1000 ft^3.

2.1.16.4.2 Aeration System.

2.1.16.4.2.1 Standard transfer efficiency, lb O_2/hp-hr.

2.1.16.4.2.2 Operating transfer efficiency, lb O_2/hp-hr.

2.1.16.4.2.3 Horsepower required, hp.

2.1.16.5 Quantities Calculations.

2.1.16.5.1 The design values for activated sludge system would be:

$$V_d = V \cdot \frac{10^6}{7.48}$$

$$HP_d = (hp)\ (V)\ (133.7)$$

where

V = volume of aeration basin million gallons.

2.1.16.5.2 Selection of number of aeration tanks and mechanical aerators per tank. The following rule will be utilized in the selection of number of aeration tanks and mechanical aerators per tank.

Q_{avg} (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NT
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.16.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.16.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.16.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.16.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.16.5.3.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.16.5.4 Mechanical aeration equipment design.

2.1.16.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.1.16.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)\,(NT)\,(NA)}$$

If HPN > 150 HP and NT = 2 or 3, then repeat the calculation with NT = NT + 1.

If HPN > 150 HP and NT ≥ 4, then repeat the calculation with NT = NT + 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.1.16.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.1.16.5.5 Design of aeration tanks.

2.1.16.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\;(NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.16.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When $HPSN \leq 100$ HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

When $HPSN > 100$ HP

$$DW = 15 \text{ ft}$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.1.16.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.1.16.5.6 Aeration tank arrangements.

2.1.16.5.6.1 Figure 2.1-24 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.1.16.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\ \frac{(Q_{avg})}{NB}$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.16.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.1.16.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\ [(2\ W + 18.5)\ (W + 17) + (2\ W + 26.5)\ (W + 25)]$$

where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.1.16.5.7.2 When NT = 3, earthwork required would be:

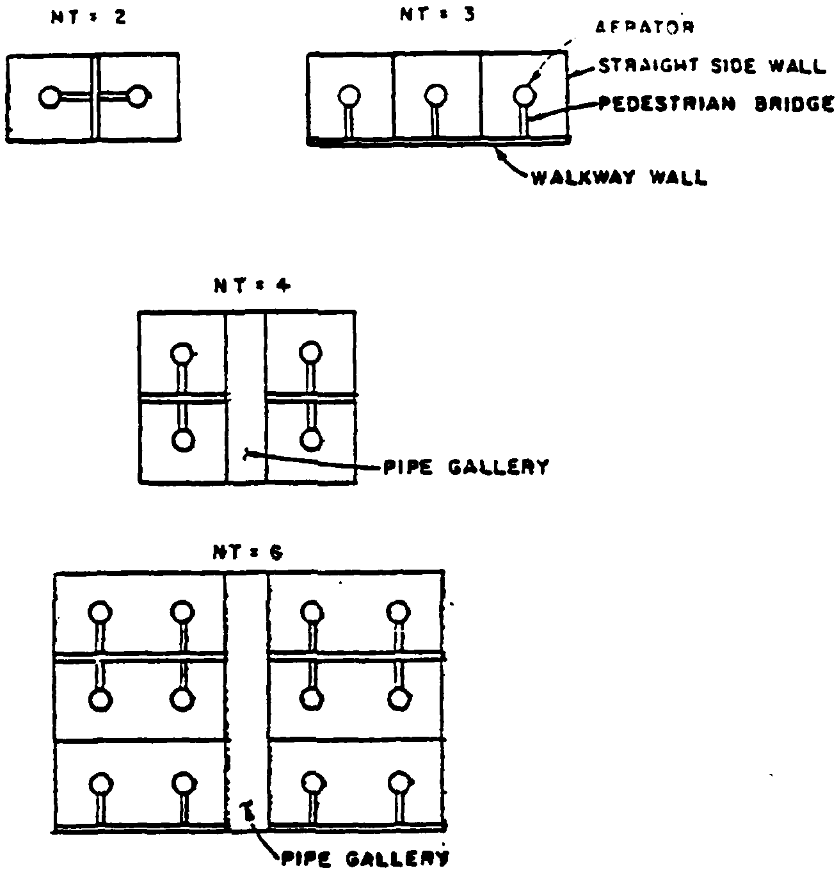
$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$

2.1.16.5.7.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.1-24 EXAMPLES OF TANK ARRANGEMENTS ACTIVATED SLUDGE PROCESSES

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB) \left[(L_s + 4) (W_s + 4) + (L_s + 12) (W_s + 12) \right]$$

where

V_{ew} = volume of earthwork, cu ft.

2.1.16.5.8 Reinforced concrete slab quantity.

2.1.16.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.1.16.5.8.2 For NT = 2,

$$V_{cs} = 1.5\ (2\ W + 14.5)\ (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.1.16.5.8.3 NT = 3,

$$V_{cs} = 1.5\ (3\ W + 16)\ (W + 13)$$

2.1.16.5.8.4 When NT $\geq$ 4,

$$V_{cs} = 1.5\ (L_s)\ (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.1.16.5.9 Reinforced Concrete Wall Quantity.

2.1.16.5.9.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-25. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.1.16.5.9.2 When NT = 2:

$$V_{cw} = W\,(1.25\ DW + 11) + (6\ W + 9)\,(1.25\ DW + 3.75)$$

2.1.16.5.9.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\,(3\ W + 6) + (1.25\ DW + 3.75)\,(7\ W + 6)$$

2.1.16.5.9.4 When NT $\geq$ 4:

$$V_{cw} = \frac{NT}{2}\,(L + 3)\,(1.25\ DW + 11) + [(0.5\ NT + 2)\,(L + 3) + 2\,(NT)\,(W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

2.1.16.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.1.16.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.1.16.5.10.2 When NT $\geq$ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5)\,(PGW)\left[\frac{(NT)\,(W)}{2} + 0.75\,(NT) + 1.5\right]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2\,(PGW)\,(NB)\,(DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

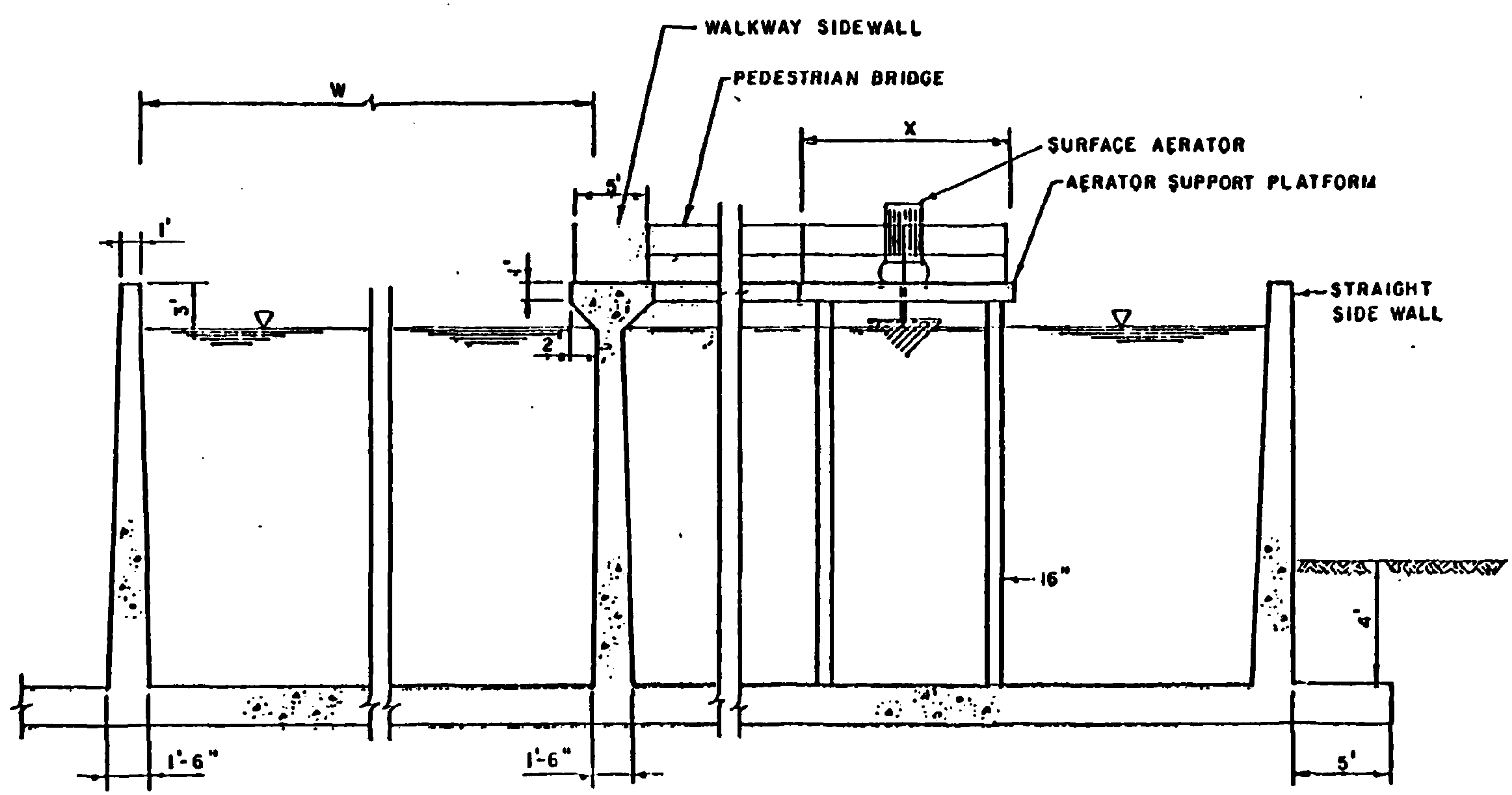


FIGURE 2.1-25 TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.1.16.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.1.16.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.1.16.5.11.2 Figure 2.1-26 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078\ (HPSN)$$

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.1.16.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6\ (DW + 2)]\ (NT)\ (NA)\ (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.1.16.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2\ (W - X)]\ (NB)\ (NT)\ (NA)$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.1.16.5.12 Summary of reinforced concrete structures.

2.1.16.5.12.1 Quantity of concrete slab.

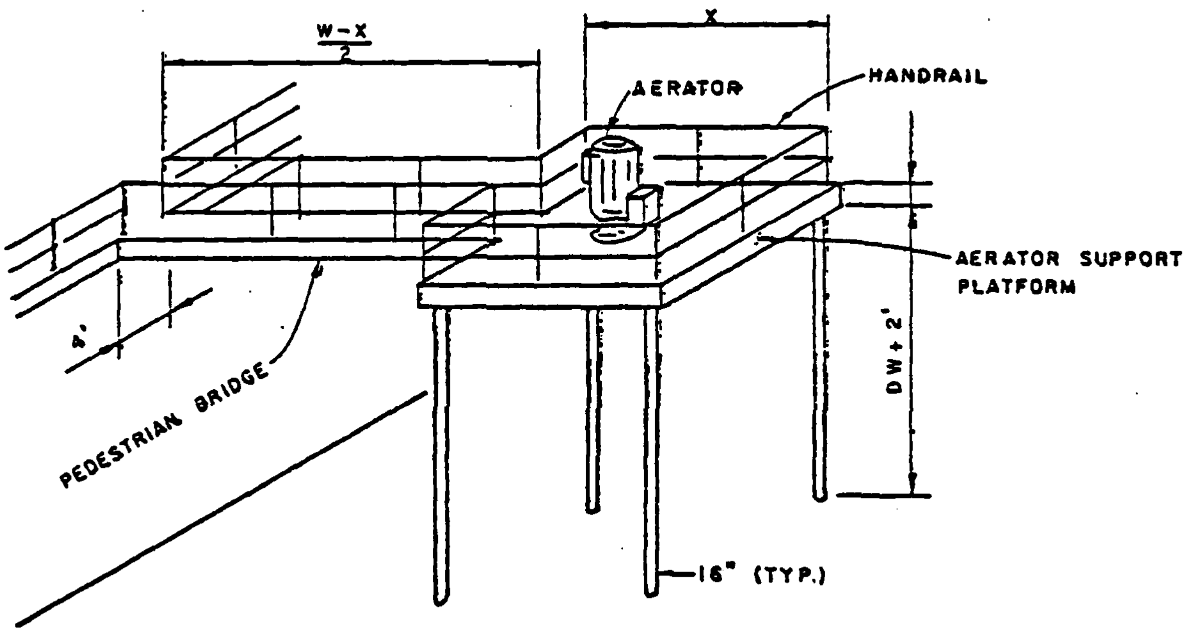


FIGURE 2.1-26 AERATOR SUPPORT PLATFORM

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.1.16.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.1.16.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.1.16.5.13.1 When NT = 2,

$$LHR = 4\ W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.1.16.5.13.2 When NT = 3,

$$LHR = 6\ W + 10 + 3 \cdot (3X + W - 4)$$

2.1.16.5.13.3 When NT $\geq$ 4,

$$\text{If } \frac{NT}{2} \text{ is an even number,}$$

$$LHR = PGW + (NT)(W) + [L + 3 - 4(NA)](NT) + (NA) \cdot (NT) \cdot (3X + W - 4) \cdot (NB)$$

$$\text{If } \frac{NT}{2} \text{ is an odd number,}$$

$$LHR = PGW + (NT)(W) + [L + 3 - 4(NA)](NT + 2) + (NA)(NT)(3X + w - 4) \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.1.16.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.1.16.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)(NT)(NA)(HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.1.16.5.14.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\,(TICA)^{0.3731}$$

When TICA ≥ 200 hp

$$OMH = 100\,(TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.1.16.5.14.3 The maintenance manpower requirement can be estimated as follows:

When TICA 100 hp

$$MMH = 106.3\ (TICA)^{0.4031}$$

When TICA 100 hp

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.1.16.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.16.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.1.16.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.16.6 Quantities Calculations Output Data.

2.1.16.6.1 Number of aeration tanks, NT.

2.1.16.6.2 Number of aerators per tank, NA.

2.1.16.6.3 Number of process batteries, NB.

2.1.16.6.4 Capacity of each individual aerator, HPSN, hp.

2.1.16.6.5 Depth of aeration tanks, DW, ft.

2.1.16.6.6 Length of aeration tanks, L, ft.

2.1.16.6.7 Width of aeration tanks, W, ft.

2.1.16.6.8 Width of pipe gallery, PGW, ft.

2.1.16.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.1.16.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.1.16.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.1.16.6.12 Quantity of handrail, LHR, ft.

2.1.16.6.13 Operation manpower requirement, OMH, MH/yr.

2.1.16.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.1.16.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.1.16.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.1.16.6.17 Correction factor for minor capital cost items, CF.

2.1.16.7 Unit Price Input Required.

2.1.16.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.1.16.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.1.16.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.1.16.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, $, optional.

2.1.16.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.1.16.7.6 Equipment installation labor rate, $/MH.

2.1.16.7.7 Crane rental rate, UPICR, $/hr.

2.1.16.7.8 Unit price of handrail, UPIHR, $/L.F.

2.1.16.8 Cost Calculations.

2.1.16.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, $.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, $/cu yd.

2.1.16.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, $.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/ cu yd.

2.1.16.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.1.16.8.4 Cost of installed aeration equipment.

2.1.16.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.1.16.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.1.16.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16,300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16,300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.16.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When HPSN $\leq$ 60 hp

$$IMH = 39 + 0.55 \, (HPSN)$$

When HPSN > 60 hp

$$IMH = 61.3 + 0.18 \, (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.16.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.16.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.16.8.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, dollars.

LABRI = labor rate, dollars/man-hour.

UPICR = crane rental rate, dollars/hr.

2.1.16.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, $ per lineal foot. A value of $25.20 per foot for the first quarter of 1977 is suggested.

2.1.16.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.1.16.8.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, dollars.

CF = correction factor for minor cost items, from second-order design output.

2.1.16.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, $/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.1.16.9 Cost Calculations Output Data.

2.1.16.9.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, dollars.

2.1.16.9.2 Operation and maintenance supply and material costs, OMMC, dollars.

2.1.17 PLUG FLOW ACTIVATED SLUDGE (DIFFUSED AERATION).

2.1.17.1 Input Data.

2.1.17.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.17.1.2 Wastewater Strength.

2.1.17.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.17.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.17.1.2.3 Suspended solids, mg/l.

2.1.17.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.17.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.17.1.3 Other Characterization.

2.1.17.1.3.1 pH.

2.1.17.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.17.1.3.3 Nitrogen,[1] mg/l.

2.1.17.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.17.1.3.5 Oils and greases, mg/l.

2.1.17.1.3.6 Heavy metals, mg/l.

2.1.17.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.17.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.17.1.4 Effluent Quality Requirements.

2.1.17.1.4.1 BOD_5, mg/l.

2.1.17.1.4.2 SS, mg/l.

2.1.17.1.4.3 TKN, mg/l.

2.1.17.1.4.4 P, mg/l.

---

[1] The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.17.1.4.5 Total nitrogen (TKN + NO_3 - N), mg/l.

2.1.17.1.4.6 Settleable solids, mg/l/hr.

2.1.17.2 Design Parameters.

2.1.17.2.1 Reaction rate constants and coefficients (average values to be used in absence of specific data).

Constants (Eckenfelder)	Range
k	0.0007 to 0.002 l/mg/hr
a	0.73
a'	0.52
b	0.075/day
f	0.4
b'	0.15/day
f'	0.53

2.1.17.2.2 Organic loading, F/M ratio.

$$F/M = (0.2\text{-}0.4)$$

2.1.17.2.3 Volumetric loading (lb BOD_5/1000 ft^3/day).

Range 20-40

2.1.17.2.4 Hydraulic detention time, t.

$$t = (4\text{-}8) \text{ hr}$$

2.1.17.2.5 Solids retention time, t_s.

$$t_s = (5\text{-}15) \text{ days}$$

2.1.17.2.6 Mixed liquor suspended solids concentration, MLSS.

MLSS = (1500-3000) mg/l.

2.1.17.2.7 Mixed liquor volatile suspended solids, MLVSS.

MLVSS = 0.7 MLSS

= (1050-2100) mg/l

2.1.17.2.8 Recycle ratio, Qr/Q.

Qr/Q = (0.25-0.5)

2.1.17.2.9 Oxygen requirements, lb O_2/lb BOD_r.

$$\text{lb } O_2/\text{lb } BOD_r \geq 1.25$$

2.1.17.2.10 Sludge production, lb solids/lb BOD_r.

$$\text{lb solids/lb } BOD_r = 0.5\text{-}0.7$$

2.1.17.2.11 Temperature coefficient, Θ.

$$\Theta = 1.0\text{-}1.03$$

2.1.17.2.12 BOD removal efficiency (80-90 percent).

2.1.17.2.13 Return sludge concentration.

2.1.17.3 Process Design Calculations.

2.1.17.3.1 Assume the following design parameters when unknown.

2.1.17.3.1.1 BOD removal rate constant (k).

2.1.17.3.1.2 Fraction of BOD synthesized (a).

2.1.17.3.1.3 Fraction of BOD oxidized for energy (a').

2.1.17.3.1.4 Endogenous respiration rate (b and b').

2.1.17.3.1.5 Mixed liquor suspended solids (MLSS).

2.1.17.3.1.6 Mixed liquor volatile suspended solids (MLVSS).

2.1.17.3.1.7 Food-to-microorganism ratio (F/M).

2.1.17.3.1.8 Nonbiodegradable fraction of VSS in influent (f).

2.1.17.3.1.9 Degradable fraction of the MLVSS (f').

2.1.17.3.1.10 Temperature correction coefficient (Θ).

2.1.17.3.2 Adjust k for temperature.

$$K_T = K_{20}\Theta^{(T-200)}$$

where

K_T = rate constant at desired temperature, °C.

K_{20} = rate constant at 20°C.

θ = temperature correction coefficient.

T = temperature, °C.

2.1.17.3.3 Determine the size of the aeration tank by first determining the detention time.

$$t = \frac{24S_o}{(X_V)(F/M)}$$

where

t = hydraulic detention time, hr.

S_o = influent BOD, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.17.3.4 Check the detention time for treatability.

$$S_e = S_o e^{-kX_V t}$$

where

S_e = BOD_5 (soluble) in effluent, mg/l.

S_o = BOD_5 in influent, mg/l.

k = BOD removal rate constant, l/mg/hr.

X_V = MLVSS, mg/l.

t = detention time, hr.

Solve for t and compare with t from above and select the larger.

2.1.17.3.5 Calculate the volume of aeration tank.

$$V = Q_{avg} \times \frac{t}{24}$$

where

V = volume of tanks, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.17.3.6 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'S_r}{t} + b'X_V$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34) + b'(X_V)(V)(8.34)$$

where

dO/dt = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed $(S_o - S_e)$, mg/l.

t = detention time, hr.

b' = endogenous respiration rate/hr.

X_V = MLVSS, mg/l.

O_2 = oxygen required, lb/day.

Q_{avg} = average daily flow, mgd.

V = volume of aeration tank, million gal.

and check the oxygen supplied per pound of BOD removed ≥ 1.25.

$$\text{lb } O_2/\text{lb BOD}_r = \frac{O_2}{Q(S_r) \times 8.34}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.17.3.7 Design aeration system.

2.1.17.3.7.1 Assume the following design parameters.

2.1.17.3.7.1.1 Standard transfer efficiency (from manufacturer), 5-8 percent (coarse bubble diffuser).

2.1.17.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx 0.9 = \alpha$.

2.1.17.3.7.1.3 0_2 saturation in waste/0_2 saturation in water $\approx$ 0.9 $= \beta$

2.1.17.3.7.1.4 Correction factor for pressure $\approx 1.0 = p$.

2.1.17.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) 0_2 saturation.

2.1.17.3.7.3 Adjuste standard transfer efficiency to operating conditions.

$$OTE = STE \frac{[(C_s)_T (\beta)(p) - C_L]}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent.

$(C_s)_T$ = 0_2 saturation at selected summer temperature T, °C, mg/l.

β = 0_2 saturation in waste/0_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = 0_2 transfer in waste/0_2 transfer in water $\approx$0.9.

T = temperature, °C.

2.1.17.3.7.4 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{0_2 (10^5)(7.48)}{(OTE\ \%)\ 0.0176 \frac{lb\ 0_2}{ft^3 air}\ 1440 \frac{min}{day}\ V}$$

where

R_a = required air flow, cfm/1000 ft^3.

V = volume of the basin, gal.

2.1.17.3.8 Calculate sludge production.

$$\Delta X_V = [aS_rQ_{avg} - bX_V(V) + (Q)(VSS)(f) + Q(SS - VSS)]8.34$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

b = endogenous respiration rate/day.

V = volume of aeration tank, million gal.

Q = flow, mgd.

VSS = volatile suspended solids in effluent, mg/l.

f = nonbiodegradable fraction of VSS in influent.

SS = suspended solids in effluent, mg/l.

Check ΔX_V solids produced against pounds of BOD removed (0.5-0.7).

$$\frac{(\text{lb solids})}{(\text{lb } BOD_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)(\%\text{ volatile})}$$

2.1.17.3.9 Determine nutrient requirements (lb/day).

for nitrogen

$$N = 0.123\,\Delta X_V$$

and phosphorus

$$P = 0.026\,\Delta X_V$$

and check against BOD:N:P = 100:5:1.

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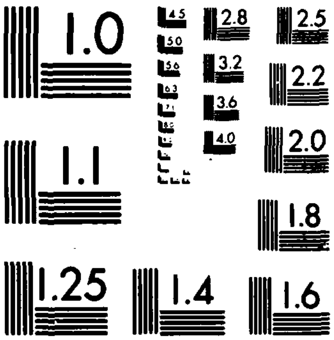

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2.1.17.3.10 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q_{avg}} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q_{avg} = average flow, mgd.

X_a = MLSS, mg/l.

X_u = SS concentration of returned sludge, mg/l.

2.1.17.3.11 Calculate solids retention time.

$$SRT = \frac{(V)(X_a)(8.34)}{\Delta X_a}$$

where

SRT = solids retention time, days.

V = volume of aeration tank, million gal.

X_a = MLSS, mg/l.

$$\Delta X_a = \frac{\Delta X_V}{\%\ \text{volatile}}$$

2.1.17.3.12 Effluent Characteristics.

2.1.17.3.12.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{eff}f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.17.3.12.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.17.3.12.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.17.3.12.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.17.3.12.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.17.3.12.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.17.4 Process Design Output Data.

2.1.17.4.1 Aeration Tank.

2.1.17.4.1.1 Reaction rate constant, l/mg/hr.

2.1.17.4.1.2 Sludge produced per BOD removed.

2.1.17.4.1.3 Endogenous respiration rate (b, b').

2.1.17.4.1.4 O_2 utilized per BOD removed.

2.1.17.4.1.5 Influent nonbiodegradable volatile suspended solids (VSS)(f).

2.1.17.4.1.6 Effluent degradable volatile suspended solids (f').

2.1.17.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.17.4.1.8 Mixed liquor suspended solids, mg/l (MLSS).

2.1.17.4.1.9 Mixed liquor volatile suspended solids, mg/l (MLVSS).

2.1.17.4.1.10 Aeration time, hr.

2.1.17.4.1.11 Volume of aeration tank, million gal.

2.1.17.4.1.12 Oxygen required, lb/day.

2.1.17.4.1.13 Sludge produced, lb/day.

2.1.17.4.1.14 Nitrogen requirement, lb/day.

2.1.17.4.1.15 Phosphorus requirement, lb/day.

2.1.17.4.1.16 Sludge recycle ratio, percent.

2.1.17.4.1.17 Solids retention time, days.

2.1.17.4.2 Aeration System.

2.1.17.4.2.1 Standard transfer efficiency, percent.

2.1.17.4.2.2 Operating transfer efficiency, percent.

2.1.17.4.2.3 Required air flow, cfm/1000 ft^3.

2.1.17.5 <u>Quantities Calculations</u>.

2.1.17.5.1 Design values for activated sludge system.

$$V_d = V \frac{10^6}{7.48}$$

$$CFM_d = (CFM)\ (V)\ (133.7)$$

where

V = volume of aeration tanks, million gallons.

2.1.17.5.2 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.17.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.17.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.17.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.17.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.17.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.17.5.4 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

2.1.17.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.1.17.5.6 Design of aeration tanks.

2.1.17.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.17.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.1.17.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.1.17.5.7 Aeration tank arrangements.

2.1.17.5.7.1 Figure 2.1-27 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = pipe gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.17.5.8 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.1.17.5.8.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\left[\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}\right]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.1.17.5.8.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\left[\frac{(15.75(NT)+15.5)\ (2L+PGW+20) + (15.75(NT)+2.5)\ (2L+PGW+28)}{2}\right]$$

2.1.17.5.9 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.1.17.5.9.1 For NT less than 4:

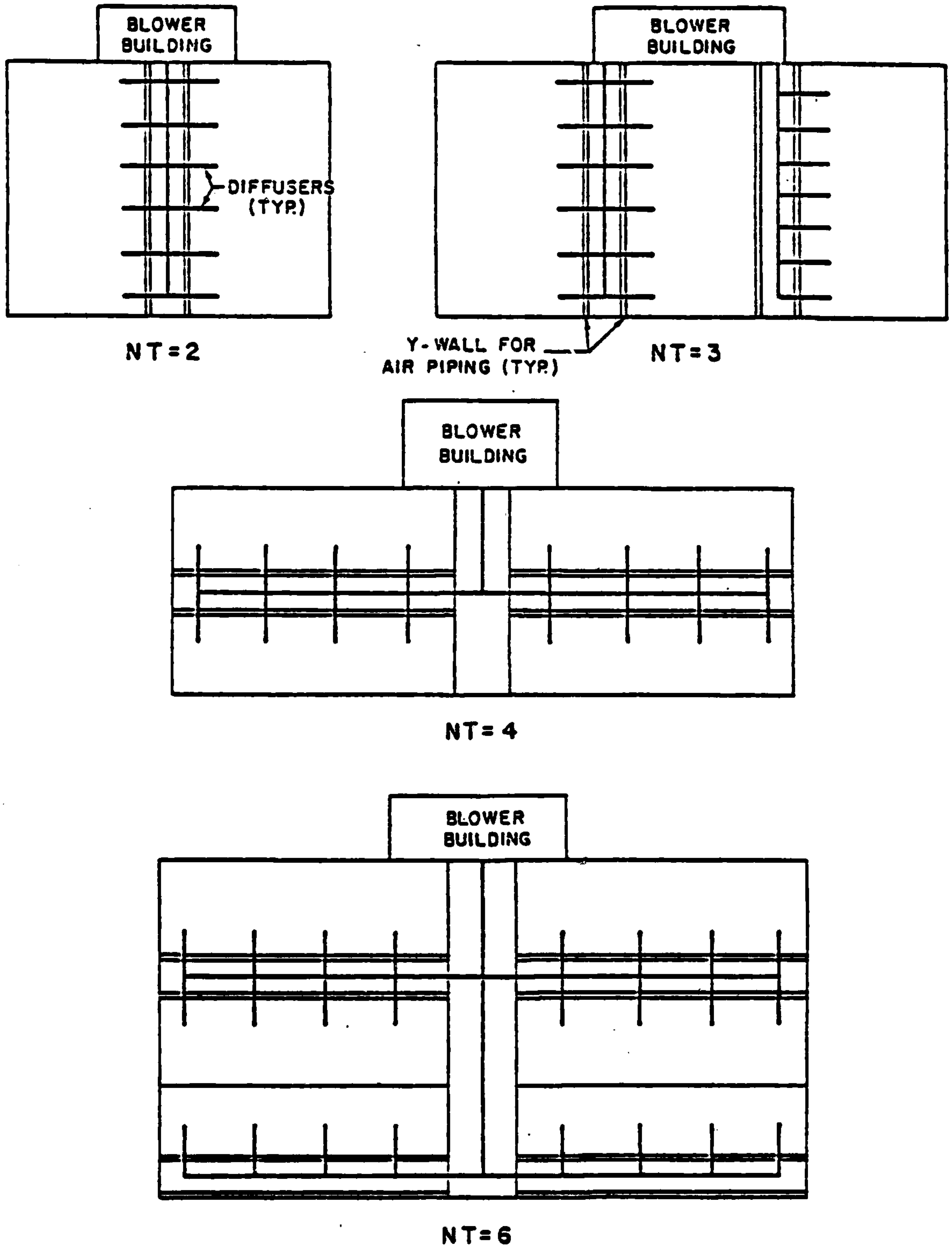


FIGURE 2.1-27 AERATION TANK ARRANGEMENT

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

2.1.17.5.9.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)\ (2L + PGW + 200)]$$

2.1.17.5.10 Reinforced concrete wall quantities.

2.1.17.5.10.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-28. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.1.17.5.10.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.17.5.10.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.17.5.10.4 When NT ≥ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

where

V_{cw} = R.C. wall quantity required, cu ft.

L = length of aeration tanks, ft.

2.1.17.5.11 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.1.17.5.11.1 If NT is less than 4:

$$LHR = [2(NT)\ (L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.1.17.5.11.2 If NT is greater than or equal to 4:

$$LHR = [2(NT)\ (L) + (4L) + 36.5(NT) + 2\ PGW + 13]\ NB$$

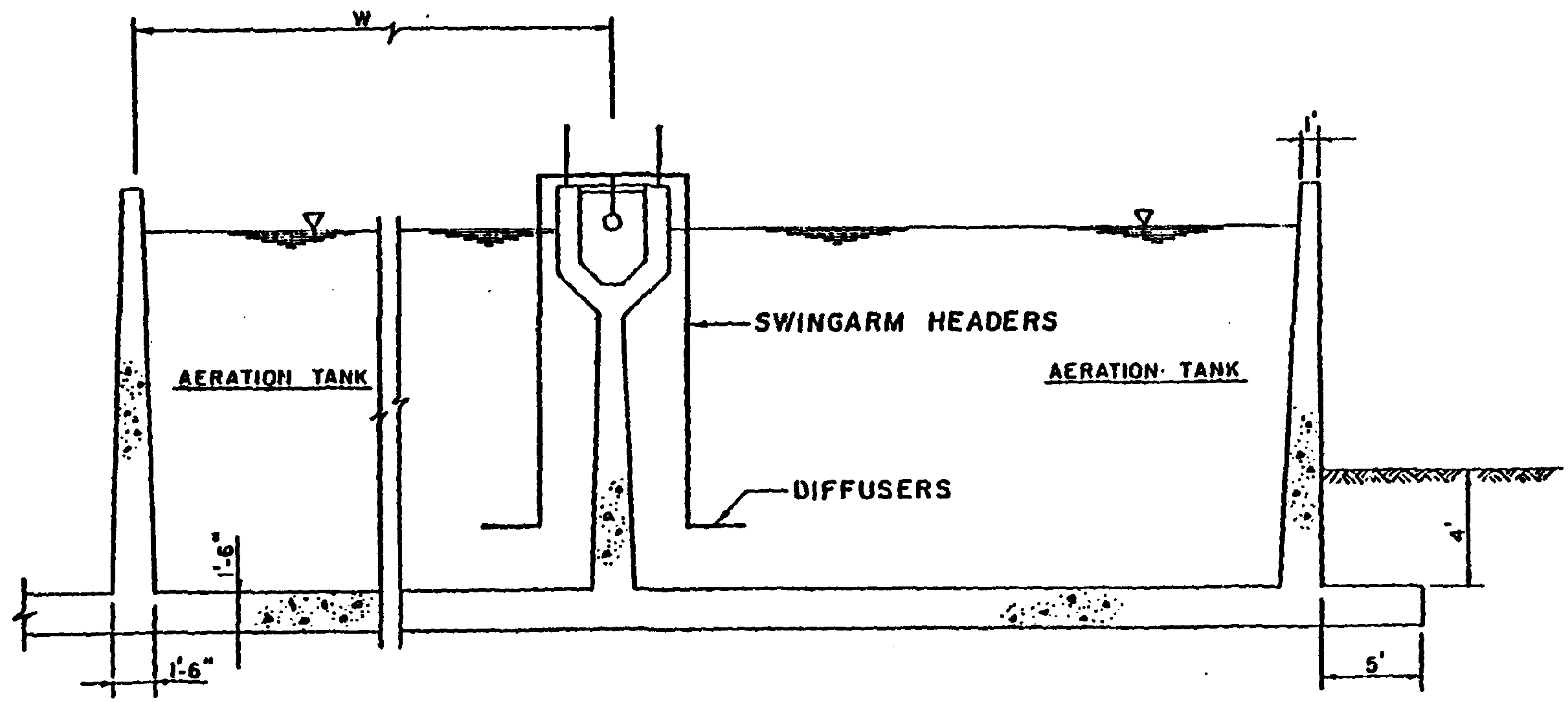


FIGURE 2.1-28 TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

where

LHR = handrail length, ft.

2.1.17.5.12 Calculate operation manpower requirements.

2.1.17.5.12.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.1.17.5.12.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.1.17.5.13 Calculate maintenance manpower requirements.

2.1.17.5.13.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.1.17.5.13.2 If CFM_d > 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.1.17.5.14 Energy requirement/for operation. The electrical energy required for operation is related to the air requirement by the following equation:

$$KWH = (CFM_d)\ (241.6)$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.1.17.5.15 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.1.17.5.16 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.17.6 Quantities Calculation Output Data.

2.1.17.6.1 Number of aeration tanks, NT.

2.1.17.6.2 Number of diffusers per tank, ND_t.

2.1.17.6.3 Number of process batteries, NB.

2.1.17.6.4 Number of swing arm headers per tank, NSA_t.

2.1.17.6.5 Length of aeration tanks, L, ft.

2.1.17.6.6 Width of pipe gallery, PGW, ft.

2.1.17.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.1.17.6.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.1.17.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.1.17.6.10 Quantity of handrail, LHR, ft.

2.1.17.6.11 Operation manpower requirement, OMH, MH/yr.

2.1.17.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.1.17.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.1.17.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.1.17.6.15 Correction factor for minor construction costs, CF.

2.1.17.7 Unit Price Input Required.

2.1.17.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.17.7.2 Unit price input R.C. wall in-place, UPICW, \$/cu yd.

2.1.17.7.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.1.17.7.4 Unit price input for handrails in-place, UPIHR, \$/ft.

2.1.17.7.5 Cost per diffuser, COSTPD, \$, (optional).

2.1.17.7.6 Cost per swing arm header, COSTPH, \$, (optional).

2.1.17.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.17.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.1.17.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.1.17.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.1.17.8 Cost Calculations.

2.1.17.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.17.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.1.17.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price R.C. slab in-place, \$/cu yd.

2.1.17.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.1.17.8.5 Cost of diffusers.

2.1.17.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.1.17.8.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, \$.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.1.17.8.6 Cost of swing arm diffuser headers.

2.1.17.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5,000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5,000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.17.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, \$.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.1.17.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.1.17.8.8 Crane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.1.17.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.1.17.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.1.17.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.1.17.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.1.17.8.10 Other costs associated with the installed equipment. This category includes the cost for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.17.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.1.17.8.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.1.17.8.13 Operation and maintenance material costs.

$$OMCC = TBCC \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/yr.

OMMP = operation and maintenance material supply costs, as percent of total bare construction cost, percent.

2.1.17.9 Cost Calculations Output Data.

2.1.17.9.1 Total bare construction cost of diffused aeration activated sludge system, TBCC, dollars.

2.1.17.9.2 Operation and maintenance material and supply costs, OMMC, dollars.

2.1.18 PLUG FLOW ACTIVATED SLUDGE (MECHANICAL AERATION).

2.1.18.1 <u>Input Data</u>.

2.1.18.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.18.1.2 Wastewater Strength.

2.1.18.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.18.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.18.1.2.3 Suspended solids, mg/l.

2.1.18.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.18.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.18.1.3 Other Characterization.

2.1.18.1.3.1 pH.

2.1.18.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.18.1.3.3 Nitrogen,[1] mg/l.

2.1.18.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.18.1.3.5 Oils and greases, mg/l.

2.1.18.1.3.6 Heavy metals, mg/l.

2.1.18.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.18.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.18.1.4 Effluent Quality Requirements.

2.1.18.1.4.1 BOD_5, mg/l.

2.1.18.1.4.2 SS, mg/l.

2.1.18.1.4.3 TKN, mg/l.

2.1.18.1.4.4 P, mg/l.

---

[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.18.1.4.5 Total nitrogen (TKN + NO_3 - N), mg/l.

2.1.18.1.4.6 Settleable solids, mg/l/hr.

2.1.18.2 Design Parameters.

2.1.18.2.1 Reaction rate constants and coefficients (average values to be used in absence of specific data).

Constants (Eckenfelder)	Range
k	0.0007 to 0.002 l/mg/hr
a	0.73
a'	0.52
b	0.075/day
f	0.4
b'	0.15/day
f'	0.53

2.1.18.2.2 Organic loading, F/M ratio.

F/M = (0.2-0.4)

2.1.18.2.3 Volumetric loading (lb BOD_5/1000 ft^3/day).

Range 20-40

2.1.18.2.4 Hydraulic detention time, t.

t = (4-8) hr

2.1.18.2.5 Solids retention time, t_s.

t_s = (5-15) days

2.1.18.2.6 Mixed liquor suspended solids concentration, MLSS.

MLSS = (1500-3000) mg/l.

2.1.18.2.7 Mixed liquor volatile suspended solids, MLVSS.

MLVSS = 0.7 MLSS

= (1050-2100) mg/l

2.1.18.2.8 Recycle ratio, Qr/Q.

Qr/Q = (0.25-0.5)

2.1.18.2.9 Oxygen requirements, lb O_2/lb BOD_r.

$$\text{lb } O_2/\text{lb } BOD_r \geq 1.25$$

2.1.18.2.10 Sludge production, lb solids/lb BOD_r.

$$\text{lb solids/lb } BOD_r = 0.5\text{-}0.7$$

2.1.18.2.11 Temperature coefficient, θ.

$$\theta = 1.0\text{-}1.03$$

2.1.18.2.12 BOD removal efficiency (80-90 percent).

2.1.18.2.13 Return sludge concentration.

2.1.18.3 <u>Process Design Calculations</u>.

2.1.18.3.1 Assume the following design parameters when unknown.

2.1.18.3.1.1 BOD removal rate constant (k).

2.1.18.3.1.2 Fraction of BOD synthesized (a).

2.1.18.3.1.3 Fraction of BOD oxidized for energy (a').

2.1.18.3.1.4 Endogenous respiration rate (b and b').

2.1.18.3.1.5 Mixed liquor suspended solids (MLSS).

2.1.18.3.1.6 Mixed liquor volatile suspended solids (MLVSS).

2.1.18.3.1.7 Food-to-microorganism ratio (F/M).

2.1.18.3.1.8 Nonbiodegradable fraction of VSS in influent (f).

2.1.18.3.1.9 Degradable fraction of the MLVSS (f').

2.1.18.3.1.10 Temperature correction coefficient (θ).

2.1.18.3.2 Adjust k for temperature.

$$K_T = K_{20}\theta^{(T-200)}$$

where

K_T = rate constant at desired temperature, ^{o}C.

K_{20} = rate constant at $20^{o}C$.

θ = temperature correction coefficient.

T = temperature, ^oC.

2.1.18.3.3 Determine the size of the aeration tank by first determining the detention time.

$$t = \frac{24S_o}{(X_V)(F/M)}$$

where

t = hydraulic detention time, hr.

S_o = influent BOD, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.18.3.4 Check the detention time for treatability.

$$S_e = S_o e^{-kX_V t}$$

where

S_e = BOD_5 (soluble) in effluent, mg/l.

S_o = BOD_5 in influent, mg/l.

k = BOD removal rate constant, l/mg/hr.

X_V = MLVSS, mg/l.

t = detention time, hr.

Solve for t and compare with t from above and select the larger.

2.1.18.3.5 Calculate the volume of aeration tank.

$$V = Q_{avg} \times \frac{t}{24}$$

where

V = volume of tanks, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.18.3.6 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'S_r}{t} + b'X_V$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34) + b'(X_V)(V)(8.34)$$

where

dO/dt = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed (S_o - S_e), mg/l.

t = detention time, hr.

b' = endogenous respiration rate/hr.

X_V = MLVSS, mg/l.

O_2 = oxygen required, lb/day.

Q_{avg} = average daily flow, mgd.

V = volume of aeration tank, million gal.

and check the oxygen supplied per pound of BOD removed $\geq$ 1.25.

$$\text{lb } O_2/\text{lb } BOD_r = \frac{O_2}{Q(S_r) \times 8.34}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.18.3.7 Design Aeration System.

2.1.18.3.7.1 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing 0.1 hp/1000 gal.

2.1.18.3.7.1.1 Standard transfer efficiency, lb/hp-hr (0 dissolved oxygen, 20°C, and tap water) (3-5 lb/hp-hr).

2.1.18.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.18.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.18.3.7.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.18.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.18.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{[(C_s)_T(\beta)(p) - C_L]}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin $\approx$ 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, °C.

2.1.18.3.7.4 Calculate horsepower requirement.

$$hp = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)(V)} \times 1000$$

where

hp = horsepower required/1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of basin, gal.

2.1.18.3.8 Calculate sludge production.

$$\Delta X_V = [aS_r Q_{avg} - bX_V(V) + (Q)(VSS)(f) + Q(SS - VSS)]\ 8.34$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

b = endogenous respiration rate/day.

V = volume of aeration tank, million gal.

Q = flow, mgd.

VSS = volatile suspended solids in effluent, mg/l.

f = nonbiodegradable fraction of VSS in influent.

SS = suspended solids in effluent, mg/l.

Check ΔX_V solids produced against pounds of BOD removed (0.5-0.7).

$$\frac{(\text{lb solids})}{(\text{lb BOD}_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)(\%\ \text{volatile})}$$

2.1.18.3.9 Determine nutrient requirements (lb/day).

for nitorgen

$$N = 0.123\,\Delta X_V$$

and phosphorus

$$P = 0.026\,\Delta X_V$$

and check against BOD:N:P = 100:5:1.

2.1.18.3.10 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q_{avg}} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q_{avg} = average flow, mgd.

X_a = MLSS, mg/l.

X_u = SS concentration of returned sludge, mg/l.

2.1.18.3.11 Calculate solids retention time.

$$SRT \quad \frac{(V)\ (X_a)\ (8.34)}{\Delta X_a}$$

where

SRT = solids retention time, days.

V = volume of aeration tank, million gal.

X_a = MLSS, mg/l.

$\Delta X_a = \dfrac{\Delta X_V}{\%\ \text{volatile}}$

2.1.18.3.12 Effluent Characteristics.

2.1.18.3.12.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.18.3.12.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.18.3.12.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.18.3.12.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.18.3.12.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.18.3.12.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.18.4 Process Design Output Data.

2.1.18.4.1 Aeration Tank.

2.1.18.4.1.1 Reaction rate constant, 1/mg/hr.

2.1.18.4.1.2 Sludge produced per BOD removed.

2.1.18.4.1.3 Endogenous respiration rate (b, b').

2.1.18.4.1.4 O_2 utilized per BOD removed.

2.1.18.4.1.5 Influent nonbiodegradable volatile suspended solids (VSS)(f).

2.1.18.4.1.6 Effluent degradable volatile suspended solids (f').

2.1.18.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.18.4.1.8 Mixed liquor suspended solids, mg/l (MLSS).

2.1.18.4.1.9 Mixed liquor volatile suspended solids, mg/l (MLVSS).

2.1.18.4.1.10 Aeration time, hr.

2.1.18.4.1.11 Volume of aeration tank, million gal.

2.1.18.4.1.12 Oxygen required, lb/day.

2.1.18.4.1.13 Sludge produced, lb/day.

2.1.18.4.1.14 Nitrogen requirement, lb/day.

2.1.18.4.1.15 Phosphorus requirement, lb/day.

2.1.18.4.1.16 Sludge recycle ratio, percent.

2.1.18.4.1.17 Solids retention time, days.

2.1.18.4.2 - Aeration System.

2.1.18.4.2.1 Standard transfer efficiency, lb O_2/hp-hr.

2.1.18.4.2.2 Operating transfer efficiency, lb O_2/hp-hr.

2.1.18.4.2.3 Horsepower required, hp.

2.1.18.5 Quantities Calculations.

2.1.18.5.1 The design values for activated sludge system would be:

$$V_d = V \cdot \frac{10^6}{7.48}$$

$$HP_d \ (hp)\ (V)\ (133.7)$$

where

V = volume of aeration basin million gallons.

2.1.18.5.2 Selection of number of aeration tanks and mechanical aerators per tank. The following rule will be utilized in the selection of number of aeration tanks and mechanical aerators per tank.

Q_{avg} (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NT
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.18.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.18.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.18.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.18.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.18.5.3.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.18.5.4 Mechanical aeration equipment design.

2.1.18.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.1.18.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)(NT)(NA)}$$

If HPN > 150 HP and NT = 2 or 3, then repeat the calculation with NT = NT + 1.

If HPN > 150 HP and NT ≥ 4, then repeat the calculation with NT = NT + 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.1.18.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.1.18.5.5 Design of aeration tanks.

2.1.18.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.18.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When $HPSN < 100$ HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

When $HPSN \geq 100$ HP

$$DW = 15 \text{ ft}$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.1.18.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.1.18.5.6 Aeration tank arrangements.

2.1.18.5.6.1 Figure 2.1-29 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.1.18.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\ \left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.18.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.1.18.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\ [(2\ W + 18.5)\ (W + 17) + (2\ W + 26.5)\ (W + 25)]$$

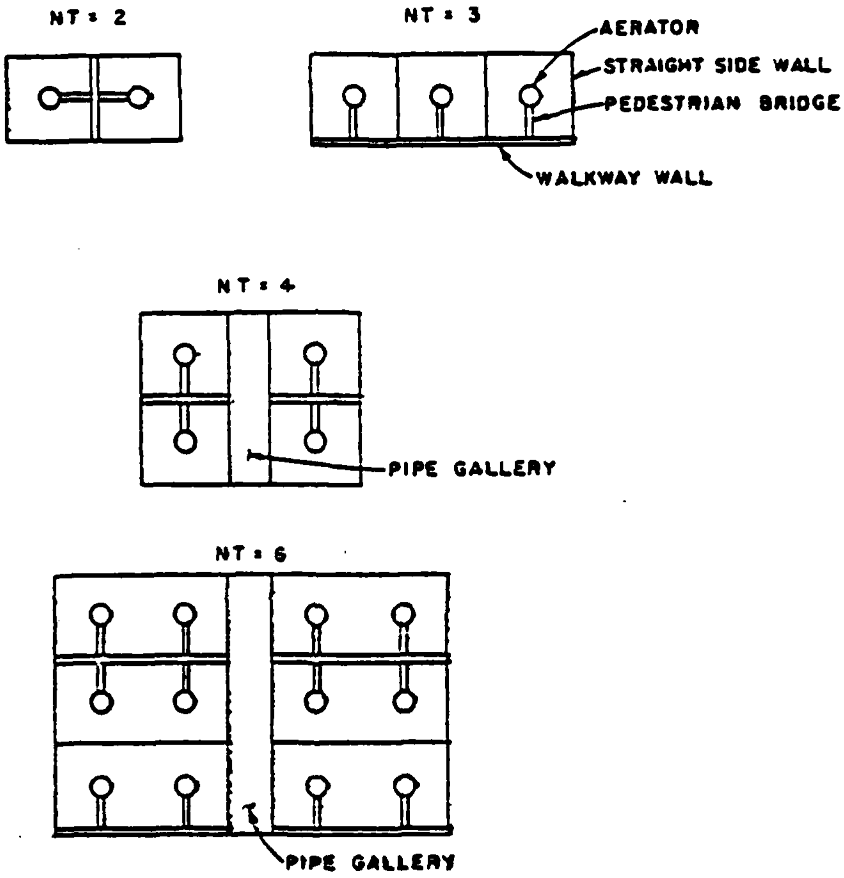
where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.1.18.5.7.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.1-29 EXAMPLES OF TANK ARRANGEMENTS ACTIVATED SLUDGE PROCESSES

2.1.18.5.7.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB)\ [(L_s + 4)\ (W_s + 4) + (L_s + 12)\ (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.1.18.5.8 Reinforced concrete slab quantity.

2.1.18.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.1.18.5.8.2 For NT = 2,

$$V_{cs} = 1.5\ (2\ W + 14.5)\ (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.1.18.5.8.3 NT = 3,

$$V_{cs} = 1.5\ (3\ W + 16)\ (W + 13)$$

2.1.18.5.8.4 When NT $\geq$ 4,

$$V_{cs} = 1.5\ (L_s)\ (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.1.18.5.9 Reinforced Concrete Wall Quantity.

2.1.18.5.9.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-30. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.1.18.5.9.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.18.5.9.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.18.5.9.4 When NT ≥ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

2.1.18.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.1.18.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.1.18.5.10.2 When NT ≥ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5)\ (PGW)\ [\frac{(NT)\ (W)}{2} + 0.75\ (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

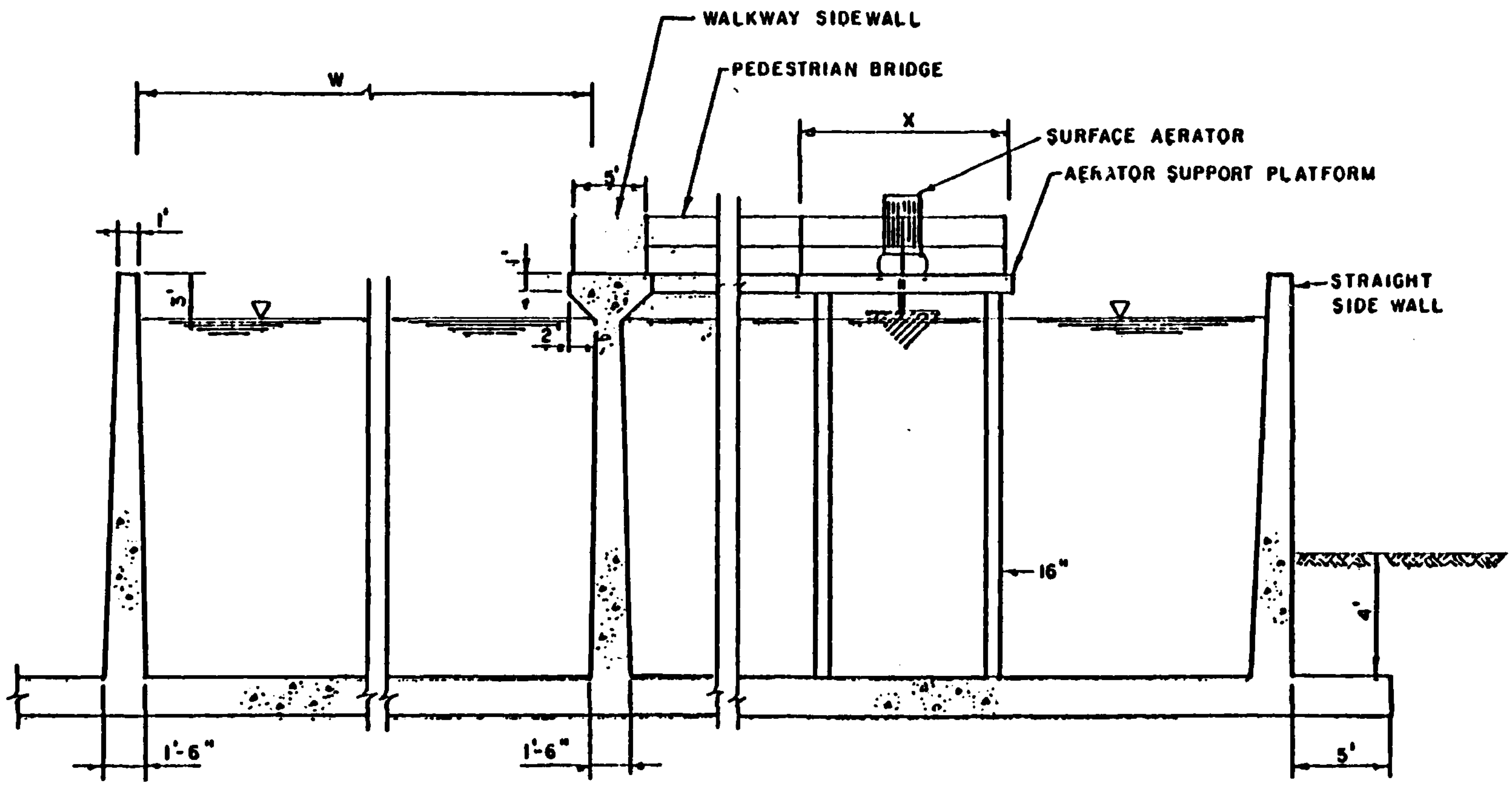


FIGURE 2.1-30 TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

and for two end walls:

$$V_{cgw} = 2\ (PGW)\ (NB)\ (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.1.18.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.1.18.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.1.18.5.11.2 Figure 2.1-31 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078\ (HPSN)$$

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.1.18.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6\ (DW + 2)]\ (NT)\ (NA)\ (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.1.18.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

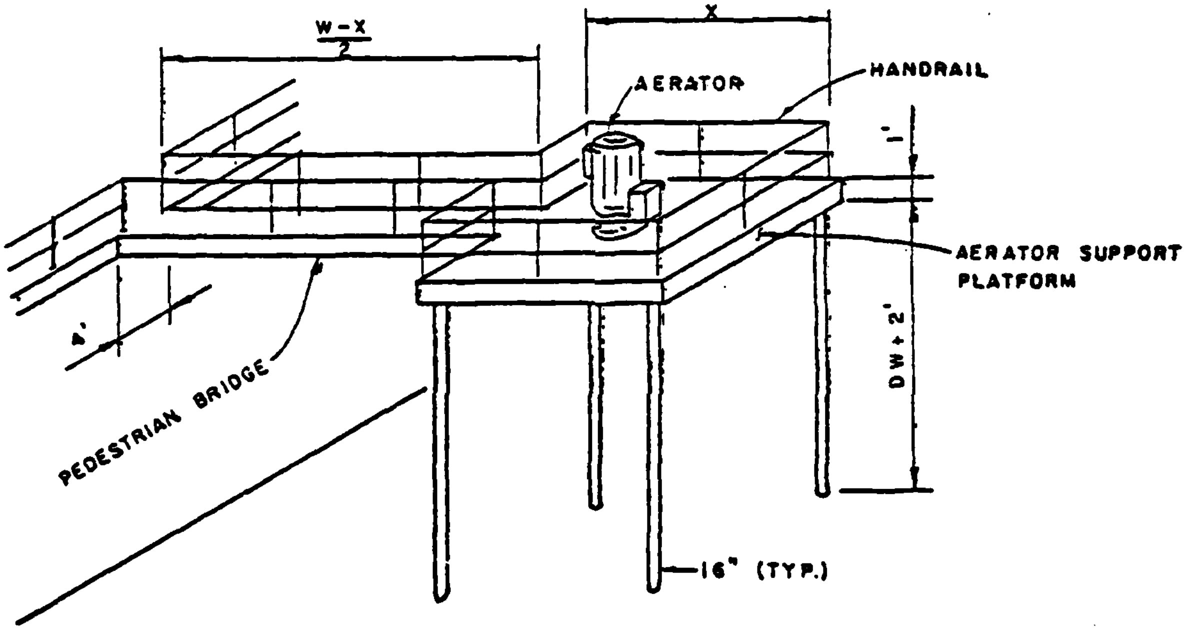


FIGURE 2.1-31 AERATOR SUPPORT PLATFORM

$$V_{cwb} = [2\ (W - X)]\ (NB)\ (NT)\ (NA)$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.1.18.5.12 Summary of reinforced concrete structures.

2.1.18.5.12.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.1.18.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.1.18.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.1.18.5.13.1 When NT = 2,

$$LHR = 4\ W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.1.18.5.13.2 When NT = 3,

$$LHR = 6\ W + 10 + 3 \cdot (3X + W - 4)$$

2.1.18.5.13.3 When NT $\geq$ 4,

If $\frac{NT}{2}$ is an even number,

$$LHR = \left\{ PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT) + (NA) \cdot (NT) \cdot (3X + W - 4) \right\} \cdot (NB)$$

If $\frac{NT}{2}$ is an odd number,

$$LHR = \left\{ PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT + 2) + (NA)\ (NT)\ (3X + w - 4) \right\} \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.1.18.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.1.18.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)\ (NT)\ (NA)\ (HPSN)$$

where

$TICA$ = total installed capacity of the aeration equipment, horsepower.

$HPSN$ = capacity of one individual aerator, horsepower.

2.1.18.5.14.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\ (TICA)^{0.3731}$$

$$\text{When TICA} \geq 200 \text{ hp}$$

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.1.18.5.14.3 The maintenance manpower requirement can be estimated as follows:

$$\text{When TICA} \leq 100 \text{ hp}$$

$$MMH = 106.3\ (TICA)^{0.4031}$$

$$\text{When TICA} > 100 \text{ hp}$$

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.1.18.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.18.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.1.18.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.18.6 Quantities Calculations Output Data.

2.1.18.6.1 Number of aeration tanks, NT.

2.1.18.6.2 Number of aerators per tank, NA.

2.1.18.6.3 Number of process batteries, NB.

2.1.18.6.4 Capacity of each individual aerator, HPSN, hp.

2.1.18.6.5 Depth of aeration tanks, DW, ft.

2.1.18.6.6 Length of aeration tanks, L, ft.

2.1.18.6.7 Width of aeration tanks, W, ft.

2.1.18.6.8 Width of pipe gallery, PGW, ft.

2.1.18.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.1.18.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.1.18.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.1.18.6.12 Quantity of handrail, LHR, ft.

2.1.18.6.13 Operation manpower requirement, OMH, MH/yr.

2.1.18.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.1.18.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.1.18.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.1.18.6.17 Correction factor for minor capital cost items, CF.

2.1.18.7 Unit Price Input Required.

2.1.18.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.18.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.1.18.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.1.18.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, \$, optional.

2.1.18.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.1.18.7.6 Equipment installation labor rate, \$/MH.

2.1.18.7.7 Crane rental rate, UPICR, \$/hr.

2.1.18.7.8 Unit price of handrail, UPIHR, \$/L.F.

2.1.18.8 Cost Calculations.

2.1.18.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.18.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, \$.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.1.18.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.1.18.8.4 Cost of installed aeration equipment.

2.1.18.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.1.18.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.1.18.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16{,}300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16{,}300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.18.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When HPSN 60 hp

$$IMH = 39 + 0.55\ (HPSN)$$

When HPSN 60 hp

$$IMH = 61.3 + 0.18\ (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.18.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.18.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.18.8.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, dollars.

LABRI = labor rate, dollars/man-hour.

UPICR = crane rental rate, dollars/hr.

2.1.18.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, $ per lineal foot. A value of $25.20 per foot for the first quarter of 1977 is suggested.

2.1.18.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.1.18.8.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, dollars.

CF = correction factor for minor cost items, from second-order design output.

2.1.18.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, $/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.1.18.9 Cost Calculations Output Data.

2.1.18.9.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, dollars.

2.1.18.9.2 Operation and maintenance supply and material costs, OMMC, dollars.

2.1.19 PURE OXYGEN ACTIVATED SLUDGE.

2.1.19.1 Input Data.

2.1.19.1.1 Wastewater flow.

2.1.19.1.1.1 Average flow, mgd.

2.1.19.1.1.2 Peak hourly flow, mgd.

2.1.19.1.2 Wastewater characteristics.

2.1.19.1.2.1 BOD_5, mg/l.

2.1.19.1.2.2 COD, mg/l.

2.1.19.1.2.3 TSS, mg/l.

2.1.19.1.2.4 VSS, mg/l.

2.1.19.2 Design Parameters.

2.1.19.2.1 MLSS, mg/l, (range 4000 - 7000, mean 5000).

2.1.19.2.2 Organic loading (F/M ratio), lb BOD_5/lb MLVSS/day (0.6 - 0.8 with peak of 2.0).

2.1.19.2.3 Hydraulic retention time, hrs (2 - 4).

2.1.19.2.4 Recycle ratio, (25 - 50 percent).

2.1.19.2.5 Effluent quality, excellent, approximately 90 percent reduction of BOD and SS.

2.1.19.3 Process Design Calculations.

2.1.19.3.1 Calculate the amount of MLVSS required.

2.1.19.3.1.1 Select designed F/M ratio of 0.7 lb BOD_5/lb MLVSS/day.

$$(MLVSS)_1 = \frac{Q_{avg} \times S_o \times 8.34}{F/M}$$

where

$(MLVSS)_1$ = mixed liquor volatile suspended solids in the aeration tanks, under average conditions, lb.

Q_{avg} = average wastewater flow, mgd.

S_o = influent BOD_5 concentration, mg/l.

F/M = food to microorganism ratio, lb BOD_5/lb MLVSS/day.

2.1.19.3.1.2 Check the peak F/M not to exceed 2.0.

$$(MLVSS)_2 = \frac{Q_p \times S_o \times 8.34}{2}$$

where

$(MLVSS)_2$ = mixed liquor suspended solids in the aeration tank under peak conditions, lb.

Q_p = peak flow, mgd.

2.1.19.3.1.3 Use the larger of the two MLVSS values as the designed MLVSS quantity, $(MLVSS)_d$.

2.1.19.3.1.4 Calculate the total mixed liquor suspended solids quantity. For the design loading range, the volatile content of the mixed liquor is approximately from 0.75 - 0.8. Thus:

$$(MLSS)_d = \frac{(MLVSS)_d}{V.C.}$$

where

$(MLSS)_d$ = designed mixed liquor suspended solids, lb.

V.C. = volatile content of mixed liquor use 0.75.

2.1.19.3.2 Effluent Characteristics.

2.1.19.3.2.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{eff}\ f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l. (from user input).

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.19.3.2.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.19.3.2.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.19.3.2.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.19.3.2.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.19.3.2.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.19.3.3 Sizing of aeration tanks.

$$V = \frac{(MLVSS)_d \times 10^6}{8.34 \times (MLVSS)_c \times 7.48}$$

where

V = volume of aeration tank, cu ft.

$(MLVSS)_c$ = MLVSS concentration in aeration tanks (use 4000 mg/l).

2.1.19.3.4 Oxygen requirement. The oxygen requirement of the activated sludge system depends on the F/M, influent COD to BOD_5 ratio. The following equations give the oxygen requirements.

When $COD/BOD_5 = 3.0$

$$O_2 = [0.953 + 0.309 (\frac{1}{F/M})] \cdot (Q_{avg}) \cdot (S_o) \cdot (8.34)$$

When $COD/BOD_5 = 2.5$

$$O_2 = [0.653 + 0.347 (\frac{1}{F/M})] \cdot (Q_{avg}) \cdot (S_o) \cdot (8.34)$$

When $COD/BOD_5 = 2.0$

$$O_2 = [0.48 + 0.32 (\frac{1}{F/M})] \cdot (Q_{avg}) \cdot (S_o) \cdot (8.34)$$

where

O_2 = oxygen requirement, lb/day.

F/M = food to microorganism ratio, lb/BOD_5/lb MLVSS/day.

2.1.19.3.5 Sludge production for municipal waste only.

2.1.19.3.5.1 Volatile sludge production.

$$X_v = [1.073 - 0.193 (\frac{1}{F/M})] \cdot (1.55) (COD/BOD_5 - 2.5) \cdot (Q_{avg}) \cdot (S_o - S_e) \cdot (8.34)$$

where

X_v = volatile sludge production, lb/day.

COD = influent COD, mg/l.

BOD_5 = influent BOD_5, mg/l.

1.55 = correction factor for other COD/BOD_5 than 2.5.

2.1.19.3.5.2 Inert sludge production.

$$X_o = (TSS - VSS) \times Q_{avg} \times 8.34$$

where

X_o = inert sludge production, lb/day.

TSS = total suspended solids concentration in influent, mg/l.

VSS = volatile suspended solids concentration in effluent, mg/l.

2.1.19.3.5.3 Total sludge production.

$$X_T = X_v + X_o$$

where

X_T = total sludge production, lb/day.

2.1.19.4 Process Design Output Data.

2.1.19.4.1 Designed MLVSS, $(MLVSS)_d$, lbs.

2.1.19.4.2 Effluent BOD_5, S_e, mg/l.

2.1.19.4.3 Volume of aeration tank, V, cu ft.

2.1.19.4.4 Oxygen requirement, O_2, lb/day.

2.1.19.4.5 Volatile sludge production, X_v, lb/day.

2.1.19.4.6 Inert sludge production, X_o, lb/day.

2.1.19.4.7 Total sludge production, X_T, lb/day.

2.1.19.5 Quantities Calculations.

2.1.19.5.1 Excess capacity factor. In order to take care of the peak loading and for conservative design, the capacities of actual units are increased by multiplying the design values by the excess capacity factors. Smith et al conducted a statistical analysis of the excess capacity factors of existing pure oxygen systems. They are used in this program.

2.1.19.5.1.1 For aeration tanks:

$$(ECF)_{at} = \frac{1.89}{(Q_{avg})^{0.07043}}$$

2.1.19.5.1.2 For oxygenation system:

$$(ECF)_{as} = \frac{1.85}{\left(\frac{O_2}{2000}\right)^{0.07334}}$$

where

$(ECF)_{at}$ = excessive capacity factor for aeration tank.

$(ECF)_{as}$ = excessive capacity factor for oxygenation system.

O_2 = oxygen requirement, lb/day.

2.1.19.5.1.3 Designed aeration tank volume, V_d:

$$V_d = (ECF)_{at} \cdot V$$

where

V_d = designed volume of aeration tank, cu ft.

V = calculated aeration tank volume, cu ft.

2.1.19.5.1.4 Designed capacity of the oxygenation system.

$$OXYGEN = \frac{O_2 \cdot (ECF)_{as}}{(2000)\ (0.9)}$$

where

OXYGEN = designed capacity of oxygenation system, ton/day.

0.9 = oxygen utilization is 90 percent for a typical pure oxygen system.

2.1.19.5.2 Reactor configuration.

2.1.19.5.2.1 In general, the aeration tanks for pure oxygen systems utilize the complete mixing reactors in series concept. Usually the number of stages in each reactor train is 3 or 4 and the minimum number of reactor trains would be 2. Based on experience, the following rule is given to determine the number of reactor trains and tanks to be used.

Q_{avg} (mgd)	No. of Trains NT	No. of Stages Per Train NS
1 - 15	2	3 or 4
15 - 30	3	3 or 4
30 - 50	4	3 or 4
50 - 60	5	3 or 4
60 - 80	6	3 or 4
80 - 100	7	3 or 4
100 - 120	8	3 or 4
120 - 150	10	3 or 4

2.1.19.5.2.2 When $Q_{avg} \leq 150$ mgd, only one battery of reactors will be used and NB = 1.

2.1.19.5.2.3 When $Q_{avg} > 150$ mgd, the system will be designed as two identical batteries. Each can handle $\frac{Q_{avg}}{2}$ and NB = 2.

where

NT = number of reactor trains.

NS = number of stages per train.

NB = number of batteries.

2.1.19.5.2.4 Dimensions. As suggested by one of the manufacturers of the pure oxygen system, the following rules are to be satisfied in the design of aeration tanks.

- Side water depth of tank, SWD = 8 to 12 ft.
- Each stage will be square construction with length of T ft, and T should be in the range of 22 to 65 ft.
- Length to depth ratio should be in the range of 0.2 to 0.37, with 0.3 the commonly used number.

Thus the length of each aeration tank stage would be:

$$T = \left(\frac{V_d}{(NB)\ (NT)\ (NS)\ (0.3)}\right)^{1/3}$$

where

T = length of one aeration stage, ft.

NS = number of stages per train; use 3 to start with.

If $T \leq 22$ ft, use 22 ft as the length.

If $T > 65$ ft, increase the number of stages, NS, to 4 and calculate new length. If the length is still larger than 65 ft, increase the number of reactor trains NT = NT + 1 and repeat, until T is in the right range of 22 ft to 65 ft.

The side water depth is calculated by:

$$SWD = 0.3 \cdot T$$

where

SWD = side water depth, ft.

Most manufacturers suggest 2.5 ft of freeboard; therefore, the total depth would be:

$$Z = SWD + 2.5$$

where

Z = total depth of aeration tanks.

2.1.19.5.3 Oxygenation system design. There are three different ways that pure oxygen can be supplied to the wastewater treatment sites. The first one is using a liquid oxygen storage tank. The second would be an on-site generation by using a PSA system and the third would be oxygen generation using the cryogenic process. Liquid oxygen is only used as backup storage in domestic wastewater treatment. PSA oxygen supply is generally used with smaller installation, usually for systems which require less than 20 tons of oxygen per day. For larger applications the cryogenic plant will be utilized. The pure oxygen is piped from the generating site to the aeration tank and the oxygen transfer is promoted by using surface aerator.

2.1.19.5.3.1 Calculate the capacity of surface aerators.

When OXYGEN $\leq$ 20 tons/day:

$$\text{Dissolution horsepower, DHP} = \frac{(\text{OXYGEN})\ (0.9)\ (2000)}{(24)(7.0)}$$

where

DHP = dissolution horsepower required, hp.

7.0 = lbs of oxygen dissolved per horsepower - hr for PSA units.

When OXYGEN > 20 tons/day:

$$\text{DHP} = \frac{(\text{OXYGEN})\ (0.9)\ (2000)}{(24)\ (8.5)}$$

where

8.5 = lbs of oxygen dissolved per horsepower-hr for cryogenic units.

2.1.19.5.3.2 Calculate the horsepower required for generation of pure oxygen.

When OXYGEN $\leq$ 20 tons/day:

$$GHP = (20.4)\ (OXYGEN)$$

When OXYGEN $>$ 20 tons/day:

$$GPH = (17.5)\ (OXYGEN)$$

where

GHP = pure oxygen generating horsepower, hp.

20.4 = energy required per ton of oxygen generated for PSA units.

17.5 = energy required per ton of oxygen generated for cryogenic units.

2.1.19.5.3.3 Calculate the total horsepower required.

$$THP = DHP + GHP$$

where

THP = total horsepower required for the pure oxygen system.

2.1.19.5.3.4 Calculate size and number of cryogenic oxygen generation units

If OXYGEN $\leq$ 100 tons/day, one unit will be used:

$$CUC = OXYGEN$$

If OXYGEN $>$ 100 tons/day, multiple units will be used:

$$CUC = \frac{OXYGEN}{N}$$

Try N = 2 first; if CUC $>$ 100, go to N = N + 1 and repeat until CUC $\leq$ 100.

where

CUC = individual cryogenic unit capacity, tons/day.

N = number of cryogenic units.

2.1.19.5.4 Calculate horsepower of individual aerators. The aerators must be one of the following sizes: 5, 7.5, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100, 125, and 150 hp.

$$HP_s = \frac{DHP}{NT \times NS \times NB}$$

Compare HP_s to the sizes listed and select the next largest size.

where

HP_s = horsepower of the individual aerators, hp.

2.1.19.5.5 Calculate earthwork required for construction. Assume that the tank will be half-buried.

$$V_{ew} = \frac{[Z/2+1]\ [NS(T)+NS+4)\ (NT(T)+NT+4)\ +\ (NS(T)+NS+Z+6)\ (NT(T)+NT+Z+6)]}{2}$$

where

V_{ew} = earthwork required for construction, cu ft.

2.1.19.5.6 Calculate reinforced concrete quantities.

2.1.19.5.6.1 Calculate quantity of R.C. slab in-place required.

$$V_{cs} = (NS(T) + NS + 4)\ (NT(T) + NT + 4)\ (NB)$$

where

V_{cs} = quantity of R.C. slab in-place required, cu ft.

2.1.19.5.6.2 Calculate quantity of R.C. wall in-place required.

$$V_{cw} = [NS(T)+NS+1]\ [NT+1] + [NT(T)+NT+1]\ [NS+1]\ \ [(Z)(NB)]$$

where

V_{cw} = quantity of R.C. wall in-place required, cu ft.

2.1.19.5.6.3 Calculate quantity of R.C. in-place for top slab.

$$V_{cts} = [NS(T)+NS+1]\ [NT(T)+NT+1]$$

where

V_{cts} = quantity of R.C. in place required for top slab, cu ft.

2.1.19.5.6.4 Calculate quantity of R.C. in-place required for effluent trough.

$$V_{cet} = 12.5\ [NT(T)+NT+1]$$

where

V_{cet} = quantity of R.C. in-place required for effluent trough.

2.1.19.5.6.5 Calculate total quantity of R.C. in-place for walls. The forming required for the top slab and effluent trough cause the cost to be the same as for walls; therefore, they will be added together to obtain a total quantity.

$$V_{cwt} = V_{cw} + V_{cts} + V_{cet}$$

where

V_{cwt} = total quantity of R.C. wall in-place required, cu ft.

2.1.19.5.7 Calculate electrical energy requirements for operation. Assume that the aerators and oxygen generation equipment operate 90 percent of the time year-round.

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times THP$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.19.5.8 Calculate operation manpower requirements. The operation manpower requirements for the pure oxygen system should be very nearly the same as that for the activated sludge process with mechanical aeration. The operation manpower is presented as a function of the total horsepower of the equipment.

2.1.19.5.8.1 If THP < 200 hp, the operation manpower is calculated by:

$$OMH = 242.4\ (THP)^{0.3731}$$

2.1.19.5.8.2 If THP ≥ 200 hp, the operation manpower is calculated by:

$$OMH = 100\ (THP)^{0.5425}$$

where

OMH = operation manpower requirement, MH/yr.

2.1.19.5.9 Calculate the maintenance manpower requirements. The maintenance manpower requirements for the pure oxygen system are of two types essentially: the aeration equipment and the oxygen generation equipment. The maintenance required for the aeration equipment has been related to aeration horsepower by Patterson and Bunker and this will be utilized to estimate the maintenance manpower for the dissolution equipment. There is no published data on maintenance for the oxygen generation equipment; however, estimates of maintenance manpower requirements have been obtained from manufacturers and will be utilized to predict this requirement.

2.1.19.5.9.1 When OXYGEN $\leq$ 20 tons/day and DHP $\leq$ 100 hp, the maintenance manpower requirement is calculated by:

$$MMH = 160 + 106.3\ (DHP)^{0.4031}$$

2.1.19.5.9.2 When OXYGEN $\leq$ 20 tons/day and DHP $>$ 100 hp, the maintenance manpower requirement is calculated by:

$$MMH = 160 + 42.6\ (DHP)^{0.5956}$$

2.1.19.5.9.3 When OXYGEN $>$ 20 tons/day and DHP $\leq$ 100 hp, the maintenance manpower requirement is calculated by:

$$MMH = 480 + 106.3\ (DHP)^{0.4031}$$

2.1.19.5.9.4 When OXYGEN $>$ 20 tons/day and DHP $>$ 100 hp, the maintenance manpower requirement is calculated by:

$$MMH = 480 + 42.6\ (DHP)^{0.5956}$$

where

MMH = maintenance manpower requirement, MH/yr.

2.1.19.5.10 Operation and maintenance material and supply costs. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material. These costs are estimated as a percent of installed costs for the aeration and oxygen generation equipment.

2.1.19.5.10.1 Operation and maintenance material and supply costs for aerators.

$$OMMPA = 4.225 - 0.975 \log (DHP)$$

where

OMMPA = operation and maintenance material and supply costs for aerators as percent of installed costs for aerators, percent.

2.1.19.5.10.2 Operation and maintenance material and supply costs for oxygen generation equipment.

$$OMMPO = 1.35\%$$

where

OMMPO = operation and maintenance material and supply costs for oxygen generation equipment as percent of in stalled cost of oxygen generation equipment, percent.

2.1.19.5.11 Other construction cost items. The majority of the costs for the pure oxygen system has been accounted for in the previous calculations. Other items such as oxygen piping, control equipment, site cleaning, painting, etc., will be estimated as a percent of the total bare construction cost. It is estimated these items would represent 10 percent of the cost.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for minor construction cost items.

2.1.19.6 Quantities Calculation Output Data.

2.1.19.6.1 Design aeration tank volume, V_d, cu ft.

2.1.19.6.2 Design capacity of oxygenation system, OXYGEN, tons/day.

2.1.19.6.3 Number of reactor trains, NT.

2.1.19.6.4 Number of stages per train, NS.

2.1.19.6.5 Number of batteries, NB.

2.1.19.6.6 Length of one aeration stage, T, ft.

2.1.19.6.7 Tank side water depth, SWD, ft.

2.1.19.6.8 Total tank depth, Z, ft.

2.1.19.6.9 Dissolution horsepower, DHP, hp.

2.1.19.6.10 Oxygen generation horsepower, GHP, hp.

2.1.19.6.11 Total horsepower requirement, THP, hp.

2.1.19.6.12 Earthwork required for construction, V_{ew}, cu ft.

2.1.19.6.13 Quantity of R.C. slab in-place required, V_{cs}, cu ft.

2.1.19.6.14 Total quantity of R.C. wall in-place required, V_{cwt}, cu ft.

2.1.19.6.15 Electrical energy required for operation, KWH, kwhr/yr.

2.1.19.6.16 Operation manpower requirement, OMH, MH/yr.

2.1.19.6.17 Maintenance manpower requirement, MMH, MH/yr.

2.1.19.6.18 Operation and maintenance material and supply costs as percent of purchase cost of equipment, OMMP, percent.

2.1.19.6.19 Correction factor for minor construction cost items, CF.

2.1.19.6.20 Horsepower of individual aerators, HP_s, hp.

2.1.19.6.22 Individual cryogenic unit capacity, CUC, tons/day.

2.1.19.6.23 Number of cryogenic units, N.

2.1.19.7 <u>Unit Price Input Required</u>.

2.1.19.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

2.1.19.7.2 Unit price input for R.C. wall in-place, UPICW, $/cu yd.

2.1.19.7.3 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

2.1.19.7.4 Standard size low speed fixed surface aerator costs, SSXSA, $, (optional).

2.1.19.7.5 Standard size PSA oxygen generation unit cost, COSTSP, $, (optional).

2.1.19.7.6 Standard size cryogenic oxygen generation cost, COSTSCR, $, (optional).

2.1.19.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.19.7.8 Equipment installation labor rate, LABRI, $/man-hour.

2.1.19.7.9 Crane rental rate, UPICR, $/hr.

2.1.19.8 Cost Calculations.

2.1.19.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.19.8.2 Cost of concrete slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of R.C. slab in-place required, cu ft.

UPICS = unit price input R.C. slab in-place, \$/cu yd.

2.1.19.8.3 Cost of concrete wall in-place.

$$COSTCS = \frac{V_{cwt}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall in-place required, cu ft.

UPICW = unit price input R.C. wall in-place, \$/cu yd.

2.1.19.8.4 Installed cost of PSA oxygen generation equipment. If OXYGEN is $<$ 20 tons/day, the PSA unit will be used.

2.1.19.8.4.1 Calculate purchase cost of PSA oxygen generation equipment.

$$COSTOE = COSTSP \times \frac{COSTRO}{100}$$

where

COSTOE = purchase cost of oxygen generation equipment, $.

COSTSP = purchase cost of standard size (10-ton) PSA oxygen generation unit, $.

COSTRO = purchase cost of oxygen generation equipment of OXYGEN capacity as percent of standard unit cost, percent.

2.1.19.8.4.2 Calculate COSTRO.

$$COSTRO = 22.96\ (OXYGEN)^{0.6531}$$

where

OXYGEN = design capacity of oxygen system, tons/day.

2.1.19.8.4.3 Cost of standard size PSA unit. The cost of the 10-ton/day PSA unit selected as the standard size unit is:

$$COSTSP = \$800,000$$

For better cost estimation COSTSP should be obtained from the vendor and treated as a unit price input. However, if it is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSP = \$800,000\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.19.8.4.4 Installed cost of PSA unit. The installation cost for PSA unit is approximately 28 percent of the equipment purchase cost.

$$IOEC = 1.28\ COSTOE$$

where

IOEC = installed cost of oxygen generation equipment, $.

2.1.19.8.5 Installed cost of cryogenic generation equipment. If OXYGEN > 20 tons/day, the cryogenic units will be used.

2.1.19.8.5.1 Calculate purchase cost of cryogenic oxygen generation equipment.

$$COSTOE = COSTSCR \times \frac{COSTRO}{100}$$

where

COSTOE = purchase cost of oxygen generation equipment, \$.

COSTSCR = purchase cost of standard size (50-ton) cryogenic oxygen generation unit, \$.

COSTRO = purchase cost of oxygen generation equipment of OXYGEN capacity as percent of standard unit cost, percent.

2.1.19.8.5.2 Calculate COSTRO.

$$COSTRO = 14.1\ (OXYGEN)^{0.5041}$$

where

OXYGEN = design capacity of oxygen generation system, tons/ day.

2.1.19.8.5.3 Cost of standard size cryogenic unit. The cost of the 50-ton/ day cryogenic unit selected as the standard size unit is:

$$COSTSCR = \$1,900,000$$

For better estimation COSTSCR should be obtained from the vendor and treated as a unit price input. However, if it is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSCR = \$1,900,000\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.19.8.5.4 Installed cost of cryogenic unit. The installation cost for cryogenic units is approximately 23 percent of the equipment purchase cost.

$$IOEC = 1.23\ (COSTOE)\ (N)$$

where

IOEC = installed cost of oxygen generation equipment, \$.

N = number of cryogenic units required.

2.1.19.8.6 Cost of installed aeration equipment.

2.1.19.8.6.1 Purchase cost of slow speed fixed surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = (SSXSA)\ (RSXSA)$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed fixed aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HP_s hp and that of the standard size aerator with 20 hp.

2.1.19.8.6.2 RSXSA. The cost ratio can be expressed as:

$$RSXSA = 0.2148\ HP_s^{0.513}$$

where

HP_s = capacity of each individual aerator, hp.

2.1.19.8.6.3 Cost of standard size aerator. The cost of fixed slow speed surface aerator for the first quarter of 1977 is:

$$SSXSA = \$16,300$$

For a better cost estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16,300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.19.8.6.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

$$\text{When } HP_s \leq 60 \text{ hp}$$

$$IMH = 39 + 0.55 (HP_s)$$

$$\text{When } HP_s > 60 \text{ hp}$$

$$IMH = 61.3 + 0.18 (HP_s)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.19.8.6.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.19.8.6.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchased equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.19.8.6.7 Installed aeration equipment cost, IAEC.

$$IAEC = [CSXSA \left(1 + \frac{PMINC}{100}\right) + IMH \cdot LABRI + CH \times UPICR] \cdot (NB) \cdot (NT) \cdot (NS)$$

where

IAEC = installed aeration equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

NB = number of batteries.

NT = number of trains.

NS = number of stages.

2.1.19.8.7 Total bare construction cost.

$$TBCC = (COSTE + COSTCS + COSTCW + IOEC + IAEC)\ CF$$

where

TBCC = total bare construction cost, $.

CF = correction factor for minor costs items.

2.1.19.8.8 Operation and maintenance material and supply costs.

$$OMMC = \frac{(OMMPA)\ (IAEC) + (OMMPO)\ (IOEC)}{100}$$

where

OMMC = operation and maintenance material and supply costs, $.

OMMPA = operation and aintenance material and supply costs for aeration equipment as percent of installed cost of aeration equipment, percent.

OMMPO = operation and maintenance material and supply costs for oxygen generation equipment as percent of installed cost of oxygen generation equipment, percent.

2.1.19.9 Cost Calculations Output Data

2.1.19.9.1 Total bare construction cost for the pure oxygen activated sludge process, TBCC, $.

2.1.19.9.2 Operation and maintenance material and supply costs, OMMC, $.

2.1.20 STEP AERATION ACTIVATED SLUDGE (DIFFUSED AERATION).

2.1.20.1 Input Data.

2.1.20.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.20.1.2 Wastewater Strength.

2.1.20.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.20.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.20.1.2.3 Suspended solids, mg/l.

2.1.20.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.20.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.20.1.3 Other Characterization.

2.1.20.1.3.1 pH.

2.1.20.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.20.1.3.3 Nitrogen,[1] mg/l.

2.1.20.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.20.1.3.5 Oils and Greases, mg/l.

2.1.20.1.3.6 Heavy metals, mg/l.

2.1.20.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.20.1.3.8 Temperature, ^{o}F or ^{o}C.

2.1.20.1.4 Effluent Quality Requirements.

2.1.20.1.4.1 BOD_5, mg/l.

2.1.20.1.4.2 SS, mg/l.

2.1.20.1.4.3 TKN, mg/l.

2.1.20.1.4.4 P, mg/l.

---

[1] The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.20.1.4.5 Total nitrogen (TKN + NO_3-N), mg/l.

2.1.20.1.4.6 Settleable solids, mg/l/hr.

2.1.20.2 Design Parameters.

2.1.20.2.1 Reaction Rate Constants and Coefficients.

Constants	Range
Eckenfelder	
k	0.0007-0.002 l/mg/hr
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
f	0.40
f'	0.53

2.1.20.2.2 F/M = (0.2-0.4).

2.1.20.2.3 Volumetric loading = 40-60.

2.1.20.2.4 t = (3-5) hr.

2.1.20.2.5 t_s = (3-7) days.

2.1.20.2.6 MLSS = (2000-3500) mg/l.

2.1.20.2.7 MLVSS = (1400-2450) mg/l.

2.1.20.2.8 Q_r/Q = (0.25-0.75).

2.1.20.2.9 lb O_2/lb $BOD_r \geq 1.25$.

2.1.20.2.10 lb solids/lb BOD_r = (0.5-0.7).

2.1.20.2.11 θ = (1.0-1.04).

2.1.20.2.12 Efficiency = (> 90 percent).

2.1.20.3 Process Design Calculations.

2.1.20.3.1 Assume the following design parameters when unknown.

2.1.20.3.1.1 BOD removal rate constant (k).

2.1.20.3.1.2 Fraction of BOD synthesized (a).

2.1.20.3.1.3 Fraction of BOD oxidized for energy (a').

2.1.20.3.1.4 Endogenous respiration rate (b and b').

2.1.20.3.1.5 Mixed liquor suspended solids (MLSS).

2.1.20.3.1.6 Mixed liquor volatile suspended solids (MLVSS).

2.1.20.3.1.7 Food-to-microorganism ratio (F/M).

2.1.20.3.1.8 Temperature correction coefficient (θ).

2.1.20.3.1.9 Nonbiodegradable fraction of VSS in influent (f).

2.1.20.3.1.10 Degradable fraction of the MLVSS (f').

2.1.20.3.2 Adjust BOD removal rate constant for temperature.

$$K_T = K_{20}\theta^{(T-20)}$$

where

K_T = rate constant for desired temperature, $^{\circ}C$.

K_{20} = rate constant at $20^{\circ}C$.

θ = temperature correction coefficient.

T = temperature, $^{\circ}C$.

2.1.20.3.3 Determine size of the aeration tank by first determining the detention time.

$$t = \frac{24S_o}{(X_V)\ (F/M)}$$

where

t = detention time, hr.

S_o = influent BOD, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.20.3.4 Check detention time for treatability.

$$\frac{S_e}{S_o} = \frac{1}{1 + kX_V t}$$

where

S_e = BOD_5 soluble in effluent, mg/l.

S_o = BOD_5 in influent, mg/l.

k = BOD removal rate constant, l/mg/hr.

X_V = MLVSS, mg/l.

t = detention time, hr.

and solve for t and compare with t in (2.1.20.3.3) above and select the larger.

2.1.20.3.5 Calculate the volume of aeration tank.

$$V = Q_{avg} \frac{t}{24}$$

where

V = volume, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.20.3.6 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'(S_r)}{t} + b'X_V$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34) + b'(X_V)(V)(8.34)$$

where

$\frac{dO}{dt}$ = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed (S_o - S_e)

t = detention time, hr.

b' = endogenous respiration rate, l/hr.

X_V = MLVSS.

O_2 = oxygen requirement, lb/day.

Q_{avg} = average flow rate, mgd.

V = volume of aeration tank, million gal.

and check the oxygen supplied against ≥ 1.25.

$$\text{lb } O_2/\text{lb } BOD_r = \frac{O_2}{Q(S_r)(8.34)}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.20.3.7 Design aeration system.

2.1.20.3.7.1 Assume the following design parameters.

2.1.20.3.7.1.1 Standard transfer efficiency, percent, from manufacturer (5-8 percent).

2.1.20.3.7.1.2 O_2 transfer in waste/O_2 transfer in water ≈ 0.9.

2.1.20.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.20.3.7.1.4 Correction factor for pressure ≈ 1.0.

2.1.20.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.20.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \; \frac{(C_s)_T(\beta)(p) - C_L}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water ≈ 0.9.

p = correction factor for pressure ≈ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

T = temperature, ^oC.

α = O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.20.3.7.4 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{O_2(10^5)(7.48)}{(OTE)0.0176 \ \frac{lb\ O_2}{ft^3\ air} \ 1440 \frac{min}{day} \ V}$$

where

R_a = required air flow, cfm/1000 ft^3.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, percent.

V = volume of basin, gal.

2.1.20.3.8 Calculate sludge production.

$$\Delta X_V = [aS_rQ_{avg} - bX_V(V) + fQ(VSS) + Q(SS - VSS)](8.34)$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

b = endogenous respiration rate, l/day.

X_V = volatile solids in raw waste, mg/l.

V = volume of tank, gal.

f = nonbiodegradable fraction of influent VSS.

Q = flow, mgd.

VSS = volatile suspended solids in effluent, mg/l.

SS = suspended solids in influent, mg/l.

2.1.20.3.9 Check ΔX_V aginst 0.5-0.7.

$$\frac{\text{lb/solids}}{(\text{lb BOD}_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)}$$

where

ΔX_V = sludge produced, lb/day.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.1.20.3.10 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q = flow, mgd.

X_a = MLSS, mg/l.

X_u = suspended solids concentration in returned sludge, mg/l.

2.1.20.3.11 Calculate solids retention time.

$$SRT = \frac{(V)(X_a)(8.34)}{\Delta X_a}$$

where

SRT = solids retention time, days.

V = volume of basin, gal.

X_a = MLSS, mg/l.

$$\Delta X_a = \frac{\Delta X_V}{\%\ \text{volatile}}$$

ΔX_V = sludge produced, lb/day.

2.1.20.3.12 Effluent Characteristics.

2.1.20.3.12.1 BOD_5.

$$BODE = Se + 0.84\ (X_V)_{eff} f'$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.20.3.12.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.20.3.12.3 Nitrogen.

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.20.3.12.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.20.3.12.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.20.3.12.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.20.3.13 Determine nutrient requirements.

for nitrogen

$$N = 0.123\,\Delta M_T \text{ (or } \Delta X_V\text{)}$$

and phosphorus

$$P = 0.026\,\Delta M_T \text{ (or } \Delta X_V\text{)}$$

where

M_T = sludge produced, lb/day.

X_V = sludge produced, lb/day.

and check against BOD:N:P = 100:5:1.

2.1.20.4 <u>Process Design Output Data</u>.

2.1.20.4.1 Aeration Tank.

2.1.20.4.1.1 Reaction rate constant, 1/mg/hr.

2.1.20.4.1.2 Sludge produced per BOD removed.

2.1.20.4.1.3 Endogenous respiration rate (b, b').

2.1.20.4.1.4 O_2 utilized per BOD removed.

2.1.20.4.1.5 Influent nonbiodegradable VSS (f).

2.1.20.4.1.6 Effluent degradable VSS (f').

2.1.20.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.20.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.20.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.20.4.1.10 Aeration time, hr.

2.1.20.4.1.11 Volume of aeration tank, million gal.

2.1.20.4.1.12 Oxygen required, lb/day.

2.1.20.4.1.13 Sludge produced, lb/day.

2.1.20.4.1.14 Nitrogen requirement, lb/day.

2.1.20.4.1.15 Phosphorus requirement, lb/day.

2.1.20.4.1.16 Sludge recycle ratio, percent.

2.1.20.4.1.17 Solids retention time, days.

2.1.20.4.2 Aeration System.

2.1.20.4.2.1 Standard transfer efficiency, percent.

2.1.20.4.2.2 Operating transfer efficiency, percent.

2.1.20.4.2.3 Required air flow, cfm/1000 ft^3.

2.1.20.5 Quantities Calculations.

2.1.20.5.1 Design values for activated sludge system.

$$V_d = V \frac{10^6}{7.48}$$

$$CFM_d = (CFM)\ (V)\ (133.7)$$

where

V = volume of aeration tanks, million gallons.

2.1.20.5.2 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.20.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.20.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.20.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.20.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.20.5.3.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.20.5.4 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

2.1.20.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.1.20.5.6 Design of aeration tanks.

2.1.20.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.20.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.1.20.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.1.20.5.7 Aeration tank arrangements.

2.1.20.5.7.1 Figure 2.1-32 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\ (\frac{Q_{avg}}{NB})$$

where

PGW = pipe gallery width, ft.

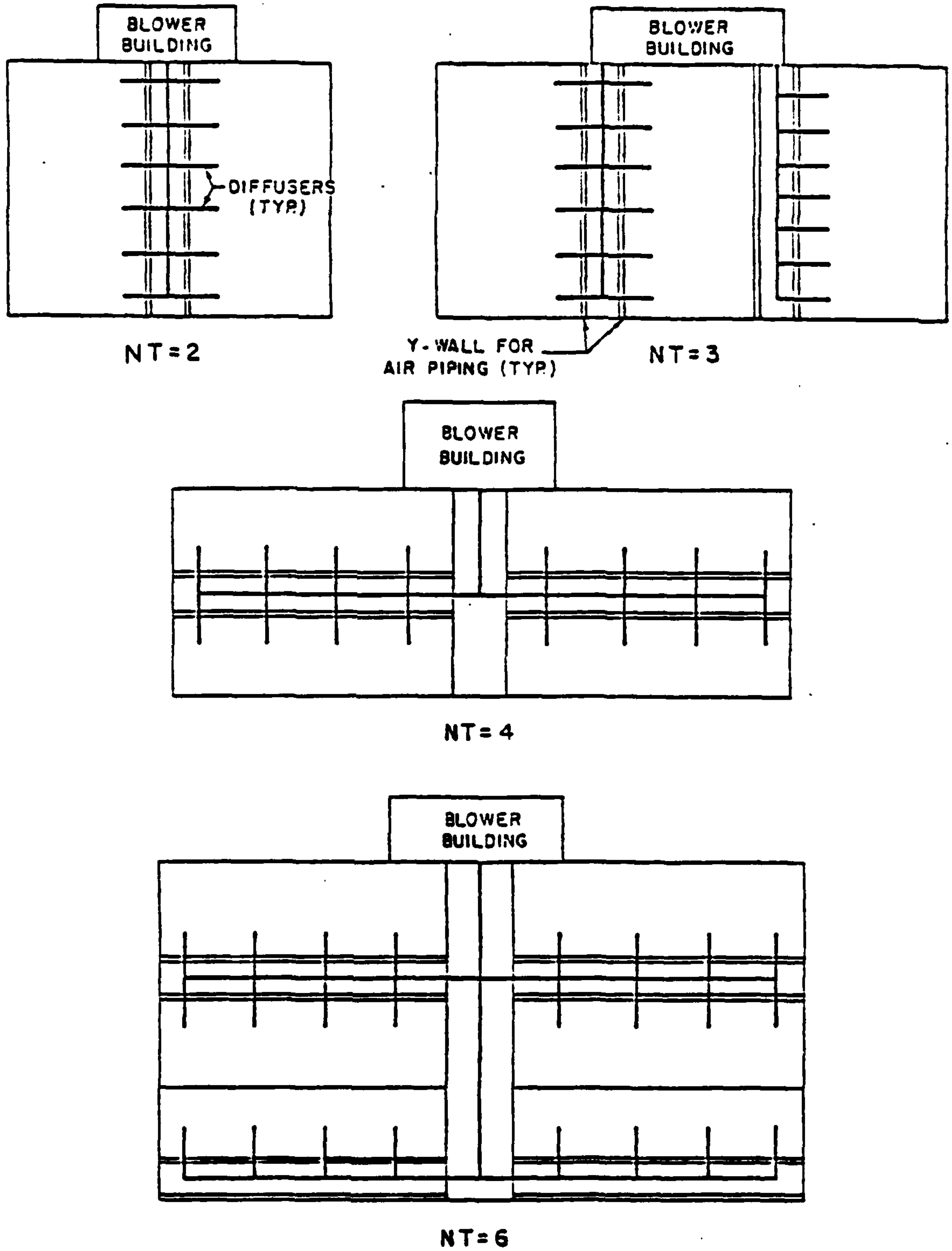


FIGURE 2.1-32 AERATION TANK ARRANGEMENT

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.20.5.8 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.1.20.5.8.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ [\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.1.20.5.8.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ [\frac{(15.75(NT)+15.5)\ (2L+PGW+20) + (15.75(NT)+2.5)\ (2L+PGW+28)}{2}]$$

2.1.20.5.9 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.1.20.5.9.1 For NT less than 4:

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

2.1.20.5.9.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)\ (2L + PGW + 200)]$$

2.1.20.5.10 Reinforced concrete wall quantities.

2.1.20.5.10.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-33. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

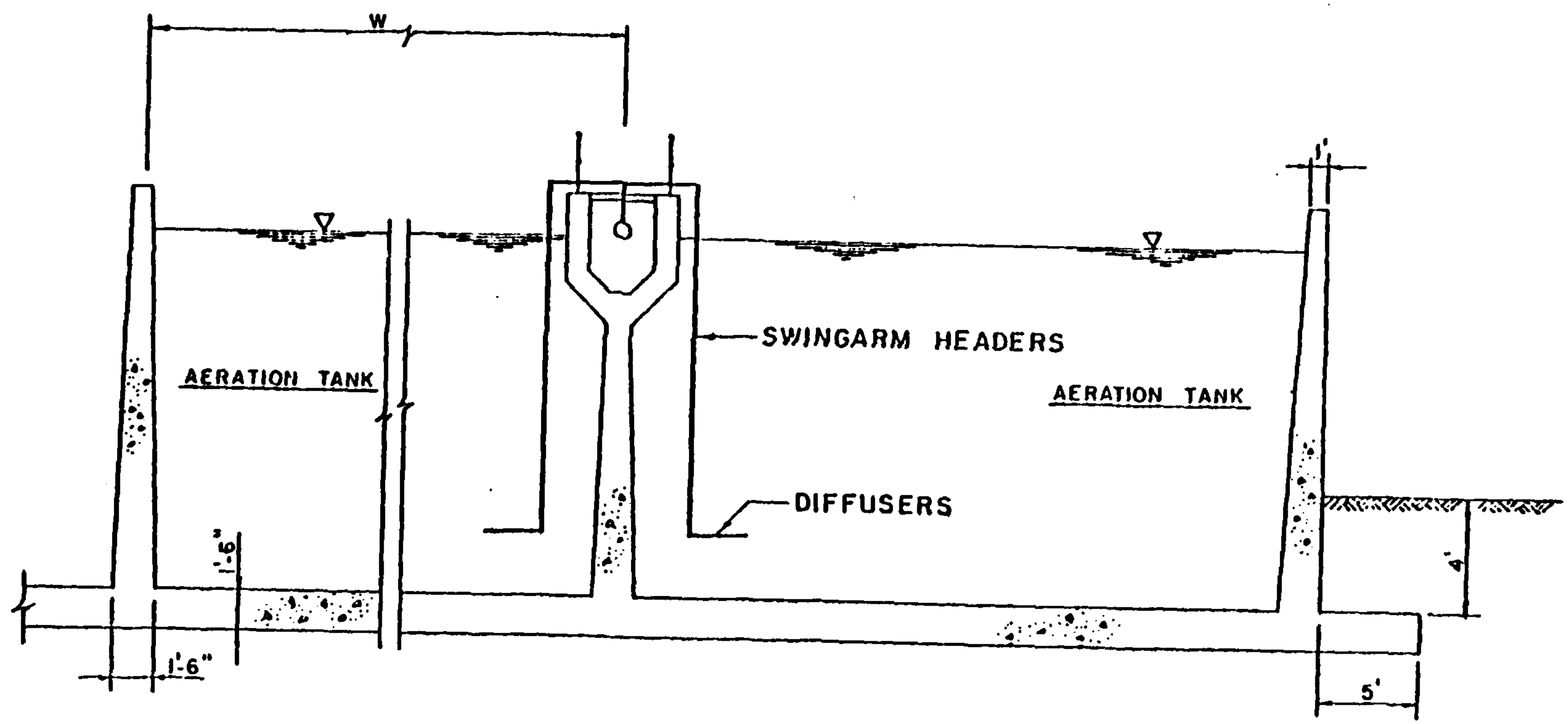


FIGURE 2.1-33 TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

2.1.20.5.10.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.1.20.5.10.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.20.5.10.4 When NT $\geq$ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

where

V_{cw} = R.C. wall quantity required, cu ft.

L = length of aeration tanks, ft.

2.1.20.5.11 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.1.20.5.11.1 If NT is less than 4:

$$LHR = [2(NT)\ (L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.1.20.5.11.2 If NT is greater than or equal to 4:

$$LHR = [2(NT)\ (L) + (4L) + 36.5(NT) + 2\ PGW + 13]\ NB$$

where

LHR = handrail length, ft.

2.1.20.5.12 Calculate operation manpower requirements.

2.1.20.5.12.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.1.20.5.12.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.1.20.5.13 Calculate maintenance manpower requirements.

2.1.20.5.13.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.1.20.5.13.2 If CFM_d 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.1.20.5.14 Energy requirement for operation. The electrical energy required for operation is related to the air requirement by the following equation:

$$KWH = (CFM_d)\ (241.6)$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.1.20.5.15 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.1.20.5.16 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.20.6 Quantities Calculation Output Data.

2.1.20.6.1 Number of aeration tanks, NT.

2.1.20.6.2 Number of diffusers per tank, ND_t.

2.1.20.6.3 Number of process batteries, NB.

2.1.20.6.4 Number of swing arm headers per tank, NSA_t.

2.1.20.6.5 Length of aeration tanks, L, ft.

2.1.20.6.6 Width of pipe gallery, PGW, ft.

2.1.20.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.1.20.6.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.1.20.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.1.20.6.10 Quantity of handrail, LHR, ft.

2.1.20.6.11 Operation manpower requirement, OMH, MH/yr.

2.1.20.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.1.20.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.1.20.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.1.20.6.15 Correction factor for minor construction costs, CF.

2.1.20.7 Unit Price Input Required.

2.1.20.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.1.20.7.2 Unit price input R.C. wall in-place, UPICW, \$/cu yd.

2.1.20.7.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.1.20.7.4 Unit price input for handrails in-place, UPIHR, \$/ft.

2.1.20.7.5 Cost per diffuser, COSTPD, \$, (optional).

2.1.20.7.6 Cost per swing arm header, COSTPH, \$, (optional).

2.1.20.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.1.20.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.1.20.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.1.20.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.1.20.8 Cost Calculations.

2.1.20.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.20.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.1.20.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price R.C. slab in-place, \$/cu yd.

2.1.20.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.1.20.8.5 Cost of diffusers.

2.1.20.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.1.20.8.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, \$.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.1.20.8.6 Cost of swing arm diffuser headers.

2.1.20.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5,000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5,000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.1.20.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, \$.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.1.20.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.1.20.8.8 Crane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.1.20.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.1.20.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.1.20.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.1.20.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.1.20.8.10 Other costs associated with the installed equipment. This category includes the cost for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.20.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.1.20.8.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.1.20.8.13 Operation and maintenance material costs.

$$OMCC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/yr.

OMMP = operation and maintenance material supply costs, as percent of total bare construction cost, percent.

2.1.20.9 Cost Calculations Output Data.

2.1.20.9.1 Total bare construction cost of diffused aeration activated sludge system, TBCC, dollars.

2.1.20.9.2 Operation and maintenance material and supply costs, OMMC, dollars.

2.1.21 STEP AERATION ACTIVATED SLUDGE (MECHANICAL AERATION).

2.1.21.1 Input Data.

2.1.21.1.1 Wastewater Flow (Average and Peak). In case of high variability, a statistical distribution should be provided.

2.1.21.1.2 Wastewater Strength.

2.1.21.1.2.1 BOD_5 (soluble and total), mg/l.

2.1.21.1.2.2 COD and/or TOC (maximum and minimum), mg/l.

2.1.21.1.2.3 Suspended solids, mg/l.

2.1.21.1.2.4 Volatile suspended solids (VSS), mg/l.

2.1.21.1.2.5 Nonbiodegradable fraction of VSS, mg/l.

2.1.21.1.3 Other Characterization.

2.1.21.1.3.1 pH.

2.1.21.1.3.2 Acidity and/or alkalinity, mg/l.

2.1.21.1.3.3 Nitrogen,[1] mg/l.

2.1.21.1.3.4 Phosphorus (total and soluble), mg/l.

2.1.21.1.3.5 Oils and Greases, mg/l.

2.1.21.1.3.6 Heavy metals, mg/l.

2.1.21.1.3.7 Toxic or special characteristics (e.g., phenols), mg/l.

2.1.21.1.3.8 Temperature, °F or °C.

2.1.21.1.4 Effluent Quality Requirements.

2.1.21.1.4.1 BOD_5, mg/l.

2.1.21.1.4.2 SS, mg/l.

2.1.21.1.4.3 TKN, mg/l.

2.1.21.1.4.4 P, mg/l.

---

[1]The form of nitrogen should be specified as to its biological availability (e.g., NH_3 or Kjeldahl).

2.1.21.1.4.5 Total nitrogen (TKN + NO_3-N), mg/l.

2.1.21.1.4.6 Settleable solids, mg/l/hr.

2.1.21.2 Design Parameters.

2.1.21.2.1 Reaction Rate Constants and Coefficients.

Constants	Range
Eckenfelder	
k	0.0007-0.002 l/mg/hr
a	0.73
a'	0.52
b	0.075/day
b'	0.15/day
f	0.40
f'	0.53

2.1.21.2.2 F/M = (0.2-0.4).

2.1.21.2.3 Volumetric loading = 40-60.

2.1.21.2.4 t = (3-5) hr.

2.1.21.2.5 t_s = (3-7) days.

2.1.21.2.6 MLSS = (2000-3500) mg/l.

2.1.21.2.7 MLVSS = (1400-2450) mg/l.

2.1.21.2.8 Q_r/Q = (0.25-0.75).

2.1.21.2.9 lb O_2/lb $BOD_r \geq 1.25$.

2.1.21.2.10 lb solids/lb BOD_r = (0.5-0.7).

2.1.21.2.11 Θ = (1.0-1.04).

2.1.21.2.12 Efficiency = (> 90 percent).

2.1.21.3 Process Design Calculations.

2.1.21.3.1 Assume the following design parameters when unknown.

2.1.21.3.1.1 BOD removal rate constant (k).

2.1.21.3.1.2 Fraction of BOD synthesized (a).

2.1.21.3.1.3 Fraction of BOD oxidized for energy (a').

2.1.21.3.1.4 Endogenous respiration rate (b and b').

2.1.21.3.1.5 Mixed liquor suspended solids (MLSS).

2.1.21.3.1.6 Mixed liquor volatile suspended solids (MLVSS).

2.1.21.3.1.7 Food-to-microorganism ratio (F/M).

2.1.21.3.1.8 Temperature correction coefficient (θ).

2.1.21.3.1.9 Nonbiodegradable fraction of VSS in influent (f).

2.1.21.3.1.10 Degradable fraction of the MLVSS (f').

2.1.21.3.2 Adjust BOD removal rate constant for temperature.

$$K_T = K_{20}\theta^{(T-20)}$$

where

K_T = rate constant for desired temperature, ^{o}C.

K_{20} = rate constant at $20^{o}C$.

θ = temperature correction coefficient.

T = temperature, ^{o}C.

2.1.21.3.3 Determine size of the aeration tank by first determining the detention time.

$$t = \frac{24S_o}{(X_V)\ (F/M)}$$

where

t = detention time, hr.

S_o = influent BOD, mg/l.

X_V = MLVSS, mg/l.

F/M = food-to-microorganism ratio.

2.1.21.3.4 Check detention time for treatability.

$$\frac{S_e}{S_o} = \frac{1}{1 + kX_V t}$$

where

S_e = BOD_5 soluble in effluent, mg/l.

S_o = BOD_5 in influent, mg/l.

k = BOD removal rate constant, l/mg/hr.

X_V = MLVSS, mg/l.

t = detention time, hr.

and solve for t and compare with t in (2.1.21.3.3) above and select the larger.

2.1.21.3.5 Calculate the volume of aeration tank.

$$V = Q_{avg} \frac{t}{24}$$

where

V = volume, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.1.21.3.6 Calculate oxygen requirements.

$$\frac{dO}{dt} = \frac{a'(S_r)}{t} + b'X_V$$

or

$$O_2 = a'(S_r)(Q_{avg})(8.34) + b'(X_V)(V)(8.34)$$

where

$\frac{dO}{dt}$ = oxygen uptake rate, mg/l/hr.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed (S_o - S_e)

t = detention time, hr.

b' = endogenous respiration rate, l/hr.

X_V = MLVSS.

O_2 = oxygen requirement, lb/day.

Q_{avg} = average flow rate, mgd.

V = volume of aeration tank, million gal.

and check the oxygen supplied against 1.25.

$$\text{lb } O_2/\text{lb } BOD_r = \frac{O_2}{Q(S_r)(8.34)}$$

where

O_2 = oxygen required, lb/day.

Q = flow, mgd.

S_r = BOD removed, mg/l.

2.1.21.3.7 Design aeration system.

2.1.21.3.7.1 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for mixing $\geq$ 0.1 hp/1000 gal.

2.1.21.3.7.1.1 Standard transfer efficiency,, lb/hp-hr (0 dissolved oxygen, 20°C, and tap water) (3-5 lb/hp-hr).

2.1.21.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.1.21.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.1.21.3.7.1.4 Correction factor for pressure $\approx$ 1.0.

2.1.21.3.7.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.1.21.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \; \frac{(C_s)_T(\beta)(p) - C_L}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, °C, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

T = temperature, ^oC.

α = 0_2 transfer in waste/0_2 transfer in water.

2.1.21.3.7.4 Calculate horsepower requirement.

$$hp = \frac{0_2}{\text{OTE} \frac{\text{lb } 0_2}{\text{hp-hr}} (24)\ (V)} \times 1000$$

where

hp = horsepower required/1000 gal.

0_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb 0_2/hp-hr.

V = volume of basin, gal.

2.1.21.3.8 Calculate sludge production.

$$\Delta X_V = [aS_rQ_{avg} - bX_V(V) + fQ(VSS) + Q(SS - VSS)](8.34)$$

where

ΔX_V = sludge produced, lb/day.

a = fraction of BOD removed synthesized to cell material.

S_r = BOD removed, mg/l.

Q_{avg} = average flow, mgd.

b = endogenous respiration rate, l/day.

X_V = volatile solids in raw waste, mg/l.

V = volume of tank, gal.

f = nonbiodegradable fraction of influent VSS.

Q = flow, mgd.

VSS = volatile suspended solids in effluent, mg/l.

SS = suspended solids in influent, mg/l.

2.1.21.3.9 Check ΔX_V aginst 0.5-0.7.

$$\frac{\text{lb/ solids}}{(\text{lb BOD}_r)} = \frac{\Delta X_V}{S_r(Q)(8.34)}$$

where

ΔX_V = sludge produced, lb/day.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.1.21.3.10 Calculate sludge recycle ratio.

$$\frac{Q_r}{Q} = \frac{X_a}{X_u - X_a}$$

where

Q_r = volume of recycled sludge, mgd.

Q = flow, mgd.

X_a = MLSS, mg/l.

X_u = suspended solids concentration in returned sludge, mg/l.

2.1.21.3.11 Calculate solids retention time.

$$SRT = \frac{(V)(X_a)(8.34)}{\Delta X_a}$$

where

SRT = solids retention time, days.

V = volume of basin, gal.

X_a = MLSS, mg/l.

$\Delta X_a = \frac{X_V}{\%\ \text{volatile}}$

ΔX_V = sludge produced, lb/day.

2.1.21.3.12 Effluent Characteristics.

2.1.21.3.12.1 BOD_5.

$$BODE = Se + 0.84\ (X_v)_{\text{eff}}{}^{\text{f}'}$$

where

BODE = effluent BOD_5 concentration, mg/l.

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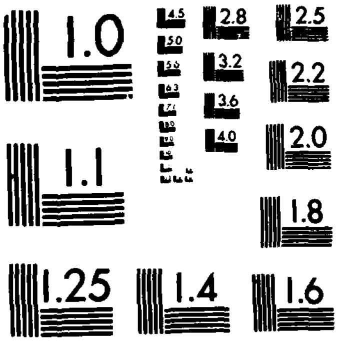
UNCLASSIFIED JAN 82 F/G 5/3 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Se = effluent soluble BOD_5 concentration, mg/l.

$(X_V)_{eff}$ = effluent volatile suspended solids, mg/l.

f' = degradable fraction of MLVSS.

2.1.21.3.12.2 COD.

$$CODE = (1.5)\ (BODE)$$

$$CODSE = (1.5)\ (Se)$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.1.21.3.12.3 *Nitrogen.*

$$TKNE = (0.7)\ TKN$$

$$NH3E = TKNE$$

where

TKNE = effluent total Kjeldahl nitrogen concentration, mg/l.

TKN = influent total Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.1.21.3.12.4 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.1.21.3.12.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.1.21.3.12.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.1.21.3.13 Determine nutrient requirements.

for nitrogen

$$N = 0.123 \Delta M_T \text{ (or } \Delta X_V)$$

and phosphorus

$$P = 0.026 \Delta M_T \text{ (or } \Delta X_V)$$

where

ΔM_T = sludge produced, lb/day.

ΔX_V = sludge produced, lb/day.

and check against BOD:N:P = 100:5:1.

2.1.21.4 Process Design Output Data.

2.1.21.4.1 Aeration Tank.

2.1.21.4.1.1 Reaction rate constant, l/mg/hr.

2.1.21.4.1.2 Sludge produced per BOD removed.

2.1.21.4.1.3 Endogenous respiration rate (b, b').

2.1.21.4.1.4 O_2 utilized per BOD removed.

2.1.21.4.1.5 Influent nonbiodegradable VSS (f).

2.1.21.4.1.6 Effluent degradable VSS (f').

2.1.21.4.1.7 lb BOD/lb MLSS-day (F/M ratio).

2.1.21.4.1.8 Mixed liquor SS, mg/l (MLSS).

2.1.21.4.1.9 Mixed liquor VSS, mg/l (MLVSS).

2.1.21.4.1.10 Aeration time, hr.

2.1.21.4.1.11 Volume of aeration tank, million gal.

2.1.21.4.1.12 Oxygen required, lb/day.

2.1.21.4.1.13 Sludge produced, lb/day.

2.1.21.4.1.14 Nitrogen requirement, lb/day.

2.1.21.4.1.15 Phosphorus requirement, lb/day.

2.1.21.4.1.16 Sludge recycle ratio, percent.

2.1.21.4.1.17 Solids retention time, days.

2.1.21.4.2 Aeration System.

2.1.21.4.2.1 Standard transfer efficiency, lb O_2/hp-hr.

2.1.21.4.2.2 Operating transfer efficiency, lb O_2/hp-hr.

2.1.21.4.2.3 Horsepower required, hp.

2.1.21.5 <u>Quantities Calculations</u>.

2.1.21.5.1 The design values for activated sludge system would be:

$$V_d = V \cdot \frac{10^6}{7.48}$$

$$HP_d = (hp)\ (V)\ (133.7)$$

where

V = volume of aeration basin million gallons.

2.1.21.5.2 Selection of number of aeration tanks and mechanical aerators per tank. The following rule will be utilized in the selection of number of aeration tanks and mechanical aerators per tank.

Q_{avg} (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NT
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.1.21.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.1.21.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.1.21.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.1.21.5.2 by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.1.21.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.1.21.5.4 Mechanical aeration equipment design.

2.1.21.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.1.21.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)(NT)(NA)}$$

If HPN > 150 HP and NT = 2 or 3, then repeat the calculation with NT = NT + 1.

If HPN > 150 HP and NT ≥ 4, then repeat the calculation with NT = NT + 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.1.21.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.1.21.5.5 Design of aeration tanks.

2.1.21.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.1.21.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

$$\text{When HPSN} < 100 \text{ HP}$$

$$DW = 4.816\ (HPSN)^{0.2467}$$

$$\text{When HPSN} \leq 100 \text{ HP}$$

$$DW = 15 \text{ ft}$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.1.21.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.1.21.5.6 Aeration tank arrangements.

2.1.21.5.6.1 Figure 2.1-34 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.1.21.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.1.21.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.1.21.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\,[(2\,W + 18.5)\,(W + 17) + (2\,W + 26.5)\,(W + 25)]$$

where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.1.21.5.7.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\,[(3\,W + 28)\,(W + 25) + (3\,W + 20)\,(W + 17)]$$

2.1.21.5.7.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\,L + PGW + 16$$

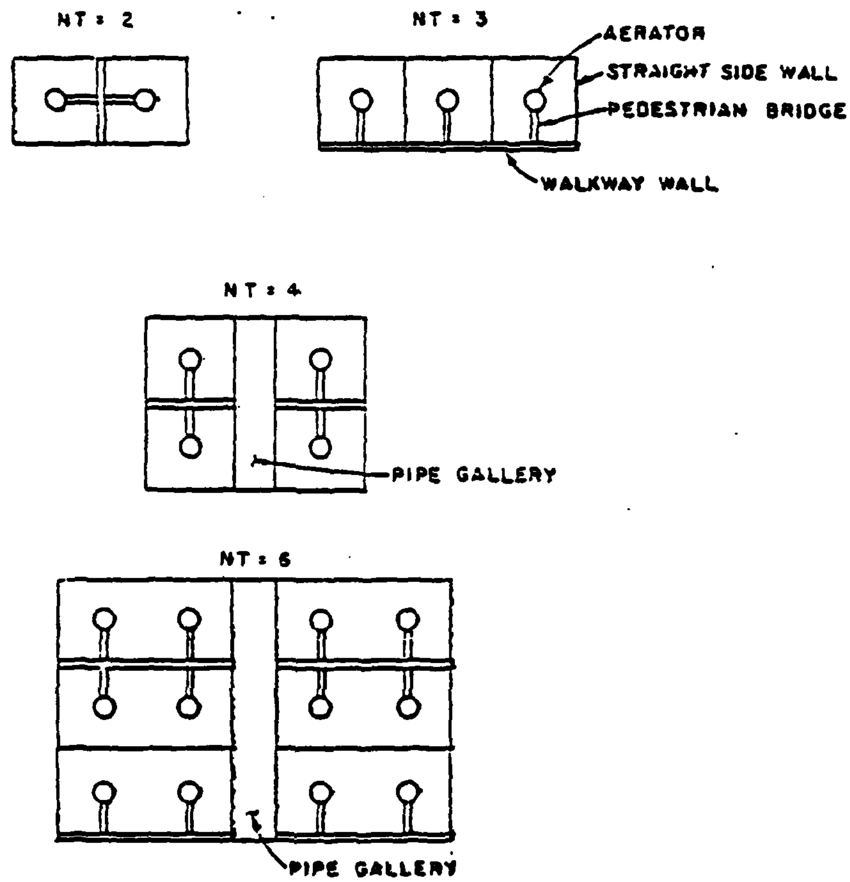
$$W_s = \tfrac{1}{2}\,(NT)\,(W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.1-34 EXAMPLES OF TANK ARRANGEMENTS
ACTIVATED SLUDGE PROCESSES

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB) [(L_s + 4) (W_s + 4) + (L_s + 12) (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.1.21.5.8 Reinforced concrete slab quantity.

2.1.21.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.1.21.5.8.2 For NT = 2,

$$V_{cs} = 1.5 (2 W + 14.5) (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.1.21.5.8.3 NT = 3,

$$V_{cs} = 1.5 (3 W + 16) (W + 13)$$

2.1.21.5.8.4 When NT $\geq$ 4,

$$V_{cs} = 1.5 (L_s) (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.1.21.5.9 Reinforced Concrete Wall Quantity.

2.1.21.5.9.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.1-35. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.1.21.5.9.2 When NT = 2:

$$V_{cw} = W (1.25 DW + 11) + (6 W + 9) (1.25 DW + 3.75)$$

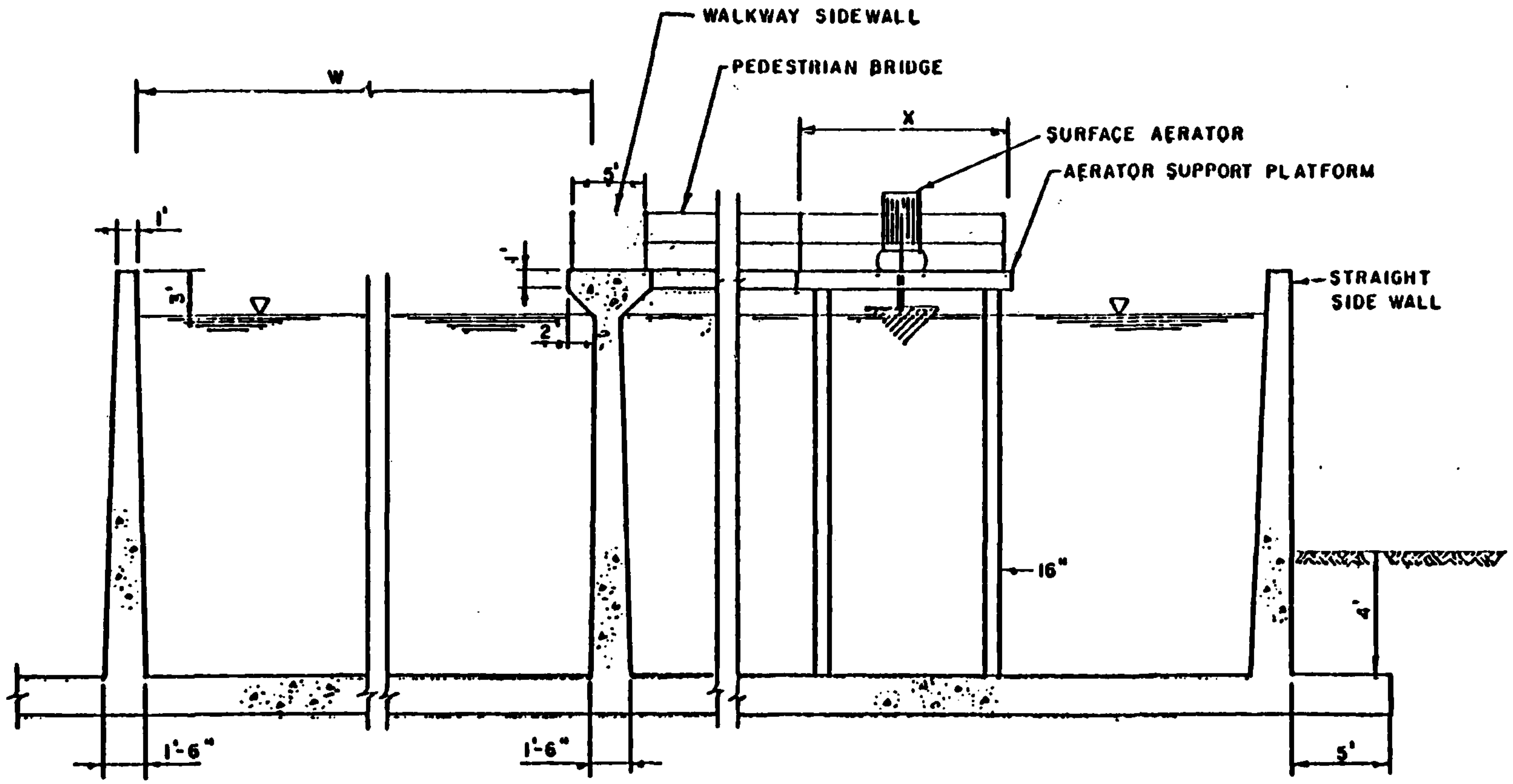


FIGURE 2.1-35 TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

2.1.21.5.9.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.1.21.5.9.4 When NT $\geq$ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

2.1.21.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.1.21.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.1.21.5.10.2 When NT $\geq$ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5)\ (PGW)\ [\frac{(NT)\ (W)}{2} + 0.75\ (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2\ (PGW)\ (NB)\ (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.1.21.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.1.21.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.1.21.5.11.2 Figure 2.1-36 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078 \text{ (HPSN)}$$

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.1.21.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6 \text{ (DW + 2)}] \text{ (NT) (NA) (NB)}$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.1.21.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2 \text{ (W - X)}] \text{ (NB) (NT) (NA)}$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.1.21.5.12 Summary of reinforced concrete structures.

2.1.21.5.12.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

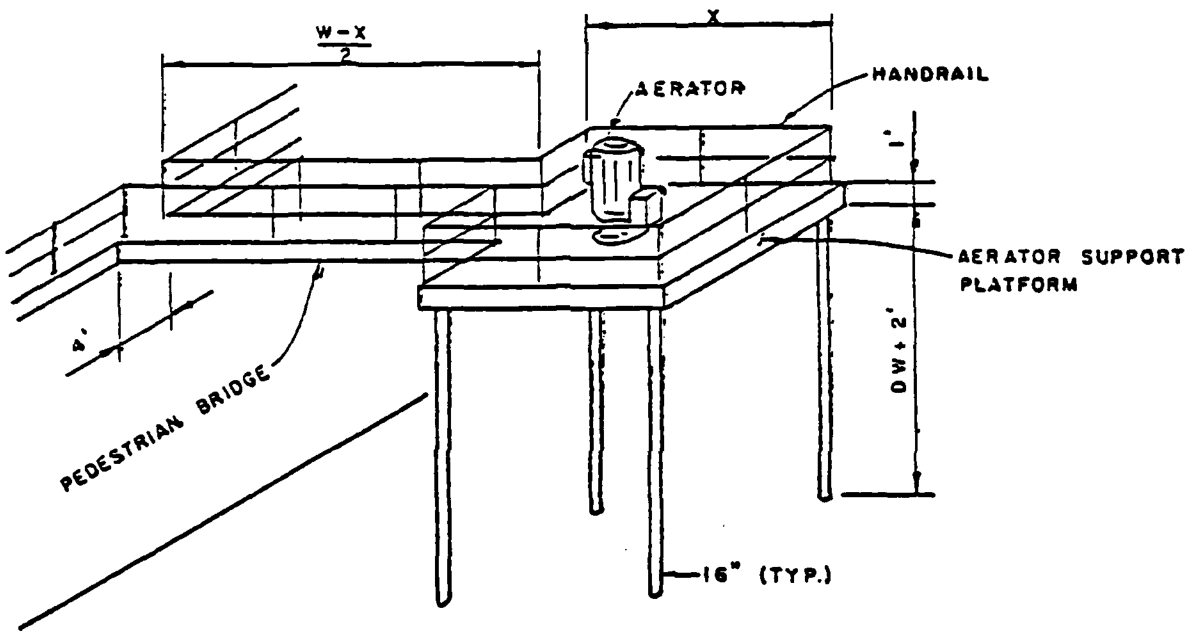


FIGURE 2.1-36 AERATOR SUPPORT PLATFORM

2.1.21.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.1.21.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.1.21.5.13.1 When NT = 2,

$$LHR = 4\ W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.1.21.5.13.2 When NT = 3,

$$LHR = 6\ W + 10 + 3 \cdot (3X + W - 4)$$

2.1.21.5.13.3 When NT $\geq$ 4,

If $\frac{NT}{2}$ is an even number,

$$LHR = \left\{ PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT) + (NA) \cdot (NT) \cdot (3X + W - 4) \right\} \cdot (NB)$$

If $\frac{NT}{2}$ is an odd number,

$$LHR = \left\{PGW + (NT)(W) + [L + 3 - 4(NA)](NT + 2) + (NA)(NT)(3X + w - 4)\right\} \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.1.21.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.1.21.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)(NT)(NA)(HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.1.21.5.14.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\,(TICA)^{0.3731}$$

When TICA ≥ 200 hp

$$OMH = 100\,(TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.1.21.5.14.3 The maintenance manpower requirement can be estimated as follows:

When TICA ≤ 100 hp

$$MMH = 106.3\,(TICA)^{0.4031}$$

When TICA 100 hp

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.1.21.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.1.21.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.1.21.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.1.21.6 Quantities Calculations Output Data.

2.1.21.6.1 Number of aeration tanks, NT.

2.1.21.6.2 Number of aerators per tank, NA.

2.1.21.6.3 Number of process batteries, NB.

2.1.21.6.4 Capacity of each individual aerator, HPSN, hp.

2.1.21.6.5 Depth of aeration tanks, DW, ft.

2.1.21.6.6 Length of aeration tanks, L, ft.

2.1.21.6.7 Width of aeration tanks, W, ft.

2.1.21.6.8 Width of pipe gallery, PGW, ft.

2.1.21.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.1.21.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.1.21.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.1.21.6.12 Quantity of handrail, LHR, ft.

2.1.21.6.13 Operation manpower requirement, OMH, MH/yr.

2.1.21.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.1.21.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.1.21.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.1.21.6.17 Correction factor for minor capital cost items, CF.

2.1.21.7 Unit Price Input Required.

2.1.21.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.1.21.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.1.21.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.1.21.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, \$, optional.

2.1.21.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.1.21.7.6 Equipment installation labor rate, \$/MH.

2.1.21.7.7 Crane rental rate, UPICR, \$/hr.

2.1.21.7.8 Unit price of handrail, UPIHR, \$/L.F.

2.1.21.8 <u>Cost Calculations</u>.

2.1.21.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.1.21.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, \$.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/ cu yd.

2.1.21.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.1.21.8.4 Cost of installed aeration equipment.

2.1.21.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.1.21.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.1.21.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16{,}300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16{,}300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.1.21.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When HPSN $\leq$ 60 hp

$$IMH = 39 + 0.55\ (HPSN)$$

When HPSN > 60 hp

$$IMH = 61.3 + 0.18\ (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.1.21.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.1.21.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.1.21.8.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, dollars.

LABRI = labor rate, dollars/man-hour.

UPICR = crane rental rate, dollars/hr.

2.1.21.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, $ per lineal foot. A value of $25.20 per foot for the first quarter of 1977 is suggested.

2.1.21.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.1.21.8.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, dollars.

CF = correction factor for minor cost items, from second-order design output.

2.1.21.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, $/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.1.21.9 <u>Cost Calculations Output Data</u>.

2.1.21.9.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, dollars.

2.1.21.9.2 Operation and maintenance supply and material costs, OMMC, dollars.

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2.2 BELT FILTER

2.2.1 Background.

2.2.1.1 Belt pressure filters are marketed in the United States by Passavant and Ralph B. Carter Company. The system is widely used in Europe, where it was jointly developed by two firms, Degemont and Phillipe. It is known as the "Flocpress" in Europe. Most of the comments related to filter presses are also applicable to belt pressure filters.

2.2.1.2 In this design the sludge is spread across a woven synthetic fiber belt, which travels horizontally for a variable length where the action of both capillarity and gravity will allow a natural drainage. This belt, after running horizontally on supporting rollers, wraps around a rubber-covered drum provided with grooves for draining away the filtrate. The action of a continuous pressure belt of cloth reinforced rubber subjects a pressure by gradually decreasing the gap between the filter belt and the pressure belt as they move forward, so that the pressure applied increases over the whole length of the filtration zone. The dried cake is then removed from the filter belt by means of a flexible scraper. A second scraper is also needed to clean the pressure belt.

2.2.1.3 Chemical conditioning is required for belt pressure filters just as it is for filter presses. One notable exception is that, while filter presses usually require media conditioning, belt pressure filters do not generally require conditioning. Chemical addition, mixing, flocculation, and gravity thickening are required. In most instances a polymer addition is helpful.

2.2.1.4 There is a tendency for sludge to flow out the edges between the belts and to limit the applied pressure to slightly more than two atmospheres as pressure increases. Because of pressure limitation, the dryness of the cake never approaches that of filter presses. Table 2.2-1 indicates typical operating perfomances of the "Flocpress".

2.2.2 Input Data.

2.2.2.1 Average wastewater flow, mgd.

2.2.2.2 Sludge volume, gallons/million gallons.

2.2.2.3 Raw sludge solids concentration, %.

2.2.2.4 Sludge produced by conditioning chemicals.

Table 2.2-1. Operating Perfomance for the Flocpress

Type of Sludge	Feed Sludge Percent Dry Solids	Output in lbs Per ft of Belt Width Per Hr	Cake Percent Dry Solids	Polymer lbs per Dry Ton
Municipal Sewage				
Raw Primary	5-10	161-268	25-35	1.8-4
Anaerobic Digested - Primary	4-10	168-235	26-36	2.0-6
Mixed Primary and Waste Activated	4-9	87-168	20-28	1.5-6
Anaerobic Digested Primary and Waste Activated	3.5-9	134-268	18-23	3.4-8
Aerobic Digested	1.5-2.5	54-101	12-18	4.0-8
Raw Slaughter House Waste	2.5-3.5	134-302	20-30	5.0-8
Lime Softening Sludge	10-15	335-670	55-70	0.4-1
Alum Sludge	3-6	54-85	14-18	1.5-5
White Water Sludge				
Paper Mill	2.5-4	67-268	20-25	2.0-4
Manufacturer of Semi-Chem Pulp				
Fibers and Sawdust	4-7	134-335	25-32	0
Sawdust	8-15	402-670	30-35	0

2.2.2.5 Cake solids, %.

2.2.2.6 Cake density, lb/ft^3.

2.2.2.7 Belt speed, ft/hr.

2.2.2.8 Loading rate, lb/ft of width/hr.

2.2.2.9 Operating schedule, hr/day.

2.2.3 <u>Design Parameters</u>.

2.2.3.1 Sludge volume per million gallons treated.

2.2.3.2 Raw sludge solids concentration, %. 1.5-15%

2.2.3.3 Conditioning sludge, from laboratory studies. Use 0.2 of sludge solids as an estimate if laboratory data are not available.

2.2.3.4 Type and amount of conditioning chemicals, %, from laboratory studies.

2.2.3.5 Cake solids. (Table 2.2-1).

2.2.3.6 Cake density. Approximately 70-80 lb/ft^3.

2.2.3.7 Belt speed, from laboratory studies. If unknown, use 10-15 ft/hr, or an average of 12 ft/hr.

2.2.3.8 Loading rate, from laboratory studies. If unknown, use 5-8 $lb/ft^2/hr$, or an average of 6.5 $lb/ft^2/hr$.

2.2.3.9 Operating schedule, hours per day. (8-16 hr).

2.2.4 <u>Process Design Calculations</u>.

2.2.4.1 Calculate the pounds of dry solids in sludge flow per day.

$$DSS = \frac{(Q_{avg})(SF)(SS)(8.34)}{100}$$

where

DSS = pounds of dry sludge solids per day.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gallons/million gallons.

SS = suspended solids in flow to pressure filter, %.

2.2.4.2 Calculate the total pounds of dry solids produced per day.

$$PDSPD = DSS + (CS)(DSS)$$

where

PDSPD = dry solids produced daily, pounds.

CS = conditioning solids, expressed as a fraction of sludge solids. (0.2)

2.2.4.3 Calculate the weight of the filter cake produced.

$$PFC = \frac{PDSPD}{CSC} \times 100$$

where

PFC = pounds of filter cake produced per day, net weight.

CSC = cake solids content, %.

2.2.4.4 Determine the cake volume.

$$CV = \frac{PFC}{CD}$$

where

CV = cake volume, ft^3/day.

CD = cake density, lb/ft^3.

2.2.4.5 Calculate the total area of pressure filter needed to dewater the specific sludge at specified loading rate.

$$AF = \frac{PDSPD}{(HPD)(SLR)}$$

where

AF = area of pressure filter, ft^2.

HPD = operating schedule for unit, hours per day.

SLR = solids loading rate, lb/ft^2/hr.

2.2.4.6 Consult manufacturer's literature to select appropriate filter, based on the area calculated and desired dewatering time (typically 3-6 hours) and the belt speed determined from laboratory studies.

2.2.5 Process Design Output Data.

2.2.5.1 Volume of filter cake, ft^3.

2.2.5.2 Cake solids content, %.

2.2.5.3 Weight of filter cake, lb/ft^3.

2.2.5.4 Area of pressure filter, ft^2.

2.2.6 Quantities Calculations. Not Used.

2.2.7 Quantities Calculations Output Data. Not Used.

2.2.8 Unit Price Input Required. Not Used.

2.2.9 Cost Calculations.

2.2.9.1 Calculate operation and maintenance cost.

$$X = \log (Q_{avg})$$

$$Z = 1.3906 - 0.73944(X) + 0.081625(X)^2$$

$$O\&M = \frac{e^Z \; Q_{avg} \times 10^3}{100}$$

where

O&M = operation and maintenance cost, \$/yr.

2.2.9.2 Calculate total bare construction cost.

$$Y = -0.69698 - 0.12594(X) + 0.095578(X)^2$$

$$TBCC = e^Y$$

where

TBCC = total bare construction cost, \$.

2.2.10 Cost Calculations Output Data.

2.2.10.1 Total bare construction cost, TBCC, \$.

2.2.10.2 Operation and maintenance cost, O&M, \$/yr.

2.2.11 Bibliography.

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2.3 BLOWERS

2.3.1 Background. Air is often required in several places in a sewage treatment facility. For this reason it is treated as a separate process. The capacity required will be determined by summing all the air requirements from the various processes; from this, the number and capacity of blowers required will be determined.

2.3.2 Input Data. The input data for this design consists of the output blower capacity from the quantities calculations for Activated Sludge Processes: Diffused Aeration System and Aerobic Sludge Digestion: Diffused Aeration.

2.3.2.1 Design capacity of blowers, from quantities calculations output of Activated Sludge Processes: Diffused Aeration System, CFM_d, scfm.

2.3.2.2 Design capacity of blowers, from quantities calculations output of Aerobic Sludge Digestion: Diffused Aeration, CFM_d, scfm.

2.3.3 Process Design Calculations. There will be no process design calculations as such since it has already been accomplished in other sections.

2.3.4 Quantities Calculations.

2.3.4.1 The sum of the air requirements from the two processes shown above represent virtually 100% of the air requirements and will be used as the design capacity (TCFM) for the blowers.

TCFM = design capacity for blower system, scfm.

2.3.4.2 Select type, number and capacity of blowers. For the purposes of this design, all blowers will be assumed to operate at 8 psig. For ease of maintenance all blowers in a single installation will be the same size. Also it is standard policy that blowers shall be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand with the single largest unit out of service. There are a number of different types of blowers which have been used to supply air for sewage treatment facilities. Each type serves a particular need. Whichever one is chosen for a given set of circumstances should be determined by a thorough economic and engineering study. For the purposes of this design the following guidelines will be used for selection.

Total Air Requirement (TCFM)	Blower Type
0 - 30,000	Rotary positive blowers.
30,000 - 72,000	Vertically-split multistage centrifugal
> 72,000	Pedestal-type single-stage centrifugal

2.3.4.2.1 $0 < TCFM \leq 30{,}000$ scfm. As shown above in this range, rotary positive blowers will be used. The number of blower (N) can be 1, 2, 3, or 4. The number of blowers will be selected by trial and error. Start with N = 1. If TCFM/N 7500 scfm go to N = N + 1 and repeat until TCFM/N 7500 scfm. This will give the number of blowers to provide the maximum air requirements. The total number of blowers (NB) will be N + 1.

$$CFMB = \frac{TCFM}{N}$$

$$NB = N + 1$$

where

CFMB = capacity of individual blowers, scfm.

N = number of blowers needed to provide maximum air requirements.

NB = total number of blowers required.

2.3.4.2.2 $30{,}000 \text{ scfm} < TCFM \leq 72{,}000$. In this range, vertically-split multistage centrifugal blowers will be utilized. The number of blowers can be 1, 2, 3 or 4. The number of blowers will be selected by trial and error. Start with N = 1. If TCFM/N 18,000 scfm go to N = N + 1 and repeat until TCFM/N 18,000 scfm. This will give the number of blowers to provide the maximum air requirements. The total number of blowers (NB) will be N + 1.

$$CFMB = \frac{TCFM}{N}$$

$$NB = N + 1$$

2.3.4.2.3 $TCFM > 72{,}000$. In this range, pedestal-type single-stage centrifugal blowers will be used. The number of blowers will be determined by trial and error. Start with N = 1. If TCFM/N 100,000 scfm go to N = N + 1 and repeat the trial; until TCFM/N 100,000 scfm. This will give the number of blowers to provide the maximum air requirements. The total number of blowers (NB) will be N + 1.

$$CFMB = \frac{TCFM}{N}$$

$$NB = N + 1$$

2.3.4.3 Determine blower building area required. The blower building area can be related to the blower capacity as follows:

$$BBA = 128.0\ (TCFM)^{0.256}$$

where

BBA = blower building area, sq ft.

2.3.4.4 Operation and maintenance manpower requirements. The operation and maintenance manpower requirements have been included in the individual processes which require air for operation, therefore it will not be included here.

2.3.4.5 Electrical energy requirement for operation. As with the O&M manpower this has been included in the processes which require air for operation.

2.3.4.6 Operation and maintenance material and supply costs. This, too, has been included in the processes which require air for operation.

2.3.4.7 Other construction cost items. The majority of the costs for the blower installation have been accounted for. Other cost items, such as painting, site cleaning and preparation, etc., can be estimated as 10 percent of the total bare construction cost.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction cost items.

2.3.5 <u>Quantities Calculation Output Data</u>.

2.3.5.1 Design capacity of blower system, TCFM, scfm.

2.3.5.2 Total number of blowers required, NB.

2.3.5.3 Capacity of individual blowers, CFMB, scfm.

2.3.5.4 Blower building area, BBA, sq ft.

2.3.5.5 Correction factor, CF.

2.3.6 <u>Unit Price Input Required</u>.

2.3.6.1 Cost of standard blower, COSTSB, $ (optional).

2.3.6.2 Current Marshall and Swift Equipment Cost Index, MSECI.

2.3.6.3 Unit price input for building cost, UPIBC, $/sq ft.

2.3.7 <u>Cost Calculations</u>.

2.3.7.1 Purchase cost of blowers and accessories. The purchase cost includes the blower, electric motor, silencers and inlet vanes, if required.

$$COSTBE = \frac{(COSTSB)\ (COSTRO)}{100}$$

where

COSTBE = purchase cost of blower of CFMB capacity, $.

COSTSB = purchase cost of standard blower, $.

COSTRO = purchase cost of blower of CFMB capacity as percent of cost of standard blower, percent.

2.3.7.2 Calculate COSTRO.

2.3.7.2.1 If $0 < TCFM \leq 30{,}000$ scfm, the rotary positive blowers will be used and COSTRO is calculated by:

$$COSTRO = 0.70\ (CFMB)^{0.6169}$$

where

CFMB = individual blower capacity, scfm.

2.3.7.2.2 If $30{,}000 < TCFM \leq 72{,}000$ scfm the vertically-split multistage centrifugal blowers will be used and COSTRO is calculated by:

$$COSTRO = 0.377\ (CFMB)^{0.5928}$$

2.3.7.2.3 If $TCFM > 72{,}000$ scfm the pedestal-type single-stage blowers will be used and COSTRO is calculated by:

$$COSTRO = 0.964\ (CFMB)^{0.4286}$$

2.3.7.3 Determine purchase cost of the standard blower.

2.3.7.3.1 If $0 < TCFM \leq 30{,}000$ scfm, the standard size blower is a rotary positive blower with a capacity of 3,000 scfm at 8 psig.

COSTSB = $16,000

2.3.7.3.2 If $30{,}000 < TCFM \leq 72{,}000$ scfm the standard size blower is a vertically-split multistage centrifugal blower with a capacity of 12,000 scfm at 8 psig.

COSTSB = $45,000

2.3.7.3.3 If $TCFM > 72{,}000$ scfm the standard size blower is a pedestal-type single-stage centrifugal blower with a capacity of 50,000 scfm at 8 psig.

$$COSTSB = \$300{,}000$$

2.3.7.3.4 For better cost estimation, COSTSB should be obtained from an equipment vendor and treated as a unit price input. However, if it is not treated as a unit price input the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSB = COSTSB \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter 1977.

2.3.7.4 Blower Installation Costs. Typically the installation cost of blowers is approximately 100% of the equipment cost. This includes costs of piping, concrete, steel, electrical, paint and installation labor. Therefore, the installed blower cost is given by:

$$IBC = COSTBE \times 2.0$$

where

IBC = installed blower costs, \$.

2.3.7.5 Blower building costs.

$$BBC = (BBA)\ (UPIBC)$$

where

BBC = blower building costs, \$.

BBA = blower building area, sq ft.

UPIBC = unit price input for building costs, dollars/sq ft.

2.3.7.6 Total bare construction cost.

$$TBCC = [(IBC)\ (NB) + BBC]\ CF$$

where

TBCC = total bare construction cost, \$.

NB = total number of blowers.

CF = correction factor for other construction cost items, from second-order design output.

2.3.8 Cost Calculations Output Data.

2.3.8.1 Total bare construction cost of blowers, TBCC, $.

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2.5 CARBON ADSORPTION

2.5.1 Background.

2.5.1.1 Activated carbon, when contacted with water containing organic material, will remove these compounds selectively by a combination of adsorption of the less polar molecules, filtration of the larger particles, and partial deposition of colloidal material on the exterior surface of the activated carbon. Adsorption results from the forces of attraction between the surface of a particle and the soluble organic materials that contact the particle. As a result of the activation process, activated carbon has numerous pores within each particle and hence a large surface area per unit weight, making it a very efficient adsorptive material. It has long been used to remove taste and odor-causing impurities from public water supplies. More recently, activated carbon adsorption has been used in wastewater treatment as a tertiary process following conventional secondary treatment or as one of several unit processes comprising physical-chemical treatment.

2.5.1.2 The most efficient and practical use of activated carbon in wastewater treatment has been in fixed beds of granular activated carbon. A typical adsorption system consists of several adsorption trains operated in parallel. Each train contains two adsorbers arranged for series flow. The wastewater is applied to the adsorbers at a flow rate ranging from 4 to 8 gpm/ft^2. Contact time (empty bed residence time) ranges from 15 to 35 minutes depending on the desired effluent quality.

2.5.1.3 To ensure minimum suspended solids collection in the adsorbers which can clog the pores and reduce adsorber capacity, the carbon adsorption system should be preceded by media filtration. Provisions must be made to regularly backwash the adsorption system to flush out accumulated suspended solids and biological growth. A good design practice is to allow for a bed expansion of from 10 to 15 percent. Flow rates during backwash should range from 10 to 15 gpm/ft^2. Biological growth can be controlled effectively by chlorination of the influent to the adsorber or by chlorination during the backwash operation.

2.5.1.4 When the active sites on the carbon particles have been filled, the effluent quality deteriorates and the carbon must be regenerated or replaced. For systems requiring regeneration of less than 400 pounds of carbon per day, it is not economical to have on-site regeneration. For larger systems, a regeneration system should be provided. A typical regeneration system would include a) hydraulic transport of the carbon to the regeneration unit, b) dewatering of spent carbon, c) heating of carbon to oxidize or volatilizate the adsorbed impurities, d) water cooling of the carbon, e) water washing and hydraulic transport back to the adsorbers, and f) scrubbing of furnace off-gasses. The most common type of furnace in use is the multiple hearth furnace.

2.5.1.5 The following sections present a design for a packed bed downflow carbon adsorption system that includes on-site carbon regeneration. Input such as the minimum contact time, optimum flow rate, head loss at various flows, backwash rate, and required carbon dosage should be obtained from on-site pilot plant carbon column tests. Where this is not possible, accepted design criteria should be used to generate the required input data. Static isotherm tests conducted in the laboratory are not sufficient.

2.5.2 Input Data.

2.5.2.1 Peak wastewater flow, Q_p, mgd.

2.5.2.2 Average flow, Q_{avg}, mgd.

2.5.2.3 Design superficial velocity, V_s, gpm/ft^2 (usually 6 gpm/ft^2).

2.5.2.4 Empty bed residence time at Q_p, T_R, minutes, (25 minutes).

2.5.2.5 Elevation of adsorber underdrain above adsorber bottom, Hu, ft, (usually 2 ft.)

2.5.2.6 Chemical oxygen demand (COD) of wastewater feed, Co, mg/l.

2.5.2.7 COD of carbon treated wastewater, C_f, mg/l.

2.5.2.8 COD/carbon loading at saturation, x/m, lb/lb. Typical value for domestic wastewater are in the range of 0.3-0.8 lbCOD/lb of Carbon.

2.5.2.9 Length to width ratio for concrete absorbers, R_{LW}.

2.5.2.10 Furnace loading rate, F, lb/day/ft^2.

2.5.3 Process Design Calculations.

2.5.3.1 Required Adsorber Characteristics

2.5.3.1.1 The required cross sectional surface area is given by:

$$A = \frac{694\ Q_p}{V_s}$$

where

A = Required cross sectional area, ft^2

Q_p = Peak flow rate, mgd

V_S = Superficial velocity, gpm/ft^2

If an optimum superficial velocity is not available from pilot studies, a value of 6 gpm/ft^2 should be used.

2.5.3.1.2 Number of absorption Trains Required. For peak flows of 10 mgd or less, pressurized downflow adsorbers constructed of steel should be used. The maximum diameter of steel adsorbers is 16 feet.

For peak flows greater than 10 mgd, gravity flow rectangular concrete adsorbers are recommended.

A minimum of two adsorption trains are required for any carbon adsorption system. Table 2.5-1 lists the number and type of adsorption trains recommended for various flow rates.

TABLE 2.5-1

Peak Flow Rate (Q_p) (mgd)	Number of Trains (N) Pressurized Adsorbers Steel Construction	Number of Trains (N) Gravity Adsorbers Concrete Construction
0.5	2	
1	2	
2	2	
4	4	
8	6	
10		4
20		4
50		4
100		8
150		8
200		10
250		12
300		12

2.5.3.1.3 The surface area of an individual adsorber is calculated by

$$A_A = \frac{A}{N}$$

where

A_A = Adsorber cross sectional area, ft^2

A = Total cross sectional area, ft^2

N = Number of trains required

2.5.3.1.4 Since there are two adsorbers in each adsorption train, the total number of adsorbers is given by:

$$N_A = 2N$$

where

N = number of adsorption trains

N_A = number of adsorbers

2.5.3.2 Adsorption Bed Depth

The overall bed depth is given by:

$$H_B = \frac{V_s T_r}{7.48}$$

where

H_B = overall adsorber bed depth, (ft)

T_r = empty bed residence time, (minutes)

V_s = superficial velocity at Q_p, (gpm/ft^2)

The empty bed residence time usually ranges from 15 to 35 minutes.

A T_r value of 25 minutes is suggested if not specified.

2.5.3.3 Backwash Requirement. Each adsorber should be backwashed daily at a maximum superficial velocity of 15 gpm/ft^2 and a minimum superficial velocity of 12 gpm/ft^2. Backwash should last 15 minutes.

2.5.3.3.1 Calculate Backwash Flow Rate

$$Q_{BW} = \frac{V_{SBW}\ A_A}{2}$$

where

Q_{BW} = Backwash flow rate (gpm)

V_{SBW} = Superficial velocity of backwash (gpm/ft^2)

A_A = Adsorber cross sectional area (ft^2)

2.5.3.3.2 Calculate Backwash Volume

$$V_{BW} = T_{BW} Q_{BW} N$$

where

V_{BW} = Total volume of backwash per day (gpd)

T_{BW} = Backwash duration (minutes)

Q_{BW} = Backwash flow rate (gpm)

N = Number of absorbers

2.5.3.4 Total Carbon Inventory. The total carbon inventory in the system is figured as first fill plus 10% and can be calculated by using the following formula:

$$Q_c = 1.10\ (A \times H_B) \times C_D$$

where

Q_c = Total carbon inventory in the system, pounds

A = Required cross sectional area, ft^2

H_B = Overall adsorber bed depth, ft

C_D = Virgin carbon density, 26 lb/ft^3

2.5.3.5 Carbon Regeneration Requirement. In a two-stage adsorption train, water is passed through adsorber A, the lead adsorber and then adsorber B, the polish adsorber. When the carbon in adsorber A is exhausted, the carbon in adsorber B is only partially spent. The carbon is adsorber A is removed for regeneration and replaced with fresh carbon. Adsorber B then becomes the lead adsorber and adsorber A becomes the polish adsorber. When adsorber B is completely exhausted, it too is regenerated and adsorber A becomes the lead adsorber again.

2.5.3.5.1 Calculate Adsorber Life.

$$T_s = \frac{(x/m)\,(A)\,(H_B/2)\,(C_D)}{8.345(C_o - C_f)\,(Q_{avg})}$$

where

T_s = Adsorber service life, days

A = Total adsorber cross sectional area, ft^2

C_D = Virgin carbon density, 26 lb/ft^3

C_o = Influent COD, mg/l

C_f = Effluent COD, mg/l

Q_{Avg} = Average flow rate, mgd

x/m = COD/carbon loading, lb/lb

H_B = Bed length of adsorber train, ft (length of two adsorbers)

2.5.3.5.2 Calculate Regeneration Rate

$$R = \frac{(H_B/2)(A)(C_{SD})}{T_s}$$

where

H_B = Bed length of adsoprtion train, ft

A = Total adsorber cross sectional area, ft^2

C_{SD} = Spent carbon density, 32 lb/ft^3

T_s = Adsorber service life, days

2.5.3.6 Effluent Characteristics. The effluent COD is specified by the user.

2.5.3.6.1 BOD_5.

$$EBOD = .5\ (C_f)$$

$$EBODS = EBOD$$

where

EBOD = effluent BOD_5, mg/l.

C_f = effluent COD, mg/l.

EBODS = effluent soluble BOD_5, mg/l.

2.5.3.6.2 CODS.

$$CODS = C_f$$

where

CODS = effluent soluble COD, mg/l.

C_f = effluent COD, mg/l.

2.5.3.6.3 Suspended Solids.

$$TSSE = (.01)(TSSI)$$

where

TSSE = total suspended solids in effluent, mg/l.

TSSI = total suspended solids in influent, mg/l.

2.5.3.6.4 Oils and grease.

$$OAG = 0.0$$

where

OAG = oil and grease in effluent, mg/l.

2.5.3.6.5 Settleable solids.

$$SETSO = 0.0$$

SETSO = settleable solids in influent, mg/l.

2.5.4 <u>Process Design Output Data</u>.

2.5.4.1 Total required cross sectional area, A, (ft^2).

2.5.4.2 Total number of adsorption trains, N.

2.5.4.3 Total number of adsorbers, N_A.

2.5.4.4 Individual adsorber cross sectional area, A_A, (ft^2).

2.5.4.5 Overall adsorber bed depth, H_B, (ft).

2.5.4.6 Backwash flow, Q_{BW}, (gpm).

2.5.4.7 Backwash volume, V_{BW}, (gal).

2.5.4.8 Total carbon inventory in the system, Q_c, (pounds).

2.5.4.9 Adsorber service life, T_s, (days).

2.5.4.10 Regeneration rate, R, (lbs/day).

2.5.4.11 Effluent BOD_5, EBOD, mg/l.

2.5.4.12 Effluent COD, C_f, mg/l.

2.5.4.13 Effluent soluble BOD_5, EBODS, mg/l.

2.5.4.14 Effluent soluble COD, CODS, mg/l.

2.5.4.15 Total suspended solids in effluent, TSSE, mg/l.

2.5.4.16 Oil and grease in effluent, OAG, mg/l.

2.5.4.17 Settleable solids in effluent, SETSO, mg/l.

2.5.5 Quantities Calculations.

2.5.5.1 Calculate adsorber diameter for 0.5 mgd $\leq Q_p <$ 10 mgd

$$D = \left(\frac{4A_A}{\pi}\right)^{0.5}$$

where

D = Adsorber diameter, ft

A_A = Individual absorber cross sectional area, ft^2.

The actual adsorber diameter should be rounded to the nearest foot.

2.5.5.2 Calculate adsorber length and width for 10 mgd $\leq Q_p \leq$ 300 mgd

$$W = \left(\frac{A_A}{R_{LW}}\right)^{0.5}$$

$$L = W\,R_{LW}$$

where

L = Inside length of gravity adsorber, ft

W = Inside width of gravity adsorber, ft

R_{LW} = Length to width ratio

The actual adsorber dimensions should be rounded to the nearest foot.

2.5.5.3 Calculate column height

$$H_c = H_v + \frac{1.4\,H_B}{2} + 1$$

$$H_c = 3 + 0.7H_B$$

where

H_v = Height of underdrain - 2 feet is sufficient

H_c = Height of one adsorber, ft

H_B = Overall carbon bed depth, ft (2 adsorbers)

2.5.5.4 Regeneration System Size. The decision as to whether to regenerate and reuse the carbon or to use it on a once through, throwaway basis is strictly based on economics. Based on current cost of multi-hearth furnace regeneration system, for plants requiring less than approximately 400 lbs/day of carbon, on-site regeneration is not economical because of the capital cost of minimum size equipment of this type. In this design procedure, a carbon regeneration rate, R, of 400 pounds per day is, thus, chosen as a demarcation point. When R is less than 400 ppd, no regeneration system is provided. Whereas, when R $\geq$ 400 ppd, the following design procedure is followed.

The spent carbon and reactived carbon storage tanks are usually sized to accommodate from 5 to 10 days supply of carbon to keep the system in operation during routine maintenance or unscheduled shutdowns. The furnace feed tank is designed to hold enough carbon for one shift of operation. The furnace feed tank controls the flow of the carbon slurry to a dewatering screw which dewaters the carbon to 40 to 50 percent water and controls the flow of carbon to the furnace. The furnace can be either a rotary kiln or multiple hearth type. For this design a six-hearth multiple hearth furnace is utilized. Two furnaces are recommended so that operation is not interrupted during maintenance shutdowns, however, one is usually used in municipal systems. Temperatures in the hearths are as follows: No. 1, 800°F, No. 2, 1000°F, No. 3, 1200°F, No. 4, 1680°F, No. 5, 1600°F, and No.6, 1680°F. Steam is introduced on the bottom three hearths at the rate of one pound steam per pound of dry carbon to give a more uniform distribution of temperatures throughout the furnace and to improve the quality of the regenerated carbon. From the furnace the carbon flows to a quench tank that both wets the reactivated carbon and provides a water seal for the furnace bottom. The gases driven off during the themal regeneration pass through an afterburner which burns the volatile and noxious gases at approximately 1200°F. A wet scrubber removes carbon dust and odorous substances.

2.5.5.4.1 The effective hearth area of the regeneration furnace is given by the following equation. Excess capacity of 50% should be provided.

$$A_R = 1.5 \frac{R}{F_R}$$

where

A_R = Effective hearth area, ft^2

R = Regeneration rate, lbs/day

F_R = Furnace loading rate, 70-80 $lbs/ft^2/day$

2.5.5.4.2 Table 2.5-2 lists standard size 6-hearth furnaces. The desired furnace size is chosen from this table.

TABLE 2.5-2

AVAILABLE MULTIHEARTH FURNACES

ID (ft)	OD (ft)	Approximate Effective Hearth Area (ft^2)
2.5	3.70	24
3.25	4.75	37
4.5	6.00	75
5.5	7.75	125
7.0	9.25	179
8.5	10.75	269
10.5	12.75	436
12.0	14.25	548
14.5	16.75	732
16.5	18.75	1,050
18.0	20.25	1,268
20.0	22.25	1,509

The largest furnace commercially available has 1509 ft^2 of hearth area. If A_R is less than 1509 ft^2, one furnace would be used,

$$NFN = 1 \text{ and } ARI = A_R,$$

and the inside diameter of the furnace is

$$ID = 0.397(ARI)^{0.541}$$

where

ARI = Hearth area of each individual furnace, ft^2

ID = Inner diameter of furnace, ft

A_R = Effective hearth area

NFN = Number of furnaces

When A_R is greater than 1509 ft^2, two furnace would be used,

$$NFN = 2,$$

and each furnace would have a hearth area of

$$A_{RI} = AR/2.$$

The internal diameter of each unit is:

$$ID = 0.397\ (A_{RI})^{0.541}$$

2.5.5.4.3 Spent Carbon Storage Tank

The required volume for spent carbon storage is given by

$$VOLSC = \frac{N_D R}{C_{SD}}$$

where

VOLSC = Volume of spent carbon storage tank, ft^3

N_D = Number of days of storage required, 5 to 10 days

R = Regeneration rate, lb/day

C_{SD} = Spent carbon density, 32 lb/ft^3

2.5.5.4.4 Reactivated Carbon Storage Tank

The required volume for reactivated carbon storage is given by:

$$VOLRC = \frac{N_D R}{C_D}$$

where

VOLRC = Volume of reactivated carbon storage tank, ft^3

N_D = Number of days of storage required, 5 to 10 days

R = Regeneration rate, lb/day

C_D = Reactivated carbon density, 26 lb/ft^3

2.5.5.5 Quantities of material associated with adsorption system. A parametric cost estimating system would be used to figure the capital cost of the pressurized adsorption system. The only design output from this section would be the surface area of building to house the adsorption system. Thus:

2.5.5.5.1 Area Requirement for Adsorber Building. The adsorber building will house adsorbers, pumping systems, and control system.

for $0.5 < Q_p \leq 10$ mgd

$$A_B = 2125 \times Q_p^{0.675} + 400\ Q_p^{0.35}$$

for $10 < Q_p \leq 300$ mgd, the adsorber system would be out doors. The only building area required is for the control room and is calculated by

$$A_B = 600\ Q_p^{0.35}$$

where

A_B = Building area for adsorption system, ft^2

Q_p = Peak flow rate, mgd

2.5.5.5.2 For an adsorption system with capacity of $10 < Q_p \leq 300$ mgd, the adsorbers will be constructed of reinforced concrete utilizing commonwall construction. Sluiceways will be integrated into the adsorber construction and utilized to transport wastewater and three-foot wide sluicegates used to isolate adsorbers. Flow through the adsorbers will be by gravity.

2.5.5.5.2.1 Reinforced concrete volume required, V_c

$$V_c = V_{c1} + V_{c2}$$

where

V_c = Volume of concrete required, yd^3

V_{c1} = Volume of concrete for adsorbers

V_{c2} = Volume of concrete for spent carbon storage

$$V_{c1} = 17.8 \times (T_R \times Q_p)^{0.712}$$

and

$$V_{c2} = 37.9 \times Q_p^{0.385}$$

2.5.5.5.2.2 Earthwork V_E

$$V_E = 146.5\ Q_p + 68.7\ Q_p^{0.599}$$

where

V_E = Volume of earthwork, yd^3

Q_p = Peak flow rate, mgd

2.5.5.5.2.3 Calculate adsorber cross sectional area for underdrain and troughs.

$$A_1 = 232\ Q_p$$

where

A_1 = Underdrain and trough area required, ft^2

Q_p = Peak flow rate, mgd

(d) Calculate the number of flow control gates (A 3 x 6 ft gate with operator and frame).

$$G = 4N$$

where

G = Number of gates required

N = Number of trains

2.5.5.6 Adsorption System O&M Requirements. The O&M requirements and costs for the carbon adsorption system excluding carbon regeneration can be related to the average flow rate (Q_{AVG}), given in million gallons per day (mgd), or to the total capital cost of the system.

2.5.5.6.1 Man-hour requirements, man-hours per year

$$MHAD = 860\ Q_{AVG}^{0.473}$$

2.5.5.6.2 Electricity requirements, KWH per year

$$KWHAD = 67{,}000\ Q_{AVG}^{1.02}$$

2.5.5.6.3 Maintenance materials requirement, percent of total cost

for 0.5 mgd < Q_{AVG} ≤ 4 mgd

$$MMAD = 0.55 - 0.664 \log_{10} Q_{AVG}$$

for 4 mgd < Q_{AVG} ≤ 20 mgd

$$MMAD = 0.193 - 0.0715 \log_{10} Q_{AVG}$$

for Q_{AVG} > 20 mgd

MMAD = 0.1% of total capital costs

MMAD = 0.1

where

MMAD = percentage of total cost of the adsorption system for each flow range

2.5.5.7 Regeneration System O&M Requirements. When R 400 lbs/day, on-site regeneration is recommended and the following O&M requirements must be included in the cost of the carbon adsorption system. If R 400 lbs/day, the spent carbon must be replaced and the only O & M cost incurred is the cost of replacement carbon.

The O & M requirements and costs for carbon regeneration can be related to the regeneration rate (R), lbs/day, the total hearth area (A_R), ft^2, or the total capital costs of the system.

2.5.5.7.1 Man-hour requirements, man-hours per year

for $R < 400$ lbs/day

$$MHRG = 0$$

for $R \geq 400$ lbs/day

$$MHRG = 90.46 A_R^{0.624}$$

2.5.5.7.2 Electricity requirements, KWH per year

for $R < 400$ lbs/day

$$KWHRG = 0$$

for $R \geq 400$ lb/day

$$KWHRG = 3110 R^{0.313}$$

2.5.5.7.3 Steam requirements, lbs per year

for $R < 400$ lb/day

$$STRG = 0$$

for $R \geq 400$ lb/day

$$STRG = 365\ (C)\ (R)$$

where

C = 0.6 lb steam per lb carbon

2.5.5.7.4 Fuel requirements, BTU per year for $R < 400$ lb/day

$BTURG = 0$

for $R \geq 400$ lb/day

$BTURG = 3{,}942{,}000{,}000\ R^{0.904}$

2.5.5.7.5 Carbon makeup requirement, lbs per year

for $R < 400$ lb/day

$CMRG = 365 \times R$

for $R \geq 400$ lb/day

$CMRG = 25.6R$

2.5.5.7.6 Maintenance materials requirement, percent

for $R < 400$ lb/day

$MMRG = 0$

for $R \geq 400$ lb/day

MMRG = 3% of total capital cost of regeneration system

$MMRG = 3$

2.5.6 Quantities Calculations Output Data.

2.5.6.1 Adsorber diameter, when $Q_p < 10$ mgd, D, (ft)

2.5.6.2 Adsorber length, when $Q_p \geq 10$ mgd, L, (ft)

2.5.6.3 Adsorber width, when $Q_p \geq 10$ mgd, W, (ft)

2.5.6.4 Adsorber column height, H_c, (ft)

2.5.6.5 Effective regeneration hearth area, A_R, (ft^2)

2.5.6.6 Individual hearth area, ARI, (ft^2)

2.5.6.7 Number of furnace required, NFN

2.5.6.8 Furnace inside diameter, ID, (ft)

2.5.6.9 Adsorption system building requirement, A_B, (ft^2)

2.5.6.10 Reinforce concrete quantity for $Q_p \geq 10$ mgd, V_c, (yd^3)

2.5.6.11 Earthwork volume, $Q_p \geq 10$ mgd, V_E, (yd^3)

2.5.6.12 Underdrain surface area, A_1, (ft^2)

2.5.6.13 Number of control gates, G

2.5.6.14 Man-power requirement for adsorption system O & M, MHAD

2.5.6.15 Electricity required for adsorption system O & M, KWHAD, (KWH/ yr)

2.5.6.16 Maintenance, material supply cost as percentage of adsorption system capital cost, MMAD, (%)

2.5.6.17 Man-power requirement for adsorption system O & M, MHRG, (MH-/ yr).

2.5.6.18 Steam requirement for carbon regeneration, STRG, (lbs/ yr)

2.5.6.19 Fuel requirement for carbon regeneration, BTURG, (BTU/ yr)

2.5.6.20 Virgin carbon makeup, CMRG, (lbs/ yr)

2.5.6.21 Maintenance material supply cost as percentage of regeneration capital cost, MMRG, (%)

2.5.7 Unit Price Input Required.

2.5.7.1 Standard size multiple hearth regeneration system cost, COSTMH, dollars (optional).

2.5.7.2 Unit price input for building construction cost, UPIBC, $/sq ft.

2.5.7.3 Unit price input for R.C. wall, UPICW, $/cu yd

2.5.7.4 Unit price input for earthwork, UPIEX, $/cu yd.

2.5.7.5 Unit price input for virgin carbon purchase, UPICC, $/ lbs.

2.5.7.6 Unit price input for underdrain system, UPIUD, $/sq ft (optional).

2.5.7.7 Unit price input for control gate, UPICG, $, (optional).

2.5.7.8 Current Marshall and Swift Equipment Cost Index, MSECI.

2.5.8 Cost Calculations.

2.5.8.1 Capital cost estimate of the pressurized adsorption system, (Q_p < 10 mgd).

2.5.8.1.1 Calculate Adsorber Cost. Each adsorber would be constructed of steel with an internal epoxy coating. An underdrain would be supplied with support/backwash distribution nozzles on nine-inch centers. Each vessel would be designed for 50 psi pressure (actual max. pressure is not to exceed 30 psi). Shut-off valves for influent, effluent and carbon removal are included. Backwash control valves are not included in the adsorber cost.

$$\text{Cost A1} = 133{,}000\ Q_p^{0.587}$$

where

Cost A1 = Adsorber cost, installed, 0.5 to 10 mgd, $.

2.5.8.1.2 Calculate wastewater feed system cost. This cost includes wet well, dry well, pumps and piping.

$$\text{Cost F1} = 35{,}500\ Q_p^{0.6}$$

where

Cost F1 = wastewater feed system cost, 0.5 to 10 mgd, $.

Q_p = Peak flow, mgd.

2.5.8.1.3 Calculate cost of Adsorber Backwash System. The backwash system consists of a wetwell to contain the full backwash volume, V_{BW}, dry well, and backwash pumps. Piping, valves and fittings are also included.

$$\text{Cost B1} = 30{,}000\ Q_p^{0.38}$$

where

Cost B1 = Backwash system cost, 0.5 to 10 mgd, $

Q_p = Peak flow, mgd

2.5.8.1.4 Calculate cost of Carbon Handling System. The carbon from a lead adsorber to the equipment interface with the regeneration system which is the feed mechanism to the furnace.

The carbon handling system includes eductors for each adsorber, piping, valves, fittings, spent carbon holding/ dewatering tank, wet well, dry well and eductor motive water pumps.

$$\text{Cost H1} = 19{,}600\ Q_p^{0.28}$$

where

Cost H1 = Cost of carbon handling system, $

Q_p = Peak flow rate, mgd

2.5.8.1.5 Calculate cost of building

$$\text{Cost B2} = A_B \times \text{UPIBC}\ .$$

where

Cost B2 = Building cost, $

UPIBC = Unit price input for building cost, \$/sq.ft

A_B = Total building area requirement, ft^2

2.5.8.1.6 First fill carbon cost

$$\text{Cost CB} = \text{UPICC} \times Q_c$$

where

Cost CB = First fill carbon cost

Q_c = Total carbon inventory in the system, lbs

UPICC = Unit price input for carbon cost, \$/lb (1st quarter 1977 cost \$0.56/lb)

2.5.8.1.7 Total base construction cost for adsorption system

$$\text{TBCAD} = \left[\frac{\text{MSECI}}{491.6}\,(\text{Cost A1} + \text{Cost F1} + \text{Cost B1} + \text{Cost H1}) + \text{Cost B2} + \text{Cost CB}\right] \times 1.42$$

where

TBCAD = Total bare construction cost for the adsortion system, \$

MSECI = Current Marshall and Swift Equipment Cost Index

491.6 = Marshall and Swift Equipment Cost Index for 1st Quarter, 1977

1.42 = Correction factor for other construction items

2.5.8.2 Capital cost estimate of the concrete adsorption system, ($Q_p > 10$ mgd)

2.5.8.2.1 Cost of reinforced concrete in place

$$\text{Cost RC} = V_c \times \text{UPICW}$$

where

Cost RC = Cost of RC in place, \$.

V_c = volume of reinforced concrete required, cu.yd.

UPICW = Unit price input for R.C. wall, in place, \$/cu.yd.

2.5.8.2.2 Cost of earthwork

$$COSTE = V_E \times UPIEX$$

where

COSTE = Cost of earthwork, \$.

V_E = Volume of earthwork, cu.yd.

UPIEX = Unit price input for earthwork, \$/cu yd.

2.5.8.2.3 First fill carbon cost

$$Cost\ CB = Q_c \times UPICC$$

where

Cost CB = First fill carbon cost, dollars

Q_c = Total carbon inventory, lbs

UPICC = Unit price input for carbon cost

2.5.8.2.4 Underdrain system cost

$$COSTUD = A_1 \times UPIUD$$

where

COSTUD = Underdrain system cost, dollars

A_1 = Underdrain and trough area required, ft^2

UPIUD = Underdrain system unit price input, \$/ft^2 (\$35/ft_2 at 1st Quarter 1977)

$$UPIUD = 35 \times \frac{MSECI}{491.6}$$

2.5.8.2.5 Control gate cost

$$COSTCG = G \times UPICG$$

where

COSTCG = Control gate cost, dollars

G = Number of control gates

UPICG = Unit price input of control gate (3 x 6 ft gate with operator and frame) A \$17,300 is used for 1st Quarter 1977

$$UPICG = \$17,300 \frac{MSECI}{491.6}$$

where

MSECI = Marshall Swift Cost Index

2.5.8.2.6 Calculate the cost of the backwash system. The backwash system inlcudes the backwash holding basins for 15 minutes supply at 15 gpm/ft^2, pumps, piping and valves, and dry well

$$COST\ B1 = 40171\ Qp^{0.637} \times \frac{MSECI}{491.6}$$

where

COST B1 = Cost of backwash system, $

Qp = Peak flow rate, mgd

MSECI = Marshall and Swift Equipment Cost Index

491.6 = MSECI for 1st Quarter 1977

2.5.8.2.7 Calculate the cost of the carbon handling system. This system includes educators, valve, piping for removing spent carbon from each lead adsorber. Pumps, wet well and dry well are included.

$$COST\ H2 = 9452\ Qp^{0.543} \times \frac{MSECI}{491.6}$$

where

COST H2 = Cost of the carbon handling system, $

Qp = Peak flow rate, mgd

2.5.8.2.8 Calculate cost of building

$$COST\ B2 = A_B \times UPIBC$$

where

COST B2 = Building cost, $.

A_B = Total building use requirement, ft^2

UPIBC = Unit price input for building cost, $/$ft^2$

2.5.8.2.9 Total bare construction cost for adsorption system

$$TBCAD = 1.42\ (COSTRC + COSTE + COSTCB + COSTUD + COSTCG + COST\ B1 + COST\ H2 + COST\ B2)$$

2.5.8.3 Capital cost estimate of the regeneration system, when the carbon regeneration rate, R, is less than 400 lbs per day. The spent carbon would either be used on a throwaway or regenerated on an off-site facility. The total bare construction cost of the regeneration system, in this case, would be zero.

$$TBCRG = 0$$

where

TBCRG = Total bare construction cost for the regeneration system, $.

2.5.8.4 Capital cost estimate of the regeneration system when R 400 lbs/day

2.5.8.4.1 Calculate COSTR

When ARI $\leq$ 400 sq.ft

$$COSTR = 0.192\ (ARI)^{0.308}$$

When ARI > 400 sq.ft

$$COSTR = 0.0272\ (ARI)^{0.602}$$

where

COSTR = Cost of furnace expressed as percent of standard size incinerator cost

ARI = Hearth area of individual furnace, sq.ft

2.5.8.4.2 Purchase cost of standard size furnace. The system includes a furnace, combustion air flow, induced draft flow, cooling in flow, controls, carbon feed system, scrubber and dust control. The unit selected as standard is a unit with a total hearth area of 400 sq.ft. The cost of this furnace for the 1st quarter of 1977 is:

$$COSTMH = \$825{,}000$$

where

COSTMH = Cost of standard size furnace system, dollars

For better estimate, COSTMH should be obtained from the manufacturer as a unit price input. If COSTMHis not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTMH = \$825{,}000 \times \frac{MSECI}{491.6}$$

where

MSECI = Current Marshall and Swift Equipment Cost Index

491.6 = MSECI, 1st quarter, 1977

2.5.8.4.3 Purchase cost of multiple hearth furnace system

where

COSTNF = COSTR x COSTMH x NFN

COSTNF = Purchase cost of furnace system

NFN = Number of furnace required

2.5.8.4.4 Calculate total bare construction cost for regeneration system

$$TBCRG = 1.25 \times COSTNF$$

where

TBCRG = Total bare construction cost for the regeneration system

2.5.8.5 Total bare construction cost of the total carbon adsorption and regeneration system

$$TBCC = TBCAD + TBCRG$$

where

TBCC = Total bare construction cost, dollars

2.5.8.6 Calculate total operation and maintenance materials and supply costs

$$OMMC = \frac{MMAD}{100} \times TBCAD + \frac{MMRG}{100} \times TBCRG$$

where

OMMC = Operation and maintenance material and supply cost, \$/yr.

MMAD = Operation and maintenance supply requirement for adsorption system as percent of total bare construction cost of the adsorption system (TBCAD)

MMRG = Operation and maintenance material and supply requirement for regeneration system as percent of total bare construction cost (TBCRG)

2.5.9 Cost Calculations Output Data.

2.5.9.1 Total bare construction cost, TBCC, dollars.

2.5.9.2 Operation and maintenance material and supply cost, OMMC, dollars/yr.

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2.7 CENTRIFUGATION

2.7.1 Background.

2.7.1.1 Centrifugration is a widely used process for concentrating and dewatering sludge for final disposal. The process offers the following advantages:

2.7.1.1.1 Capital cost is low in comparison with other mechanical equipment.

2.7.1.1.2 Operating costs are moderate, provided flocculants are not required.

2.7.1.1.3 The unit is totally enclosed, thus odors are minimized.

2.7.1.1.4 The unit is simple and will fit in a small place.

2.7.1.1.5 Chemical conditioning of the sludge is often not required.

2.7.1.1.6 The unit is flexible and can process a wide variety of solids.

2.7.1.1.7 Minimum supervision is required.

2.7.1.2 Disadvantages associated with centrifugation are:

2.7.1.2.1 Without the use of chemicals, solids capture is often poor.

2.7.1.2.2 Chemical costs can be substantial.

2.7.1.2.3 Trash must often be removed from the centrifuge feed by screening.

2.7.1.2.4 The percentage of cake solids is often lower than that resulting from vacuum filtration.

2.7.1.2.5 Maintenance costs are higher than vacuum filtration.

2.7.1.2.6 Fine solids (in concentrate) that escape the centrifuge may resist settling when recycled to the head of the treatment plant and gradually build up in concentration, eventually raising effluent solids level.

2.7.1.3 Centrifuges applicable to sludge thickening and dewatering fall into three general classifications: disc, basket, and the currently popular solid-bowl. Basically, centrifuges separate solids from liquids through sedimentation and centrifugal force. Process variables in centrifugation include feed rates, sludge solids characteristics, feed consistency, and chemical additives. Machine variables include bowl design, bowl speed, pool volume, and conveyor speed.

2.7.1.4 The main objectives in centrifuge design are cake dryness and solids recovery. The effect of the various parameters on these two factors are summarized in Table 2.7-1. Operating data reported in the literature indicate that raw primary and digested primary sludge dewater easily. With polymer addition, a centrifuge (fig. 2.7-1) can produce 25-40 percent cake solids with better than 90 percent recovery.

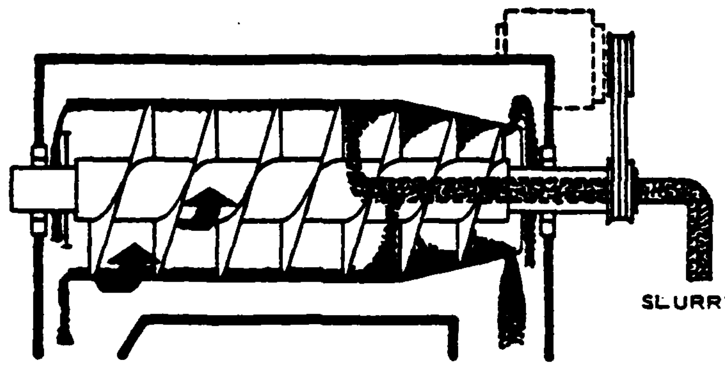

Figure 2.7-1. Schematic of a centrifuge.

Waste activated sludge, however, is difficult to thicken or dewater with centrifugation. High polymer dosages will be required to produce 8-10 percent cake solids and 90 percent recovery.

2.7.1.5 Design criteria for centrifugation systems are scarce. One criterion used in determining the size of centrifuge required is the power requirement per gallon per minute of inflow (0.5-2.0 hp per gpm). Generally, a power requirement of 1.0 hp/gpm of inflow is applicable to most centrifuges commercially manufactured.

2.7.2 Input Data.

2.7.2.1 Sludge flow, gpd.

2.7.2.2 Sludge concentration, percent solids.

2.7.3 Design Parameters.

2.7.3.1 Centrifuges power requirement, hp/gpm ($\approx$1.0).

2.7.3.2 Operation per day, hr.

2.7.3.3 Operation per week, days.

2.7.3.4 Number of units.

2.7.3.5 Excess capacity factor, 1.25.

2.7.3.6 Chemical dosage, percent of dry weight of solids.

2.7.4 <u>Process Design Calculations</u>.

2.7.4.1 Calculate total power required.

$$THP = \frac{(SF)\ (CPR)\ (7)\ (ECF)\ (24)}{(60)\ (DPW)\ (HPD)}$$

where

THP = total power required, hp.

SF = sludge flow, gpd.

CPR = centrifuge power requirement, hp/gpm (1.0).

DPW = operation per week, days.

ECF = excess capacity factor, 1.25.

HPD = operation per day, hr.

2.7.4.2 Calculate power per unit. Use duplicate units.

$$hp/unit = \frac{THP}{NU}$$

where

THP = total power required, hp.

NU = number of units.

2.7.4.3 Compare horsepower with manufacturer's specification and select centrifuge that meets the requirements.

2.7.4.4 Calculate chemical requirements.

$$CR = \frac{(SF)(7)(C_d)(8.34\ lb/gal)(24)}{(HPD)(DPW)(100)}\ \frac{100 - C_i}{100}$$

where

CR = chemical requirements, lb/hr.

SF = sludge flow, gpd.

C_d = chemical dosage, percent of dry weight of solids fed to filter.

HPD = operation per day, hr.

DPW = operation per week, days.

C_i = initial moisture content of sludge, percent.

2.7.4.5 Calculate output sludge quantity.

$$SFO = \frac{(SF)\ (100 - Ci)\ (.9)}{9}$$

where

SFO = sludge flow out, gpd.

SF = sludge flow, gpd.

Ci = initial moisture content of sludge, %.

9 = final sol[illegible] content of sludge, %.

.9 = fraction ot solids captured.

2.7.5 Process Design Output Data.

2.7.5.1 Power required/unit.

2.7.5.2 Number of units.

2.7.5.3 Chemical requirements, lb/hr.

2.7.5.4 Sludge flow, gal/day.

2.7.5.5 Initial solid concentrate, percent.

2.7.5.6 Operation per day, hr.

2.7.5.7 Operation per week, days.

Table 2.7-1. Summary of the Effect of Various Parameters on Centrifuge Perfomance

To Increase Cake Dryness	To Increas^ Solids Recovery
Increase bowl speed	Increase bowl speed
Decrease pool volume	Increase pool volume
Decrease conveyor speed	Decrease conveyor speed
Increase feed rate	Decrease feed rate
Decrease feed consistency	Increase temperature
Increase temperature	Use flocculants
Do not use flocculants	Increase feed consistency

2.7.6 Quantities Calculations.

2.7.6.1 Calculate area of centrifuge building. The building area for the centrifuges was developed based on actual size of commercial units and accessories.

2.7.6.1.1 If the number of units (NU) is one (1), then the building area is:

$$AB = 200 \text{ sq ft}$$

2.7.6.1.2 If the number of units (NU) is greater than one (1), then the building area is:

$$AB = 200 + 5.3\ (NU)\ (HP)^{0.738}$$

where

A_B = area of centrifuge building, sq ft.

NU = number of centrifuges.

HP = power required per unit, HP.

2.7.6.2 Calculate dry solids produced.

$$DSTPD = \frac{(SF)\ (8.34)\ (Ci)}{(2000)\ (100)}$$

where

DSTPD = dry solids produced, tpd.

SF = sludge flow, gpd.

Ci = initial solids concentration, percent.

2.7.6.3 Calculate operational labor.

2.7.6.3.1 If DSTPD is between .1 tpd and 1.0 tpd, the operational labor is calculated by:

$$OMH = 1520\ (DSTPD)^{0.3293}$$

2.7.6.3.2 If DSTPD is between 1.0 tpd and 10.0 tpd, the operational labor is calculated by:

$$OMH = 1520\ (DSTPD)^{0.4994}$$

2.7.6.3.3 If DSTPD is greater than 10.0 TPD, the operational labor is calculated by:

$$OMH = 632\ (DSTPD)^{0.8751}$$

where

OMH = operation labor, MH/yr.

DSTPD = dry solid produced, tpd.

2.7.6.4 Calculate maintenance labor.

2.7.6.4.1 If DSTPD is between .1 tpd and 1.0 tpd, the maintenance labor is calculated by:

$$MMH = 264\ (DSTPD)^{0.3424}$$

2.7.6.4.2 If DSTPD is between 1.0 tpd and 10.0 tpd, the maintenance labor is calculated by:

$$MMH = 264\ (DSTPD)^{0.4815}$$

2.7.6.4.3 If DSTPD is greater than 10.0 tpd, the maintenance labor is calculated by:

$$MMH = 112.0\ (DSTPD)^{0.8573}$$

where

MMH = maintenance labor, man-hour/yr.

2.7.6.5 Calculate electrical energy required for operation.

$$KWH = 33{,}000\ (DSTPD)^{0.9248}$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.7.6.6 Calculate other operation and maintenance material costs. This item includes repair and replacement material costs. It is expressed as a percent of total bare construction cost. This does not include chemical costs.

$$OMMP = 2.27\ (DSTPD)^{0.1868}$$

where

OMMP = percent of total bare construction cost for the operation and maintenance material costs, percent.

2.7.6.7 Other minor construction items. Items such as piping, electrical conduits, etc., would be approximately 15 percent of the total construction costs.

$$CF = \frac{1}{.85} = 1.18$$

where

CF = correction factor for minor costs.

2.7.7 Quantities Calculations Output Data.

2.7.7.1 Area of centrifuge building, A_B, sq ft.

2.7.7.2 Power required per unit, HP, hp.

2.7.7.3 Number of units, NU.

2.7.7.4 Operational labor, OMH, man-hour/yr.

2.7.7.5 Maintenance labor, MMH, man-hour/yr.

2.7.7.6 Electrical energy required for operation, KWH, kwhr/yr.

2.7.7.7 Other operation and maintenance costs, OMMP, percent.

2.7.7.8 Correction factor for other construction costs, CF.

2.7.8 Unit Price Input Required.

2.7.8.1 Cost of building, UPIBC, dollars/sq ft.

2.7.8.2 Unit price for 50 hp centrifuge, COSTSC, dollars (optional).

2.7.9 Cost Calculations.

2.7.9.1 Cost of centrifuge building.

$$COSTB = A_B \times UPIBC$$

where

COSTB = cost of centrifuge building, dollars.

A_B = area of centrifuge building, sq ft.

UPIBC = unit price input for building construction costs, \$/sq ft.

2.7.9.2 Purchase cost of centrifuge. The purchase cost of centrifuges is calculated by:

$$COSTC = COSTSC \times \frac{COSTR}{100}$$

where

COSTC = cost of centrifuge selected, \$.

COSTSC = cost of standard size centrifuge of 50 hp, \$.

COSTR = cost of centrifuge selected expressed as percent of standard size unit cost, percent.

2.7.9.2.1 Standard size unit cost. The 1st quarter 1977 price for COSTSC is \$165,000. For better estimation COSTCS should be o btained from the equipment vendors and treated as a unit price input. If this is not done, the following equation will be utilized to compensate for inflation.

$$COSTSC = 165{,}000 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = MSECI value for 1st quarter 1977.

2.7.9.2.2 Calculate COSTR.

$$COSTR = 2.567\ (HP)^{0.9362}$$

where

COSTR = cost of centrifugal selected expressed as percent of standard size unit cost, percent.

HP = power required per unit, hp.

2.7.9.3 Other equipment and installation costs. Other equipment required to make the dewatering process complete includes conveyors, polymer feed system, pumps, and conditioning tanks. It is estimated by the equipment vendors and contractors that the purchase and installation costs of this equipment are approximately equal to the purchase cost of the centrifuge.

$$OICOST = COSTC$$

where

OICOST = other equipment and installation costs, dollars.

2.7.9.4 Calculate total bare construction cost:

$$TBCC = (CF)\ (COSTC + COSTB + OICOST)$$

where

TBCC = total bare construction cost, dollars.

2.7.9.5 Operation and maintenance material costs.

$$OMMC = \frac{OMMP}{100}\ (TBCC)$$

where

OMMC = operation and maintenance material costs, dollars.

OMMP = percent of total bare construction cost for the operation and maintenance material costs, percent.

2.7.10 Cost Calculations Output Data.

2.7.10.1 Total bare construction cost, TBCC, $.

2.7.10.2 Operation and maintenance cost, OMMC, $/yr.

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2.9 CHEMICAL COAGULATION

2.9.1 Background.

2.9.1.1 Chemical coagulation involves the aggregation of small particles into large, more readily settleable conglomerates. Chemical coagulation is a common process used in water treatment for the removal of turbidity and color. In wastewater treatment, chemical coagulation has been used to remove colloidal and suspended matter fom raw wastes, remove phosphorus, remove algae from oxidation pond effluents, and enhance sludge dewaterability.

2.9.1.2 Wastewater can be coagulated using any of the coagulants common for water treatment. The most widely used chemicals include: iron salts (ferric chloride, ferric sulfate, ferrous chloride, and ferrous sulfate), aluminum salts (alum and sodium aluminate), lime and synthetic polymers. The choice of chemicals should be based on careful evaluation of the wastewater characteristics, availability and cost of the coagulant, and sludge handling and disposal characteristics. Jar tests must be conducted to determine the coagulant dosage, the optimum conditions for coagulation, the quality of the effluent, and the characteristics of the chemical sludge.

2.9.1.3 Rapid mixing, flocculation and sedimentation are the major processes involved in the unit process of chemical coagulation. These steps may be accomplished in separate basins or in the integrated flocculator-clarifier type of unit. Only the circular integrated mixing chamber-flocculator-clarifier type system will be utilized due to its popularity in recent years and its economy.

2.9.1.4 Chemical coagulation may be used as a part of a physicochemical treatment system or for phosphorus removal from wastewaters generated at recreation areas.

2.9.2 Input Data.

2.9.2.1 Wastewater flow.

2.9.2.1.1 Average daily flow, mgd.

2.9.2.1.2 Peak hourly flow, mgd.

2.9.2.2 Wastewater characteristics.

2.9.2.2.1 BOD, total and soluble, mg/l.

2.9.2.2.2 COD, total and soluble, mg/l.

2.9.2.2.3 Phosphorus, mg/l.

2.9.2.2.4 Suspended solids, mg/l.

2.9.2.2.5 pH.

2.9.2.2.6 Alkalinity, mg/l.

2.9.2.3 Coagulant to be used and its dosage, mg/l.

2.9.3 Design Parameters.

2.9.3.1 Desired quality of treated effluent, mg/l.

2.9.3.2 Coagulant dosage, mg/l (jar test).

2.9.3.3 Detention time of rapid mix basin, (1-3 min).

2.9.3.4 Detention time of flocculation basin, (15-60 min).

2.9.3.5 Surface loading rates for clarification, gpd/sq ft.

2.9.3.6 Sludge production, lb/day.

2.9.3.7 Sludge quantity, pgd.

2.9.4 Process Design Calculations.

2.9.4.1 Selection of coagulant and calculation of dosing requirement.

$$CR = (CD)\ (Q_{avg})\ (8.34)$$

where

CR = coagulant requirement, lb/day.

CD = coagulant dosage (from input data), mg/l.

Q_{avg} = average daily flow, mgd.

2.9.4.2 Sludge production when alum is used as coagulant.

2.9.4.2.1 Suspended solid removal.

$$X_s = Q_{avg} \cdot [(SS)_{inf} - (SS)_{eff}] \cdot 8.34$$

where

X_s = sludge produced due to suspended solids removed, lb/day.

$(SS)_{inf}$ = influent suspended solids, mg/l.

$(SS)_{eff}$ = effluent suspended solids, mg/l.

2.9.4.2.2 Aluminum phosphate sludge.

$$X_P = Q_{avg} [P_{inf} - P_{eff}] \cdot 8.34 \cdot 4.0$$

X_P = $ALPO_4$ sludge production, lb/day.

P_{inf} = influent phosphorus concentration, mg/l as P.

P_{eff} = effluent phosphorus concentration, mg/l as P.

2.9.4.2.3 Alum required for phosphorus precipitation.

$$AP = [P_{inf} - P_{eff}] \cdot 9.6$$

where

AP = alum required for phosphorus precipitation, mg/l.

2.9.4.2.4 Alum available to form hydroxide precipitate.

$$AOH = CD - AP$$

where

AOH = alum available to form aluminum hydroxide, mg/l.

$$X_{OH} = Q_{avg} \cdot AOH \cdot (8.34) \cdot (0.263)$$

where

X_{OH} = $Al(OH)_3$ sludge production, lbs/day.

2.9.4.2.5 Total sludge production when alum is used as coagulant is

$$X_{Al} = X_s + X_P + X_{OH}$$

where

X_{Al} = total sludge production, lb/day.

2.9.4.3 Sludge production when ferric chloride is used as coagulant.

2.9.4.3.1 Suspended solids removal.

$$X_s = Q_{avg} \cdot [(SS)_{inf} - (SS)_{eff}] \cdot (8.34)$$

2.9.4.3.2 $FePO_4$ sludge production.

$$X_P = Q_{avg} [P_{inf} - P_{eff}] (8.34) (4.9)$$

2.9.4.3.3 $Fe(OH)_3$ sludge production.

$$FP = [P_{inf} - P_{eff}] \cdot (5.23)$$

where

FP = $FeCl_3$ requirement for phosphorus removal, mg/l.

then

$$FOH = CD - FP$$

where

FOH = $FeCl_3$ available for precipitation as hydroxide.

thus

$$X_{OH} = FP \cdot Q_{avg} \cdot (8.34) \cdot (0.65)$$

2.9.4.3.4 Total sludge production when $FeCl_3$ is used as coagulant.

$$X_{Fe} = X_s + X_p + X_{OH}$$

where

X_{Fe} = total sludge production, lb/day.

2.9.4.4 Sludge production when lime is used as coagulant.

2.9.4.4.1 Suspended solids removal.

$$X_s = Q_{avg} \cdot [(SS)_{inf} - (SS)_{eff}] \cdot (8.34)$$

2.9.4.4.2 Chemical sludge production. The chemical reactions involved in the lime precipitation process are quite complex. Factors such as raw waste, hardness, alkalinity, calcium and magnesium ratio, pH, and others, can significantly affect the sludge quantity. For preliminary estimation purposes, the following equation can be utilized:

$$X_{Ca} = Q_{avg} \cdot (8.34) \cdot (CD) \cdot (1.5)$$

where

X_{Ca} = chemical sludge production due to lime precipitation, lb/day.

CD = chemical dosage as quicklime, mg/l.

2.9.4.4.3 Total sludge production.

$$X_{CaT} = X_s + X_{Ca}$$

where

X_{CaT} = total sludge production due to lime addition, lb/day.

2.9.4.5 Selection of floccular-clarifier.

2.9.4.5.1 This type system integrates the rapid mixing tank, flocculator and clarifier into one unit. It is sometimes referred to as an upflow clarifier or a sludge blanket clarifier. Figure 2.9-1 illustrates the components of such a unit.

2.9.4.5.2 The design of the upflow clarifier is based on overflow rate, (Table 2.9-1), Q_{of}. For lime coagulation:

$$Q_{of} = 630 \text{ gpd/sq ft based on } Q_{avg}.$$

For alum coagulation.

$$Q_{of} = 360 \text{ gpd/sq ft.}$$

For iron salt coagulation.

$$Q_{of} = 500 \text{ gpd/sq ft.}$$

2.9.4.5.3 Unit selection. The number of units to be employed depends upon the wastewater quantity to be handled. The following assumptions will be followed in the determination of number of units, N.

Flow Range MGD, Q_{avg}	Number of Units, N
0.5 1.0	2
1.0 - 10.0	2
10.0 - 24.0	4
24.0 - 50.0	8
50.0 - 70.0	12
70.0 -100.0	16

When Q_{avg} is larger than 100 mgd, the system will be designed as several batteries of units.

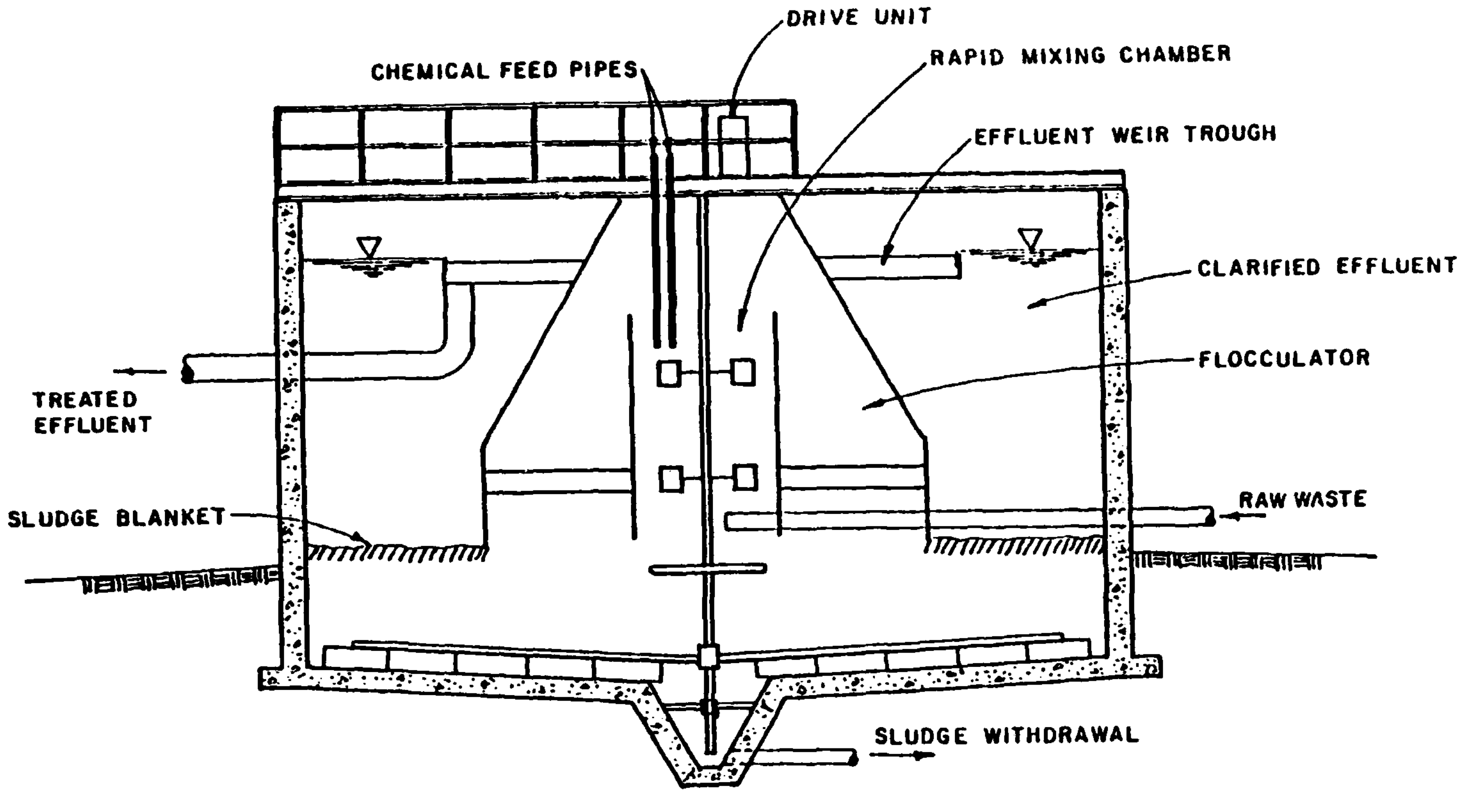

FIGURE 2.9-1. TYPICAL FLOCCULATOR/CLARIFIER

Table 2.9-1. Recommended Surface-Loading Rates For Various Suspensions

Suspension	Loading Rate, gpd/ft^2 Range	Peak Flow
Untreated wastewater	600 to 1200	1200
Alum floc[a]	300 to 600	600
Iron floc[a]	500 to 800	800
Lime floc[a]	500 to 1200	1200

[a] Mixed with the settleable suspended solids in the untreated wastewater and colloidal or other suspended solids swept out by the floc.

If $100 \leq Q_{avg} \leq 200$

Number of process batteries, NB, would be 2.

The system would be designed as two identical batteries with flow to each battery at $Q_{avg}/2$.

Select the number of units per battery as described above using flow to be $Q_{avg}/2$.

If $Q_{avg} > 200$

Number of process batteries, NB, would be 3.

The system would be designed as three identical batteries with flow to each battery at $Q_{avg}/3$.

Select the number of units per battery as described above using flow to be $Q_{avg}/3$.

2.9.4.5.4 Sizing individual unit. Wastewater to be handled by one unit, Q_u.

$$Q_u = \frac{Q_{avg}}{N \cdot NB}$$

where

Q_u = wastewater to be handled by one unit, mgd.

N = number of units.

2.9.4.5.5 The surface area of each unit, A_s.

$$A_s = \frac{Q_u \cdot 10^6}{Q_{of}}$$

where

A_s = surface area of each individual unit, sq ft.

Q_{of} = design overflow rate, dependent on the type of coagulant used, gpd/sq ft.

2.9.4.5.6 The diameter of this unit then:

$$DIA = \frac{4 \cdot A_s}{3.1416}^{0.5}$$

where

DIA = diameter of the unit, ft.

2.9.4.5.7 The depth of the unit, SWD, is a function of the diameter and can be expressed as:

$$SWD = 10.67 + (0.067) \cdot (DIA)$$

where

SWD = side water depth, ft.

2.9.4.5.8 The available sizes from off-the-shelf items range from 10 ft to 200 ft.

So if the calculated DIA is larger than 200 ft, the number of units will have to be increased to N = N + 1.

2.9.4.6 Effluent Characteristics.

2.9.4.6.1 BOD_5.

$$BODE = (BOD5)(1.0 - \frac{BODR}{100})$$

If BODE < BOD5S then BODE = BOD5S

where

BODE = effluent BOD_5, mg/1.

BODR = BOD_5 removal rate, %.

BOD5 = influent BOD_5, mg/1.

BOD5S = effluent soluble BOD_5, mg/1.

2.9.4.6.2 COD.

$$CODE = (COD)\ (1 - \frac{CODR}{100})$$

If CODE < CODS then CODE = CODS

where

CODE = effluent COD concentration, mg/l.

COD = influent COD concentration, mg/l.

CODR = COD removal rate, %.

CODS = effluent soluble COD concentration, mg/l.

2.9.4.6.3 Phosphorus.

$$PO4E = PO4\ (1.0 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

2.9.4.6.4 Suspended solids.

$$SSE = SSI\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

2.9.4.6.5 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNR}{100})$$

If TKNE < NH3 then TKN = NH3

where

TKNE = effluent TKN concentration, mg/l.

TKN = influent TKN concentration, mg/l.

TKNR = TKN removal rate, %.

NH3 = effluent ammonia concentration.

2.9.4.6.6 Oil and grease.

$$OAG = 0.0$$

where

OAG = effluent oil and grease concentration, mg/l.

2.9.4.6.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids concentration, mg/l.

2.9.5 Process Design Output Data.

2.9.5.1 Coagulant type.

2.9.5.2 Coagulant requirement, CR, lb/day.

2.9.5.3 Sludge production, X, lb/day.

2.9.5.4 Number of clarifiers per battery, N.

2.9.5.5 Number of batteries, NB.

2.9.5.6 Diameter of each unit, DIA, ft.

2.9.5.7 Side water depth of clarifier, SWD, ft.

2.9.5.8 Effluent BOD_5 concentration, BODE, mg/l.

2.9.5.9 Effluent COD concentration, CODE, mg/l.

2.9.5.10 Effluent phosphorus concentration, PO4E, mg/l.

2.9.5.11 Effluent suspended solids concentration, SSE, mg/l.

2.9.5.12 Effluent TKN concentration, TKNE, mg/l.

2.9.5.13 Effluent oil and grease concentration, OAG, mg/l.

2.9.6 Quantities Calculations.

2.9.6.1 Earthwork required for construction. The procedure to estimate the earthwork requirement is the same as that for circular clarifier.

$$V_{ew} = (1.15)\ (NB)\ (N)\ [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = earthwork required for construction, cu ft.

1.15 = addition of 15% more for safety factor.

2.9.6.2 Reinforced concrete quantities.

2.9.6.2.1 Reinforced concrete slab quantity, V_{csn}, for one tank.

$$V_{csn} = (0.825)\ (DIA + 4)^2 \cdot (\frac{t_s}{12})$$

where

V_{cs} = quantity of R.C. slab in-place, cu ft.

t_s = thickness of the slao, inches, can be obtained by

$$t_s = 7.9 + 0.25\ SWD$$

2.9.6.2.2 Reinforced concrete wall quantity, V_{cwn}, for one tank.

$$V_{cwn} = (3.14)\ (SWD + 1.5) \cdot (DIA)\ (\frac{t_w}{12})$$

where

V_{cwn} = quantity of R.C. wall in-place, cu ft.

t_w = wall thickness, inches, and can be estimated by:

$$t_w = 7 + (0.5)\ (SWD)$$

2.9.6.2.3 Quantity of concrete for splitter boxes, V_{cb}.

$$V_{cb} = 100 \cdot N^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter box, cu ft.

N = number of units.

2.9.6.2.4 Total quantity of R.C. in-place.

$$\text{Wall:}\quad V_{cw} = [N \cdot V_{cwn} + V_{cb}] \cdot NB$$

$$\text{Slab:}\quad V_{cs} = [N \cdot V_{csn}] \cdot NB$$

2.9.6.3 <u>Mechanism Horsepower</u>. Mechanism horsepower is a function of the capacity of the unit, expressed as million gallons per day which can be handled.

$$Q_{hu} = \frac{Q_{of} \cdot A_s}{10^6}$$

where

Q_{hu} = flow can be handled by one unit, mgd.

The total connective horsepower associated with the mixing, scraping, and recirculation mechanisms, MPH, can be estimated as:

$$MHP = \frac{14}{11} \cdot Q_{hu} \qquad \text{For } Q_{hu} \quad 11.0 \text{ mgd}$$

$$MHP = 14 + (0.06) \cdot (Q_{hu} - 11) \qquad \text{For } Q_{hu} \quad 11.0 \text{ mgd}$$

where

MHP = mechanism horsepower installed, Hp.

2.9.6.4 Electrical energy required for operation, KWH.

$$KWH = N \cdot MHP \cdot (24) \cdot (365) \cdot (0.9) \cdot (0.85) \cdot (NB)$$

where

KWH = electric energy requirement, kwhr/yr.

2.9.6.5 <u>Operation and Maintenance Manpower Requirement</u>.

Operation man-hour requirement, OMH.

$$OMH = [3285 + (142) \cdot N^{1.365}] \cdot NB$$

where

OMH = operation man-hour per year.

N = number of units.

Maintenance man-hour requirement, MMH.

$$MMH = [1151 + (64) \cdot N^{1.365}] \cdot NB$$

where

MMH = maintenance man-hour requirement.

2.9.6.6 Other Operation and Maintenance Material Costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of total installed costs of the upflow clarifier system.

OMMP = 1.0%

OMMP = percent of the installed upflow clarifier costs for operation and maintenance material costs.

2.9.6.7 Chemical Quantity. Total chemical requirement per year can be estimated as:

2.9.6.7.1 For lime coagulation.

$$LIME = (CR) \cdot (365)$$

where

LIME = quantity of lime purchased per year, lb/yr.

2.9.6.7.2 For alum coagulation.

$$ALUM = (CR) \cdot (365)$$

where

ALUM = quantity of alum purchased per year, lb/yr.

2.9.6.7.3 For iron salt coagulation.

$$IRON = (CR) \cdot (365)$$

where

IRON = quantity of iron salt ($FeCl_3$) purchased per year, lb/yr.

2.9.6.8 Other construction cost items.

2.9.6.8.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

2.9.6.8.2 Other minor cost items such as piping, site cleaning, control panel, etc., would be 15 percent.

2.9.6.8.3 CF correction factor would be $\frac{1}{0.85} = 1.18$.

2.9.7 Quantities Calculation Output Data.

2.9.7.1 Quantity of earthwork, V_{ew}, cu ft.

2.9.7.2 Quantity of R.C. wall in-place, V_{cw}, cu ft.

2.9.7.3 Quantity of R.C. slab in-place, V_{cs}, cu ft.

2.9.7.4 Horsepower for mechanisms, MHP, hp.

2.9.7.5 Electrical energy requirements, KWH, kwhr/yr.

2.9.7.6 Operational manpower, OMH, man-hour/yr.

2.9.7.7 Maintenance manpower, MMH, man-hour/yr.

2.9.7.8 Other operation and maintenance material costs, OMMP, percent.

2.9.7.9 Lime requirement, LIME, lb/yr.

2.9.7.10 Alum requirement, ALUM, lb/yr.

2.9.7.11 Iron salt requirement, IRON, lb/yr.

2.9.7.12 Correction factor for construction costs, CF.

2.9.8 Unit Price Inputs Required.

2.9.8.1 Cost of earthwork, UPIEX, $/cu yd.

2.9.8.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.9.8.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.9.8.4 Standard size flocculator-clarifier mechanism (60-feet diameter) cost, COSTCL, $ (optional).

2.9.8.5 Marshall and Swift equipment cost index, MSECI.

2.9.8.6 Equipment installation labor rate, $/MH.

2.9.8.7 Crane rental rate, UPICR, \$/hr.

2.9.9 <u>Cost Calculations</u>.

2.9.9.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.9.9.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cw}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in-place, \$.

V_{cw} = quantity of concrete wall, cu ft.

UPICW = unit price input cost of concrete wall in-place, \$/cu yd.

2.9.9.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cs}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of concrete slab, cu ft.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.9.9.4 Cost of installed equipment.

2.9.9.4.1 Purchase cost of clarifier mechanism. The purchase cost of mechanisms can be obtained by using the following equation.

$$COSTCM = COSTCL * COSTRO$$

where

COSTCM = purchase cost of mechanism with diameter DIA feet, \$.

COSTCL = purchase cost of standard size mechanism with diameter of 60 ft.

COSTRO = ratio of cost of mechanism with diameter of DIA, feet and the cost of standard size clarifier.

2.9.9.4.2 COSTRO. The cost ratio can be expressed as:

$$COSTRO = (0.0164) * DIA$$

2.9.9.4.3 Cost of standard size mechanism. The cost of mechanism for a 60-foot diameter upflow clarifier for the first quarter of 1977 is:

$$COSTCL = \$110{,}000$$

For better estimate, COSTCL should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTCL = 110{,}000 * \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift equipment cost index.

491.6 = Marshall and Swift Cost Index 1st quarter 1977.

2.9.9.4.4 Equipment installation man-hour requirement. The man-hour requirement for field erection of clarifier mechanism can be estimated as:

$$IMH = 200 + (25) * (DIA)$$

where

IMH = installation man-hour requirement, MH.

2.9.9.4.5 Crane requirement for installation, CH.

$$CH = (0.1) * IMH$$

CH = crane time requirement for installation, hr.

2.9.9.4.6 Other minor costs associated with the installed equipment. This category includes the costs for electric wiring, piping, painting, etc., and can be added as percent of purchased equipment cost.

$$PMINC = 5\%$$

where

PMINC = percentage of purchasing costs of equipment as minor costs, percent.

2.9.9.4.7 Installed equipment costs, IEC.

$$IEC = [COSTCM\ (1 + \frac{PMINC}{100}) + IMH \times LABRI + CH \times UPICR] \cdot (N)\ (NB)$$

where

IEC = installed equipment costs, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.9.9.5 Other cost items. This category includes cost of piping, walkways, electrical control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.9.9.6 Total bare construction costs, TBCC.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC) \cdot CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

2.9.9.7 Operation and maintenance material costs. Since this item of the operation and maintenance costs is expressed as a percentage of the total bare construction costs, it can be calculated by:

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = percentage of the total bare construction costs as operation and maintenance material cost, percent.

2.9.10 <u>Cost Calculations Output Data</u>.

2.9.10.1 Total bare construction costs of the flocculator-clarifier cost, TBCC, $.

2.9.10.2 Operation and maintenance material costs, OMMC, $.

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2.11 CHEMICAL FEED SYSTEMS

2.11.1 Background. Various organic and inorganic chemical compounds are used in wastewater treatment processes. Four of the most widely used chemicals; lime, alum, iron salts and synthetic polymers, are discussed in this section. The discussion of other chemical compounds can be added as the situation warrants. In this section the general properties of these chemicals will be briefly described and the capital costs of the chemical feed systems will be provided in parametric forms. The using of the parametric cost curves is justified by the fact that chemical feed systems are usually a part of other unit processes. Lime feed system, for example, is only a minor part of the chemical coagulation process. Using the parametric cost estimating procedure will not introduce significant error in the capital cost estimate of the whole project.

2.11.2 General Description Alum Feed System.

2.11.2.1 Alum is utilized in wastewater treatment primarily as a coagulant. It is also used in phosphate removal processes. Alum is available in either dry or liquid forms. The commercial dry alum most often used in wastewater treatment is known as "filter alum" and has the approximate chemical formula $Al_2(SO_4)_3 \times 14 H_2O$ and a molecular weight of about 600. The available grades of dry filter alum and their corresponding bulk density and angle of repose may be seen as follows:

Grade	Angle of Repose	Bulk Density lb/cu.ft.
Lump	--	62 - 68
Ground	43°	60 - 71
Rice	38°	57 - 71
Powdered	65°	38 - 45

The liquid form is available at a solution strength of about 49 percent as $Al_2(SO_4)_3 \cdot 14 H_2O$. The solution weighs about 11 lb/gal at 60°F. Alum solution must be stored or conveyed in corrosion resistant material such as rubber-lined steel, fiberglass or type 316 stainless steel.

2.11.2.2 Alum storage and feed system. The storage and feed equipment for dry alum is similar to that for the hydrated lime, except corrosion resistant material is used in the solution tank, piping and pumping systems. Liquid alum is usually the preferred form to be used in municipal wastewater treatment plants due to its ease of handling. The storage and feed system for liquid alum usually consists of sheltered storage tanks and metering pump for feeding. A schematic diagram of liquid alum storage and feed system is given in Figure 2.11-1.

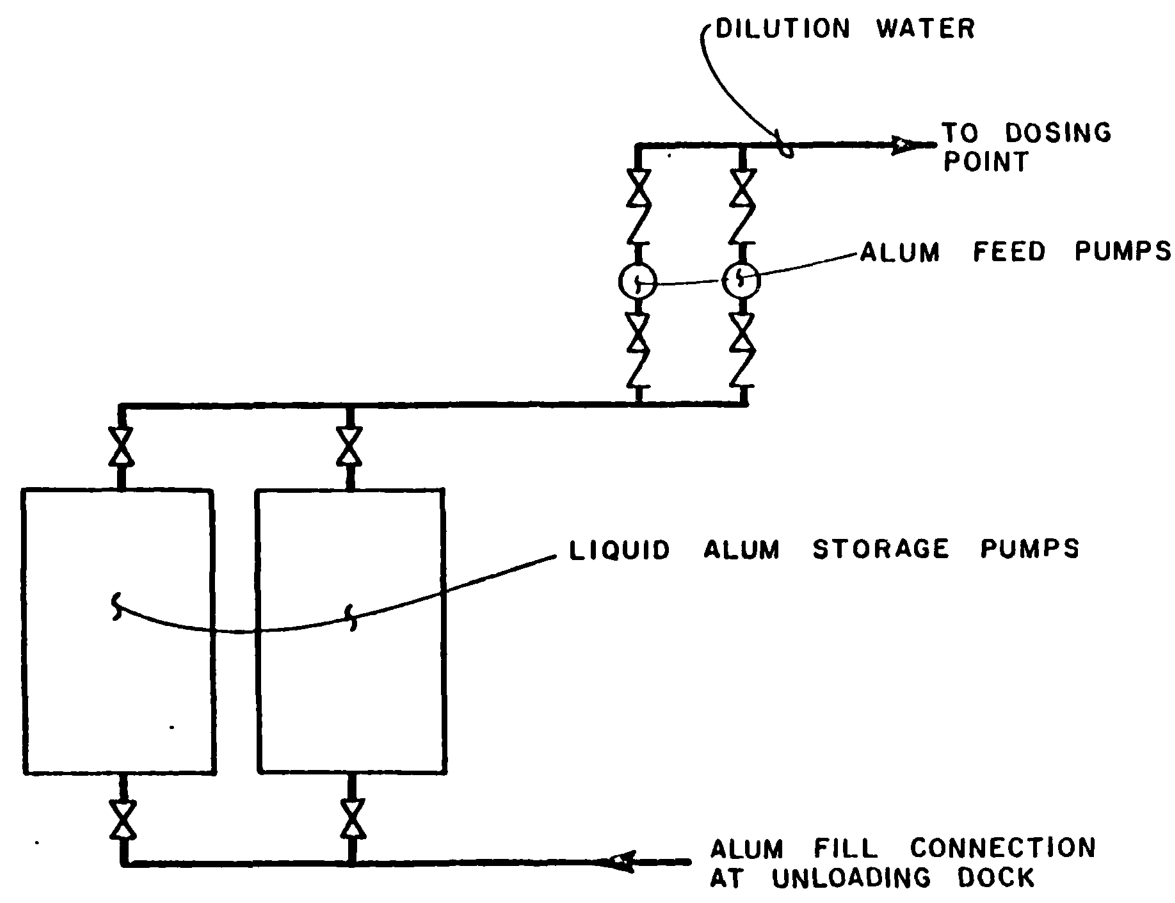

FIGURE 2.11-1. LIQUID ALUM STORAGE & FEED SYSTEM SCHEMATIC

2.11.3 General Description Iron Salt Feed System.

Iron compounds have pH coagulation ranges and floc characteristics similar to aluminum sulfate. The cost of iron compound may often be less than the cost of alum. However, the iron compounds are corrosive and often present difficilties in dissolving. Generally, iron salts are used only in sludge conditioning and, in lesser degree, in the precipitation of phosphates from waste streams. In certain locations, iron salts are available in the form of waste pickling liquor. In most instances, the availability of waste pickling liquor will depend on the proximity to steel processing plants. The storage and feeding systems for iron salts are similar to those utilized in handling alum.

2.11.4 General Description Lime Feed System.

Chemical lime is produced by the calcination of high quality lime stones. Either high calcium quicklime or dolomitic quicklime is the end product, depending on the chemical makeup of the raw material. These quicklimes are further treated by adding water to produce their hydrated counterparts termed hydrated lime or slaked lime. Both the high calcium and the dolomitic types have been utilized in water and wastewater treatment extensively. Each has particular advantages over the other which should be weighed for a given application. Table 2.11-1 summarizes technical information on the various forms of lime.

2.11.4.1 Lime applications in wastewater treatment. Limes are used in wastewater treatment mainly for the following functions:

2.11.4.1.1 Lime-coagulation in physical-chemical wastewater treatment processes.

2.11.4.1.2 Supplementation of alkalinity in biological nitrification processes.

2.11.4.1.3 Phosphate percipitation by lime addition.

2.11.4.1.4 pH elevation of wastewaters for the removal of ammonia by air-stripping process.

2.11.4.1.5 Regeneration of spent clinoptilolite, an ammonia selective ion exchange zeolite.

2.11.4.1.6 Sludge conditioning prior to dewatering processes.

TABLE 2.11-1

CHARACTERISTICS OF VARIOUS LIME FORMS

CHEMICAL	SHIPPING DATA		PHYSICAL AND CHEMICAL CHARACTERISTICS			
Common Name Formula	*Available Forms*	*Containers and Requirements*	*Appearance and Properties*	*Weight lb/cu ft (Bulk Density)*	*Commercial Strength*	*Solubility in Water g/100 ml at 25°C*
Quicklime CaO	Pebble Crushed Lump Ground Pulverized	Moisture proof bags, 80-100 lb Wood bbl Bulk C/L Store dry Max. 60 days Keep container closed	White (light gray, tan) lumps to powder Unstable, caustic irritant Slakes to hydroxide slurry evolving heat Air slakes to $CaCO_3$ Sat. Sol. pH 12.4	55-75 To calculate hopper capacity - use 60 Sp. G., 3.2-3.4	70-96% CaO (Below 88% can be poor quality)	Reacts to form $Ca(OH)_2$ Each lb of quicklime will form 1.16-1.32 lb. of $Ca(OH)_2$, with 2-12% grit, depending on purity
Recovered Lime CaO	Pellets	Bulk delivery direct from kiln to storage bin	Light gray, tan Same properties as quicklime		75-90% CaO	Same as quicklime
Dolomitic Lime $CaO \cdot MgO$ (MgO content varies)	Pebble Crushed Lump Ground Pulverized	Bags, 50-60 lb Bulk C/L bbl	Same appearance and properties as quicklime, except MgO slakes slowly	Pebble, 60-65 Ground, 50-75 Lump, 50-65 Powder, 37-63 Avg. 60 Sp. G., 3.2-3.4	CaO 55-57.5% MgO 37.6-40.5%	Slakes to form $Ca(OH)_2$ slurry plus MgO, which slakes slowly
Hydrated Lime $Ca(OH)_2$	Powder (passes 200 mesh)	Bags, 50 lb. Bbl, 100 lb. Bulk, C/L (Store dry)	White, 200-400 mesh, powder, free of lumps Caustic, dusty irritant Absorbs H_2O and CO_2 from air to form $Ca(HCO_3)_2$ Sat. Sol. pH 12.4	35 to 50 To calculate hopper capacity use 40 Some 20-30 use 23 Sp. G., 2.3-2.4	$Ca(OH)_2$ 82-98% CaO 62-74% (Std. 70%)	0.18 at 0°C 0.16 at 20°C 0.15 at 30°C 0.077 at 100°C
Carbide Lime $Ca(OH)_2$	Powder 70-90% (200 mesh) Slurry	Bulk	Coarse, gray powder Gray slurry (35% solids)	35 to 55	95% $Ca(OH)_2$	Same as $Ca(OH)_2$
Dolomitic Hydrated Lime $Ca(OH)_2 + Mg(OH)_2$ Content of MgO and $Mg(OH)_2$ varies	Monohydrated powder slaked at atmos. press. Dihydrate powder slaked at high press. & temp.	Bags, 50 lb. Bbl Bulk, C/L (Store dry)	Tan to white powder free of lumps (-200 mesh) Caustic, dusty irritant Sat. Sol. pH 12.4	Monohydrate 25-37 Dihydrate 27-43 To calculate hopper capacity, use 40 Sp. G., 2.65-2.75	Monohydrate $Ca(OH)_2$-62% MgO-34% Dihydrate $Ca(OH)_2$-54% $Mg(OH)_2$-42% (approx.)	Same as $Ca(OH)_2$
Limestone (Unburned lime) $CaCO_3$	Powder Granules Ground	Bags, 50 lb. 80 lb. 100 lb. Drums Bulk, C/L	White amorphous powder Sat. Sol. pH 9-9.5	Powder 35 to 60 Granules 100 to 115 Sp. G., 2.65-2.75	96-99%	0.0013 at 20°C 0.002 at 100°C
Dolomite $CaCO_3 \cdot MgCO_3$	Lump or crushed Granular Ground Powder	Bags, 50 lb. Drums Bulk, C/L	White, gray, tan Sat. Sol. pH 9-9.5	87 to 95 Sp. G., 2.8-2.9	Varies	Approx. same as limestone

2.11.4.2 Lime storage and feed equipment description. As indicated in Table 2.11-1, lime may be supplied in a variety of foms and containers. The types and sizes of lime feed equipment are detemined by the dosage rate and type of lime supplied. For preliminary design and cost estimating purposes, the following generalizations are offered in the selection of lime feed equipment. These rules of thumb are based on the considerations of economics and ease of operation.

2.11.4.2.1 When lime consumption is in the range of 50-500 lb/day, bagged hydrated lime will be used. The suggested storage and feed equipment is shown in Figure 2.11-2.

2.11.4.2.2 When lime consumption is in the range of 500-6000 lb/day, bulk hydrated lime will be used. A schematic diagram of the storage and feed system is given in Figure 2.11-3.

2.11.4.2.3 When lime consumption is above approximately 6000 lb/day, bulk quantity of quicklime will be used. A schematic diagram of lime storage, slaking and feed equipment is shown in Figure 2.11-4.

2.11.5 General Description Polymer Feed System

Polyelectrolytes are playing an ever increasing role in wastewater treatment. They have been utilized as flocculation aids, sludge conditioners, coagulants and filtration aids. All synthetic polymers can be classified on the basis of the type of charge on the polymer chain. Thus polymers possessing negative charges are called anionic while those carrying positive charges are cationic. Certain compounds carry no electrical charge and are called nonioic polyelectrolytes. There is a great variety of polymer commercially available on the market. The extensive use of jar testing is mandatory to determine the specific polymer that will perfom well in each individual application. The polymers usually are available both in dry fom and liquid fom. In order to feed the dry polymer, a system which consists of dry chemical feeder, polymer dispenser, dissolving-aging tank, holding tank and metering pump, is usually required. For liquid polymer dosing, a simple system with dilution tank and metering pump is all that is required for feeding.

2.11.6 Alum Feed System

2.11.6.1 Input Data

2.11.6.1.1 Alum dosage rate, lb/day as $Al_2(SO_4)_3 \cdot 14\ H_2O$.

2.11.6.2 Design Parameters.

2.11.6.2.1 Alum dosage rate, lb/day as $Al_2(SO_4)_3 \cdot 14\ H_2O$.

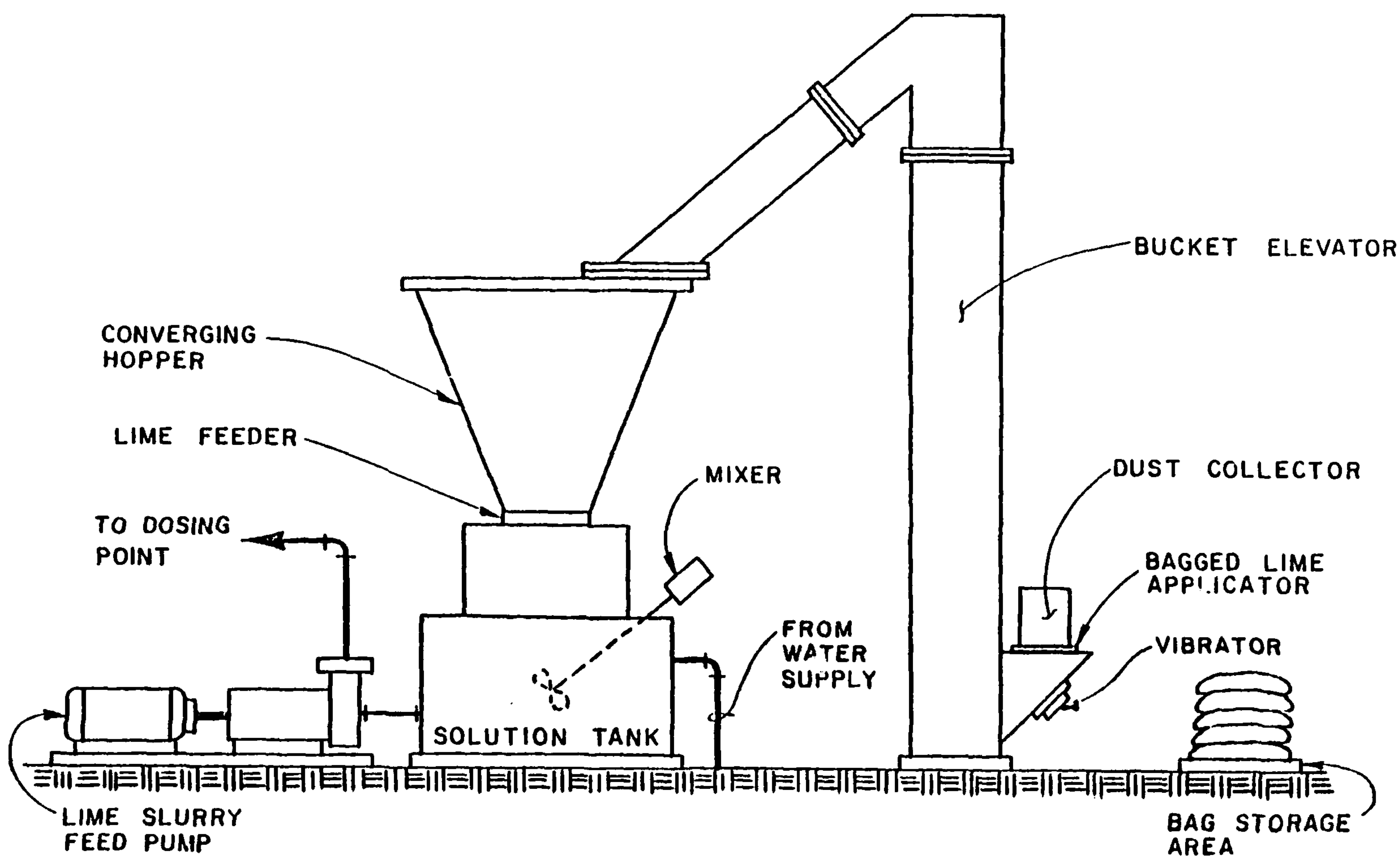

FIGURE 2.11-2 BAGGED LIME FEEDING SYSTEM SCHEMATIC

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL. 6/18

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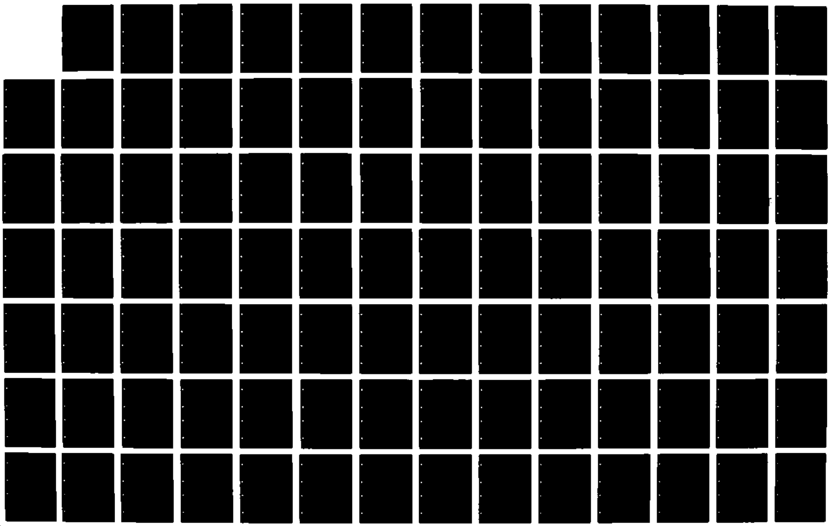

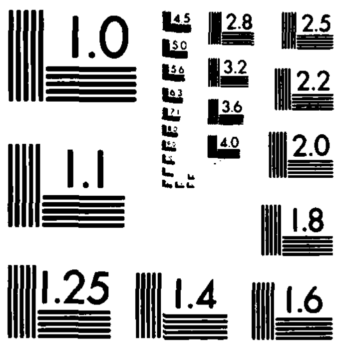

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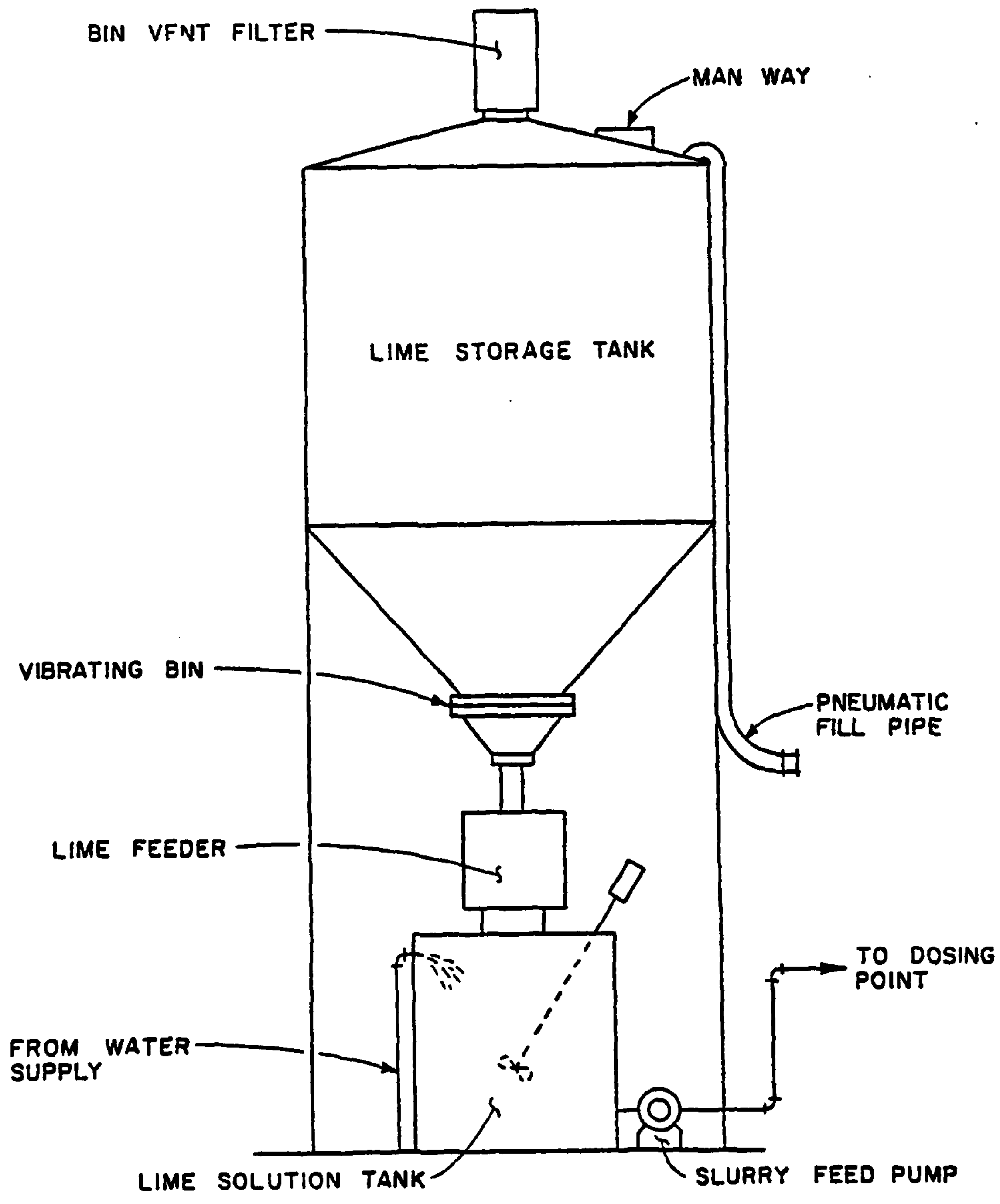

FIGURE 2.11-3. HYDRATED LIME STORAGE SILO AND FEED EQUIPMENT

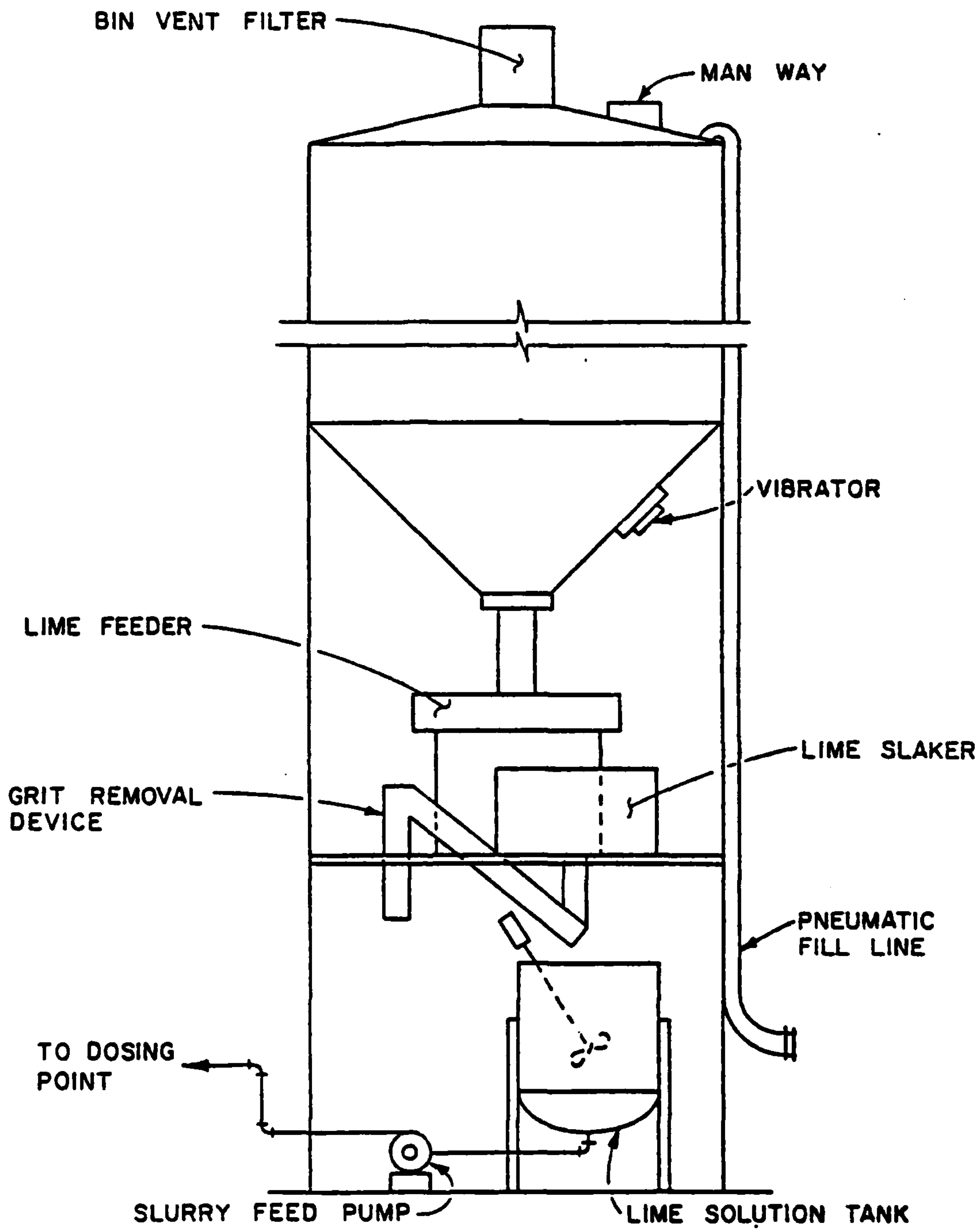

FIGURE 2.11-4. QUICKLIME STORAGE SILO, SLAKING AND FEEDING EQUIPMENT

2.11.6.3 Process Design Calculations. The costing for this unit process is parametric and is determined by the dosage rate, therefore no process design calculations are required.

2.11.6.4 Process Design Output Data. Not used.

2.11.6.5 Quantities Calculations.

2.11.6.5.1 Operation and Maintenance Manpower.

2.11.6.5.1.1 The operation and maintenance manpower is estimated as a function of gallons of liquid chemical fed per day. It is assumed that liquid alum contains 0.4902 lb of aluminum per gallon.

$$LCV = \frac{(ALUM)\ (27)}{0.4902\ (603)}$$

where

27/603 = conversion from alum to filter aluminum.

LCV = liquid chemical solution fed per day, gpd.

AL = alum dosage in lb/day as $Al_2(SO_4)_3 \cdot 14H_2O$.

2.11.6.5.1.2 Calculate O&M manpower.

2.11.6.5.1.2.1 If LCV < 90 gpd.

$$OMMH = 600$$

2.11.6.5.1.2.2 If 90 ≤ LCV < 350 gpd.

$$OMMH = 189.2\ (LCV)^{0.2565}$$

2.11.6.5.1.2.3 If 350 ≤ LCV < 1050 gpd.

$$OMMH = 33.4\ (LCV)^{0.5527}$$

2.11.6.5.1.2.4 If 1050 ≤ LCV ≤ 10,000 gpd.

$$OMMH = 51.8\ (LCV)^{0.4894}$$

2.11.6.5.1.2.5 If LCV > 10,000 gpd.

$$OMMH = 12.2\ (LCV)^{0.647}$$

where

OMMH = operation and maintenance manpower, MH/yr.

LCV = liquid chemical solution fed per day, gpd.

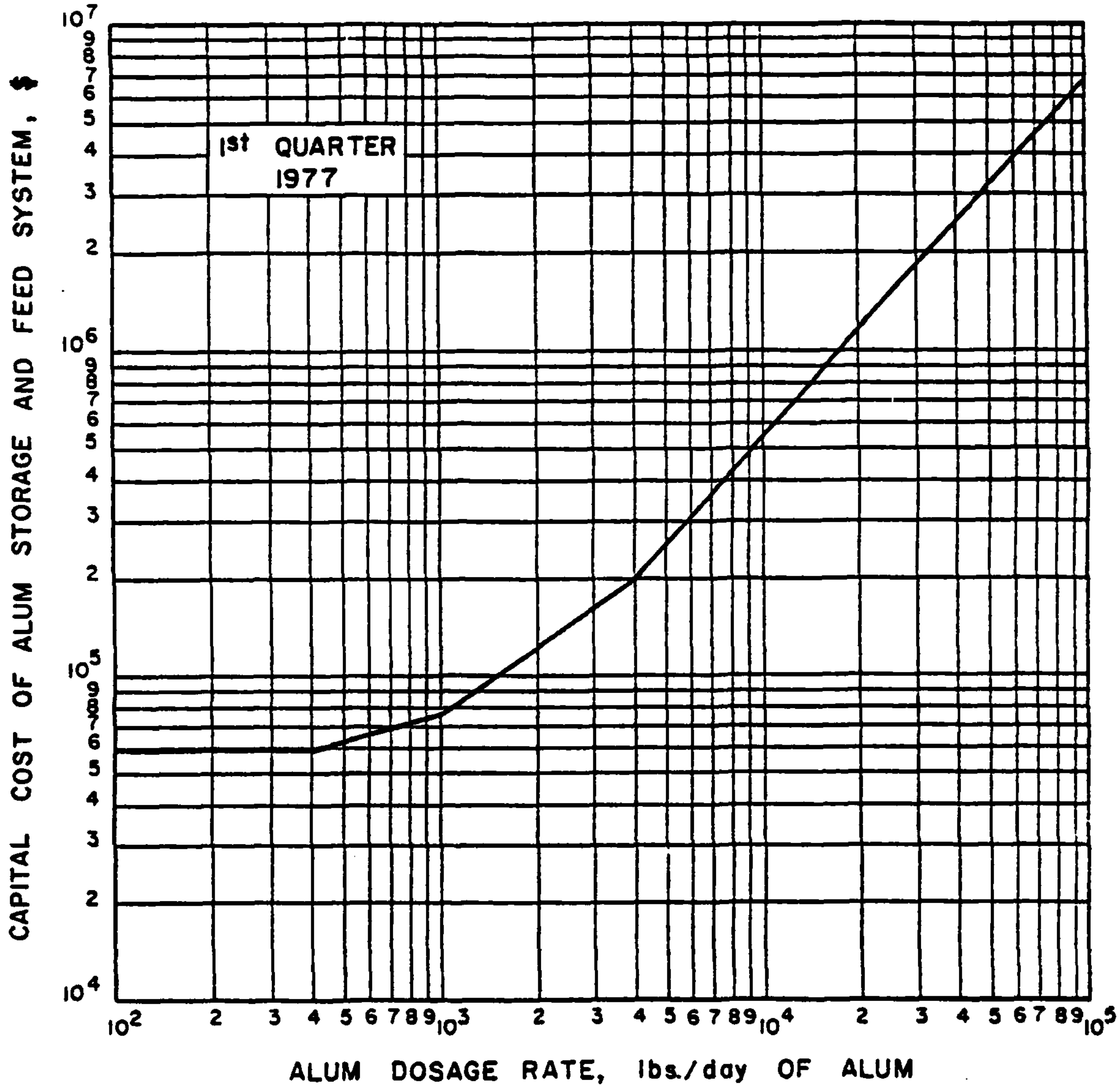

FIGURE 2.11-5. CAPITAL COST OF ALUM STORAGE AND FEED SYSTEM

2.11.6.5.2 Operation and maintenance material and supply costs. The O&M material and supply costs for the alum feed system are not available in the literature. However, information for various components of the system are available. By combining the information for the components an average value of approximately 2 percent of the capital cost was obtained.

$$OMMP = 0.02$$

where

OMMP = O&M material and supply cost as fraction of capital cost, fraction.

2.11.6.5.3 Electrical energy required. The electrical energy required for this process is considered negligible in comparison with overall treatment facility energy usage.

2.11.6.6 Quantities Calculations Output Data.

2.11.6.6.1 Operation and maintenance manpower required, OMMH, MH/yr.

2.11.6.6.2 O&M material and supply costs as fraction of capital costs, OMMP, fraction.

2.11.6.7 Unit Price Input Required. None as parametric costing is used.

2.11.6.8 Cost Calculations.

2.11.6.8.1 Capital cost of alum storage and feed system. A cost curve relating the capital expenditure for liquid alum feed system to the dosage rate is shown in Figure 2.11-5. This is obtained by upgrading an existing curve to first quarter 1977 costs by means of EPA cost index. In order to use this curve, the alum feed rate has to be expressed as equivalent amount of aluminum. Thus

$$AL = ALUM \times \frac{27}{603}$$

where

AL = alum dosage rate expressed as equivalent aluminum, lb/day.

ALUM = alum dosage rate, lb/day as $Al_2(SO_4)_3 \cdot 14\ H_2O$.

27 = molecular weight of alum.

603 = molecular weight of filter alum.

2.11.6.8.2 The cost of alum feed system can be estimated by:

When AL 400 lb/day

$$CALFED = \frac{LCAT}{132} x\ 59{,}000$$

When $400 \leq AL < 1000$ lb/day

$$CALFED = \frac{LCAT}{132} (10{,}260) \cdot (AL)^{0.289}$$

When $1000 \leq AL \leq 4000$ lb/day

$$CALFED = \frac{LCAT}{132} \cdot (661.7) \cdot (AL)^{0.689}$$

When $AL > 4000$ lb/day

$$CALFED = \frac{LCAT}{132} \cdot (2.419)(AL)^{1.365}$$

where

CALFED = capital cost of alum storage and feed system, \$.

LCAT = EPA Cost Index for Larger City Advanced Treatment.

2.11.6.8.3 Calculate O&M material and supply cost.

$$OMMC = (CALFED)\ (OMMP)$$

where

OMMC = O&M material and supply costs, \$/yr.

CALFED = capital cost of alum storage and feed system, \$.

OMMP = O&M material and supply costs as fraction of capital cost, fraction.

2.11.6.9 Cost Calculations Output Data.

2.11.6.9.1 Capital cost of alum storage and feed systems, CALFED, \$.

2.11.6.9.2 O&M material and supply costs, \$/yr.

2.11.7 Iron Salt Feed Systems

2.11.7.1 Input Data

2.11.7.1.1 Iron salt dosage rate, lb/day as $FeCl_3$.

2.11.7.2 Design Parameters.

2.11.7.2.1 Iron salt dosage rate, lb/day as $FeCl_3$.

2.11.7.3 Process Design Calculations. The costing for this unit process is parametric and is determined by the dosage rate, therefore no process design calculations are required.

2.11.7.4 Process Design Output Data. Not Used.

2.11.7.5 Quantities Calculations.

2.11.7.5.1 Calculate operation and maintenance manpower.

2.11.7.5.1.1 The O&M manpower has been related in the literature to the amount of equivalent iron feed rate each day in a liquid solution. It will be assumed that liquid ferric chloride contains 4.11 lbs. of iron per gallon.

$$FE = IRON \frac{55.8}{162.2}$$

$$LCV = \frac{FE}{4.11}$$

where

IRON = ferric chloride dosage rate, lb/day.

FE = iron salt dosage rate expressed as equivalent iron molecules, lb/day.

55.8 = molecular weight of iron.

162.2 = molecular weight of $FeCl_3$.

LCV = liquid chemical solution feed per day, gpd.

2.11.7.5.1.2 If LCV < 90 gpd.

$$OMMH = 600$$

2.11.7.5.1.3 If $90 \leq LCV < 350$ gpd.

$$OMMH = 189.2\ (LCV)^{0.2565}$$

2.11.7.5.1.4 If $350 \leq LCV < 1050$ gpd.

$$OMMH = 33.4\ (LCV)^{0.5527}$$

2.11.7.5.1.5 If 1050 LCV 10,000 gpd.

$$OMMH = 51.8\ (LCV)^{0.4894}$$

2.11.7.5.1.6 If LCV 10,000 gpd.

$$OMMH = 12.2\ (LCV)^{0.647}$$

where

OMMH = O&M manpower requirement, MH/yr.

LCV = liquid chemical solution feed per day, gpd.

2.11.7.5.2 Calculate operation and maintenance material and supply cost. The operation and maintenance material and supply costs for the iron salt feed system are not available in the literature. However, information for the various components of the system are available and a combination of these valves indicates the cost to be approximately 2 percent of the capital cost.

$$OMMP = 0.02$$

where

OMMP = O&M material and supply cost as fraction of capital cost, fraction.

2.11.7.5.3 Electrical energy requirement. The electrical energy requirement for this system is insignificant in comparison with the energy requirement for the entire treatment facility.

2.11.7.6 <u>Quantities Calculations Output Data</u>.

2.11.7.6.1 Operation and maintenance manpower requirements OMMH, MH/yr.

2.11.7.6.2 O&M material and supply costs as a fraction of capital cost, OMMP, fraction.

2.11.7.7 <u>Unit Price Input Required</u>. None as parametric costing is used.

2.11.7.8 Cost Calculations.

2.11.7.8.1 Capital cost of iron salt storage and feed system. A cost curve relating the capital expenditure for liquid $FeCl_3$ feed system to the dosage rate is shown in Figure 2.11-6. This is obtained by upgrading an existing curve to first quarter 1977 costs by means of EPA cost index. In order to use this curve, the iron salt feed rate has to be expressed as equivalent amount of iron.

$$FE = IRON \cdot \frac{55.8}{162.2}$$

where

FE = iron salt dosage rate expressed as equivalent iron molecules, lb/day.

IRON = ferric chloride dosage rate, lb/day.

55.8 = molecular weight of iron.

162.2 = molecular weight of $FeCl_3$.

The cost of iron salts storage and feed system can be esimated by the following equations.

2.11.7.8.1.1 When FE < 1000 lb/day

$$CFEFDD = \frac{LCAT}{132} \times 59{,}000$$

2.11.7.8.1.2 When 1000 ≤ FE < 4000 lb/day

$$CFEFED = \frac{LCAT}{132} (3352) \ (FE)^{0.4152}$$

2.11.7.8.1.3 When 4000 ≤ FE ≤ 10,000 lb/day.

$$CFEFED = \frac{LCAT}{132} \ (86.92) \ (FE)^{0.8857}$$

2.11.7.8.1.4 When FE > 10,000 lb/day

$$CFEFED = \frac{LCAT}{132} (0.458) \ (FE)^{1.425}$$

where

Fe = iron salt dosage rate expressed as equivalent iron molecules, lb/day.

CFEFED = capital costs of iron salt feed system, $.

LCAT = current EPA cost index for Larger City Advanced Treatment.

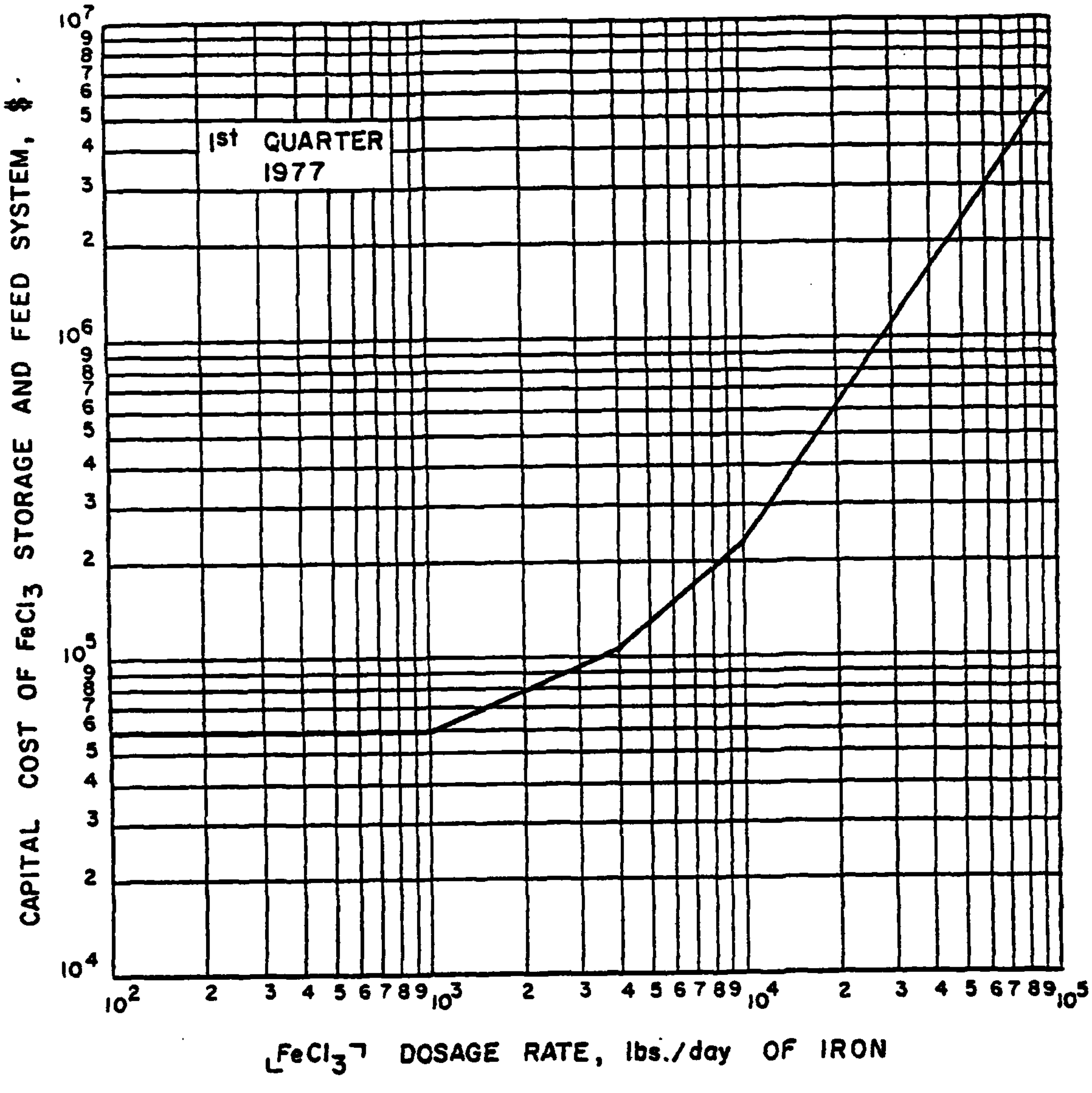

FIGURE 2.11-6. CAPITAL COST OF $FeCl_3$ STORAGE AND FEED SYSTEM

2.11.7.8.2 Calculate O&M material and supply costs.

$$OMMC = (CFEFED)\ (OMMP)$$

where

OMMC = O&M material and supply costs, \$/yr.

CFEFED = capital cost of iron salt feed system, \$.

OMMP = O&M material and supply costs as fraction of capital cost, fraction.

2.11.7.9 Cost Calculations Output Data

2.11.7.9.1 Capital cost of iron salt feed system, CFEFED, \$.

2.11.7.9.2 O&M material and supply costs, OMMC, \$/yr.

2.11.8 Lime Feed System

2.11.8.1 Input Data

2.11.8.1.1 Lime dosage rate, lb/day as $Ca(OH)_2$.

2.11.8.2 Design Parameters

2.11.8.2.1 Lime dosage rate, lb/day as $Ca(OH)_2$.

2.11.8.3 Process Design Calculations. The costing for this unit process is parametric and is determined by the dosage rate, therefore no process design calculations are required.

2.11.8.4 Process Design Output Data. Not Used.

2.11.8.5 Quantities Calculations.

2.11.8.5.1 Calculate operation and maintenance manpower.

2.11.8.5.1.1 The O&M manpower can be estimated as a function of gallons of liquid chemical fed per day. It is assumed that a lime solution contains 0.5 lb of $Ca(OH)_2$ per gallon.

$$LCV = \frac{LIME}{0.5}$$

where

LCV = liquid chemical solution fed per day, gpd.

LIME = lime dosage rate in lb/day as $Ca(OH)_2$, lb/day.

2.11.8.5.1.2 The operation and maintenance manpower is a function of gallons of liquid chemical used.

2.11.8.5.1.2.1 If LCV 90 gpd.

$$OMMH = 600 + 92.5\ (LCV)^{0.2827}$$

2.11.8.5.1.2.2 If 90 LCV 350 gpd.

$$OMMH = 189.2\ (LCV)^{0.2565} + 92.5\ (LCV)^{0.2827}$$

2.11.8.5.1.2.3 If 350 LCV 1050 gpd.

$$OMMH = 33.4\ (LCV)^{0.5527} + 92.5\ (LCV)^{0.2827}$$

2.11.8.5.1.2.4 If 1050 LCV 10,000 gpd.

$$OMMH = 51.8\ (LCV)^{0.4894} + 92.5\ (LCV)^{0.2827}$$

2.11.8.5.1.2.5 If LCV 10,000 gpd

$$OMMH = 12.2\ (LCV)^{0.647} + 92.5\ (LCV)^{0.2827}$$

where

OMMH = operation and maintenance manpower required, MH/yr.

LCV = liquid chemical solution fed per day, gpd.

2.11.8.5.2 Operation and maintenance material and supply costs. The operation and maintenance material and supply costs for the lime feed system are not available in the literature. However, infomation for the various components of the system are available and a combination of these values indicates the cost to be approximately 2 percent of the capital costs.

$$OMMP = 0.02$$

where

OMMP = O&M material and supply costs as fraction of capital cost, fraction.

2.11.8.5.3 Electrical energy requirement. The electric energy requirement for this sytem is considered insignificant in comparison with the energy requirement for the entire treatment facility.

2.11.8.6 Quantities Calculations Output Data.

2.11.8.6.1 Operation and maintenance manpower requirements, OMMH, MH/yr.

2.11.8.6.2 O&M material and supply costs as a fraction of capital costs, OMMP, fraction.

2.11.8.7 Unit Price Input Required. None as parametric costing is used.

2.11.8.8 Cost Calculations.

2.11.8.8.1 Lime storage and feed equipment capital costs. By using the rules set up in the preceding subsection and a lime storage capacity of 30 days, a generalized cost curve is given in Figure 2.11-7. This cost curve was generated based on experience and certain estimated data. It gives an approximate estimate of the first quater 1977 cost of lime feed systems.

2.11.8.8.1.1 If LIME 750 lb/day.

$$\mathrm{CCLIME} = \frac{\mathrm{LCAT}}{132} \times \$26,100$$

2.11.8.8.1.2 If LIME 750 lb/day.

$$\mathrm{CCLIME} = \frac{\mathrm{LCAT}}{132} \times 326.7 \cdot (\mathrm{LIME})^{0.6614}$$

where

LIME = lime feed rate as lb/day of $Ca(OH)_2$.

CCLIME = capital costs of lime feed system, $.

LCAT = current EPA cost index for Larger City Advanced Treatment.

132 = LCAT value at first quarter 1977.

2.11.8.8.2 Calculate O&M material and supply costs.

$$\mathrm{OMMC} = (\mathrm{OMMP})(\mathrm{CCLIME})$$

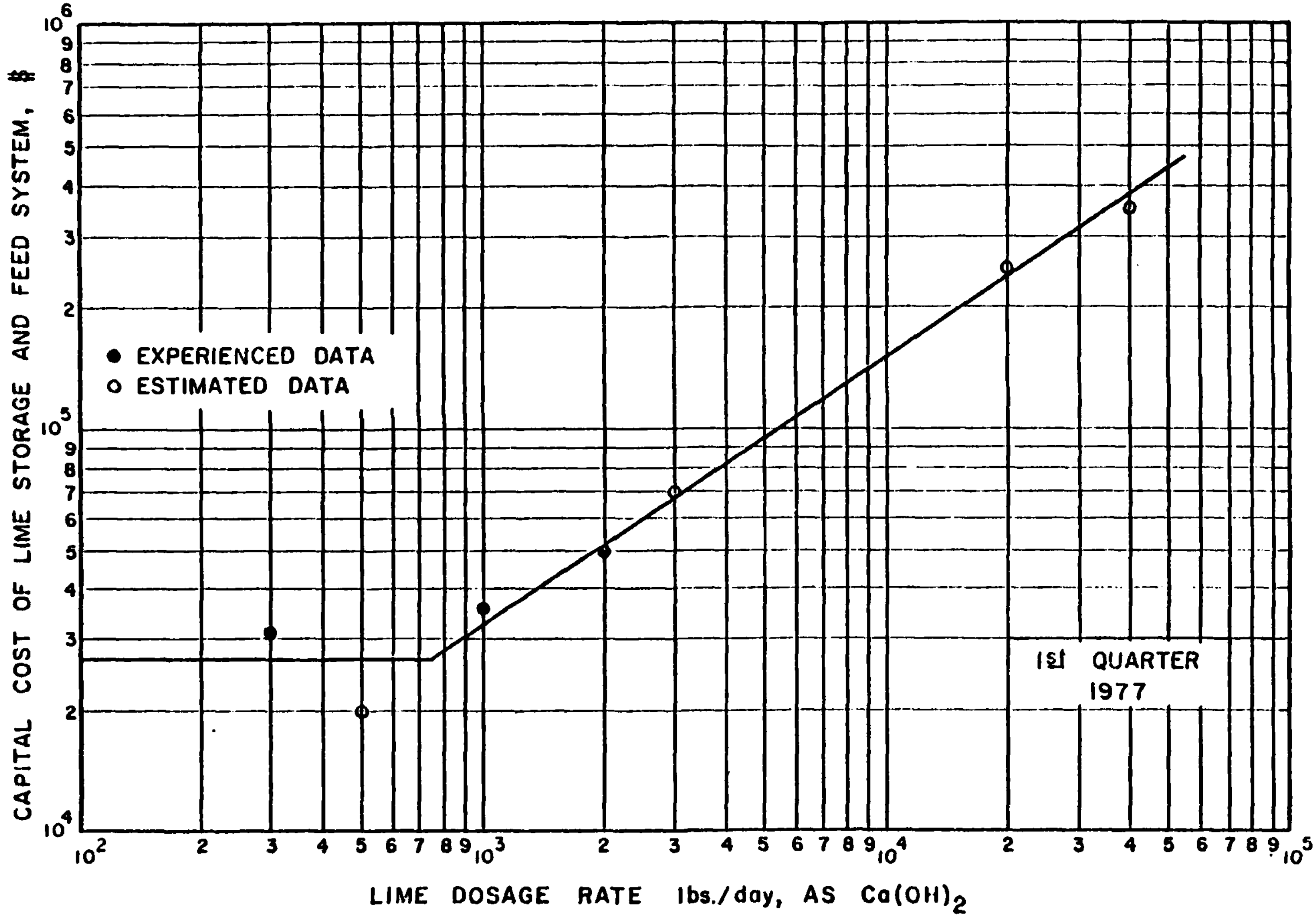

FIGURE 2.11-7. LIME STORAGE AND FEED SYSTEM CONSTRUCTION COSTS

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as fraction of capital costs, fraction

CCLIME = capital costs of lime feed system, \$.

2.11.8.9 Cost Calculations Output Data.

2.11.8.9.1 Capital costs of lime feed system, CCLIME, \$.

2.11.8.9.2 O&M material and supply costs, OMMC, \$/yr.

2.11.9 Polymer Feed System.

2.11.9.1 Input Data.

2.11.9.1.1 Polymer dosage rate, lb/day.

2.11.9.2 Design Parameters.

2.11.9.2.1 Polymer dosage rate, lb/day.

2.11.9.3 Process Design Calculations. The costing for this unit process is parametric and is determined by the polymer dosage rate, therefore, no process design calculations are required.

2.11.9.4 Process Design Output Data. Not Used.

2.11.9.5 Quantities Calculations.

2.11.9.5.1 Calculate operation and maintenance manpower.

2.11.9.5.1.1 The O&M manpower can be estimated as a function of gallons of liquid chemical fed per day. It is assumed that the solution of polymer and water has a concentration of 0.25 percent.

$$LCV = \frac{(PLMER)\ (100)}{(0.25)\ (8.34)}$$

where

LCV = liquid chemical solution fed per day, gpd.

PLMER = polymer dosage rate, lb/day.

2.11.9.5.1.2 If LCV < 90 gpd.

$$OMMH = 600 + 92.5\ (LCV)^{0.2827}$$

2.11.9.5.1.3 If $90 \leq LCV < 350$ gpd.

$$OMMH = 189.2\ (LCV)^{0.2565} + 92.5(LCV)^{0.2827}$$

2.11.9.5.1.4 If $350 \leq LCV < 1050$ gpd.

$$OMMH = 33.4\ (LCV)^{0.4894} + 92.5(LCV)^{0.2827}$$

2.11.9.5.1.5 If $1050 \leq LCV \leq 10{,}000$ gpd,

$$OMMH = 51.8\ (LCV)^{0.4894} + 92.5\ (LCV)^{0.2827}$$

2.11.9.5.1.6 If $LCV > 10{,}000$ gpd,

$$OMMH = 12.2\ (LCV)^{0.647} + 92.5\ (LCV)^{0.2827}$$

where

OMMH = operation and maintenance manpower, MH/yr.

LCV = liquid chemical solution fed per day, gpd.

2.11.9.5.2 Operation and Maintenance Material and Supply Costs. The O&M material and supply costs for the polymer feed system are not available in the literature. However, infomation for the various components of the system are available and a combination of these values indicates that the cost to be approximately 2 percent.

$$OMMP = 0.02$$

where

OMMP = O&M material and supply as fraction of capital costs, fraction.

2.11.9.5.3 Electrical energy requirement. The electrical energy requirement for this unit process is considered insignificant in comparison to the energy requirement for the entire facility.

2.11.9.6 <u>Quantities Calculations Output Data.</u>

2.11.9.6.1 Operation and maintenance manpower requirements, OMMH, MH/yr.

2.11.9.6.2 O&M material and supply cost as a fraction of capital cost, OMMP, fraction.

2.11.9.7 Unit Price Input Required. None as parametric costing is used.

2.11.9.8 Cost Calculations.

2.11.9.8.1 Capital cost of polymer feed system. The cost of polymer feed system is obtained by upgrading an existing curve to 1977 costs by means of EPA Cost Index. It is shown in Figure 2.11-8. The cost of polymer feed system can be estimated by:

2.11.9.8.1.1 When PLMER $<$ 30 lb/day,

$$CPMFED = \frac{LCAT}{132} (4524) \cdot (PLMER)^{0.4075}$$

2.11.9.8.1.2 When PLMER $\geq$ 30 lb/day,

$$CPMFED = \frac{LCAT}{132} (1018) (PLMER)^{0.8562}$$

where

PLMER = polymer dosage, lb/day.

CPMFED = capital cost of polymer feed system, \$.

2.11.9.8.2 O&M material and supply costs.

$$OMMC = (CPMFED)(OMMP)$$

where

OMMC = O&M material and supply costs, \$/yr.

CPMFED = capital cost of polymer feed system, \$.

OMMP = O&M material and supply costs as fraction of capital cost, fraction.

2.11.9.9 Cost Calculations Output Data.

2.11.9.9.1 Capital cost of polymer feed system, CPMFED, \$.

2.11.9.9.2 O&M material and supply cost, OMMC, \$/yr.

2.11.10 Bibliography.

2.11.10.1 AWWA, Water Quality and Treatment, 3rd edition, McGraw-Hill, 1971.

2.11.10.2 Bauer Engineering, Inc., "Survey-Scope Study of Wastewater Management, Chicago-South and Lake Michigan Area", Feb. 8, 1972.

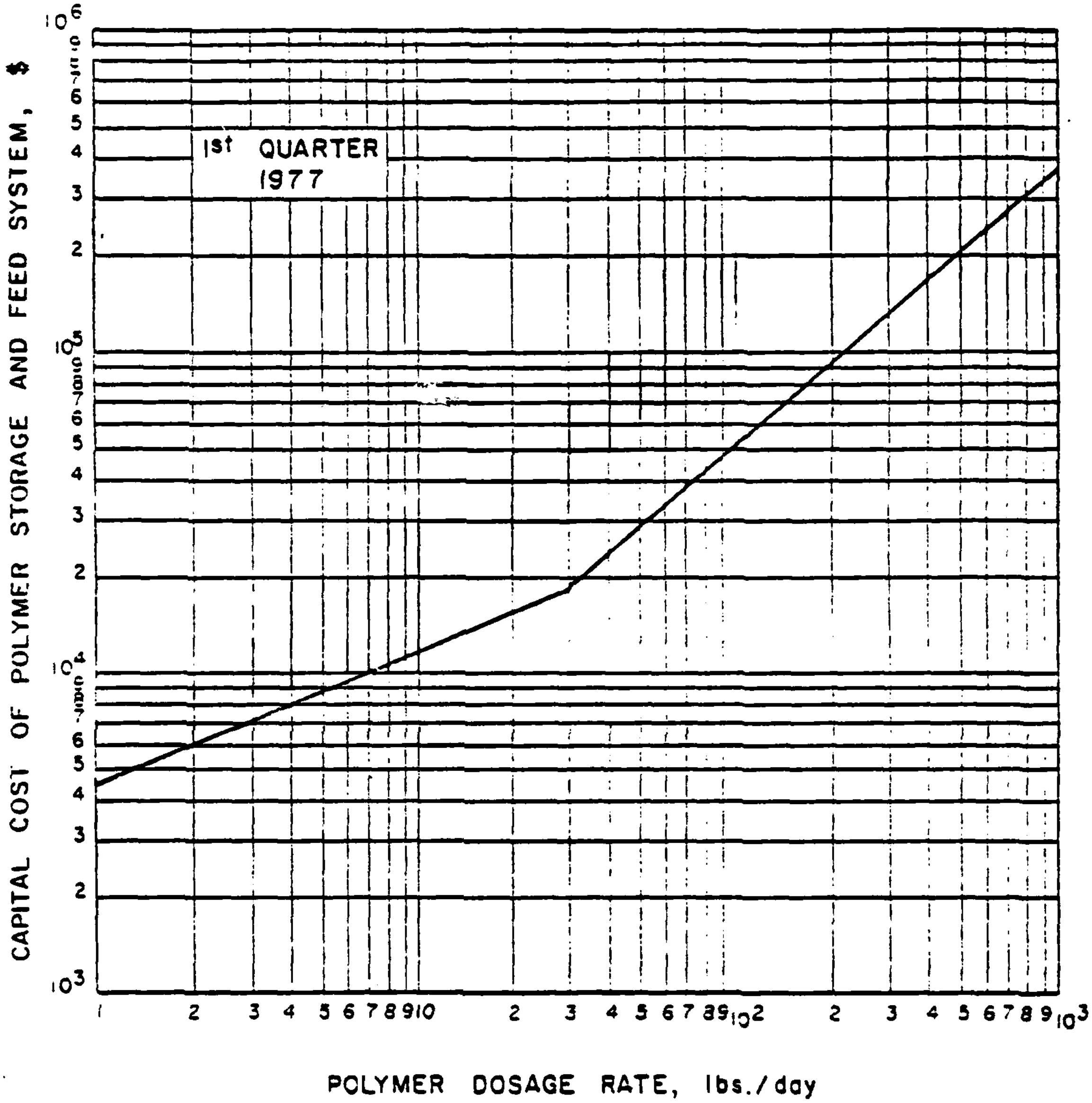

FIGURE 2.11-8. CAPITAL COST OF POLYMER STORAGE AND FEED SYSTEM

2.11.10.3 Blecker and Nichols, "Capital and Operation Costs of Pollution Control Equipment Modules, Vol II Data Manual", EPA-R5-73-023b, 1973.

2.11.10.4 Culp and Culp, Advanced Wastewater Treatment, Van Nostrand Reinhold, 1971.

2.11.10.5 National Lime Association, "Lime, Handling, Application and Storage in Treatment Processes," Bulletin 213, 1971.

2.11.10.6 Smith, R., "Cost of Phosphorus Removal in Conventional Wastewater Treatment Plants by Means of Chemical Addition", EPA Interoffice Memo, May 30, 1977.

2.11.10.7 U. S. EPA Manual of Practice, "Process Design Manual for Suspended Solids Removal", EPA 625/1-75-003a, 1975.

2.13 CHLORINATION

2.13.1 Background.

2.13.1.1 Disinfection is the selective destruction of pathogenic organisms; sterilization is the complete destruction of all microorganisms. Disinfection may be considered as one of the most important processes in wastewater treatment. This practice used in wastewater treatment has resulted in the virtual disappearance of waterborne diseases.

2.13.1.2 Disinfection may be accomplished through the use of chemical agents, physical agents, mechanical means, and radiation. In wastewater treatment the most commonly used disinfectant is chlorine; however, other halogens, ozone, and ultraviolet radiation have been used. Because of its almost universal use, only disinfection using chlorine gas is considered.

2.13.1.3 The rate of disinfection by chlorine depends on several factors, including chlorine dosage, contact time, presence of organic matter, pH, and temperature. The recommended chlorine dosage for disinfection purposes is that which produces a chlorine residual of 0.5 to 1 mg/l after a specified contact time. Effective contact time of not less than 15 min at peak flow is recommended. Typical chlorine dosages recommended for disinfection and odor control are presented in Table 2.13-1.

2.13.1.4 The most common forms of chlorine used in wastewater treatment plants are calcium and sodium hypochlorites and chlorine gas. Hypochlorites are recommended for small treatment plants where simplicity and safety are more important than cost. Chlorine gas may be applied as a gas, or mixed with water to form a solution, a method used almost exclusively in wastewater treatment.

2.13.1.5 The design of the chlorine contact tank plays an important role in the degree of effectiveness produced from chlorination. Factors which must be considered in the design include method of chlorine addition, degree of mixing, minimization of short circuiting, and elimination of solids settling. A recent study indicated that, to minimize short-circuiting, the basin outlet may be designed as a sharp-creasted weir that spans the entire width of the basin outlet. The longitudinal baffling of a serpentine flow basin was superior to cross-baffling; a length-to-width ratio of 40 to 1 was necessary to reach maximum plug flow performance regardless of the type of baffling.

2.13.2 Input Data.

2.13.2.1 Chlorine contact tank influent flow, mgd.

2.13.2.2 Peak flow, mgd.

2.13.2.3 Average flow, mgd.

2.13.3 **Design Parameters.**

2.13.3.1 Contact time at maximum flow, min.

2.13.3.2 Length-to-width ratio.

2.13.3.3 Number of tanks.

2.13.3.4 Chlorine dosage, mg/l.

2.13.4 **Process Design Calculations.**

2.13.4.1 Select contact time at peak flow and calculate volume of contact tank.

$$VCT = \frac{Q_p(CT) \times 10^6}{(24)(60)}$$

where

VCT = volume of contact tank, gal.

Q_p = peak flow, mgd.

CT = contact time at maximum flow, min.

2.13.4.2 Select a side water depth and calculate surface area.

$$SA = \frac{(VCT)}{(7.48)(SWD)}$$

where

SA = surface area, ft^2.

VCT = volume of contact tank, gal.

SWD = side water depth $\approx$ 8 ft.

2.13.4.3 Select a length-to-width ratio and calculate dimensions.

$$CTW = \left(\frac{SA}{RLW}\right)^{1/2}$$

$$CTL = \frac{SA}{CTW}$$

where

CTW = contact tank width, ft.

SA = surface area, ft^2.

RLW = length-to-width ratio.

CTL = contact tank length, ft.

2.13.4.4 Select chlorine dosage (Table 2.13-1) and calculate chlorine requirements.

$$CR = (Q_{avg})(CD)(8.34)$$

where

CR = chlorine requirement, lb/day.

Q_{avg} = average flow, mgd.

CD = chlorine dosage, mg/l.

2.13.4.5 Calculate peak chlorine requirements.

$$PCR = (CR)\frac{Q_p}{Q_{avg}}$$

where

PCR = peak chlorine requirements, lb/day.

CR = chlorine requirements, lb/day.

Q_p = peak flow, mgd.

Q_{avg} = average flow, mgd.

2.13.4.6 Effluent Quality. It is assumed that none of the characteristics are changed by this unit process. The following equation by Shelleck, Collins and White is used to predict coliform reduction.

$$COLIFR = [1.0 - ((1.0 + (.23)(CD)(CT))^{-3}]\,100$$

where

COLIFR = coliform reduction, %.

CD = chlorine dosage, mg/l.

CT = contact time, minutes.

2.13.5 Process Design Output Data.

2.13.5.1 Maximum flow, mgd.

2.13.5.2 Average flow, mgd.

2.13.5.3 Contact time, min.

2.13.5.4 Volume of contact tank, gal.

2.13.5.5 Average chlorine requirement, lb/day.

2.13.5.6 Peak chlorine requirement, lb/day.

2.13.5.7 Tank dimensions.

Table 2.13-1. Typical Chlorine Dosages for Disinfection and Odor Control

Effluent from	Dosage Range mg/l
Untreated wastewater (prechlorination)	6 to 25
Primary sedimentation	5 to 20
Chemical precipitation plant	2 to 6
Trickling filter plant	3 to 15
Activated sludge plant	2 to 8
Multimedia filter following activated sludge plant	1 to 5

2.13.6 <u>Quantities Calculations</u>

2.13.6.1 Calculate size and number of chlorine cylinders.

2.13.6.1.1 If the chlorine requirement (CR) is between 0 and 50 lb/day, use 150-lb cylinders. Assume that a minimum of 30 days supply will be kept on hand. The minimum number of cylinders, regardless of chlorine requirement, will be two 150-lb cylinders.

$$N_c = \frac{CR\ (30)}{(150)}$$

If N_c is not an integer, use the next larger integer.

where

N_c = number of chlorine cylinders required.

CR = chemical requirement, lb/day.

2.13.6.1.2 If the chlorine requirement is greater than 50 lb/day, use 1-ton cylinders. The minimum number of cylinders is 2.

$$N_c = \frac{CR\ (30)}{(2000)}$$

If N_c is not an integer, use the next larger integer.

where

N_c = number of cylinders.

CR = chlorine requirement, lb/day.

2.13.6.2 Calculate chlorination building area. The chlorination building is sized according to the area required for the anticipated equipment to be used for the various chlorine requirements.

2.13.6.2.1 If CR is less than 2000 lb/day,

$$BA_c = 220 \text{ sq ft}$$

2.13.6.2.2 If CR is greater than 2000 lb/day,

$$N_u = \frac{CR}{8000}$$

If N_u is not an integer, use the next larger intager.

$$BA_c = 360 \times N_u$$

where

BA_c = chlorination building area, sq ft.

CR = chlorine requirement, lb/day.

N_u = number of chlorinators and evaporators required.

2.13.6.3 Calculate chlorine storage area.

2.13.6.3.1 If CR is less than 50 lb/day,

$$A_s = 16 \times N_c$$

2.13.6.3.2 If CR is greater than 50 lb/day,

$$A_s = 140 \times N_c$$

where

N_c = number of chlorine cylinders required.

A_s = area of chlorine storage building, sq ft.

2.13.6.4 Chlorine contact tank.

2.13.6.4.1 The basic configuration of the chlorine contact tanks is shown in Figure 2.13-1.

2.13.6.4.2 Dimensions for the various size contact tanks are shown in Table 2.13-2. Quantities of reinforced concrete and the earthwork were calculated using the dimensions shown in Table 2.13-2.

2.13.6.5 Calculate earthwork for chlorine contact tank.

2.13.6.5.1 If Q_{avg} is between 0.50 mgd and 3.75 mgd, the volume of earthwork is calculated by:

$$V_{ew} = (3700)\ (Q_{avg})^{0.3782}$$

2.13.6.5.2 If Q_{avg} is between 3.75 mgd and 100 mgd, the volume of earthwork is calculated by:

$$V_{ew} = 1870\ (Q_{avg})^{0.8873}$$

2.13.6.5.3 If Q_{avg} is between 100 mgd and 200 mgd, the volume of earthwork is calculated by:

$$V_{ew} = 2021.9\ (Q_{avg})^{0.8873}$$

2.13.6.5.4 If Q_{avg} is between 200 mgd and 300 mgd, the volume of earthwork is calculated by:

$$V_{ew} = 2116.4\ (Q_{avg})^{0.8873}$$

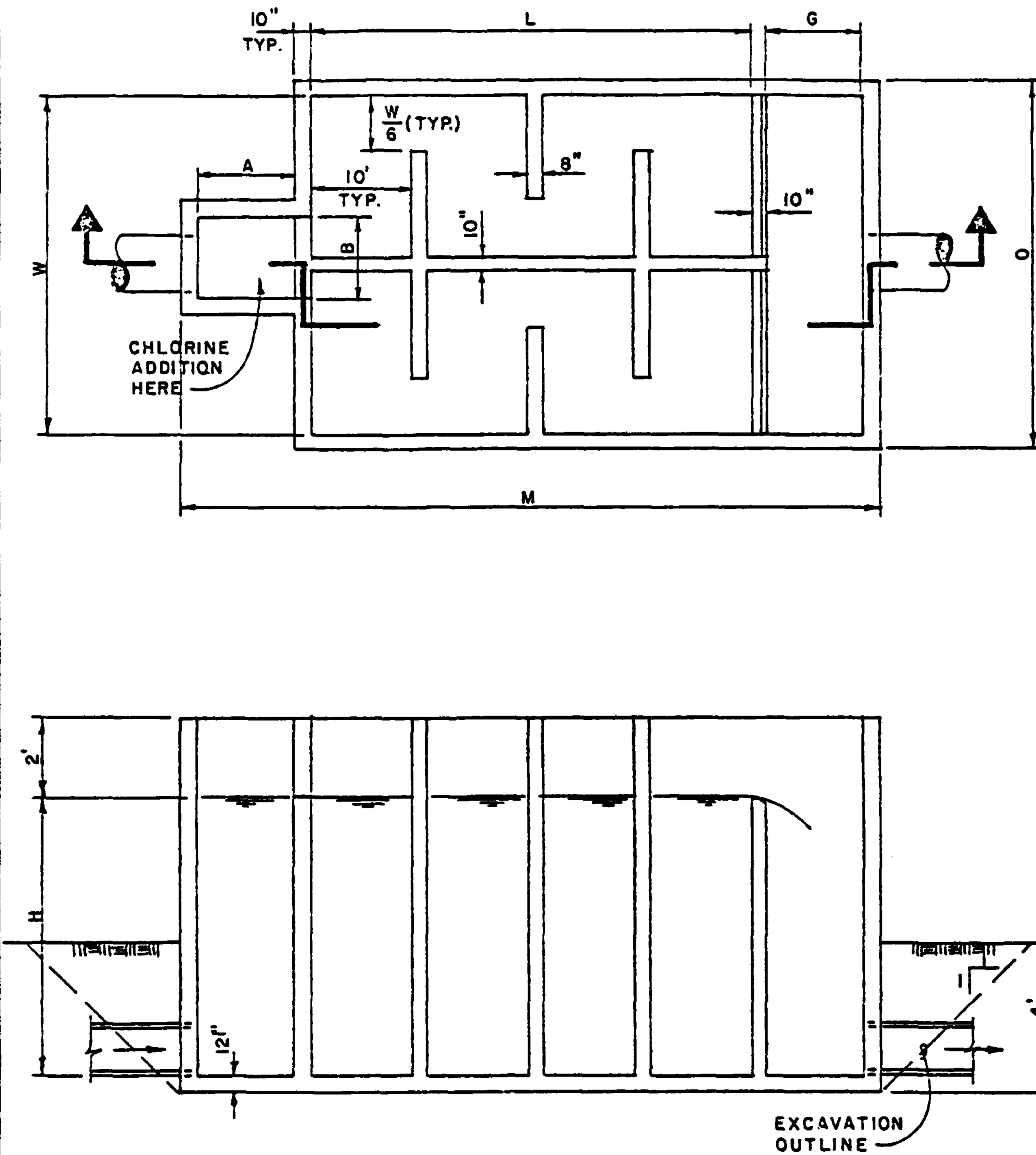

FIGURE 2.13-1 TYPICAL CHLORINE CONTACT CHAMBER

Table 2.13-2

CHLORINE CONTACT CHAMBER DIMENSIONS

Flow Range	Rectangular Chlorine Contact Chamber Dimensions					
	A (ft)	B (ft)	h (ft)	L (ft)	W (ft)	G (ft)
0.5 - 1.00	2.5	2.5	5	40	10	2.5
1.00 - 2.50	4	4	10	40	10	4
2.50 - 5.00	4	7.5	10	55	15	4
5.00 - 10.00	4	15	10	85	20	4
10.00 - 20.00	4	30	10	92	40	4
20.00 - 40.00	5	50	10	150	50	5
40.00 - 80.00	8	60	10	230	65	8
80.00 - 100.00	8	77	10	280	80	8

where

Q_{avg} = average daily flow, mgd.

V_{ew} = volume of earthwork, cu ft.

2.13.6.6 Calculate the volume of reinforced concrete required for the walls of the contact tank.

2.13.6.6.1 If Q_{avg} is between 0.5 mgd and 3.75 mgd, the volume of concrete required is calculated by:

$$V_{cw} = (2000)\ (Q_{avg})^{0.2545}$$

2.13.6.6.2 If Q_{avg} is between 3.75 mgd and 100 mgd, the volume of concrete required is calculated by:

$$V_{cw} = (1100)\ (Q_{avg})^{0.7108}$$

2.13.6.6.3 If Q_{avg} is between 100 mgd and 200 mgd, the volume of concrete required is calculated by:

$$V_{cw} = (1344.1)\ (Q_{avg})^{0.7108}$$

2.13.6.6.4 If Q_{avg} is between 200 mgd and 300 mgd, the volume of concrete required is calculated by:

$$V_{cw} = (1511.4)\ (Q_{avg})^{0.7108}$$

where

Q_{avg} = average daily flow, mgd.

V_{cw} = volume of reinforced concrete for wall, cu ft.

2.13.6.7 Calculate the volume of reinforced concrete required for the slab of the contact tank.

2.13.6.7.1 If Q_{avg} is between 0.5 mgd and 3.75 mgd, the volume of concrete required is calculated by:

$$V_{cs} = (700)\ (Q_{avg})^{0.4078}$$

2.13.6.7.2 If Q_{avg} is between 3.75 mgd and 100 mgd, the volume of concrete required is calculated by:

$$V_{cs} = (340)\ (Q_{avg})^{0.9554}$$

2.13.6.7.3 If Q_{avg} is between 100 mgd and 200 mgd, the volume of concrete required is calculated by:

$$V_{cs} = (350.7)\ (Q_{avg})^{0.9554}$$

2.13.6.7.4 If Q_{avg} is between 200 mgd and 300 mgd, the volume of concrete required is calculated by:

$$V_{cs} = (357.1)\ (Q_{avg})^{0.9554}$$

where

Q_{avg} = average daily flow, mgd.

V_{cs} = volume of reinforced concrete for slab, cu ft.

2.13.6.8 Other construction cost items for the chlorine contact chamber.

2.13.6.8.1 From the calculations, approximately 85% of the construction costs of the chlorine contact chamber have been accounted for.

2.13.6.8.2 Other minor items such as grass seeding, site cleaning, piping, etc., would be 15%.

2.13.6.8.3 The correction factor would be $\frac{1}{0.85} = 1.18$.

2.13.6.9 Calculate chlorine requirement for one year.

$$CRTPY = \frac{(CR)\ (365)}{2000}$$

where

CRTPY = chlorine requirement per year, ton/yr.

CR = chlorine requirement, lb/day.

2.13.6.10 Calculate operational labor.

$$OMH = 100\ (CRTPY)^{0.5316}$$

where

OMH = operational labor, man-hours/yr.

CRTPY = chlorine requirement per year, ton/yr.

2.13.6.11 Calculate maintenance labor.

2.13.6.11.1 If the chemical requirement is between 1 ton/yr and 10 ton/yr,

$$MMH = (27)\ (CRTPY)^{0.3141}$$

2.13.6.11.2 If the chemical requirement is between 10 ton/yr and 50 ton/yr.

$$MMH = (18.06)\ (CRTPY)^{0.4914}$$

2.13.6.11.3 If the chemical requirement is greater than 50 ton/yr,

$$MMH = (2.8)\ (CRTPY)^{0.9682}$$

where

MMH = maintenance man-hour requirements, man-hours/yr.

2.13.6.12 Calculate the energy requirement for operation.

2.13.6.12.1 If Q_{avg} is between 0.5 mgd and 5 mgd, the energy requirement is constant.

$$KWH = 118,000$$

2.13.6.12.2 If Q_{avg} is greater than 5 mgd, the energy requirement is found by:

$$KWH = (83,000)\ (Q_{avg})^{0.1991}$$

where

KWH = electrical energy requirement for operation, kwhr/yr.

Q_{avg} = average daily flow, mgd.

2.13.6.13 Calculate the material and supply costs for operation and maintenance.

2.13.6.13.1 Material and supply costs include the materials and replacement parts required to keep the chlorination facilities in proper operating conditions. These costs are estimated as a percent of the construction costs from the following equations.

2.13.6.13.2 If CR is between 0 lb/day and 300 lb/day, the material and supply costs are found by:

$$OMMP = 6.255\ (CR)^{-0.0797}$$

2.13.6.13.3 If CR is greater than 300 lb/day, the material and supply costs are found by:

$$OMMP = 15.01\ (CR)^{-0.2332}$$

where

OMMP = material and supply costs for operation and maintenance as percent of construction costs, percent.

CR = chlorine requirement, lb/day.

2.13.7 Quantities Calculations Output Data.

2.13.7.1 Chlorination building area, BA_c, sq ft.

2.13.7.2 Area of chlorine storage building, A_s, sq ft.

2.13.7.3 Volume of earthwork, V_{ew}, cu ft.

2.13.7.4 Volume of reinforced concrete for walls, V_{cw}, cu ft.

2.13.7.5 Volume of reinforced concrete for slab, V_{cs}, cu ft.

2.13.7.6 Correction factor for construction cost of chlorine contact chamber, CF.

2.13.7.7 Operational labor, OMH, man-hours/yr.

2.13.7.8 Maintenance labor, MMH, man-hours/yr.

2.13.7.9 Energy requirement for operation, KWH, kwhr/yr.

2.13.7.10 Material and supply costs for operation and maintenance, OMMP, percent.

2.13.8 Unit Price Input Required.

2.13.8.1 Standard size chlorinator cost, COSTCLE, $ (optional).

2.13.8.2 Chlorination building cost, UPIBC, $/sq ft.

2.13.8.3 Excavation cost, UPIEX, $/cu yd.

2.13.8.4 Reinforced concrete wall cost, UPICW, $/cu yd.

2.13.8.5 Reinforced concrete slab cost, UPICS, $/cu yd.

2.13.8.6 Current Marshall and Swift Equipment Cost Index, MSECI.

2.13.9 <u>Cost Calculations</u>.

2.13.9.1 Calculate installed equipment costs.

2.13.9.1.1 The equipment purchase cost includes chlorinators, evaporators, scales, leak detector, flow recorder, booster pump, and residual analyser as required according to the size of the facility.

2.13.9.1.1.1 If CR is between 0 lb/day and 50 lb/day.

$$COSTCE = 4.33 \times COSTCLE$$

2.13.9.1.1.2 If CR is between 50 lb/day and 500 lb/day,

$$COSTCE = 5.93 \times COSTCLE$$

2.13.9.1.1.3 If CR is between 500 lb/day and 1000 lb/day.

$$COSTCE = 7.15 \times COSTCLE$$

2.13.9.1.1.4 If CR is between 1000 lb/day and 1500 lb/day.

$$COSTCE = 7.41 \times COSTCLE$$

2.13.9.1.1.5 If CR is between 1500 lb/day and 2000 lb/day.

$$COSTCE = 11.33 \times COSTCLE$$

2.13.9.1.1.6 If CR is between 2000 lb/day and 8000 lb/day,

$$COSTCE = 20.26 \times COSTCLE$$

2.13.9.1.1.7 If CR is greater than 8000 lb/day,

$$COSTCE = \left[\left(\frac{CR}{8000} + 1\right)(13.48) + 3\right] COSTCLE$$

where

CR = chlorine requirement, lb/day.

COSTCE = purchase cost of chlorination equipment, $.

COSTCLE = purchase cost of standard size chlorinator. Chlorinator with 2000 lb/day capacity, $.

2.13.9.1.2 Cost of standard size chlorinator. The approximate cost of a 2000 lb/day chlorinator for the first quarter of 1977 is:

$$COSTCLE = \$2,700$$

For better estimation COSTCLE should be obtained from equipment vendor and treated as unit price input. If COSTCLE is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTCLE = \$2,700 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index first quarter 1977.

2.13.9.1.3 Installation costs of chlorination equipment and other minor equipment. This includes the cost for installing the chlorination equipment and also other costs such as electrical piping, painting, etc. It is expressed as a percent of the equipment costs.

$$ICOST = (0.3)\ COSTCE$$

where

ICOST = installation and minor equipment costs, \$.

COSTCE = purchase cost of chlorination equipment, \$.

2.13.9.1.4 Calculate installed equipment costs.

$$IEC = COSTCE + ICOST$$

where

IEC = installed equipment costs, \$.

2.13.9.2 Calculate cost of chlorination building.

$$COSTCB = BA_C \times UPIBC$$

where

COSTCB = cost of chlorination building, \$.

BA_C = chlorination building area, sq ft.

UPIBC = unit price input building cost, \$/sq ft.

2.13.9.3 Calculate cost of chlorine storage building.

$$COSTS = A_s \times UPIBC \times 0.5$$

where

COSTS = cost of chlorine storage building, \$.

A_s = area of chlorine storage building, sq ft.

0.5 = factor to convert UPIBC to unit cost of storage building.

2.13.9.4 Calculate cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost for excavation, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for excavation, \$/cu yd.

2.13.9.5 Calculate cost of reinforced concrete walls.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of concrete walls, \$.

V_{cw} = volume of reinforced concrete for walls, cu ft.

UPICW = unit price input for concrete walls, \$/cu yd.

2.13.9.6 Calculate cost of reinforced concrete for slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of concrete slab, \$.

V_{cs} = volume of reinforced concrete for slab, cu ft.

UPICS = unit price input for concrete slab, \$/cu yd.

2.13.9.7 Calculate construction cost for chlorine contact chamber.

$$COSTCC = (COSTE + COSTCW + COSTCS)\ (CF)$$

where

COSTCC = construction cost for chlorine contact chamber.

2.13.9.8 Calculate total bare construction cost.

$$TBCC = IEC + COSTCB + COSTS + COSTCC$$

where

TBCC = total bare construction cost, $.

2.13.9.9 Calculate operation and maintenance material costs.

$$OMMC = (TBCC)\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs.

2.13.10 Cost Calculations Output Data.

2.13.10.1 Total bare construction cost, TBCC, $.

2.13.10.2 O&M material and supply costs, OMMC, $/yr.

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2.15 CLARIFICATION

2.15.1 Background. Sedimentation is a solid-liquid separation process designed primarily to remove the suspended particles that are heavier than water. This process is the most popular and most widely used process in waste treatment. Sedimentation removes grit, removes the settleable fraction of the suspended solids from raw waste in primary clarifiers, separates the biological floc from the mixed liquor in the final clarifier of a biological treatment system, separates the chemical floc from the supernatent in physical-chemical systems, and concentrates sludges in thickeners. Sedimentation may be classified into four categories depending on the characteristics of the suspension.

2.15.1.1 Discrete Settling. Suspended solids in this case are discrete particles which retain their identity, size, shape, and settling velocity during the settling process. The main factor influencing the efficiency of the process is the overflow rate expressed as gal/ft^2/day. All particles with settling velocities greater than the design overflow rate will be removed. Particles with settling velocities less than the design overflow rate will be removed in proportion to the ratio of their settling velocities to the design overflow rate. This type of settling normally occurs in a grit chamber.

2.15.1.2 Flocculant Settling. In this type of settling, particles flocculate and agglomerate during settling with changes in size, shape, and density. The settling velocity increases as the particles grow larger. The settling characteristics of the flocculant suspension can be determined through laboratory settling tests. Efficiency of removal is influenced by both the overflow rate and detention time. Flocculant settling normally occurs in a primary clarifier.

2.15.1.3 Zone Settling. This type of settling normally occurs with activated sludge. Particles adhere to each other and settle as a blanket, forming a distinct solid-liquid interface at the top of the settling zone. Removal efficiency is influenced by mass loading, overflow rate, and detention time. Batch sedimentation tests are usually performed to evaluate these characteristics for industrial waste suspensions.

2.15.1.4 Compression Settling. In this case, the concentration of the suspension is so great that the particles rest on each other at the bottom of the sedimentation basin. This type of settling occurs in a sludge thickener.

2.15.2 <u>General Description Primary Clarification.</u>

2.15.2.1 Primary clarifiers are normally used in conjunction with biological waste treatment systems to remove the settleable solids and a fraction of the BOD, thereby reducing the load on the biological systems. Efficiently designed and operated, primary clarifiers can remove 50 to 65 percent of the suspended solids and 25 to 35 percent of the 5-day BOD.

2.15.2.2 Primary clarifiers are usually keyed to the overflow rates (gal/day/ft^2) and detention times. Detention times of 2 to 3 hr, based on average flows, are recommended. Surface loading rates usually depend on the characteristics of the suspension to be separated. Batch laboratory studies may be used to determine the optimum design parameters for a specific suspension. Typical values for various suspensions are reported in Table 2.15-1. Ten State Standards recommends a surface loading not to exceed 600 gal/day/ft^2 for small plants (1 mgd or less). Relationship of overflow rates, detention times, and tank depths are presented in Table 2.15-2.

2.15.2.3 Outlet design and arrangement have been reported to affect the efficiency of the primary clarifier. Weir loadings, defined in the Ten State Standards, should not exceed 10,000 gpd/ft for small plants (1 mgd or less) and 15,000 gpd/ft for larger plants.

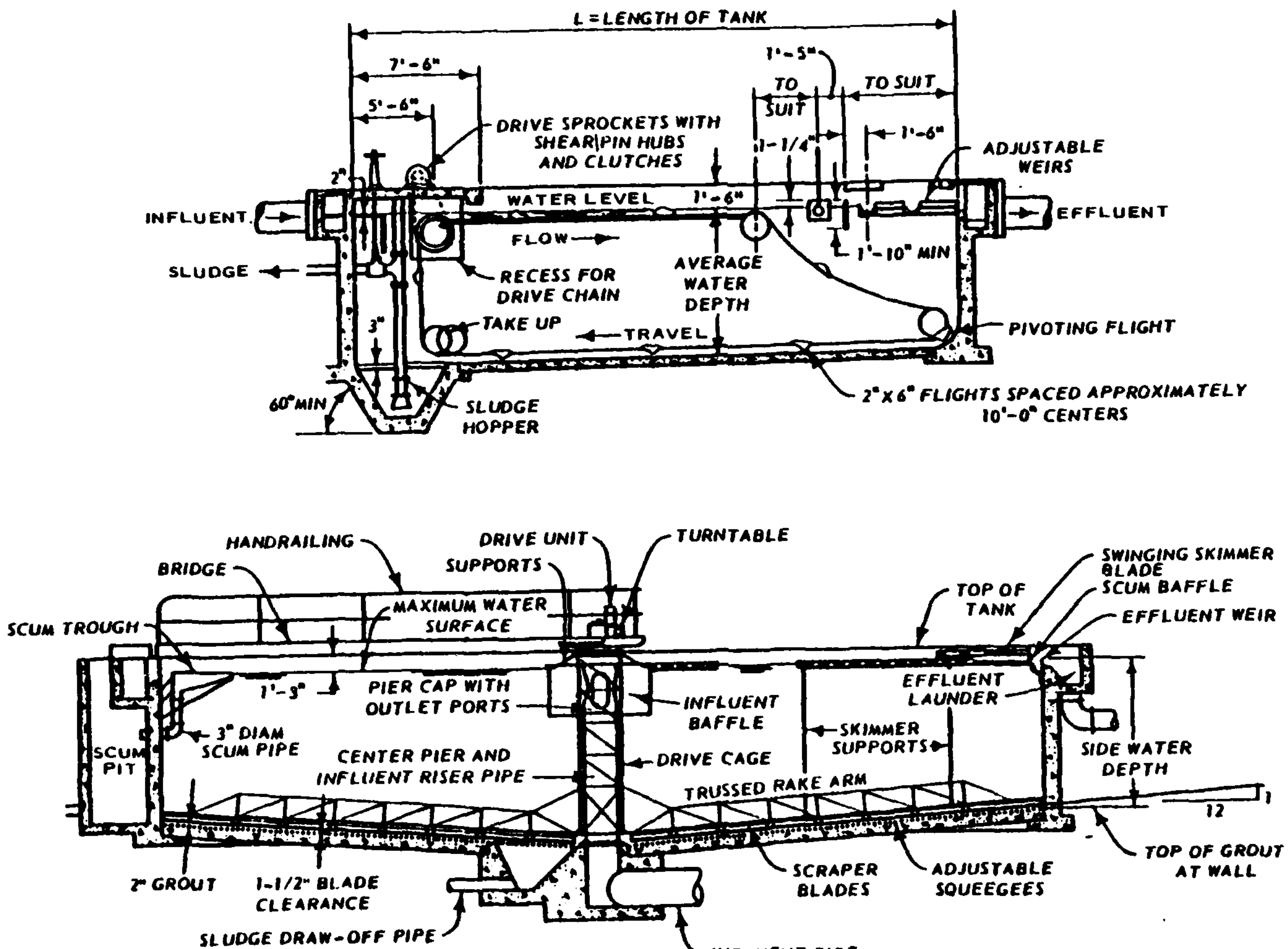
2.15.2.4 Primary clarifiers, rectangular or circular (fig. 2.15-1), are usually cleaned mechanically. Two tanks should be provided to allow for maintenance and repair work.

2.15.2.5 The volume of sludge produced in primary settling tanks depends on several factors which include the characteristics of the raw waste, the design of the clarifier, the condition of the removed solids, and the period between sludge removals. Sludge should be removed continuously or at least once per shift (more frequently in hot weather) to avoid deterioration of the effluent quality. Specific gravities of several types of sludge are shown in Table 2.15-3.

2.15.2.6 Primary sedimentation may be used in both biological treatment systems and physical-chemical systems. Figure 2.15-1 shows typical primary sedimentation tanks.

2.15.3 <u>General Description Secondary Clarification.</u>

2.15.3.1 The final clarifier performs a vital role in a secondary waste treatment system. In the activated sludge process, the final clarifier must provide an effluent low in suspended solids and an underflow of sufficient concentration to maintain a sufficient population of active microbial mass in the aeration tank. Final clarifiers are, therefore, designed to provide clarification, as well as thickening.



From Metcalf and Eddy, 1972

FIGURE 2.15-1. SCHEMATICS OF TYPICAL PRIMARY SEDIMENTATION TANKS

2.15.3.2 In addition to being governed by the overflow rate and detention time, the design of final clarifiers must be based on solid loading rates (lb solids/ft^2/day). Surface overflow rates and detention times recommended by the Ten State Standards are presented in Table 2.15-4. Typical values of overflow rates recommended for the design of secondary clarifiers include 600 gal/ft^2/day for smaller plants (up to 1 mgd), and up to 800 gal/ft^2/day for larger plants. The design calculations should consider the peak incoming wastewater flow; the return sludge withdrawal takes place at a point very near the inlet to the tank. Solids loading rates for various mixed liquor suspended solids are illustrated in Figure 2.15-4. Typical solids loading rates reported range from 12 to 30 lb/ft^2/day. Solid concentration of the underflow ranges from 0.8 to 1.20 percent, by weight.

2.15.3.3 The performance of the final clarifiers is also affected by the method of sludge withdrawal. The preferred sludge collection mechanism is a vacuum- or suction-type draw off. The plow-type collectors with the chain and flight mechanism in rectangular basins or the bridge with attached plows in circular basins ineffectively concentrate the waste activated sludge.

2.15.3.4 The bibliography contains excellent discussions on the design and operations of final clarifiers.

Table 2.15-1. Recommended Surface-Loading Rates for Various Suspensions

Suspension	Loading Rate, gpd/ft^2	
	Range	Peak Flow
Untreated wastewater	600 to 1200	1200
Alum floc[a]	360 to 600	600
Iron floc[a]	540 to 800	800
Lime floc[a]	540 to 1200	1200

[a]Mixed with the settleable suspended solids in the untreated wastewater and colloidal or other suspended solids swept out by the floc.

Table 2.15-2. Detention Times for Various Surface-Loading Rates and Tank Depths

Surface-Loading Rate, gpd/ft^2	Detention Time, hr			
	7-ft Depth	8-ft Depth	10-ft Depth	12-ft Depth
400	3.2	3.6	4.5	5.4
600	2.1	2.4	3.0	3.6
800	1.6	1.8	2.25	2.7
1000	1.25	1.4	1.8	2.2

Table 2.15-3. Specific Gravity of Raw Sludge Produced from Various Types of Sewage

Type of Sewerage System	Strength of Sewage	Specific Gravity
Sanitary	Weak	1.02
Sanitary	Medium	1.03
Combined	Medium	1.05
Combined	Strong	1.07

Table 2.15-4. Design Requirements for Final Settling Tanks

Types of Process	Average Design Flow, mgd	Detention Time, hr.	Surface Settling Rates gal/day/ft^2
Conventional, modified, or "high rate" and step aeration	To 0.5	3.0	600
	0.5 to 1.5	2.5	700
	1.5 and up	2.0	800
Contact stabilization	To 0.5	3.6	500
	0.5 to 1.5	3.0	600
	1.5 and up	2.5	700
Extended aeration	To 0.05	4.0	300
	0.05 to 0.15	3.6	300
	0.15 and up	3.0	600

Note: The inlets and sludge collection and withdrawal facilities shall be designed to minimize density currents and assure rapid return of sludge to the aeration tanks. Multiple units capable of independent operation are desirable and shall be provided in all plants where design flows exceed 0.1 mgd unless other provision is made to assure continuity of treatment.

The detention time, surface settling rate, and weir overflow rate should be adjusted for the various processes to minimize the problems with sludge loadings, density currents, inlet hydraulic turbulence, and occasional poor sludge settleability.

2.15.4 Primary Clarification - Circular.

2.15.4.1 Input Data.

2.15.4.1.1 Wastewater flow.

2.15.4.1.1.1 Average flow, mgd.

2.15.4.1.1.2 Peak flow, mgd.

2.15.4.1.2 Suspended solids, mg/l.

2.15.4.1.3 Volatile suspended solids, percent.

2.15.4.1.4 BOD_5 concentration, mg/l.

2.15.4.2 Design Parameters.

2.15.4.2.1 Overflow rate, gpd/ft^2. See Tables 2.15-1 and 2.15-2.

2.15.4.2.2 Detention time, hr. See Table 2.15-1.

2.15.4.2.3 Specific gravity of sludge. See Table 2.15-3.

2.15.4.2.4 Solids content of underflow, percent (4 to 6 percent).

2.15.4.2.5 Removal efficiencies.

2.15.4.2.5.1 Suspended solids, percent.

2.15.4.2.5.2 BOD_5, percent.

2.15.4.2.6 Weir loading, gpd/ft (10,000 to 15,000 gpd/ft).

2.15.4.3 Process Design Calculations.

2.15.4.3.1 Select an overflow rate by using Table 2.15-1 or by laboratory methods and calculate surface area.

$$SA = \frac{Q_p \times 10^6}{OFR}$$

where

SA = surface area, ft^2

Q_p = peak flow, mgd.

OFR = overflow rate, gal/ft^2/day

2.15.4.3.2 Select detention time and calculate volume (see Table 2.15-2).

$$V = (Q_{avg})\ (t) \times \frac{1}{7.48} \times \frac{1}{24} \times 10^6$$

where

V = volume of tank, ft^3.

Q_{avg} = average flow, mgd.

t = time, hr

2.15.4.3.3 Calculate side water depth.

$$SWD = \frac{V}{SA}$$

where

SWD = side water depth, ft

V = volume, ft^3

SA = surface area, ft^2

2.15.4.3.4 Check solid loading rate.

$$SLR = \frac{(Q_{avg})\ (SSI)\ (8.34)}{(SA)}$$

where

SLR = solid loading rate, $lb/ft^2/day$

Q_{avg} = average flow, mgd.

SSI = influent solids concentration, mg/l.

SA = surface area, ft^2

2.15.4.3.5 Select weir loading rate and calculate weir length.

$$WL = \frac{Q_P}{WLR} \times 10^6$$

where

WL = weir length, ft

Q_p = peak flow, mgd

WLR = weir loading rate, gal/ft/day

2.15.4.3.6 Determine percentage of suspended solids removed from Figure 2.15-2.

2.15.4.3.7 Calculate amount of primary sludge produced.

$$PSP = (Q_{avg}) (SSI) (SSR) (10^{-2}) (8.34)$$

where

PSP = primary sludge produced, lb/day

Q_{avg} = average flow, mgd

SSI = influent solids concentration, mg/l

SSR = suspended solids removed, percent (50 to 60 percent for municipal systems)

2.15.4.3.8 Select underflow concentration (3 to 6 percent) and sludge specific gravity (Table 2.15-3), and calculate the volume flow of primary sludge produced.

$$VPSP = \frac{PSP(100)}{(\text{specific gravity}) (UC) (8.34)}$$

where

VPSP = volume flow of primary sludge produced, gal/day

PSP = primary sludge produced, gal/day

UC = underflow concentration (3 to 6 percent)

2.15.4.3.9 Effluent Characteristics.

2.15.4.3.9.1 BOD. Specify a BOD_5 removal rate or select a removal rate from Figure 2.15-3.

$$BODE = (BODI) (1 - \frac{BODR}{100})$$

$$S_e = (BODSI)$$

If BODE < S_e set BODE = S_e

where

BODE = effluent BOD_5 concentration, mg/l.

BODI = influent BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, %.

S_e = effluent soluble BOD_5 concentration, mg/l.

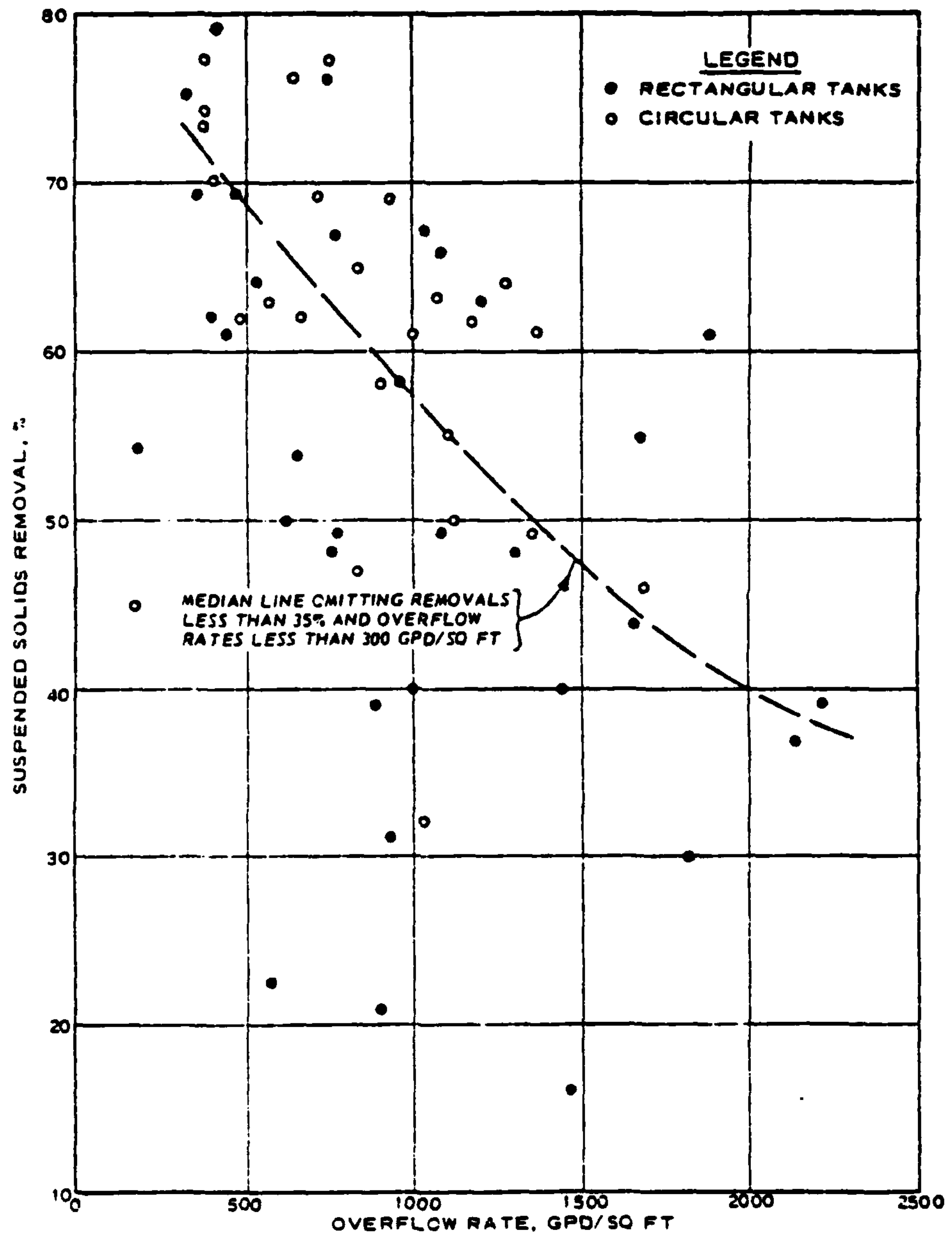


FIGURE 2.15-2. SUSPENDED SOLIDS REMOVAL VERSUS OVERFLOW RATE FOR PRIMARY CLARIFIERS

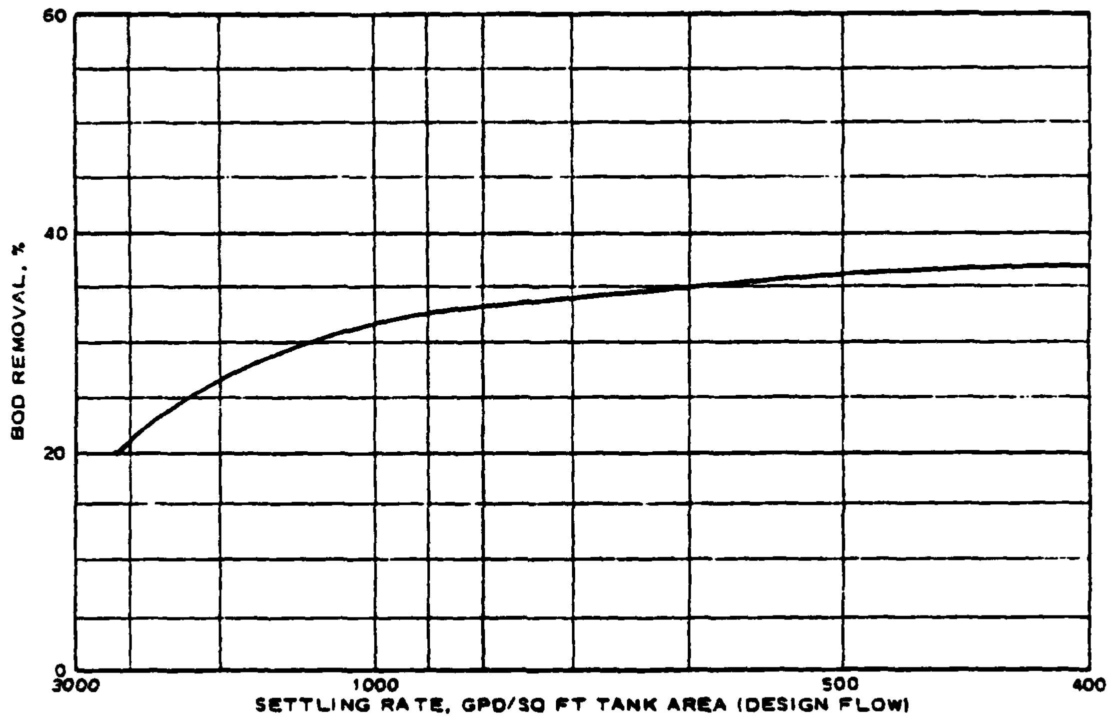


FIGURE 2.15-3. BOD REMOVAL RATE IN PRIMARY CLARIFIER

BODSI = influent soluble BOD_5 concentration, mg/l.

2.15.4.3.9.2 COD.

$$CODE = 1.5\ (BODE)$$

$$CODSE = 1.5\ (S_e)$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.15.4.3.9.3 Suspended solids.

$$SSE = (SSI)\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

2.15.4.3.9.4 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNR}{100})$$

If TKNE   NH3E set TKNE = NH3E

where

TKNE = effluent total Kjedahl nitrogen concentration, mg/l.

TKN = influent total Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

TKNR = total Kjedahl nitrogen removal rate, %.

2.15.4.3.9.5 Phosphorus

$$PO4E = (PO4)\ (1 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

2.15.4.3.9.6 Oil and Grease

$$OAGE = OAG$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

2.15.4.3.9.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids.

2.15.4.4 <u>Process Design Output Data.</u>

2.15.4.4.1 Overflow rate, gal/ft^2/day.

2.15.4.4.2 Surface area, ft^2.

2.15.4.4.3 Side water depth, ft.

2.15.4.4.4 Detention time, hr.

2.15.4.4.5 Solid loading, lb/ft^2/day.

2.15.4.4.6 Weir loading, gal/ft/day.

2.15.4.4.7 Weir length, ft.

2.15.4.4.8 Volume of sludge produced, gal/day.

2.15.4.4.9 Suspended solids removal, percent.

2.15.4.4.10 BOD removal, percent.

2.15.4.4.11 COD removal, percent.

2.15.4.4.12 TKN removal, percent.

2.15.4.4.13 PO_4 removal, percent.

2.15.4.4.14 Effluent BOD, mg/l.

2.15.4.4.15 Effluent suspended solids, mg/l.

2.15.4.4.16 Effluent COD, mg/l.

2.15.4.4.17 Effluent TKN, mg/l.

2.15.4.5 Quantities Calculations.

2.15.4.5.1 Unit selection. The number of units to be employed depends upon the wastewater quantity to be handled. The following assumptions will be followed in the determination of number of units, N.

Flow Range MGD, Q_{avg}		Number of Units
0.5	1.0	2
1.0	10.0	2
10.0	24.0	4
24.0	50.0	8
50.0	70.0	12
70.0	100.0	16

2.15.4.5.1.1 When Q_{avg} is larger than 100 mgd, the system will be designed as several batteries of units.

2.15.4.5.1.2 If $100 < Q_{avg} \leq 200$

Number of process batteries, NB, would be 2.

The system would be designed as two identical batteries with flow to each battery at $Q_{avg}/2$.

Select the number of units per battery as described above using flow to be $Q_{avg}/2$.

2.15.4.5.1.3 If $Q_{avg} > 200$

Number of process batteries, NB, would be 3.

The system would be designed as three identical batteries with flow to each battery at $Q_{avg}/3$.

Select the number of units per battery as described above using flow to be $Q_{avg}/3$.

2.15.4.5.2 Sizing individual unit.

2.15.4.5.2.1 The surface area of each unit.

$$SAU = \frac{SA}{N \cdot NB}$$

where

SAU = surface area per unit, sq ft.

N = number of units.

2.15.4.5.2.2 The diameter of each unit.

$$DIA = \sqrt{\frac{4 \cdot SAU}{3.1416}}$$

If DIA is not an integer, use the next larger integer.

where

DIA = diameter of the unit, ft.

2.15.4.5.2.3 The available sizes from off-the-shelf items range from 10 ft to 200 ft; so if the calculated DIA is larger than 200 ft, the number of units will have to be increased to N = N + 1.

2.15.4.5.3 Earthwork required for construction.

$$V_{ew} = (1.15)\ N\ [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = earthwork required for construction, cu ft.

1.15 = addition of 15% more for safety factor.

2.15.4.5.4 Reinforced concrete quantities.

2.15.4.5.4.1 Reinforced concrete slab quantity, V_{csn}, for one tank.

$$V_{csn} = (0.825)\ (DIA + 4)^2 \cdot \left(\frac{t_s}{12}\right)$$

where

V_{cs} = quantity of R.C. slab in-place, cu ft.

t_s = thickness of the slab, inches, can be obtained by

$$t_s = 7.9 + 0.25\ SWD$$

where

SWD = sidewater depth, ft.

2.15.4.5.4.2 Reinforced concrete wall quantity, V_{cwn}, for one tank.

$$V_{cwn} = (3.14)\ (SWD + 1.5) \cdot (DIA)\ (\frac{t_w}{12})$$

where

V_{cwn} = quantity of R.C. wall in-place, cu ft.

t_w = wall thickness, inches, and can be estimated by

$$t_w = 7 + (0.5)\ (SWD)$$

2.15.4.5.4.3 Quantity of concrete for splitter boxes, V_{cb}

$$V_{cb} = 100 \cdot N^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter box, cu ft.

N = number of units.

2.15.4.5.4.4 Total quantity of R.C. in-place.

$$\text{Wall:}\quad V_{cw} = [N \cdot V_{cwn} + V_{cb}] \cdot NB$$

$$\text{Slab:}\quad V_{cs} = [N \cdot V_{csn}] \cdot NB$$

2.15.4.5.5 Calculate maintenance manpower requirements.

2.15.4.5.5.1 If SA is less than 1000 sq ft, then the maintenance manpower required is:

$$MMH = 200$$

2.15.4.5.5.2 If SA is between 1000 and 3000 sq ft, then the maintenance manpower required is:

$$MMH = 30.3\ (SA)^{0.2733}$$

2.15.4.5.5.3 If SA is greater than 3000 sq ft, then the maintenance manpower required is:

$$MMH = 2.05\ (SA)^{0.6098}$$

where

MMH = maintenance manpower required, man-hours/yr.

SA = surface area, sq ft.

2.15.4.5.6 Calculate operation manpower required.

2.15.4.5.6.1 If SA is less than 1000 sq ft, then the operations manpower required is:

$$OMH = 350$$

2.15.4.5.6.2 If SA is between 1000 and 3000 sq ft, then the operation manpower required is:

$$OMH = 37.1\ (SA)^{0.3247}$$

2.15.4.5.6.3 If SA is greater than 3000 sq ft, then the operation manpower required is:

$$OMH = 4.0\ (SA)^{0.6020}$$

where

OMH = operation manpower required, man-hours/yr.

2.15.4.5.7 Calculate electrical energy required.

2.15.4.5.7.1 If SA is less than 1670 sq ft, then the electrical energy required is:

$$KWH = 7500$$

2.15.4.5.7.2 If SA is between 1670 and 16,700 sq ft, then the electrical energy required is:

$$KWH = 2183.3\ (SA)^{0.1663}$$

2.15.4.5.7.3 If SA is greater than 16,700 sq ft, then the electrical energy required is:

$$KWH = 38.4\ (SA)^{0.5818}$$

where

KWH = electrical energy required, kwhr/yr.

2.15.4.5.8 Other operation and maintenance material costs. (c) This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of the total bare construction cost of the sedimentation system.

$$OMMP = 1\%$$

where

OMMP = percent of sedimentation system total bare construction costs for operation and maintenance material supply.

2.15.4.5.9 Other construction cost items.

2.15.4.5.9.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

2.15.4.5.9.2 Other minor cost items such as piping, site cleaning, control panel, etc., would be 15 percent of the total installed cost.

2.15.4.5.9.3 The correction factor would be:

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor cost items.

2.15.4.6 <u>Quantities Calculations Output Data.</u>

2.15.4.6.1 Surface area, SA, sq ft.

2.15.4.6.2 Number of units, N.

2.15.4.6.3 Number of batteries, NB.

2.15.4.6.4 Surface area per unit, SAU, sq ft.

2.15.4.6.5 Diameter of unit, DIA, ft.

2.15.4.6.6 Earthwork required, V_{ew}, cu ft.

2.15.4.6.7 Total quantity of R.C. wall required, V_{cw}, cu ft.

2.15.4.6.8 Total quantity of R.C. slab required, V_{cs}, cu ft.

2.15.4.6.9 Maintenance manpower required, MMH, man-hour/yr.

2.15.4.6.10 Operation manpower required, OMH, man-hour/yr.

2.15.4.6.11 Electrical energy required, KWH, kwhr/yr.

2.15.4.6.12 Other operation and maintenance material costs, OMMP, percent.

2.15.4.6.13 Correction factor for construction costs, CF.

2.15.4.7 Unit Price Inputs Required.

2.15.4.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.15.4.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.15.4.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.15.4.7.4 Standard size clarifier mechanism (90-ft diameter) cost, COSTES, \$ (optional).

2.15.4.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.15.4.7.6 Equipment installation labor rate, LABRI, \$/man-hour.

2.15.4.7.7 Crane rental rate, UPICR, \$/hour.

2.15.4.8 Cost Calculations.

2.15.4.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.15.4.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} \cdot UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. required for walls, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.15.4.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of R.C. required for slab, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.15.4.8.4 Cost of installed equipment.

2.15.4.8.4.1 Purchase cost of clarifier equipment. The purchase cost of the clarifier mechanism can be obtained from the following equation:

$$COSTCM = COSTES \times \frac{COSTRO}{100}$$

where

COSTCM = purchase cost of mechanism with diameter of DIA ft, \$.

COSTES = purchase cost of standard size mechanism with diameter of 90 ft, \$.

COSTRO = cost of mechanism with diameter of DIA ft, as percent of cost of standard size mechanism, percent.

2.15.4.8.4.2 Calculate COSTRO.

$$COSTRO = 2.16\ (DIA)^{0.8515}$$

2.15.4.8.4.3 Cost of standard size mechanism. The cost of the mechanism for a 90-ft diameter clarifier for the first quarter of 1977 is:

$$COSTES = \$75,000$$

For better cost estimation, COSTES should be obtained from equipment vendor and treated as a unit price input. If COSTES is not treated as a unit price input, the cost will be automatically updated by using the Marshall and Swift Equipment Cost Index.

$$\text{COSTES} = 75{,}000 \cdot \frac{\text{MSECI}}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index 1st quarter 1977.

2.15.4.8.4.4 Installation man-hours for clarifier mechanism. The man-hour requirement for field erection of clarifier mechanism can be estimated by:

$$\text{IMH} = 2.04\ (\text{DIA})$$

where

IMH = installation man-hour requirement, man-hours.

2.15.4.8.4.5 Crane requirement for installations, CH.

$$\text{CH} = (0.1)\ (\text{IMH})$$

where

CH = crane time requirement for installation, hr.

2.15.4.8.4.6 Other minor costs associated with the installed equipment. This category includes the cost for electrical controls, influent pipe, effluent weirs, scum baffles, special materials, painting, etc., and can be added as percent of purchase equipment cost.

$$\text{PMINC} = 15\%$$

where

PMIC = percentage of purchase cost of equipment as minor costs, percent.

2.15.4.8.4.7 Installed equipment costs.

$$\text{IEC} = (\text{N})\ (\text{NB})\ [\text{COSTCS}\ [1 + \frac{\text{PMINC}}{100}] + (\text{IMH})\ (\text{LABRI}) + (\text{CH})\ (\text{UPICR})]$$

where

IEC = installed equipment costs, \$.

LABRI = labor rate, \$/man-hour.

UPICR = crane rental rate, \$/hr.

2.15.4.8.5 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.15.4.8.6 Operation and maintenance material costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = percentage of total bare construction cost as operation and maintenance material costs, percent.

2.15.4.9 Cost Calculations Output Data.

2.15.4.9.1 Total bare construction cost of the circular clarifier, TBCC, \$.

2.15.4.9.2 Operation and maintenance material costs, OMMC, \$/yr.

2.15.5 Primary Clarification - Rectangular.

2.15.5.1 Input Data.

2.15.5.1.1 Wastewater flow.

2.15.5.1.1.1 Average flow, mgd.

2.15.5.1.1.2 Peak flow, mgd.

2.15.5.1.2 Suspended solids, mg/l.

2.15.5.1.3 Volatile suspended solids, percent.

2.15.5.1.4 BOD_5 concentration, mg/l.

2.15.5.2 Design Parameters.

2.15.5.2.1 Overflow rate, gpd/ft^2. See Tables 2.15-1 and 2.15-2.

2.15.5.2.2 Detention time, hr. See Table 2.15-2.

2.15.5.2.3 Specific gravity of sludge. See Table 2.15-3.

2.15.5.2.4 Solids content of underflow, percent (4 to 6 percent).

2.15.5.2.5 Removal efficiencies.

2.15.5.2.5.1 Suspended solids, percent.

2.15.5.2.5.2 BOD_5, percent.

2.15.5.2.6 Weir loading, gpd/ft (10,000 to 15,000 gpd/ft).

2.15.5.3 Process Design Calculations.

2.15.5.3.1 Select an overflow rate by using Table 2.15-1 or by laboratory methods and calculate surface area.

$$SA = \frac{Q_p \times 10^6}{OFR}$$

where

SA = surface area, ft^2

Q_p = peak flow, mgd.

OFR = overflow rate, gal/ft^2/day

2.15.5.3.2 Select detention time and calculate volume (see Table 2.15-2).

$$V = (Q_{avg})\ (t) \times \frac{1}{7.48} \times \frac{1}{24} \times 10^6$$

where

V = volume of tank, ft^3.

Q_{avg} = average flow, mgd.

t = time, hr

2.15.5.3.3 Calculate side water depth.

$$SWD = \frac{V}{SA}$$

where

SWD = side water depth, ft

V = volume, ft^3

SA = surface area, ft^2

2.15.5.3.4 Check solid loading rate.

$$SLR = \frac{(Q_{avg})\ (SSI)\ (8.34)}{(SA)}$$

where

SLR = solid loading rate, $lb/ft^2/day$

Q_{avg} = average flow, mgd.

SSI = influent solids concentration, mg/l.

SA = surface area, ft^2

2.15.5.3.5 Select weir loading rate and calculate weir length.

$$WL = \frac{Q_p}{WLR} \times 10^6$$

where

WL = weir length, ft

Q_p = peak flow, mgd

WLR = weir loading rate, gal/ft/day

2.15.5.3.6 Determine percentage of suspended solids removed from Figure 2.15-2.

2.15.5.3.7 Calculate amount of primary sludge produced.

$$PSP = (Q_{avg})\ (SSI)\ (SSR)\ (10^{-2})\ (8.34)$$

where

PSP = primary sludge produced, lb/day

Q_{avg} = average flow, mgd

SSI = influent solids concentration, mg/l

SSR = suspended solids removed, percent (50 to 60 percent for municipal systems)

2.15.5.3.8 Select underflow concentration (3 to 6 percent) and sludge specific gravity (Table 2.15-3), and calculate the volume flow of primary sludge produced.

$$VPSP = \frac{PSP(100)}{(\text{specific gravity})\ (UC)\ (8.34)}$$

where

VPSP = volume flow of primary sludge produced, gal/day

PSP = primary sludge produced, gal/day

UC = underflow concentration (3 to 6 percent)

2.15.5.3.9 Effluent Characteristics.

2.15.5.3.9.1 BOD. Specify a BOD_5 removal rate or select a removal rate from Figure 2.15-3.

$$BODE = (BODI)\ (1 - \frac{BODR}{100})$$

$$S_e = (BODSI)$$

If BODE S_e set BODE = S_e

where

BODE = effluent BOD_5 concentration, mg/l.

BODI = influent BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, %.

S_e = effluent soluble BOD_5 concentration, mg/l.

BODSI = influent soluble BOD_5 concentration, mg/l.

2.15.5.3.9.2 COD.

$$CODE = 1.5\ (BODE)$$

$$CODSE = 1.5\ (S_e)$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.15.5.3.9.3 Suspended solids.

$$SSE = (SSI)\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

2.15.5.3.9.4 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNR}{100})$$

If TKNE NH3E set TKNE = NH3E

where

TKNE = effluent total Kjedahl nitrogen concentration, mg/l.

TKN = influent total Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

TKNR = total Kjedahl nitrogen removal rate, %.

2.15.5.3.9.5 Phosphorus

$$PO4E = (PO4)\ (1 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

2.15.5.3.9.6 Oil and Grease

$$OAGE = OAG$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

2.15.4.3.9.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids.

2.15.5.4 <u>Process Design Output Data</u>.

2.15.5.4.1 Overflow rate, gal/ft^2/day.

2.15.5.4.2 Surface area, ft^2.

2.15.5.4.3 Side water depth, ft.

2.15.5.4.4 Detention time, hr.

2.15.5.4.5 Solid loading, lb/ft^2/day.

2.15.5.4.6 Weir loading, gal/ft/day.

2.15.5.4.7 Weir length, ft.

2.15.5.4.8 Volume of sludge produced, gal/day.

2.15.5.4.9 Suspended solids removal, percent.

2.15.5.4.10 BOD removal, percent.

2.15.5.4.11 COD removal, percent.

2.15.5.4.12 TKN removal, percent.

2.15.5.4.13 PO_4, removal, percent.

2.15.5.4.14 Effluent BOD, mg/l.

2.15.5.4.15 Effluent suspended solids, mg/l.

2.15.5.4.16 Effluent COD, mg/l.

2.15.5.4.17 Effluent TKN, mg/l.

2.15.5.5 <u>Quantities Calculations</u>.

2.15.5.5.1 Calculate number and size of units. It is assumed that the units will be 20 ft wide and will be a maximum of 260 ft in length. The minimum number of units that will be used is 2.

$$L = \frac{SA}{20 \cdot N}$$

Begin with N = 2; if L is greater than 260, then try N = N + 1 and repeat until L is less than 260. If L is not an integer, use the next larger integer.

where

L = length of rectangular clarifier, ft.

N = number of rectangular clarifiers.

2.15.5.5.2 Earthwork required for construction. The volume of earthwork required for rectangular clarifiers can be estimated by the following equation.

$$V_{ew} = N[69.3L + 4/3(660L^2 + 8580L + 19{,}800)^{1/2} + 988]$$

where

V_{ew} = volume of earthwork required for construction, cu ft.

2.15.5.5.3 Reinforced concrete quantities.

2.15.5.5.3.1 Reinforced concrete wall quantity.

$$V_{cw} = V_{ew} + V_{sw} + V_{iec} + V_{ts}$$

$$V_{ew} = (44.7\ SWD + 134)N$$

$$V_{sw} = [10\ SWD + (SWD)\ (L) + 1.5L + 30]\ (N + 1)$$

$$V_{iec} = 220\ N$$

$$V_{ts} = (90 + 2L)\ N$$

where

V_{cw} = total volume of R.C. wall in-place, cu ft.

V_{ew} = volume of R.C. wall for end walls, cu ft.

V_{sw} = volume of R.C. wall for side wall, cu ft.

V_{iec} = volume of R.C. wall for influent and effluent channels, cu ft.

V_{ts} = volume of R.C. wall for top slab, cu ft.

SWD = side water depth, ft.

L = length of clarifier, ft.

N = number of clarifiers.

2.15.5.5.3.2 Reinforced concrete slab quantity.

$$V_{cs} = N(22L + 150)$$

where

V_{cs} = quantity of R.C. wall slab in-place, cu ft.

2.15.5.5.4 Calculate maintenance manpower requirements.

2.15.5.5.4.1 If SA is less than 1000 sq ft, then the maintenance manpower required is:

$$MMH = 200$$

2.15.5.5.4.2 If SA is between 1000 and 3000 sq ft, then the maintenance manpower required is:

$$MMH = 30.3\ (SA)^{0.2733}$$

2.15.5.5.4.3 If SA is greater than 3000 sq ft, then the maintenance manpower required is:

$$MMH = 2.05\ (SA)^{0.6098}$$

where

MMH = maintenance manpower required, man-hours/yr.

SA = surface area, sq ft.

2.15.5.5.5 Calculate operation manpower required.

2.15.5.5.5.1 If SA is less than 1000 sq ft, then the operation manpower required is:

$$OMH = 350$$

2.15.5.5.5.2 If SA is between 1000 and 3000 sq ft, then the operation manpower required is:

$$OMH = 37.1\ (SA)^{0.3247}$$

2.15.5.5.5.3 If SA is greater than 3000 sq ft, then the operation manpower required is:

$$OMH = 4.0\ (SA)^{0.6020}$$

where

OMH = operation manpower required, man-hours/yr.

2.15.5.5.6 Calculate electrical energy required.

2.15.5.5.6.1 If SA is less than 1670 sq ft, then the electrical energy required is:

$$KWH = 7500$$

2.15.5.5.6.2 If SA is between 1670 and 16,700 ft, then the electrical energy required is:

$$KWH = 2183.3\ (SA)^{0.1663}$$

2.15.5.5.6.3 If SA is greater than 16,700 sq ft, then the electrical energy required is:

$$KWH = 38.4\ (SA)^{0.5818}$$

where

KWH = electrical energy required, kwhr/yr.

2.15.5.5.7 Other operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of the total bare construction cost of the sedimentation system.

$$OMMP = 1\%$$

where

OMMP = percent of sedimentation system total bare construction costs for operation and maintenance material supply.

2.15.5.5.8 Other construction cost items.

2.15.5.5.8.1 From the above estimation approximately 85 percent of the construction costs have been accounted for.

2.15.5.5.8.2 Other minor construction costs such as piping, site cleaning, control panel, etc., would be 15 percent of the total construction costs.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor cost items.

2.15.5.6 <u>Quantities Calculation Output Data.</u>

2.15.5.6.1 Surface area, SA, sq ft.

2.15.5.6.2 Number of units, N.

2.15.5.6.3 Surface area per unit, sq ft.

2.15.5.6.4 Length of units, L, ft.

2.15.5.6.5 Earthwork required, V_{ew}, cu ft.

2.15.5.6.6 Quantity of R.C. wall required, V_{cw}, cu ft.

2.15.5.6.7 Quantity of R.C. slab required, V_{cs}, cu ft.

2.15.5.6.8 Maintenance manpower required, MMH, man-hours/yr.

2.15.5.6.9 Operation manpower required, OMH, man-hours/yr.

2.15.5.6.10 Electrical energy required, KWH, kwhr/yr.

2.15.5.6.11 Other operation and maintenance material costs, OMMP, percent.

2.15.5.6.12 Correction factor for construction costs, CF.

2.15.5.7 <u>Unit Price Input Required</u>.

2.15.5.7.1 Cost of earthwork, COSTE, \$/cu yd.

2.15.5.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.15.5.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.15.5.7.4 Standard rectangular clarifier mechanism cost, COSTRC, \$ (optional).

2.15.5.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.15.5.7.6 Equipment installation labor rate, LABRI, \$/man-hours.

2.15.5.7.7 Crane rental rate, UPICR, \$/hr.

2.15.5.8 <u>Cost Calculations</u>.

2.15.5.8.1 Cost of earthwork.

$$COSTE + \frac{V_{ew}}{27} \; UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.15.5.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. required for walls, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.15.5.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} (UPICS)$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. required for slab, cu ft.

UPICS = unit price input for R.C. Slab in-place, \$/cu yd.

2.15.5.8.4 Cost of installed equipment.

2.15.5.8.4.1 Purchase cost of clarifier equipment. The purchase cost of the rectangular clarifier mechanism can be obtained from the following equation.

$$COSTCM = COSTES \cdot \frac{COSTRO}{100}$$

where

COSTCM = purchase cost of mechanism 20 ft wide and length L feet, \$.

COSTRC = purchase cost of standard size mechanism 20 ft wide and 120 ft long, \$.

COSTRO = cost of mechanism of length L as a percent of cost of the standard size mechanism, percent.

2.15.5.8.4.2 Calculate COSTRO.

$$COSTRO = (.31)(L) + 63$$

2.15.5.8.4.3 Cost of standard size clarifier mechanism. The cost of the mechanism for a clarifier 20 ft wide and 120 ft long for the first quarter of 1977 is:

$$COSTRC = \$42,000$$

For better cost estimation, COSTRC should be obtained from equipment vendors and treated as a unit price input. If COSTES is not treated as a unit price input, the cost will be automatically updated by using the Marshall and Swift Equipment Cost Index.

$$COSTRC = 42,000 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index 1st quarter, 1977.

2.15.5.8.4.4 Installation man-hours for clarifier mechanism. The man-hour requirement for field erection of clarifier mechanism can be estimated by:

$$IMH = 0.978\ (L) + 80$$

where

IMH = installation man-hour requirement for clarifier mechanism, man-hours.

2.15.5.8.4.5 Crane requirement for installation.

$$CH = (.05)\ (IMH)$$

where

CH = crane time requirement for installation, hrs.

2.15.5.8.4.6 Other minor costs associated with the installed equipment. This category includes the cost of electrical wiring, drive unit assembly, tee rails, painting, etc., and can be added as a percent of the purchased equipment cost.

$$PMINC = 15\%$$

where

PMINC = percentage of purchase cost of equipment as minor costs, percent.

2.15.5.8.4.7 Installed equipment cost.

$$IEC = [COSTCM\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + CH(UPICR)]N$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/man-hour.

UPICR = crane rental rate, \$/hr.

2.15.5.8.5 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.15.5.8.6 Operation and maintenance material costs.

$$OMMC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = percentage of total bare construction cost as operation and maintenance material cost, percent.

2.15.5.9 Cost Calculation Output Data.

2.15.5.9.1 Total bare construction cost of rectangular clarifier, TBCC, \$.

2.15.5.9.2 Operation and maintenance costs, OMMC, \$.

2.15.6 Secondary Clarification - Circular

2.15.6.1 Input Data.

2.15.6.1.1 Wastewater flow.

2.15.6.1.1.1 Average daily flow, mgd.

2.15.6.1.1.2 Peak flow, mgd.

2.15.5.1.2 Mixed liquor suspended solids, mg/l.

2.15.6.2 Design Parameters.

2.15.6.2.1 Solids loading rate ($lb/ft^2/day$) ($30\ lb/\ ft^2/day$).

2.15.6.2.2 Surface overflow rate.

2.15.6.2.2.1 Small plants 600 $gal/ft^2/day$.

2.15.6.2.2.2 Larger plants 800 $gal/ft^2/day$.

2.15.6.2.3 Sludge specific gravity from Table 2.15-3.

2.15.6.2.4 Underflow concentration (UC), percent.

2.15.6.2.4.1 For activated sludge UC = 0.8-1.2 percent.

2.15.6.2.4.2 For trickling filter UC = 2-4 percent.

2.15.6.2.5 Weir overflow rate = 10,000-15,000 gpd/ft/day.

2.15.6.2.6 Detention time = 2-4 hr.

2.15.6.3 Process Design Calculations.

2.15.6.3.1 Select a solids loading rate and calculate the surface area.

$$SA = \frac{(Q_{avg})\ (MLSS)\ (8.34)}{SLR}$$

where

SA = surface area, ft^2.

Q_{avg} = average flow, mgd.

$MLSS$ = mixed liquor suspended solids, mg/l.

SLR = solid loading rate, $lb/ft^2/day$.

2.15.6.3.2 Check the maximum overflow rate.

$$OFR = \frac{Q_p \times 10^6}{SA}$$

where

OFR = maximum overflow rate, gal/ft^2/day.

Q_p = peak flow, mgd.

SA = surface area, ft^2.

If OFR is within range, proceed to next step; if OFR is outside range, assume OFR and recalculate SA as follows.

$$SA = \frac{Q_p \times 10^6}{OFR}$$

2.15.6.3.3 Assume detention time and calculate volume.

$$V = (Q_{avg})\ (10^6)\left(\frac{t}{24}\right)\left(\frac{1}{7.48}\right)$$

where

V = volume, ft^3.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.15.6.3.4 Calculate side water depth.

$$SWD = \frac{V}{SA}$$

where

SWD = side water depth, ft.

V = volume, ft^3.

SA = surface area, ft^2.

2.15.6.3.5 Select weir overflow rate and calculate weir length.

$$WL = \frac{Q_p \times 10^6}{WOFR}$$

where

WL = weir length, ft.

Q_p = peak flow, mgd.

WOFR = weir overflow rate, gal/ft/day.

2.15.6.3.6 Effluent Characteristics.

2.15.6.3.6.1 BOD. Specify a BOD_5 removal rate or select a removal rate from Figure 2.15-3.

$$BODE = (BODI)\ (1 - \frac{BODR}{100})$$

$$S_e = (BODSI)$$

If $BODE < S_e$ set $BODE = S_e$

where

BODE = effluent BOD_5 concentration, mg/l.

BODI = influent BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, %.

S_e = effluent soluble BOD_5 concentration, mg/l.

BODSI = influent soluble BOD_5 concentration, mg/l.

2.15.6.3.6.2 COD.

$$CODE = 1.5\ (BODE)$$

$$CODSE = 1.5\ (S_e)$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.15.6.3.6.3 Suspended solids.

$$SSE = (SSI)\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

2.15.6.3.6.4 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNR}{100})$$

If TKNE < NH3E set TKNE = NH3E

where

TKNE = effluent total Kjedahl nitrogen concentration, mg/l.

TKN = influent total Kjedahl nitrogeu concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

TKNR = total Kjedahl nitrogen removal rate, %.

2.15.6.3.6.5 Phosphorus

$$PO4E = (PO4)\ (1 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

2.15.6.3.6.6 Oil and Grease

$$OAGE = OAG$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

2.15.6.3.6.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids.

2.15.6.4 Process Design Calculations Output Data.

2.15.6.4.1 Solids loading rate, $lb/ft^2/day$.

2.15.6.4.2 Surface area, ft^2.

2.15.6.4.3 Overflow rate, $gal/ft^2/day$.

2.15.6.4.4 Detention time, hr.

2.15.6.4.5 Weir overflow rate, gal/ft/day.

2.15.6.4.6 Tank side water depth, ft.

2.15.6.4.7 Weir length, ft.

2.15.6.4.8 Volume of wasted sludge, gal/day.

2.15.6.4.9 Underflow concentration, percent.

2.15.6.4.10 Effluent total BOD, mg/l.

2.15.6.5 Quantities Calculations.

2.15.6.5.1 Unit selection. The number of units to be employed depends upon the wastewater quantity to be handled. The following assumptions will be followed in the detemination of number of units, N.

Flow Range MGD, Q_{avg}		Number of Units
0.5	1.0	2
1.0	10.0	2
10.0	24.0	4
24.0	50.0	8
50.0	70.0	12
70.0	100.0	16

2.15.6.5.1.1 When Q_{avg} is larger than 100 mgd, the system will be designed as several batteries of units.

2.15.6.5.1.2 If $100 < Q_{avg} \leq 200$

Number of process batteries, NB, would be 2.

The system would be designed as two identical batteries with flow to each battery at $Q_{avg}/2$.

Select the number of units per battery as described above using flow to be $Q_{avg}/2$.

2.15.6.5.1.3 If $Q_{avg} > 200$

Number of process batteries, NB, would be 3.

The system would be designed as three identical batteries with flow to each battery at $Q_{avg}/3$.

Select the number of units per battery as described above using flow to be $Q_{avg}/3$.

2.15.6.5.2 Sizing individual unit.

2.15.6.5.2.1 The surface area of each unit.

$$SAU = \frac{SA}{N \cdot NB}$$

where

SAU = surface area per unit, sq ft.

N = number of units.

2.15.6.5.2.2 The diameter of each unit.

$$DIA = \frac{4 \cdot SAU}{3.1416}$$

If DIA is not an integer, use the next larger integer.

where

DIA = diameter of the unit, ft.

2.15.6.5.2.3 The available sizes from off-the-shelf items range from 10 ft to 200 ft; so if the calculated DIA is larger than 200 ft, the number of units will have to be increased to N = N + 1.

2.15.6.5.3 Earthwork required for construction.

$$V_{ew} = (1.15)\ N\ [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = earthwork required for construction, cu ft.

1.15 = addition of 15% more for safety factor.

2.15.6.5.4 Reinforced concrete quantities.

2.15.6.5.4.1 Reinforced concrete slab quantity, V_{csn}, for one tank.

$$V_{csn} = (0.825)\ (DIA + 4)^2 \cdot (\frac{t_s}{12})$$

where

V_{cs} = quantity of R.C. slab in-place, cu ft.

t_s = thickness of the slab, inches, can be obtained by

$$t_s = 7.9 + 0.25\ SWD$$

where

SWD = sidewater depth, ft.

2.15.6.5.4.2 Reinforced concrete wall quantity, V_{cwn}, for one tank.

$$V_{cwn} = (3.14)\ (SWD + 1.5) \cdot (DIA)\ (\frac{t_w}{12})$$

where

V_{cwn} = quantity of R.C. wall in-place, cu ft.

t_w = wall thickness, inches, and can be estimated by

$$t_w = 7 + (0.5)\ (SWD)$$

2.15.6.5.4.3 Quantity of concrete for splitter boxes, V_{cb}

$$V_{cb} = 100 \cdot N^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter box, cu ft.

N = number of units.

2.15.6.5.4.4 Total quantity of R.C. in-place.

$$\text{Wall:}\quad V_{cw} = [N \cdot V_{cwn} + V_{cb}] \cdot NB$$

$$\text{Slab:}\quad V_{cs} = [N \cdot V_{csn}] \cdot NB$$

2.15.6.5.5 Calculate maintenance manpower requirements.

2.15.6.5.5.1 If SA is less than 1000 sq ft, then the maintenance manpower required is:

$$MMH = 200$$

2.15.6.5.5.2 If SA is between 1000 and 3000 sq ft, then the maintenance manpower required is:

$$MMH = 30.3\ (SA)^{0.2733}$$

2.15.6.5.5.3 If SA is greater than 3000 sq ft, then the maintenance manpower required is:

$$MMH = 2.05\ (SA)^{0.6098}$$

where

MMH = maintenance manpower required, man-hours/yr.

SA = Surface area, sq ft.

2.15.6.5.6 Calculate operation manpower required.

2.15.6.5.6.1 If SA is less than 1000 sq ft, then the operations manpower required is:

$$OMH = 350$$

2.15.6.5.6.2 If SA is between 1000 and 3000 sq ft, then the operation manpower required is:

$$OMH = 37.1\ (SA)^{0.3247}$$

2.15.6.5.6.3 If ASA is greater than 3000 sq ft, then the operation manpower required is:

$$OMH = 4.0\ (SA)^{0.6020}$$

where

OMH = operation manpower required, man-hours/yr.

2.15.6.5.7 Calculate electrical energy required.

2.15.6.5.7.1 If SA is less than 1670 sq ft, then the electrical energy required is:

$$KWH = 7500$$

2.15.6.5.7.2 If SA is between 1670 and 16,700 sq ft, then the electrical energy required is:

$$KWH = 2183.3\ (SA)^{0.1663}$$

2.15.6.5.7.3 If SA is greater than 16,700 sq ft, then the electrical energy required is:

$$KWH = 38.4\ (7SA)^{0.5818}$$

where

KWH = electrical energy required, kwhr/yr.

2.15.6.5.8 Other operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of the total bare construction cost of the sedimentation system.

$$OMMP = 1\%$$

where

OMMP = percent of sedimentation system total bare construction costs for operation and maintenance material supply.

2.15.6.5.9 Other construction cost items.

2.15.6.5.9.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

2.15.6.5.9.2 Other minor cost items such as piping, site cleaning, control panel, etc., would be 15 percent of the total installed cost.

2.15.6.5.9.3 The correction factor would be:

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor cost items.

2.15.6.6 <u>Quantities Calculations Output Data.</u>

2.15.6.6.1 Surface area, SA, sq ft.

2.15.6.6.2 Number of units, N.

2.15.6.6.3 Number of batteries, NB.

2.15.6.6.4 Surface area per unit, SAU, sq ft.

2.15.6.6.5 Diameter of unit, DIA, ft.

2.15.6.6.6 Earthwork required, V_{ew}, cu ft.

2.15.6.6.7 Total quantity of R.C. wall required, V_{cw}, cu ft.

2.15.6.6.8 Total quantity of R.C. slab required, V_{cs}, cu ft.

2.15.6.6.9 Maintenance manpower required, MMH, man-hour/yr.

2.15.6.6.10 Operation manpower required, OMH, man-hour/yr.

2.15.6.6.11 Electrical energy required, KWH, kwhr/yr.

2.15.6.6.12 Other operation and maintenance material costs, OMMP, percent.

2.15.6.6.13 Correction factor for construction costs, CF.

2.15.6.7 <u>Unit Price Inputs Required.</u>

2.15.6.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.15.6.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.15.6.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.15.6.7.4 Standard size clarifier mechanism (90-ft diameter) cost, COSTES, \$ (optional).

2.15.6.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.15.6.7.6 Equipment installation labor rate, LABRI, \$/man-hour.

2.15.6.7.7 Crane rental rate, UPICR, \$/hour.

2.15.6.8 <u>Cost Calculations.</u>

2.15.6.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.15.6.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} \cdot UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. required for walls, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.15.6.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of R.C. required for slab, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.15.6.8.4 Cost of installed equipment.

2.15.6.8.4.1 Purchase cost of clarifier equipment. The purchase cost of the clarifier mechanism can be obtained from the following equation:

$$COSTCM = COSTES \times \frac{COSTRO}{100}$$

where

COSTCM = purchase cost of mechanism with diameter of DIA ft, \$.

COSTES = purchase cost of standard size mechanism with diameter of 90 ft, \$.

COSTRO = cost of mechanism with diameter of DIA ft, as percent of cost of standard size mechanism, percent.

2.15.6.8.4.2 Calculate COSTRO.

$$COSTRO = 2.16\ (DIA)^{0.8515}$$

2.15.6.8.4.3 Cost of standard size mechanism. The cost of the mechanism for a 90-ft diameter clarifier for the first quarter of 1977 is:

$$COSTES = \$75,000$$

For better cost estimation, COSTES should be obtained from equipment vendor and treated as a unit price input. If COSTES is not treated as a unit price input, the cost will be automatically updated by using the Marshall and Swift Equipment Cost Index.

$$COSTES = 75,000 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index 1st quarter 1977.

2.15.6.8.4.4 Installation man-hours for clarifier mechanism. The man-hour requirement for field erection of clarifier mechanism can be estimated by:

$$IMH = 2.04\ (DIA)$$

where

IMH = installation man-hour requirement, man-hours.

2.15.6.8.4.5 Crane requirement for installations, CH.

$$CH = (0.1)\ (IMH)$$

where

CH = crane time requirement for installation, hr.

2.15.6.8.4.6 Other minor costs associated with the installed equipment. This category includes the cost for electrical controls, influent pipe, effluent weirs, scum baffles, special materials, painting, etc., and can be added as percent of purchase equipment cost.

$$PMINC = 15\%$$

where

PMIC = percentage of purchase cost of equipment as minor costs, percent.

2.15.6.8.4.7 Installed equipment costs.

$$IEC = (N)\ (NB)\ [COSTCS\ [1 + \frac{PMINC}{100}] + (IMH)\ (LABRI) + (CH)\ (UPICR)]$$

where

IEC = installed equipment costs, $.

LABRI = labor rate, $/man-hour.

UPICR = crane rental rate, $/hr.

2.15.6.8.5 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, $.

CF = correction factor for minor cost items.

2.15.6.8.6 Operation and maintenance material costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, $/yr.

OMMP = percentage of total bare construction cost as operation and maintenance material costs, percent.

2.15.6.9 <u>Cost Calculations Output Data.</u>

2.15.6.9.1 Total bare construction cost of the circular clarifier, TBCC, $.

2.15.6.9.2 Operation and maintenance material costs, OMMC, $.

2.15.7 Secondary Clarification - Rectangular.

2.15.7.1 Input Data.

2.15.7.1.1 Wastewater flow.

2.15.7.1.1.1 Average daily flow, mgd.

2.15.7.1.1.2 Peak flow, mgd.

2.15.7.1.2 Mixed liquor suspended solids, mg/l.

2.15.7.2 Design Parameters.

2.15.7.2.1 Solids loading rate ($lb/ft^2/day$) (30 $lb/ft^2/day$).

2.15.7.2.2. Surface overflow rate.

2.15.7.2.2.1 Small plants 600 $gal/ft^2/day$.

2.15.7.2.2.2 Larger plants 800 $gal/ft^2/day$.

2.15.7.2.3 Sludge specific gravity from Table 2.15-3.

2.15.7.2.4 Underflow concentration (UC), percent.

2.15.7.2.4.1 For activated sludge UC = 0.8-1.2 percent.

2.15.7.2.4.2 For trickling filter UC = 2-4 percent.

2.15.7.2.5 Weir overflow rate = 10,000-15,000 gpd/ft/day.

2.15.7.2.6 Detention time = 2-4 hr.

2.15.7.3 Process Design Calculations.

2.15.7.3.1 Select a solids loading rate and calculate the surface area.

$$SA = \frac{(Q_{avg})\ (MLSS)\ (8.34)}{SLR}$$

where

SA = surface area, ft^2.

Q_{avg} = average flow, mgd.

$MLSS$ = mixed liquor suspended solids, mg/l.

SLR = solid loading rate, $lb/ft^2/day$.

2.15.7.3.2 Check the maximum overflow rate.

$$OFR = \frac{Q_p \times 10^6}{SA}$$

where

OFR = maximum overflow rate, gal/ft^2/day.

Q_p = peak flow, mgd.

SA = surface area, ft^2.

If OFR is within range, proceed to next step; if OFR is outside range, assume OFR and recalculate SA as follows.

$$SA = \frac{Q_p \times 10^6}{OFR}$$

2.15.7.3.3 Assume detention time and calculate volume.

$$V = (Q_{avg})\ (10^6)\ \frac{t}{24}\ \ \frac{1}{7.48}$$

where

V = volume, ft^3.

Q_{avg} = average daily flow, mgd.

t = detention time, hr.

2.15.7.3.4 Calculate side water depth.

$$SWD = \frac{V}{SA}$$

where

SWD = side water depth, ft.

V = volume, ft^3.

SA = surface area, ft^2.

2.15.7.3.5 Select weir overflow rate and calculate weir length.

$$WL = \frac{Q_p \times 10^6}{WOFR}$$

where

WL = weir length, ft.

Q_p = peak flow, mgd.

WOFR = weir overflow rate, gal/ft/day.

2.15.7.3.6 Effluent Characteristics.

2.15.7.3.6.1 BOD. Specify a BOD_5 removal rate or select a removal rate from Figure 2.15-3.

$$BODE = (BODI)\ (1 - \frac{BODR}{100})$$

$$S_e = (BODSI)$$

If BODE $<$ S_e set BODE = S_e

where

BODE = effluent BOD_5 concentration, mg/l.

BODI = influent BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, %.

S_e = effluent soluble BOD_5 concentration, mg/l.

BODSI = influent soluble BOD_5 concentration, mg/l.

2.15.7.3.6.2 COD.

$$CODE = 1.5\ (BODE)$$

$$CODSE = 1.5\ (S_e)$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.15.7.3.6.3 Suspended solids.

$$SSE = (SSI)\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

2.15.7.3.6.4 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNR}{100})$$

If TKNE < NH3E set TKNE = NH3E

where

TKNE = effluent total Kjedahl nitrogen concentration, mg/l.

TKN = influent total Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

TKNR = total Kjedahl nitrogen removal rate, %.

2.15.7.3.6.5 Phosphorus

$$PO4E = (PO4)\ (1 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

2.15.7.3.6.6 Oil and Grease

$$OAGE = OAG$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

2.15.7.3.6.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids.

2.15.7.4 <u>Process Design Calculations Output Data.</u>

2.15.7.4.1 Solids loading rate, $lb/ft^2/day$.

2.15.7.4.2 Surface area, ft^2.

2.15.7.4.3 Overflow rate, $gal/ft^2/day$.

2.15.7.4.4 Detention time, hr.

2.15.7.4.5 Weir overflow rate, gal/ft/day.

2.15.7.4.6 Tank side water depth, ft.

2.15.7.4.7 Weir length, ft.

2.15.7.4.8 Volume of wasted sludge, gal/day.

2.15.7.4.9 Underflow concentration, percent.

2.15.7.4.10 Effluent total BOD, mg/l.

2.15.7.5 <u>Quantities Calculations.</u>

2.15.7.5.1 Calculate number and size of units. It is assumed that the units will be 20 ft wide and will be a maximum of 260 ft in length. The minimum number of units that will be used is 2.

$$L = \frac{SA}{20 \cdot N}$$

Begin with N = 2; if L is greater than 260, then try N = N + 1 and repeat until L is less than 260. If L is not an integer, use the next larger integer.

where

L = length of rectangular clarifier, ft.

N = number of rectangular clarifiers.

2.15.7.5.2 Earthwork required for construction. The volume of earthwork required for rectangular clarifiers can be estimated by the following equation.

$$V_{ew} = N[69.3L + 4/3(660L^2 + 8580L + 19,800)^{1/2} + 988]$$

where

V_{ew} = volume of earthwork required for construction, cu ft.

2.15.7.5.3 Reinforced concrete quantities.

2.15.7.5.3.1 Reinforced concrete wall quantities.

$$V_{cw} = V_{ew} + V_{sw} + V_{iec} + V_{ts}$$

$$V_{ew} = (44.7\ SWD + 134)N$$

$$V_{sw} = [10\ SWD + (SWD)\ (L) + 1.5L + 30]\ (N + 1)$$

$$V_{iec} = 220\ N$$

$$V_{ts} = (90 + 2L)\ N$$

where

V_{cw} = total volume of R.C. wall in-place, cu ft.

V_{ew} = volume of R.C. wall for end walls, cu ft.

V_{sw} = volume of R.C. wall for side wall, cu ft.

V_{iec} = volume of R.C. wall for influent and effluent channels, cu ft.

V_{ts} = volume of R.C. wall for top slab, cu ft.

SWD = side water depth, ft.

L = length of clarifier, ft.

N = number of clarifiers.

2.15.7.5.3.2 Reinforced concrete slab quantity.

$$V_{cs} = N(22L + 150)$$

where

V_{cs} = quantity of R.C. wall slab in-place, cu ft.

2.15.7.5.4 Calculate maintenance manpower requirements.

2.15.7.5.4.1 If SA is less than 1000 sq ft, then the maintenance manpower required is:

$$MMH = 200$$

2.15.7.5.4.2 If SA is between 1000 and 3000 sq ft, then the maintenance manpower required is:

$$MMH = 30.3\ (SA)^{0.2733}$$

2.15.7.5.4.3 If SA is greater than 3000 sq ft, then the maintenance manpower required is:

$$MMH = 2.05\ (SA)^{0.6098}$$

where

MMH = maintenance manpower required, man-hours/yr.

SA = surface area, sq ft.

2.15.7.5.5 Calculate operation manpower required.

2.15.7.5.5.1 If SA is less than 1000 sq ft, then the operation manpower required is:

$$OMH = 350$$

2.15.7.5.5.2 If SA is between 1000 and 3000 sq ft, then the operation manpower required is:

$$OMH = 37.1\ (SA)^{0.3247}$$

2.15.7.5.5.3 If SA is greater than 3000 sq ft, then the operation manpower required is:

$$OMH = 4.0\ (SA)^{0.6020}$$

where

OMH = operation manpower required, man-hours/yr.

2.15.7.5.6 Calculate electrical energy required.

2.15.7.5.6.1 If SA is less than 1670 sq ft, then the electrical energy required is:

$$KWH = 7500$$

2.15.7.5.6.2 If SA is greater than 16,700 sq ft, then the electrical energy required is:

$$KWH = 2183.3\ (SA)^{0.1663}$$

2.15.7.5.6.3 If SA is greater than 16,700 sq ft, then the electrical energy required is:

$$KWH = 38.4\ (SA)^{0.5818}$$

where

KWH = electrical energy required, kwhr/yr.

2.15.7.5.7 Other operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of the total bare construction cost of the sedimentation system.

$$OMMP = 1\%$$

where

OMMP = percent of sedimentation system total bare construction costs for operation and maintenance material supply.

2.15.7.5.8 Other construction cost items.

2.15.7.5.8.1 From the above estimation approximately 85 percent of the construction costs have been accounted for.

2.15.7.5.8.2 Other minor construction costs such as piping, site cleaning, control panel, etc., would be 15 percent of the total construction costs.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor cost items.

2.15.7.6 <u>Quantities Calculation Output Data.</u>

2.15.7.6.1 Surface area, SA, sq ft.

2.15.7.6.2 Number of units, N.

2.15.7.6.3 Surface area per unit, sq ft.

2.15.7.6.4 Length of units, L, ft.

2.15.7.6.5 Earthwork required, V_{ew}, cu ft.

2.15.7.6.6 Quantity of R.C. wall required, V_{cw}, cu ft.

2.15.7.6.7 Quantity of R.C. slab required, V_{cs}, cu ft.

2.15.7.6.8 Maintenance manpower required, MMH, man-hours/yr.

2.15.7.6.9 Operation manpower required, OMH, man-hours/yr.

2.15.7.6.10 Electrical energy required, KWH, kwhr/yr.

2.15.7.6.11 Other operation and maintenance material costs, OMMP, percent.

2.15.7.6.12 Correction factor for construction costs, CF.

2.15.7.7 Unit Price Input Required.

2.15.7.7.1 Cost of earthwork, COSTE, $/cu yd.

2.15.7.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.15.7.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.15.7.7.4 Standard rectangular clarifier mechanism cost, COSTRC, $ (optional).

2.15.7.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.15.7.7.6 Equipment installation labor rate, LABRI, $/man-hours.

2.15.7.7.7 Crane rental rate, UPICR, $/hr.

2.15.7.8 Cost Calculations.

2.15.7.8.1 Cost of earthwork.

$$COSTE + \frac{V_{ew}}{27} \text{ UPIEX}$$

where

COSTE = cost of earthwork, $.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, $/cu yd.

2.15.7.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} \text{ UPICW}$$

where

COSTCW = cost of R.C. wall in-place, $.

V_{cw} = volume of R.C. required for walls, cu ft.

UPICW = unit price input for R.C. wall in-place, $/cu yd.

2.15.7.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \text{ (UPICS)}$$

where

COSTCS = cost of R.C. slab in-place, $.

V_{cs} = volume of R.C. required for slab, cu ft.

UPICS = unit price input for R.C. Slab in-place, $/cu yd.

2.15.7.8.4 Cost of installed equipment.

2.15.7.8.4.1 Purchase cost of clarifier equipment. The purchase cost of the rectangular clarifier mechanism can be obtained from the following equation.

$$COSTCM = COSTES \cdot \frac{COSTRO}{100}$$

where

COSTCM = purchase cost of mechanism 20 ft wide and length L feet, $.

COSTRC = purchase cost of standard size mechanism 20 ft wide and 120 ft long, \$.

COSTRO = cost of mechanism of length L as a percent of cost of the standard size mechanism, percent.

2.15.7.8.4.2 Calculate COSTRO.

$$COSTRO = (.31)\ (L) + 63$$

2.15.7.8.4.3 Cost of standard size clarifier mechanism. The cost of the mechanism for a clarifier 20 ft wide and 120 ft long for the first quarter of 1977 is:

$$COSTRC = \$42{,}000$$

For better cost estimation, COSTRC should be obtained from equipment vendors and treated as a unit price input. If COSTES is not treated as a unit price input, the cost will be automatically updated by using the Marshall and Swift Equipment Cost Index.

$$COSTRC = 42{,}000\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index 1st quarter, 1977.

2.15.7.8.4.4 Installation man-hours for clarifier mechanism. The man-hour requirement for field erection of clarifier mechanism can be estimated by:

$$IMH = 0.978\ (L) + 80$$

where

IMH = installation man-hour requirement for clarifier mechanism, man-hours.

2.15.7.8.4.5 Crane requirement for installation.

$$CH = (.05)\ (IMH)$$

where

CH = crane time requirement for installation, hrs.

2.15.7.8.4.6 Other minor costs associated with the installed equipment. This category includes the cost of electrical wiring, drive unit assembly, tee rails, painting, etc., and can be added as a percent of the purchased equipment cost.

$$PMINC = 15\%$$

where

PMINC = percentage of purchase cost of equipment as minor costs, percent.

2.15.7.8.4.7 Installed equipment cost.

$$IEC = [COSTCM\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + CH(UPICR)]N$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/man-hour.

UPICR = crane rental rate, \$/hr.

2.15.7.8.5 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.15.7.8.6 Operation and maintenance material costs.

$$OMMC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = percentage of total bare construction cost as operation and maintenance material cost, percent.

2.15.7.9 <u>Cost Calculation Output Data.</u>

2.15.7.9.1 Total bare construction cost of rectangular clarifier, TBCC, \$.

2.15.7.9.2 Operation and maintenance costs, OMMC, \$.

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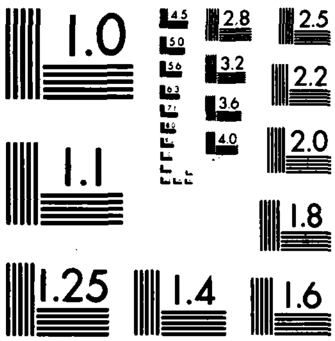

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

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2.17 DENITRIFICATION

2.17.1 Background.

2.17.1.1 Nitrogen in wastewater can exist in four foms: organic nitrogen, ammonia nitrogen, nitrite nitrogen and nitrate nitrogen. The prevalent foms present in untreated sewage are organic nitrogen and ammonia nitrogen. Organic nitrogen exists in both soluble and particulate foms.

2.17.1.2 Discharge of ammonia nitrogen to a receiving stream causes depletion of the stream's dissolved oxygen content as the ammonia nitrogen is oxidized to nitrate. In addition, ammonia nitrogen can adversely affect fish life under certain environmental conditions. As a result, many wastewater treatment plants employ a nitrification process to convert ammonia nitrogen to nitrate nitrogen prior to discharge. Nitrification is a biological oxidation process which can be perfomed in either a suspended or attached growth mode.

2.17.1.3 However, the nitrogen in nitrate is available as a nutrient for biological growth. Thus, the discharge of nitrate can contribute to biostimulation of surface waters, resulting in affects such as algal blooms and eutrophication. As a result, a denitrification system must be employeed at certain treatment plants. The biological process of denitrification involves the conversion of nitrate nitrogen to a gaseous nitrogen species. The gaseous product, which is relatively unavailable for biological growth, is primarily nitrogen gas but also may contain some quantities of nitrous oxide or nitric oxide.

2.17.1.4 Denitrification is a two-step biological process. Nitrate is converted to nitrite, which in turn is reduced to nitrogen gas. This two-step process is temed "dissimilation". A relatively broad range of bacteria, including Psuedomonas, Micrococcus, Archromobacter and Bacillus, can accomplish denitrification. These bacteria can use either nitrate or oxygen to oxidize organic material. Since the use of oxygen is more energetically favorable than nitrate, denitrification must be conducted in the absence of oxygen (anoxic condition) to ensure that nitrate, rather than oxygen, is used in the oxidation of the organic material.

2.17.1.5 In order for denitrification to occur, a carbon source must be available for oxidation. Since the process typically occurs after carbonaceous material in the raw wastewater has been removed, an external carbon source must be added to the denitrification system. Since methanol (CH_3OH) is the carbon source most often used in practice, its use will be incorporated in this model. It is important that only sufficient methanol is added in denitrification to accomplish the nitrate removal, as excess dosing causes organics to appear in the effluent unless control measures are taken.

2.17.1.6 The two-step conversion of nitrate nitrogen to nitrogen gas occurring in the denitrification process is represented by Equations 1 and 2. Equation 3 is the overall reaction, and is the sum of Equations 1 and 2. The chemical reaction for synthesis of the denitrifying organisms is presented in Equation 4.

(1) $NO_3^- + 0.33\ CH_3OH \rightarrow NO_2^- + 0.67\ H_2O + 0.33\ CO_2$

(2) $NO_2^- + 0.5\ CH_3OH \rightarrow 0.5\ N_2 + 0.5\ CO_2 + 0.5\ H_2O + OH^-$

(3) $NO_3^- + 0.83\ CH_3OH \rightarrow 0.5\ N_2 + .83\ CO_2 + 1.17\ H_2O + OH^-$

(4) $3NO_3^- + 14\ CH_3OH + 4\ H_2CO_3 \rightarrow 3\ C_5H_7O_2N + 20H_2O + 3\ HCO_3^-$

2.17.1.7 The theoretical methanol requirement for nitrate reduction and cell synthesis is calculated as 2.47 mg methanol per mg nitrate nitrogen. However, additional methanol is needed for reduction of any nitrite present and elimination of any remaining oxygen. These factors increase the required methanol dosage to a value of 3 mg methanol per mg nitrate nitrogen.

2.17.1.8 The pH affect of denitrification must be examined. In the conversion of nitrate nitrogen to nitrogen gas, bicarbonate is produced and the carbonic acid concentration is reduced, thus increasing the wastewater alkalinity. This increase is estimated to be 3.0 mg alkalinity ($CaCO_3$) produced per mg nitrogen reduced.

2.17.1.9 As a result, denitrification will tend to at least partially reverse the pH depression occurring in the nitrification process. It has been determined that the highest denitrification rates occur in the pH range from 7.0 to 7.5. Denitrification rates are depressed below pH 6.0 and above pH 8.0.

2.17.1.10 The model of denitrification will include both suspended and attached modes of operation. A common assumption for both systems is that the influent to the process is a nitrified secondary effluent.

2.17.2 General Description Attached Growth Denitrification.

2.17.2.1 Several different attached growth processes are available for denitrification. Most schemes use an upflow configuration with the media submerged. Reactors are classified as either packed bed or fluidized bed types. Packed bed media can be high-porosity plastic modules and dumped rings or low-porosity fine media. Fluidized bed units use high-porosity fine media. The system investigated in this study will be a submerged, packed bed reactor filled with high porosity corrugated plastic sheet modules.

2.17.3 General Description Suspended Growth Denitrification.

2.17.3.1 Suspended growth systems should be operated in the plug flow mode. The contents of the denitrification reactor must be kept mixed, but not aerated. Liquor from the reactor must receive clarification in order to collect biomass for return to the reactor vessel and to produce a clear effluent from the process. Because the nitrogen gas released in the process often becomes attached to the biological solids, a nitrogen release step is included between the reactor and the sedimentation facilities. The removal of the attached nitrogen gas bubbles can be perfomed either in aerated channels connecting the biological reactor to the sedimentation facilities or in a separate tank in which the biological solids are aerated for a short period of time.

2.17.4 Denitrification (Attached Growth).

2.17.4.1 Input Data.

2.17.4.1.1 Average wastewater flow, MGD (Qave)

2.17.4.1.2 Peak wastewater flow, MGD (Qpeak)

2.17.4.1.3 Wastewater temperature, ^{o}C

2.17.4.1.4 Influent nitrate nitrogen, mg/l $(NO_3\text{-}N)inf$

2.17.4.1.5 Allowable effluent nitrate nitrogen, mg/l $(NO_3\text{-}N)eff$

2.17.4.1.6 Wastewater dissolved oxygen content, mg/l

2.17.4.1.7 Wastewater alkalinity, mg/l as $CaCo_3$

2.17.4.1.8 Water viscosity, centipoise

2.17.4.2 Design Parameters.

2.17.4.2.1 pH 7.0 to 7.5.

2.17.4.2.2 Methanol requirement = 3 lb methanol/1b $(NO_3\text{-}N)$ inf.

2.17.4.2.3 Alkalinity production = 3 mg as $CaCO_3$/mg $NO_3\text{-}N$ removed.

2.17.4.2.4 Surface removal rate (SR), lb nitrate nitrogen removed/ft^2 media surface area/day.

2.17.4.2.5 Dentrification media specific surface (SS), ft^2/ft^3.

2.17.4.2.6 Dentrification media void volume (VV), %.

2.17.4.2.7 Wastewater application rate (AR), gpm/ft^2 of column cross-sectional area.

2.17.4.2.8 Water backwash rate = 10 gpm/ft^2.

2.17.4.2.9 Denitrification media depth, minimum is 8 feet.

2.17.4.3 Process Design Calculations.

2.17.4.3.1 Select the applicable surface removal rate (SR) for the minimum wastewater temperature (°C) - see Figure 2.17-1.

2.17.4.3.2 Determine the total media surface area required (TSA).

$$TSA = \frac{(Qpeak)(8.34)\ [(NO_3\text{-}N)inf - (NO_3\text{-}N)eff]}{SR}$$

2.17.4.3.3 Select the media specific surface (SS) value to be used:

2.17.4.3.4 Calculate the total volume of media needed (VM) in ft^3.

$$VM = \frac{TSA}{SS}$$

2.17.4.3.5 Select the design wastewater application rate (AR) in gpm/sf.

2.17.4.3.6 Determine the total column cross-sectional area (CSSA) in ft^2.

$$CSSA = \frac{Q_{avg} \times 10^6}{1440\ (AR)}$$

2.17.4.3.7 Determine the application rate at peak flow (AR_p).

$$AR_p = \frac{Qp \times 10^6}{(1440)(CSSA)}$$

2.17.4.3.8 Determine the media depth (D) in ft.

$$D = \frac{VM}{CSSA}$$

2.17.4.3.9 If D < 8; use 8' depth. Recalculate actual media volume (VM_A).

$$VM_A = (CSSA)(8')$$

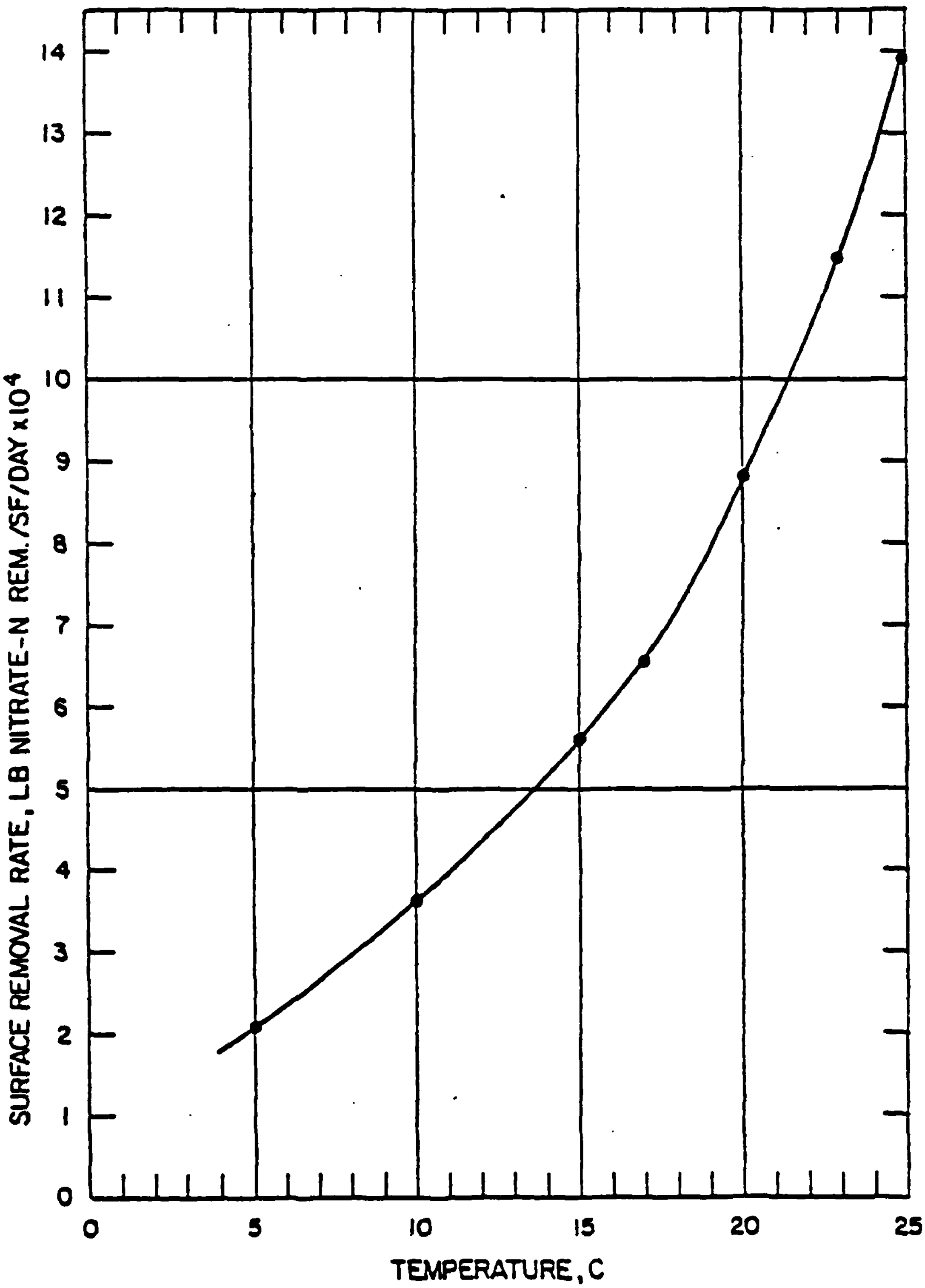

FIGURE 2.17-1

SURFACE DENITRIFICATION RATE FOR SUBMERGED MEDIA COLUMNS

2.17.4.3.10 Determine methanol requirement (M) in lb CH_3OH/day.

$$M = \frac{3 \text{ lb } CH_3OH}{\text{lb } (NO_3\text{-}N)\text{inf}} \times Q_{avg} \times 8.34 \text{ lb/gal} \times (NO_3\text{-}N)\text{inf}$$

2.17.4.3.11 Calculate alkalinity production (AP) in mg/l as $CaCO_3$.

$$AP = \frac{3 \text{ mg as } CaCO_3}{\text{mg } (NO_3\text{-}N) \text{ removed}} \left[(NO_3\text{-}N)\text{inf} - (NO_3\text{-}N)\text{eff}\right]$$

2.17.4.3.12 Calculate total water backwash requirement.

$$BV = 10\ (CSSA)$$

2.17.4.3.13 Effluent Characteristics

2.17.4.3.13.1 BOD_5. The values for BOD_5 are set as follows:

$$BODE = 10 \text{ mg/l}$$

$$S_e = 7 \text{ mg/l}$$

where

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.17.4.3.13.2 COD.

$$CODE = 1.5\ BODE$$

$$CODSE = 1.5\ (S_e)$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.17.4.3.13.3 Suspended Solids.

$$SSE = 20 \text{ mg/l}$$

where

SSE = effluent suspended solids, mg/l.

2.17.4.3.13.4 Nitrogen

NO3E is specified by user

$$NO2E = 0.0$$

where

NO3E = effluent NO_3 concentration, mg/l.

NO2 = effluent NO_2 concentration, mg/l.

2.17.4.3.13.5 Phosphorus

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.17.4.3.13.6 pH.

$$PH = 7.2$$

where

PH = effluent pH.

2.17.4.3.13.7 Settleable solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.17.4.3.13.8 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.17.4.4 Process Design Output Data.

2.17.4.4.1 Total media surface area required, ft^2.

2.17.4.4.2 Total media volume required, ft^3.

2.17.4.4.3 Denitrification column cross-sectional area, ft^2.

2.17.4.4.4 Denitrification media depth, ft.

2.17.4.4.5 Methanol requirement.

2.17.4.4.6 Alkalinity production.

2.17.4.4.7 Peak wastewater application rate.

2.17.4.4.8 Total water backwash requirements, gpm.

2.17.4.5 Quantities Calculations.

2.17.4.5.1 Selection of number of denitrification column units. The following rules will be used:

Q_{avg}-MGD	Number of Columns (NC)
0.5 - 2	2
2 - 5	2
5 - 10	4
10 - 20	6
20 - 30	8
30 - 40	12
40 - 50	16
50 - 70	24
70 - 100	32

When Q_{avg} is larger than 100 MGD, several batteries of columns will be used. See next section for details. Various example layouts for a single battery are shown in Figure 2.17-2.

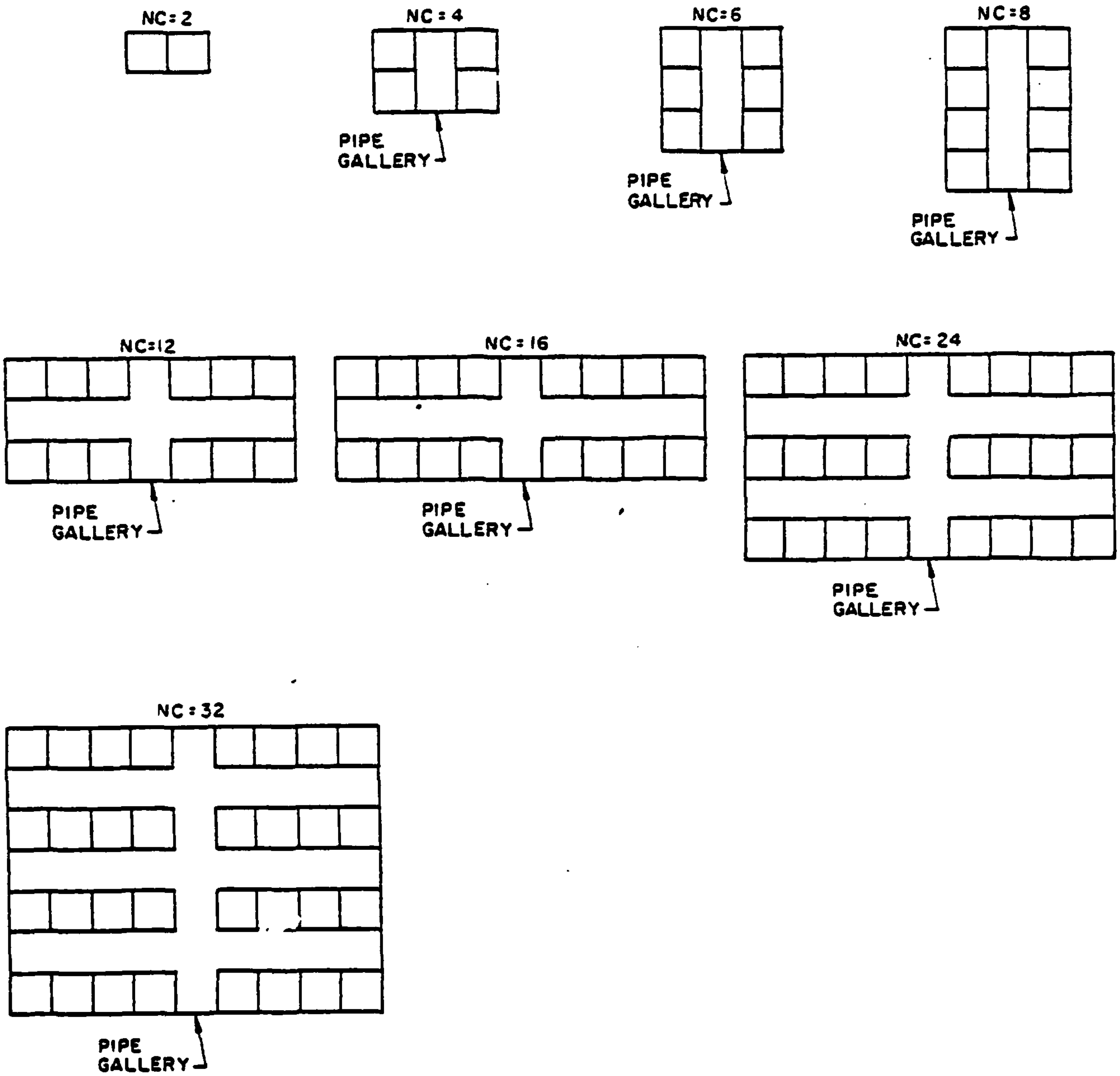

FIGURE 2.17-2. EXAMPLES OF COLUMN ARRANGEMENTS
COLUMN DENITRIFICATION PROCESS

2.17.4.5.2 Selection of number of columns and number of batteries when Q_{avg} is larger than 100 MGD. It is general practice in designing larger sewage treatment plants that several batteries of units, instead of a single group of units, are used. This is due to land availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.17.4.5.2.1 When $Q_{avg} \leq$ 100 MGD, only one battery of columns will be used. Thus,

$$NBC = 1$$

where

NBC = number of batteries of units.

2.17.4.5.2.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of denitrification columns. Each battery would handle half of the wastewater. The number of columns in each battery would be selected according to the rules established in Section 2.17.4.5.1 (above) by using half the design flow as Q_{avg}. Thus,

$$NBC = 2$$

2.17.4.5.2.3 When $Q_{avg} > 200$ mgd, the design will use three batteries, each handling one-third of the wastewater. Thus,

$$NBC = 3$$

2.17.4.5.3 Sizing of denitrification columns.

2.17.4.5.3.1 Height of columns. The overall column height is equal to the sum of the media depth, distribution system height and freeboard above the media.

$$HC = UH + D + F$$

where

HC = overall column height, ft.

UH = distribution system height, ft.
(use 8' as default value)

D = media depth, ft. (minimum is 8').

F = freeboard above media, ft.
(use 3' as default value)

2.17.4.5.3.2 Length and width of columns. In all cases, each column will be square, so that

$$L_c = W_c$$

where

L_c = column cross-sectional length, ft.

W_c = column cross-sectional width, ft.

These values are calculated from:

$$L_c = W_c = \left(\frac{CSSA}{NC}\right)^{0.5}$$

where

CSSA = total column cross-sectional area, sf.

2.17.4.5.3.3 Size of piping gallery. As seen in Figure 2.17-2, a pipe gallery will be provided when the number of columns is equal to or larger than four. The purpose of the piping gallery is to house various piping systems and control equipment. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximate this width. It is expressed as:

$$PGW = 20 + (0.3)\left(\frac{Q_{avg}}{NBC}\right)$$

where

PGW = piping gallery width, ft.

NBC = number of batteries of columns.

2.17.4.5.3.4 Size of underdrain system. The height of underdrain system was assumed to be 8'. The cross-sectional area of the underdrains is equal to that of the columns. Thus,

$$L_u = L_c$$

$$W_u = W_c$$

therefore

$$L_u = W_u = \left(\frac{CSSA}{NC}\right)^{0.5}$$

2.17.4.5.4 Methanol feed system.

2.17.4.5.4.1 Determine liquid methanol feed rate.

$$M_1 = M \times \frac{1\ gal}{5.9\ lb} \times \frac{1\ day}{1440\ min}$$

where

M_1 = liquid methanol feed rate, gpm.

M = methanol feed requirement (lb/day).

2.17.4.5.4.2 Determine methanol storage volume. Storage should be provided for approximately three weeks.

$$VSTR = (M_1)\left(\frac{1440\ min}{day}\right)(7\ day)\ (3\ weeks)$$

where

VSTR = total methanol storage volume, gal.

2.17.4.5.5 Earthwork required for construction. It is assumed that column bottom would be 3 feet below ground level. Thus, the earthwork required would be estimated by the following equations:

2.17.4.5.5.1 When NC = 2, the earthwork required would be:

$$V_{ew} = 3\ [(2W_c + 18.5)(W_c + 17) + (2W_c + 26.5)(W_c + 25)]$$

where

V_{ew} = quantity of earthwork required, cu ft.

W_c = width of each column.

2.17.4.5.5.2 When NC = 4, 6 or 8, the width and length of the concrete slab for the whole column battery can be calculated by:

$$L_B = \tfrac{1}{2}\ (NC)(L_c) + 16$$

$$W_B = 2(W_c) + PGW + 16$$

when NC = 12 or 16

$$L_B = \tfrac{1}{2}\ (NC)(L_c) + PGW + 16$$

$$W_B = 2(W_c) + PGW + 16$$

when NC = 24

$$L_B = 1/3\ (NC)(L_c) + PGW + 16$$

$$W_B = 3\ (W_c) + 2(PGW) + 16$$

When NC = 32

$$L_B = 1/4\ (NC)\ (L_c) + PGW + 16$$

$$W_B = 4(W_c) + 3(PGW) + 16$$

The earthwork for these cases is estimated by:

$$V_{ew} = 3 \times NBC\ [(L_B + 4)(W_B + 4) + (L_B + 12)(W_B + 12)]$$

2.17.4.5.6 Reinforced concrete slab quantity, it is assumed that a 2' -0" thick slab will be used in this program regardless of the size of the system.

2.17.4.5.6.1 For NC = 2,

$$V_{cs} = 2.0\ (2W_c + 14.5)(W_c + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.17.4.5.6.2 When NC $\geq$ 4

$$V_{cs} = (2)(L_B)(W_B)$$

where

L_B = slab length, ft.

W_B = slab width, ft.

2.17.4.5.7 Reinforced concrete wall quantity. It is assumed that a 1'-0" thick wall will be used in this program regardless of the size of the syster.

2.17.4.5.7.1 For NC = 2.

$$V_{cw} = (HC)(7L_c)$$

where

V_{cw} = R.C. wall quantity, cu ft.

HC = total column height, ft.

L_c = column cross-sectional length, ft.

2.17.4.5.7.2 For NC = 4, 6 or 8

$$V_{cw} = 2[(HC)\ (4 + 3\ (\frac{NC}{2} - 1))\ L_c]$$

2.17.4.5.7.3 For NC = 12 of 16

$$V_{cw} = 4[(HC)\ (4 + 3\ (\frac{NC}{4} - 1))\ L_c]$$

2.17.4.5.7.4 For NC = 24

$$V_{cw} = 6\ [(HC)\ (4 + 3\ (\frac{NC}{6} - 1))\ L_c]$$

2.17.4.5.7.5 For NC = 32

$$V_{cw} = 8[(HC)\ (4 + 3\ (\frac{NC}{8} - 1))\ L_c]$$

2.17.4.5.8 Reinforced concrete for piping gallery slab has been estimated in the column slab calculations. Only the concrete for ceilings and end slab is necessary. Assume these are 1'-6" thick.

2.17.4.5.8.1 When NC = 2

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.17.4.5.8.2 When NC = 4, 6 or 8

$$V_{cg} = 1.5\ (PGW)\ [\frac{(NC)(W_c)}{2} + 0.75(NC) + 1.5]$$

2.17.4.5.8.3 When NC = 12 or 16

$$V_{cg} = 1.5\ PGW\ [\ (\frac{NC}{2} + 3)\ W_c + 0.75(NC) + 1.5]$$

2.17.4.5.8.4 When NC = 24

$$V_{cg} = 1.5\ PGW\ [(\frac{2\ NC}{3} + 5)\ W_c + 0.75(NC) + 1.5]$$

2.17.4.5.8.5 When NC = 32

$$V_{cg} = 1.5\ PGW\ [(\frac{3\ NC}{4} + 7)\ W_c + 0.75(NC) + 1.5]$$

2.17.4.5.9 Summary of reinforced concrete structures.

2.17.4.5.9.1 Quantity of concrete slab

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of denitrification columns.

2.17.4.5.9.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg}$$

where

V_{cwt} = total quantity of R.C. wall for the construction of denitrification tanks.

2.17.4.5.10 Operation and maintenance manpower requirements. The manday per year requirement is a function of the design average flow of the denitrification system.

2.17.4.5.10.1 The maintenance labor can be estimated as follows:

When $Q_{avg} \leq 10$ mgd

$$ML = 1896.0\ (Q_{avg})^{0.278}$$

When $Q_{avg} > 10$ MGD

$$ML = 675.2\ (Q_{avg})^{0.727}$$

where

ML = maintenance labor, MH/yr.

2.17.4.5.10.2 The operation labor can be estimated as follows:

When $Q_{avg} \leq 10$ mgd

$$OL = 3351.2\ (Q_{avg})^{0.446}$$

When $Q_{avg} > 10$ mgd

$$OL = 4344.8\ (Q_{avg})^{0.3332}$$

where

OL = operation labor, MH/yr.

2.17.4.5.11 Energy requirements for operation. The power consumption for the denitrification system is related to the average design flow by the following:

$$KWH = 170,000\ (Q_{avg})^{0.92}$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.17.4.5.12 Operation and maintenance material and supply cost. It is assumed that 0.5% of the total bare construction cost of the denitrification system would be required annually for the replacement and repair material cost.

2.17.4.5.13 Other construction cost items. Using the above calculations, the majority of cost items for the denitrification process have been evaluated. Other cost items, such as control equipment, painting, piping, site cleaning and preparation, etc., can be estimated as a percentage of the total bare construction cost. This percentage value has been shown to vary from 10-20 percent of the total construction cost of the system. The value depends greatly on site location and system complexity. For a generalized model, an average value of 15% would be adequate. Thus,

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor to account for the minor cost items.

2.17.4.6 Quantities Calculation Output Data.

2.17.4.6.1 Number of columns, NC.

2.17.4.6.2 Number of batteries, NBC.

2.17.4.6.3 Height of columns, HC, ft.

2.17.4.6.4 Length of column, L_c, ft.

2.17.4.6.5 Width of column, W_c, ft.

2.17.4.6.6 Width of pipe gallery, PGW, ft.

2.17.4.6.7 Length of underdrain system, L_u, ft.

2.17.4.6.8 Width of underdrain system, W_u, ft.

2.17.4.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.17.4.6.10 Reinforced concrete slab quantity, V_{cs}, cu ft.

2.17.4.6.11 Reinforced concrete wall quantity, V_{cw}, cu ft.

2.17.4.6.12 Reinforced concrete for pipe gallery, V_{cg}, cu ft.

2.17.4.6.13 Operation labor, OL, MH/yr.

2.17.4.6.14 Maintenance labor, ML, MH/yr.

2.17.4.6.15 Energy requirements, KWH, kwhr/yr.

2.17.4.6.16 Correction factor for minor capital cost items, CF.

2.17.4.6.17 Percentage for O&M material and supply cost, OMMP, %.

2.17.4.7 <u>Unit Price Input Required</u>.

2.17.4.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.17.4.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.17.4.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.17.4.7.4 Marshall and Swift Equipment Cost Index, MSECI.

2.17.4.8 <u>Cost Calculations</u>.

2.17.4.8.1 Cost of earthwork, COSTE, dollars.

$$COSTE = \frac{V_{ew}}{27} \times UPIEX$$

where

COSTE = cost of earthwork, $.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, cu yd.

2.17.4.8.2 Cost of concrete wall in-place.

$$COSTCW = \frac{V_{cwt}}{27} \times UPICW$$

where

COSTCW = cost of concrete wall in-place, $.

V_{cwt} = quantity of R.C. Wall, cu ft.

UPICW = unit price input of concrete wall in-place, $/cu yd.

2.17.4.8.3 Cost of concrete slab in-place.

$$COSTCS = \frac{V_{cst}}{27} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.17.4.8.4 Cost of methanol feed system.

2.17.4.8.4.1 Methanol, CH_3OH, has a variety of names such as methyl alcohol, carbinol and wood alcohol and is normally supplied pure (99.9%). It is a colorless liquid, non-corrosive (except to aluminum and lead) at normal atmospheric temperatures. It has a density of 6.59 lbs per gallon at 20°C. Fire and explosion are primary dangers of methanol. Methanol can be received in 55 gallon metal drums, tank wagon, tank cars or tank truck. The recommended storage and feed system in municipal plants includes a methanol storage tank, feed pump and control system.

2.17.4.8.4.2 Methanol feed system cost. A cost curve is used for this purpose. This cost curve was generated based on estimated data. It gives an approximate estimate of the first quarter 1977 cost of methanol feed system. Thus, the cost of methanol feed system:

$$CMAFS = \frac{LCAT}{132} \times 1290\ (M)^{0.417}$$

where

CMAFS = capital cost of methanol feed system, \$.

LCAT = current EPA cost index for larger city advanced treatment.

M = methanol feed rate, pounds/day.

2.17.4.8.5 Cost of denitrification media. The media cost is calculated by using a unit cost (\$/cu ft media). This unit cost is \$3.00/cu ft in second quarter 1977 dollars.

$$TMC = \$3.00/cf\ (VM_A) \times \frac{MSECI}{491.6}$$

where

TMC = total installed media cost, \$.

VM_A = total media volume, cu ft.

MSECI = current Marshall Swift Equipment Cost Index.

2.17.4.8.6 Cost of distribution system and effluent weir troughs. This cost is based on a unit price of \$40/sf of cross-sectional area based on second quarter 1977 dollars.

$$CDEW = (\$40/sf)(CSSA) \times \frac{MSECI}{491.6}$$

where

CDEW = cost of distribution system and effluent weir troughs.

CSSA = total column cross-sectional area.

2.17.4.8.7 Other cost items. This cost includes the cost of process piping, control instrumentation, site work, etc. Costs can be adjusted by multiplying the correction factor CF by the sum of the other costs.

2.17.4.8.8 Total bare construction costs.

$$TBCC = (COSTE + COSTCW + COSTCS + CMAFS + TMC + CDEW) \times CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

2.17.4.8.9 Operation and maintenance material and supply cost.

$$OMMC = (0.005)(TBCC)$$

where

OMMC = operation and maintenance material and supply costs, \$/yr.

2.17.4.9 <u>Cost Calculations Output Data.</u>

2.17.4.9.1 Total bare construction cost of the denitrification process, TBCC, \$.

2.17.4.9.2 Operation and maintenance supply and material costs, OMMC, \$.

2.17.5 <u>Denitrification (Suspended Growth).</u>

2.17.5.1 <u>Input Data.</u>

2.17.5.1.1 Average wastewater flow, mgd (Qave).

2.17.5.1.2 Peak wastewater flow, mgd (Qpeak).

2.17.5.1.3 Wastewater temperature, ^oC.

2.17.5.1.4 Influent nitrate nitrogen, mg/l. (NO_3-N) inf.

2.17.5.1.5 Allowable effluent nitrate nitrogen, mg/l (NO_3-N) eff.

2.17.5.1.6 Wastewater dissolved oxygen content, mg/l.

2.17.5.1.7 Wastewater alkalinity, mg/l as $CaCo_3$.

2.17.5.1.8 Water viscosity, centipoise.

2.17.5.2 <u>Design Parameters.</u>

2.17.5.2.1 pH 7.0 to 7.5.

2.17.5.2.2 MLSS (X) = 1,000 to 2,000 mg/l.

2.17.5.2.3 MLVSS (X_1) = 0.8X.

2.17.5.2.4 Methanol requirement = 3 lb methanol/lb (NO_3-N) inf.

2.17.5.2.5 Alkalinity production = 3 mg as $CaCO_3$/mg NO_3-N removed.

2.17.5.2.6 Denitrifier gross yield (Y_D) = 0.6-1.2 lb VSS grown per lb removed.

2.17.5.2.7 Half saturation constant (Ko) = 0.16 mg/l NO_3-N at 20^oC for system with solids recycle.

2.17.5.2.8 Decay coefficient (K_D) = 0.04 day^{-1}.

2.17.5.2.9 Peak nitrate removal rate (q_D) = lb NO_3-N removed/lb MLVSS - day (assumes neither methanol nor nitrate is a limiting factor) - see Figure 2.17-3.

2.17.5.2.10 Final clarifier effluent suspended solids = 20 mg/l.

2.17.5.2.11 Final clarifier effluent BOD_5 = 10 mg/l.

2.17.5.2.12 Final clarifier sludge concentration (Xw) = 10,000 mg/l.

2.17.5.2.13 Final clarifier design hydraulic loading = 600 gpd/sf.

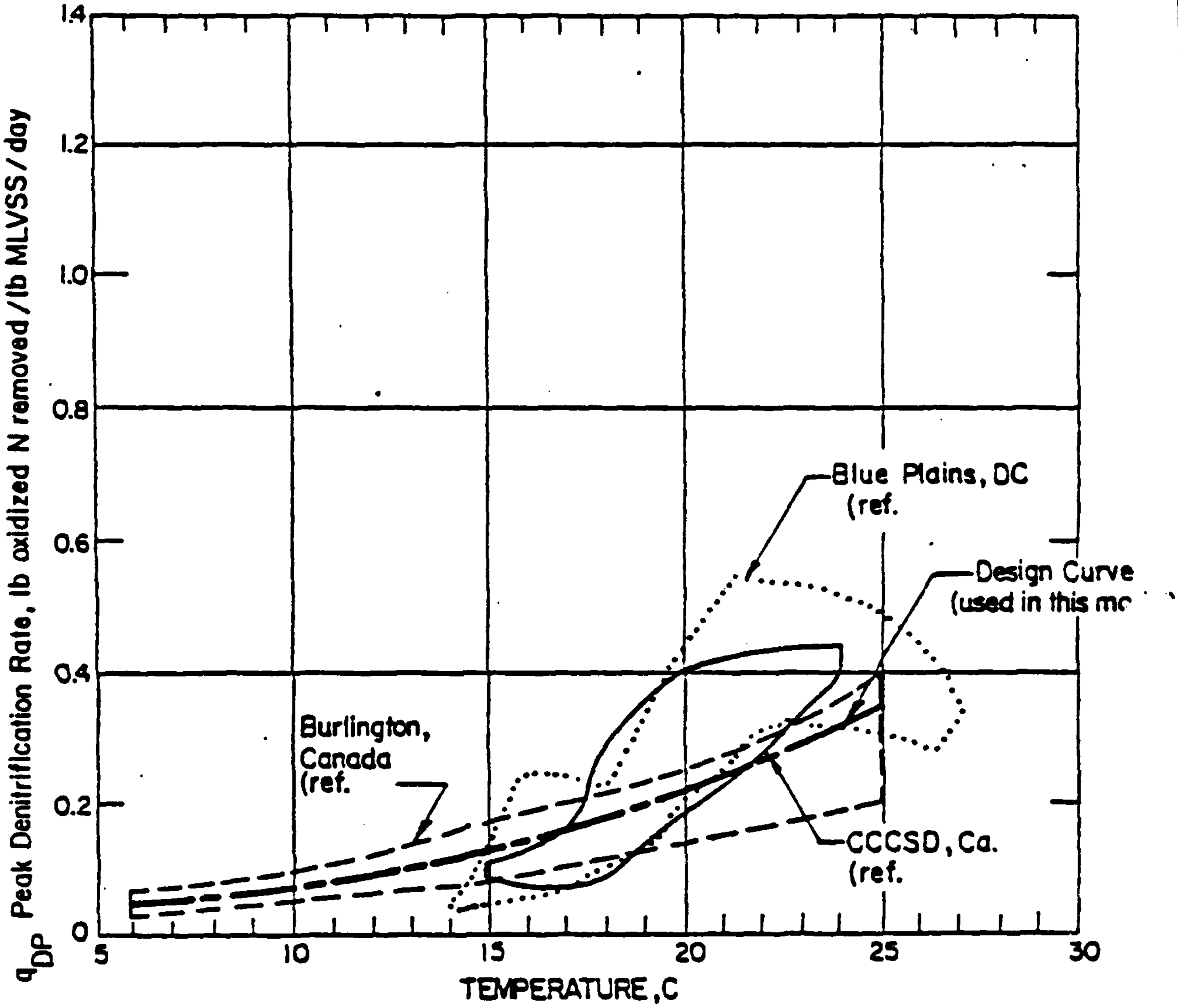

FIGURE 2.17-3 OBSERVED DENITRIFICATION RATES FOR SUSPENDED GROWTH SYSTEMS USING METHANOL

2.17.5.2.14 Final clarifier peak solids loading = 30 lbs/sf-day.

2.17.5.2.15 Return sludge rate Q_R/Qave) = 1.0.

2.17.5.3 Process Design Calculations.

2.17.5.3.1 Select the applicable peak denitrification rate (q_D) for the wastewater temperature (^{o}C) - see Figure 2.17.3.

2.17.5.3.2 Calculate the design denitrification rate (q_o).

$$q_o = q_D \frac{(NO_3\text{-}N)eff}{K_o + (NO_3\text{-}N)eff}$$

where

K_o = half saturation constant.

K_o = 0.16 mg/l NO_3-N.

2.17.5.3.3 Calculate the design sludge age (θ_c^D).

$$\theta_c^D = \frac{1}{\dfrac{Y_D q_o\ [(NO_3\text{-}N)inf - (NO_3\text{-}N)eff]}{(NO_3\text{-}N)inf - (NO_3\text{-}N)eff + K_o \ln \dfrac{(NO_3\text{-}N)inf}{(NO_3\text{-}N)eff}} - K_D}$$

where

Y_D = denitrifier gross yield.

K_D = decay coefficient.

2.17.5.3.4 Select a MLSS (X) value and calculate MLVSS (X_1).

$$X_1 = 0.8X$$

2.17.5.3.5 Calculate the hydraulic detention time (T) in days.

$$T = \frac{[(NO_3\text{-}N)inf - (NO_3\text{-}N)eff]\ Y_D \theta_c^D}{(X)\ (1 + K_D \theta_c^D)}$$

2.17.5.3.6 Calculate denitrification reactor volume (V) in million gallons.

$$V = Q_{ave} \times T$$

2.17.5.3.7 Determine sludge wasting schedule.

$$I = 8.34 \frac{lb}{gal} (X) (V)$$

where

I = total lb solids in denitrification reactor.

$S = I/\theta_c{}^D$

where

S = total sludge wasted, lb/day.

$$W = \frac{S}{(8.34\ lb/g) \times X_w}$$

where

W = waste sludge flow rate, mgd.

X_w = final clarifier sludge concentration = 10,000 mg/l.

Note: For purposes of preliminary design, it is assumed that no solids are lost in the final clarifier effluent.

2.17.5.3.8 Calculate methanol requirements (M) in lbs CH_3OH/day.

$$M = \frac{3\ lb\ CH_3OH}{lb\ (NO_3\text{-}N)inf} \times Q_{ave} \times 8.34 \frac{lb}{gal} \times (NO_3\text{-}N)inf$$

2.17.5.3.9 Calculate alkalinity production (AP) in mg/l as $CaCO_3$.

$$AP = \frac{3\ mg\ as\ CaCO_3}{mg\ (NO_3\text{-}N\ removed)} [(NO_3\text{-}N)inf - (NO_3\text{-}N)eff]$$

2.17.5.3.10 Size aerated stabilization tank (VA) in million gallons.

$$VA = Q_{ave} \times \frac{TA}{24}$$

where

TA = aerated stabilization tank detention time in hours.

= 1 hour.

2.17.5.3.11 Size mechanical mixing system for denitrification reactor. Assume sufficient power is needed to provide complete mixing both tanks.

$$P/V = 0.00475\ u_c{}^{0.3} X^{0.298}$$

where

P/V = hp/1000 gal tank volume.

u_c = water viscosity in centipoise for wastewater temperature (^oC).

X = MLSS in reactor, mg/l.

$$P = \frac{P}{V} \times V.$$

where

P = total denitrification reactor power requirement.

2.17.5.3.12 Size mechanical aeration system for aerated stabilization tank.

$$\frac{PA}{VA} = 1.0 \text{ to } 2.0 \text{ hp/1000 cf tank volume}$$

$$PA = \frac{PA}{VA} \times \frac{VA}{7.48 \times 10^3}$$

where

PA = total stabilization tank power requirement.

2.17.5.3.13 Size return sludge facilities.

$$Q_R/Q_{ave} = 1.0$$

$$\therefore Q_R = 1.0\ (Q_{ave})$$

2.17.5.3.14 Effluent Characteristics.

2.17.5.3.14.1 Suspended Solids.

SSE is specified by the user

where

SSE = effluent suspended solids concentration.

2.17.5.3.14.2 BOD_5.

$$BODE = S_e + 0.84\ (f')\ (X_v)_{eff}$$

$$S_e \text{ is specified by the user}$$

where

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

f' = degradable fraction of MLVSS.

$(X_v)_{eff}$ = effluent volatile suspended solids concentration, mg/l.

2.17.5.3.14.3 COD.

$$CODE = 1.5\ BODE$$

$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.17.5.3.14.4 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.17.5.3.14.5 Nitrogen.

NO3E is specified by the user

$$NO2E = 0.0$$

where

NO3E = effluent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

2.17.5.3.14.6 pH.

$$PH = 7.2$$

where

PH = effluent pH.

2.17.5.3.14.7 Settleable solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.17.5.3.14.8 Oil and grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.17.5.4 Process Design Output Data.

2.17.5.4.1 Design denitrification rate (q_o), lb NO_3-N removed/lb VSS-day.

2.17.5.4.2 Design sludge age (θ_c^D), days.

2.17.5.4.3 Denitrification reactor detention time (T), hours.

2.17.5.4.4 MLSS (X), mg/l.

2.17.5.4.5 MLVSS (X_1), mg/l.

2.17.5.4.6 Denitrification reactor volume (V), M.Gal.

2.17.5.4.7 Waste sludge quantity (S), lb/day.

2.17.5.4.8 Waste sludge flow rate (W), mgd.

2.17.5.4.9 Methanol requirement (M), lb/day.

2.17.5.4.10 Alkalinity production (AP), mg/l as $CaCO_3$.

2.17.5.4.11 Aerated stabilization tank volume (VA), M.Gal.

2.17.5.4.12 Power requirements for denitrification tank mixing.

2.17.5.4.13 Power requirements for aerated stabilization tank mixing.

2.17.5.4.14 Return sludge system capacity.

2.17.5.5 Quantities Calculations.

2.17.5.5.1 Selection of numbers of denitrification tanks and mechanical mixers per tank.

The following rule will be utilized in the selection of numbers of tanks and mechanical mixers per tank.

Q_{ave} (MGD)	Number of Tanks (NTD)	Number of Mixers Per Tank (NM)
0.5 - 2	2	1
2 - 4	2	1
4 - 10	2	2
10 - 20	3	3
20 - 30	4	3
30 - 40	4	5
40 - 50	6	4
50 - 70	8	4
70 - 100	10	5

When Q_{ave} is larger than 100 mgd, several batteries of tanks will be used. See Section 2.17.5.5.4 for details.

2.17.5.5.2 Selection of numbers of aerated stabilization tanks and mechanical aerators per tank.

Q_{ave} (MGD)	Number of Tanks (NTA)	Number of Aerators Per Tank (NA)
0.5 - 2	1	1
2 - 4	2	1
4 - 10	2	1
10 - 20	2	1
20 - 30	3	1
30 - 40	4	1
40 - 50	4	1
50 - 70	4	2
70 - 100	4	3

When Q_{ave} is larger than 100 mgd, several batteries of tanks will be used. See Section 2.17.5.5.3 for details.

2.17.5.5.3 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.17.5.5.3.1 When $Q_{avg} \leq 100$ mgd, only one battery of tanks will be used. Thus,

$$NB = 1$$

where

NB = number of batteries of units.

2.17.5.5.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of basins. Each battery would handle half of the wastewater. The number of denitrification or stabilization tanks in each battery would be selected according to the rules established in subsections 2.17.5.5.2 and 2.17.5.5.3 by using half the design flow as Q_{avg}. Thus,

$$NB = 2$$

2.17.5.5.3.3 When $Q_{avg} > 200$ mgd, the design will be perfomed to use three batteries of basins, each handling one-third of the wastewater. Thus,

$$NB = 3$$

2.17.5.5.4 Mechanical aeration equipment design for aerated stabilization tank.

2.17.5.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP, and 150 HP.

2.17.5.5.4.2 Horsepower for each individual aerator:

$$HPNA = \frac{(PA)}{(NBA)(NTA)(NA)}$$

where

HPNA = horsepower of each aerator, horsepower.

PA = horsepower required for the aerated stabilization tanks, horsepower.

NBA = number of batteries of stabilization tanks.

NTA = number of aeration tanks per battery.

NA = number of aerators per tank.

2.17.5.5.4.3 Sizing of aerators. Compare HPNA with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPNA. The capacity of the selected unit would be designated as HPSNA. Thus the total capacity of the aeration units would be:

$$HPTA = (NBA) \times (NTA) \times (NA) \times (HPSNA)$$

where

HPTA = total capacity of selected aerators for stabilization tanks, horsepower.

Comparing HPTA with PA, the designed horsepower would be correct.

If $PA \leq HPTA$, the selection would be correct.

If $PA > HPTA$, the HPSNA value should be adjusted to the next larger off-the-shelf sizes. Repeat this procedure until $PA \leq HPTA$ is met.

2.17.5.5.5 Mechanical mixing equipment design for denitrification tanks.

2.17.5.5.5.1 A turbine-flocculator device would be used to mix the denitrification tanks without introducing oxygen to the wastewater. These units consist of a submerged impellor attached to a vertical shaft. The shaft is rotated by a variable-speed drive, mounted on beams spanning the tank. The impellor consists of vertical, radical steel blades extending above and below a horizontal steel disc, which is attached to the drive shaft. Maximum standard size of these units is 5 hp. The 5 hp unit will provide mixing for a volume of 167,000 gallons. Other standard size units are 1/2, 3/4, 1, 1.5, 2 and 3 hp.

2.17.5.5.5.2 Horsepower for each individual mixer.

$$HPNM = \frac{(P)}{(NBD)(NTD)(NM)}$$

where

HPNM = horsepower for each individual mixer.

P = horsepower required for the denitrification reactors, hp.

NBD = number of batteries.

NTD = number of tanks per battery.

NM = number of mixers per tank.

2.17.5.5.5.3 Sizing of mixers. Compare HPNM with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPNM. The capacity of the selected unit is designated as HPSNM. Thus,

$$HPTM = (NBD)(NTD)(NM)(HPSNM)$$

where

PTM = total capacity of selected mixers for denitrification tanks, hp.

Compating HPTM with P, the designed horsepower requirement.

If $P \leq HPTM$, the selection is correct.

If $P > HPTM$, the HPSNM value would be adjusted to the next larger standard size. Repeat this procedure until $P \leq HPTM$.

2.17.5.5.6 Methanol feed system.

2.17.5.5.6.1 Determine the liquid methanol feed rate.

$$ML = M \times \frac{1 \text{ gal}}{5.9 \text{ lb}} \times \frac{1 \text{ day}}{1440 \text{ min}}$$

where

M_L = liquid methanol feed rate, gpm.

M = methanol feed requirement lb/day.

2.17.5.5.6.2 Determine methanol storage volume. Storage should be provided for approximately three weeks.

$$VSTR = (M_L) \left(\frac{1440 \text{ min}}{\text{day}}\right) \left(\frac{7 \text{ day}}{\text{week}}\right) (3 \text{ week})$$

where

VSTR = total methanol storage volume, gal.

2.17.5.5.7 Design of aerated stabilization tanks.

2.17.5.5.7.1 Volume of each individual tank would be:

$$VNA = \frac{(VA)}{(NBA)(NTA)}$$

where

VNA = volume of single stabilization tank, cu ft.

2.17.5.5.7.2 Depth of stabilization tanks. The depth of an aerated basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current would occur. If the water depth is too deep, insufficient mixing would be extended to the bottom of the tank and sludge accumulation would occur. Thus, proper selection of liquid depth of a basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When HPSNA $\leq$ 100 HP

$$DWA = 4.816 \, (HPSNA)^{0.2467}$$

When HPSNA > 100 HP

$$DWA = 15 \text{ ft}$$

where

DWA = water depth of the stabilization tanks, ft.

HPSNA = capacity of each aerator, HP.

2.17.5.5.7.3 Width and length of stabilization tank. The ratio between length and width of a tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, LA/WA = 1.

If NA = 2. Rectangular tank construction, LA/WA = 2.

If NA = 3. Rectangular tank construction, LA/WA = 3.

If NA = 4. Rectangular tank construction, LA/WA = 4.

and

$$LA/WA = NA$$

where

NA = number of aerators per tank.

LA = length of tank, ft.

WA = width of tank, ft.

After the volume, depth and LA/WA ratio of the tank are determined, the width of the tank can be calculated by:

$$WA = \left[\frac{VNA}{(DWA)(NA)}\right]^{1/2}$$

The length of the tank would be:

$$LA = (NA)\ (WA)$$

2.17.5.5.8 Design of denitrification tanks.

2.17.5.5.8.1 Volume of each individual tank would be:

$$VND = \frac{(V)}{(NBD)\ (NTD)}$$

where

VND = volume of single denitrification tank, cf.

V = total denitrification tank volume

2.17.5.5.8.2 Depth of tank. The relationship between the basin depth and the capacity of the mixers can be expressed as:

$$DWD = 1.45\ HPSNM + 5$$

where

DWD = water depth of the denitrification tanks, ft.

HPSNM = capacity of each mixer, hp.

2.17.5.5.8.3 Width and length of stabilization tank: the ratio between length and width is dependent on the number of mixers to be installed in the tank, NM.

If NM = 1. Square tank: LD/WD = 1

NM = 2. Rectangular tank: LD/WD = 2

NM = 3. Rectangular tank: LD/WD = 3

NM = 4. Rectangular tank: LD/WD = 4

NM = 5. Rectangular tank: LD/WD = 5

and

$$LD/WD = NM$$

where

NM = number of mixers per tank.

LD = length of tank, ft.

WD = width of tank, ft.

After the volume, depth and LD/WD ratio of the tank are determined, the width of the tank can be calculated by:

$$WD = \frac{VND}{(DWP)\ (NM)}$$

The length of the denitrification tank would be:

$$LD = (NM)\ (WD)$$

2.17.5.5.9 Treatment tank arrangements - either denitrification or aerated stabilization tanks.

2.17.5.5.9.1 Figure 2.17-4 shows the schematic diagram of the arrangements. Piping galleries will be provided when the number of tanks is equal to or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

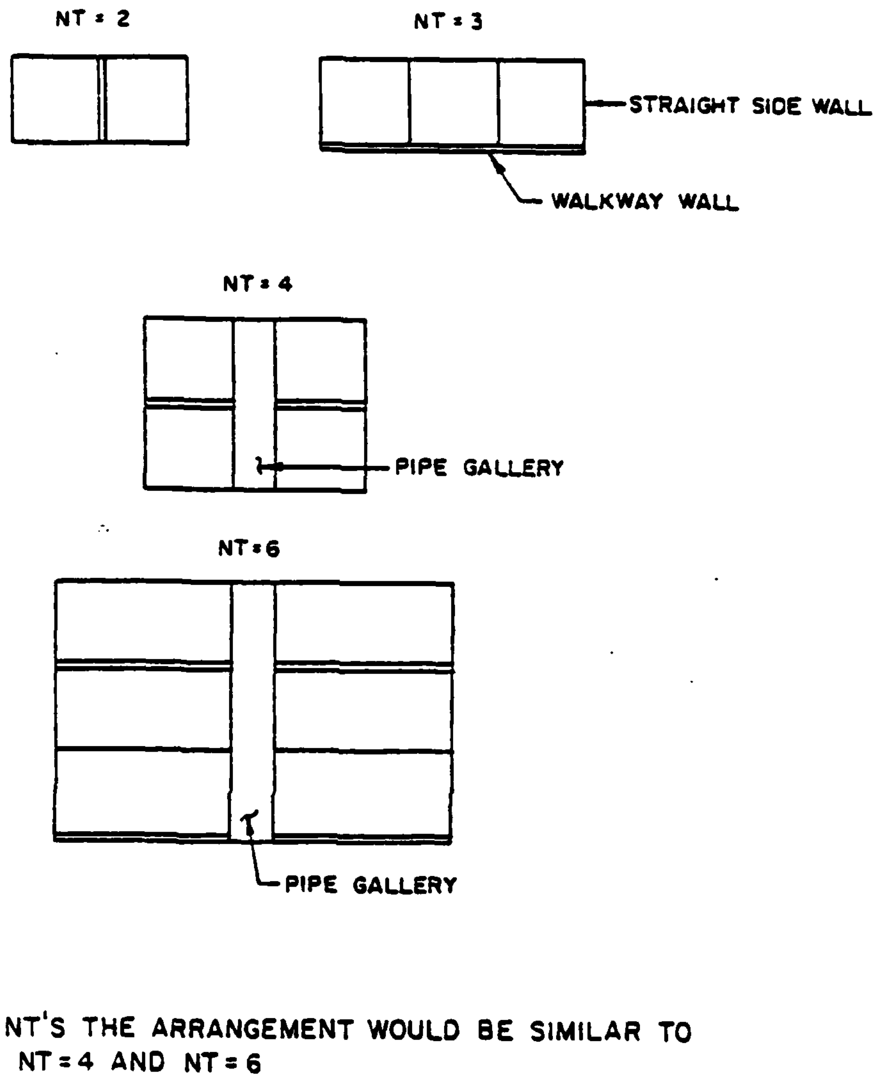

FIGURE 2.17-4. EXAMPLES OF TANK ARRANGEMENTS DENITRIFICATION PROCESSES

2.17.5.5.9.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries (either denitrification or stabilization).

2.17.5.5.10 Typical wall construction.

2.17.5.5.10.1 The wall constructions are different for complete mix and plug flow systems. The denitrification tanks should be designed as a plug flow system. The aerated stabilization tanks would utilize the complete mix scheme. In order to achieve complete mix, the inflow to the denitrification stabilization tanks would be distributed uniformly along one side of the aeration tank, flowing across the width of the tank and being discharged along the other side wall. Thus, a Y-wall construction will be used so that the top section of the wall can be an open channel for influent and/or effluent discharges. Figure 2.17-5, shows a typical section of the complete mix denitrification tank.

2.17.5.5.10.2 In using the plug system, influent to the denitrification basin will be piped to one end of the tank and discharged at the other end. Thus, it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.17-6. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.17.5.5.11 Earthwork required for construction. It is assumed that tank bottom would be 3 feet below ground level. Thus, the earthwork required would be estimated by the following equations:

2.17.5.5.11.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\,[(2\,W + 18.5)\,(W + 17) + (2\,W + 26.5)\,(W + 25)]$$

where

NT = number of denitrification or stabilization tanks.

V_{ew} = quantity of earthwork required, cu ft.

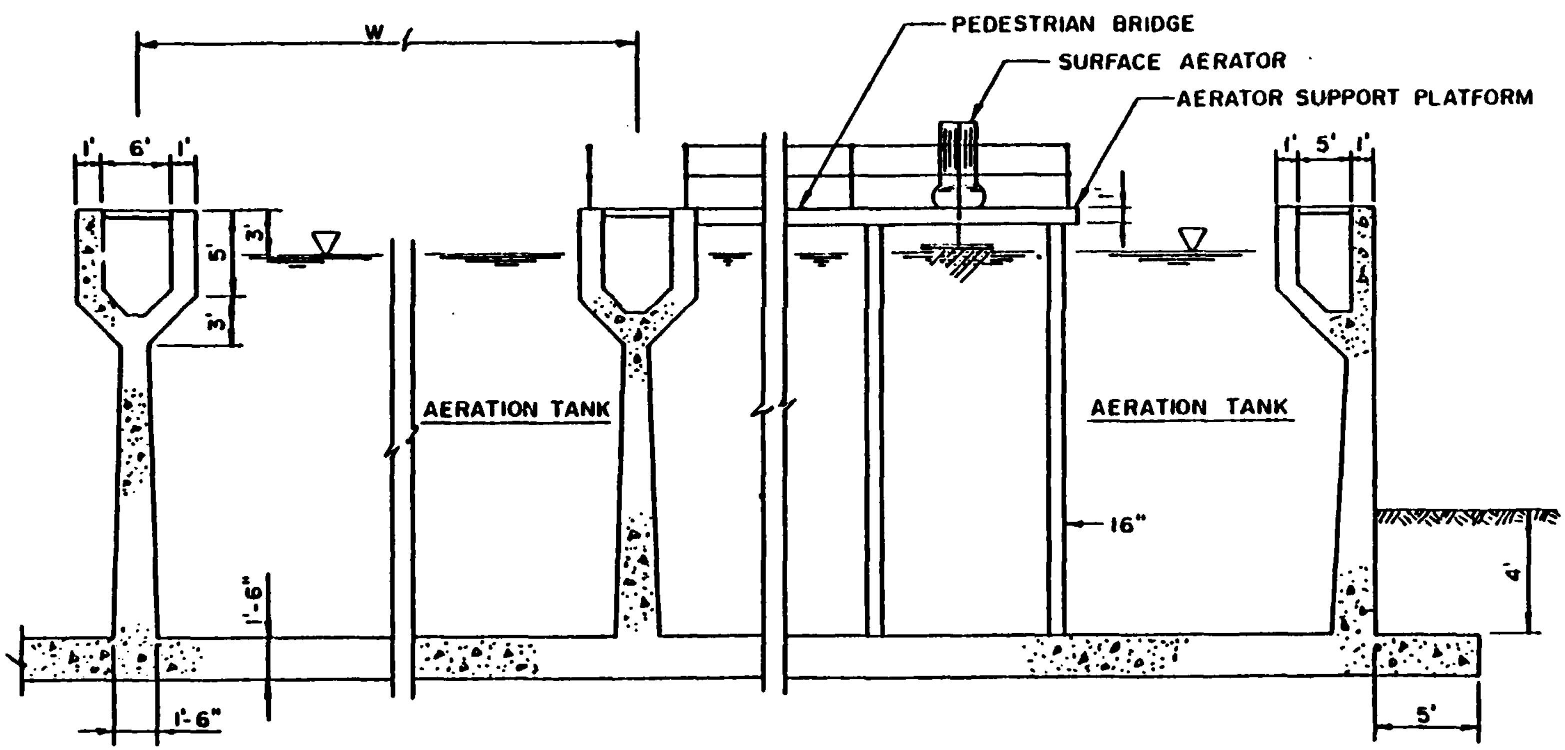

FIGURE 2.17-5
TYPICAL SECTION OF COMPLETE MIX AERATED STABILIZATION TANKS

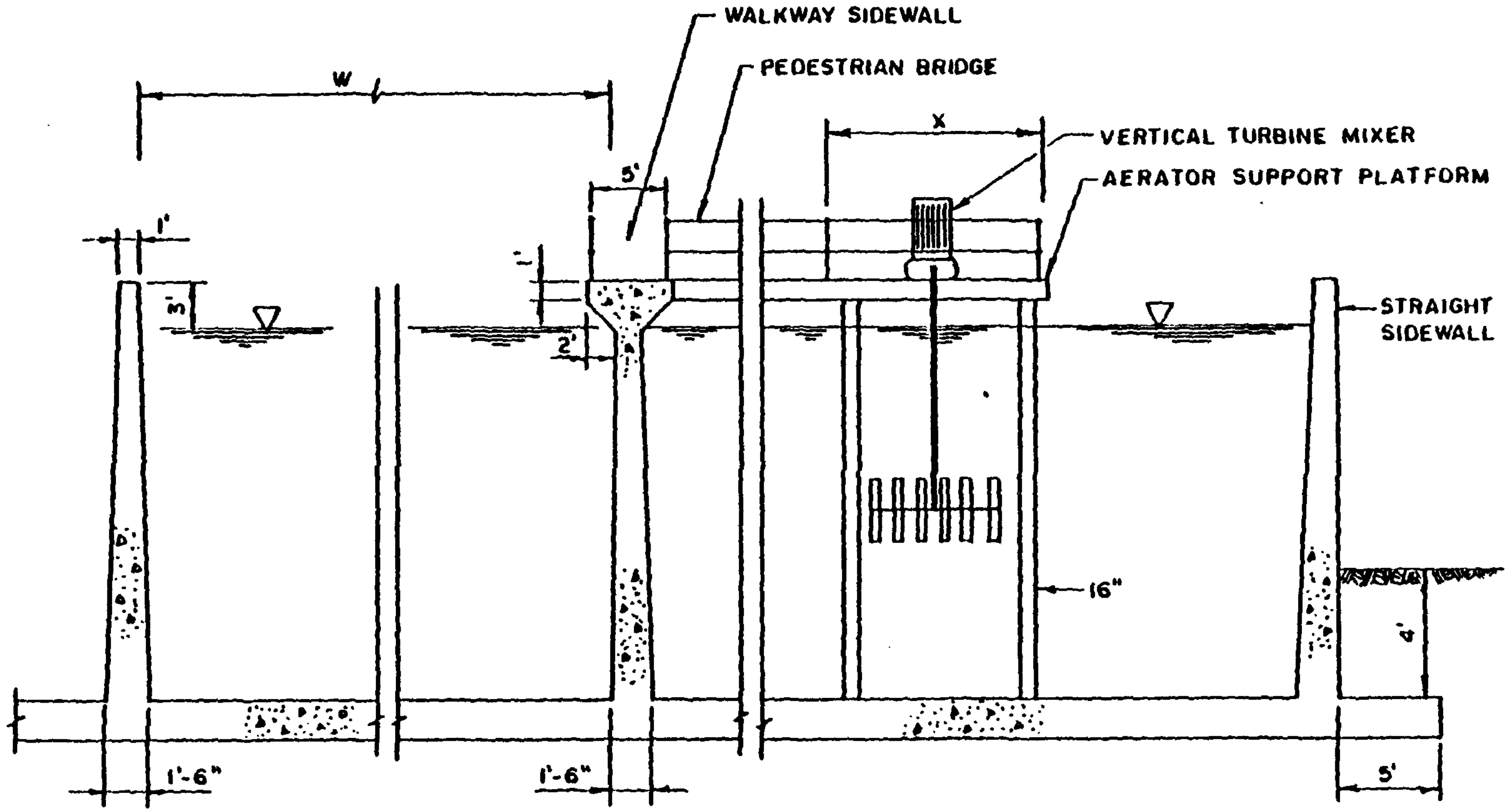

FIGURE 2.17-6 TYPICAL SECTION OF PLUG FLOW DENITRIFICATION TANKS

W = width of tank, ft.

2.17.5.5.11.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$

2.17.5.5.11.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus, the earthwork can be estimated by:

$$V_{ew} = 3 \times (NB)\ [(L_s + 4)\ (W_s + 4) + (L_s + 12)\ (W_s + 12)]$$

2.17.5.5.12 Reinforced concrete slab quantity.

2.17.5.5.12.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.17.5.5.12.2 For NT = 2,

$$V_{cs} = 1.5\ (2\ W + 14.5)\ (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.17.5.5.12.3 NT = 3,

$$V_{cw} = 1.5\ (3\ W + 16)\ (W + 13)$$

2.17.5.5.12.4 When NT $\geq$ 4.

$$V_{cs} = 1.5\ (L_s)\ (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.17.5.5.13 Reinforced concrete wall quantity.

2.17.5.5.13.1 For complete mix flow stabilization tanks,

2.17.5.5.13.1.1 When NTA = 2,

$$V_{cw} = WA\ [8.75\ DWA + 88]$$

where

V_{cw} = R.C. wall quantity, cu ft.

WA = width of individual tank, ft.

DWA = water depth of tank, ft.

2.17.5.5.13.1.2 When NTA = 3,

$$V_{cw} = 6\ (WA + 2)\ (1.25\ DWA + 13.45) + 5\ WA\ [DWA + 3]$$

2.17.5.5.13.1.3 When NTA > 3,

$$V_{cw} = (NBA) \quad 4 \times LA \times [1.25\ (DWA + 3) + 9.7] + (NTA - 2) \times LA \times (1.25\ DWA + 31) + 2.5\ (NTA) - (WA)\ (DWA + 3)$$

2.17.5.5.13.2 For plug flow denitrification tanks,

2.17.5.5.13.2.1 When NTD = 2,

$$V_{cw} = WD\ (1.25\ DWD + 11) + (6\ WD + 9)\ (1.25\ DWD + 3.75)$$

2.17.5.5.13.2.2 When NTD = 3,

$$V_{cw} = (1.25\ DWD + 11)\ (3\ WD + 6) + (1.25\ DWD + 3.75)\ (7\ WD + 6)$$

2.17.5.5.13.2.3 When NTD ≥ 4,

$$V_{cw} = \frac{NTD}{2}\ (LD + 3)\ (1.25\ DWD + 11) + [(0.5\ NTD + 2)\ (LD + 3) + 2\ (NTD)\ (WD)] \times (1.25\ DWD + 3.75) \times (NBD)$$

2.17.5.5.14 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the tank slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.17.5.5.14.1 When NT < 4

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.17.5.5.14.2 When NT ≥ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be

$$V_{cgc} = (NB) \times (1.5)\ (PGW)\ [\frac{(NT)\ (W)}{2} + 0.75\ (N) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2\ (PGW)\ (NB)\ (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus, total R.C. volume for piping gallery construction would be:

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.17.5.5.15 Reinforced concrete quantity for aerator/mixer supporting platform construction.

2.17.5.5.15.1 Number of aerator/mixer supporting platforms. Each unit will be supported by an individual platform.

2.17.5.5.15.2 Figure 2.17-7 shows a typical supporting platform for the aeration/mixing equipment. The width of the platform would be a function of the capacity of the unit to be supported. The following experience formula is given to approximate this relationship.

$$Z = 5 + 0.078\ (HPSN)$$

where

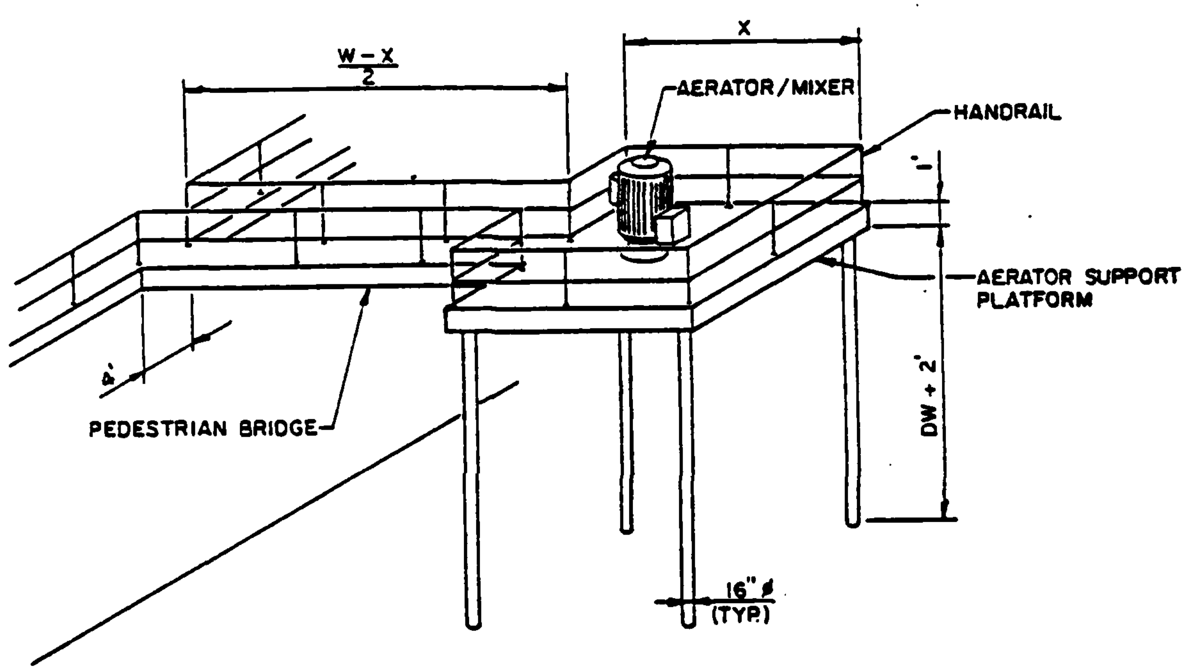

FIGURE 2.17-7 AERATOR/MIXER SUPPORT PLATFORM

Z = width of the platform, ft.

HPSN = horsepower of the mechanical aerator/mixer, hp.

2.17.5.5.15.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [Z^2 + 5.6\ (DW + 2)]\ (NT)\ (N)\ (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the basin, ft.

N = number of mechanical aerators or mixers.

2.17.5.5.15.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2\ (W - X)]\ (NB)\ (NT)\ (N)$$

where

V_{cst} = total quantity of R.C. slab for the construction of tanks, cu ft.

2.17.5.5.16 Summary of reinforced concrete structures.

2.17.5.5.16.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction tanks, cu ft.

2.17.5.5.16.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of tanks, cu ft.

V_{cw} = quantity of tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator/mixer supporting platfoms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.17.5.5.17 Quantity of handrail for saiety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator/mixer platfoms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.17.5.5.17.1 When NT = 2.

$$LHR = 4\,W + 11 + 2 \times (3Z + W - 4)$$

where

LHR = handrail length, ft.

W = tank width, ft.

Z = width of aerator/mixer supporting platfom, ft.

2.17.5.5.17.2 When NT = 3

$$LHR = 6\,W + 10 + 3 \times (3Z + W - 4)$$

2.17.5.5.17.3 When NT ≥ 4

If $\frac{NT}{2}$ is an even number

$$HRL = \left\{PGW + (NT)\,(W) + [L + 3 - 4\,(N)]\,(NT) + (N) \times (NT) \times (3Z + W - 4)\right\} \times (NB)$$

If $\frac{NT}{2}$ is an odd number

$$HRL = \left\{PGW + (NT)\,(W) + [L + 3 - 4\,(N)]\,(NT + 2) + (N)\,(NT)\,(3Z + W - 4)\right\} \times (NB)$$

where

PGW = width of the piping gallery, ft.

N = number of mixers or aerators per tank.

2.17.5.5.18 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration and mixing equipment.

2.17.5.5.18.1 Calculate the total installed capacity of the aeration and mixing equipment.

$$TICA = (NBA)\ (NTA)\ (NA)\ (HPSNA) + (NBD)\ (NTD)\ (NM)\ (HPSNM)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSNA = capacity of one individual aerator, horsepower.

HPSNM = capacity of one individual mixer, horsepower.

2.17.5.5.18.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 hp

$$OMH = 242.4\ (TICA)^{0.3731}$$

When TICA ≥ 200 hp

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.17.5.5.18.3 The maintenance manpower requirement can be estimated as follows:

When TICA < 100 hp

$$MMH = 106.3\ (TICA)^{0.4031}$$

When TICA ≥ 100 hp

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.17.5.5.19 Energy requirement for operation. By assuming that all the aerators and mixers will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.17.5.5.20 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint and repair material, etc. These costs are estimated as a percent of installed costs for the aeration/mixing equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration and mixing equipment, horsepower.

2.17.5.5.21 Other construction cost items. Using the above calculation, the majority of cost items of the denitrification process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary 4 to 15 percent of the total construction cost of the system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.17.5.6 Quantities Calculations Output Data.

2.17.5.6.1 Number of denitrification tanks, NTD.

2.17.5.6.2 Number of aeration stabilization tanks, NTA.

2.17.5.6.3 Number of aerators per stabilization tank, NA.

2.17.5.6.4 Number of mixers per denitrification tank, NM.

2.17.5.6.5 Number of process batteries, NBA, NBD.

2.17.5.6.6 Capacity of each individual aerator, HPSNA, hp.

2.17.5.6.7 Capacity of each individual mixer, HPSNM, hp.

2.17.5.6.8 Depth of stablization tanks, DWA, ft.

2.17.5.6.9 Depth of denitrification tanks, DWD, ft.

2.17.5.6.10 Length of stabilization tanks, LA, ft.

2.17.5.6.11 Length of denitrification tanks, LD, ft.

2.17.5.6.12 Width of stabilization tanks, WA, ft.

2.17.5.6.13 Width of denitrification tanks, WD, ft.

2.17.5.6.14 Width of pipe gallery, PGWA or PGWD, ft.

2.17.5.6.15 Earthwork required for construction, V_{ew}, cu ft.

2.17.5.6.16 Total quantity of R.C. Slab, V_{cst}, cu ft.

2.17.5.6.17 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.17.5.6.18 Quantity of handrail, LHR, ft.

2.17.5.6.19 Operation manpower requirement, OMH, man-hour/yr.

2.17.5.6.20 Maintenance, manpower requirement, MMH, man-hour/yr.

2.17.5.6.21 Electrical energy for operation, KWH, kwhr/yr.

2.17.5.6.22 Percentage for O&M material and supply cost, OMMP, percent.

2.17.5.6.23 Correction factor for minor capital cost items, CF.

2.17.5.7 <u>Unit Price Input Required</u>.

2.17.5.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.17.5.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.17.5.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.17.5.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, \$, optional.

2.17.5.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.17.5.7.6 Equipment installation labor rate, LABRI, \$/MH.

2.17.5.7.7 Crane rental rate, UPICR, \$/hr.

2.17.5.7.8 Unit price of handrail, UPIHR, \$/LF.

2.17.5.8 <u>Cost Calculations</u>.

2.17.5.8.1 Cost of earthwork.

$$COSTE + \frac{V_{ew}}{27} \times UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.17.5.8.2 Cost of concrete wall in-place.

$$COSTCW = \frac{V_{cwt}}{27} \times UPICW$$

where

COSTCW = cost of concrete wall in-place, \$.

V_{cwt} = quantity of R.C. wall, cu ft.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.17.5.8.3 Cost of concrete slab in-place.

$$COSTCS = \frac{V_{cst}}{27} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.17.5.8.4 Cost of installed mechanical aeration equipment.

2.17.5.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = (RSXSA)\ (SSXSA)$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSNA hp and that of the standard size aerator with 20 hp.

2.17.5.8.4.2 RSXSA. The cost ratio can be expressed as:

$$RSXSA = 0.2148\ (HPSNA)^{0.513}$$

where

HPSNA = capacity of each individual aerator, hp.

2.17.5.8.4.3 Cost of standard size aerator. The cost of the standard size pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16{,}300$$

For a better estimate, SSXSA should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16{,}300 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, First quarter 1977.

2.17.5.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When HPSNA $\leq$ 60 hp

$$IMHA = 39 + 0.55\ (HPSNA)$$

When HPSNA $>$ 60 hp

$$IMHA = 61.3 + 0.18\ (HPSNA)$$

where

IMHA = installation man-hour requirement, MH.

2.17.5.8.4.5 Crane requirement for installation.

$$CHA = (0.1) \times IMHA$$

where

CHA = crane time requirement for installation, hr.

2.17.5.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.17.5.8.4.7 Installed equipment cost, IECA.

$$IECA = [CSXSA\ (1 + PMINC) + IMHA \times LABRI + CHA \times UPICR]$$

$$\times\ (NBA) \times (NTA) \times (NA)$$

where

ICEA = installed equipment cost, $.

LABRI = labor rate, $/MH.

UPICR = crane rental rate, $/hr.

2.17.5.8.5 Cost of installed mechanical mixing equipment.

2.17.5.8.5.1 Purchase cost of vertical turbine mixers. The purchase cost of mixers can be obtained by using the following equation:

$$CSXM = (RSXM)\ (SSXM)$$

where

CSXM = purchase cost of each mixer.

SSXM = purchase cost of a standard size (5 hp) mixer.

RSXM = ratio of cost of mixers with capacity of HPSNM (hp) and that of the standard size mixer (5 hp).

2.17.5.8.5.2 RSXM. The cost ratio can be expressed as:

$$RSXM = 0.67 + 0.067 \times (HPSNM)$$

where

HPSNM = capacity of each individual mixer, hp.

2.17.5.8.5.3 Cost of standard size mixer. For the most accurate estimate, SSXM should be obtained from a suitable equipment vendor. For first quarter 1977, SSXM = \$7,740. If SSXM is not obtained from the vendor the cost will be updated by the Marshall and Swift Equipment Cost Index.

$$SSXM = 7,740 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Cost Index for 1st quarter, 1977.

2.17.5.8.5.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted mixers can be estimated as:

$$IMHM = 61.3 + 0.18\ (HPSNM)$$

where

IMHM = installation man-hour requirement.

2.17.5.8.5.5 Crane requirement for installation.

$$CHM = (0.1) \times IMHM$$

where

CHM = crane time for installation, hr.

2.17.5.8.5.6 Other costs associated with the installed equipment. This category includes the costs for electrical wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = % of purchase cost of equipment as minor installation cost.

2.17.5.8.5.7 Installed equipment cost, IECM.

$$IECM = [(CSXM)(1 + PMINC) + IMHM \times LABRI +$$

$$CHM \times UPICR] \times (NBD)(NTD)(NM)$$

where

IECM = installed equipment cost.

2.17.5.8.6 Cost of methanol feed system.

2.17.5.8.6.1 Background. Methanol, CH_3OH, has a variety of names such as methyl alcohol, carbinol and wood alchohol and is normally supplied pure (99.9%). It is colorless liquid, non-corrosive (except to aluminum and lead) at normal atmospheric temperatures. It has a density of 6.59 lbs per gallon at 20°C. Fire and explosion are the primary dangers of methanol. Methanol can be received in 55 gallon metal drums, tank wagon, tank cars or tank truck. The recommended storage and feed system in municipal plants is schematically shown in Figure 2.17-8. The total system includes a methanol storage tank, feed pump and control system.

2.17.5.8.6.2 Methanol feed system cost. A cost curve is used for this purpose. (See Figure 2.17-9). This cost curve was generated based on estimated data. It gives an appropriate estimate of the first quarter 1977 cost of methanol feed system. Thus, the cost of methanol feed system:

$$CMAFS = \frac{LCAT}{132} \times 1290\ (M)^{0.417}$$

where

CMAFS = capital cost of methanol feed system, $.

LCAT = current EPA cost index for larger city advanced treatment.

M = methanol feed rate, pounds/day.

2.17.5.8.7 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = installed handrail cost, $.

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, $ per linear foot.

2.17.5.8.8 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

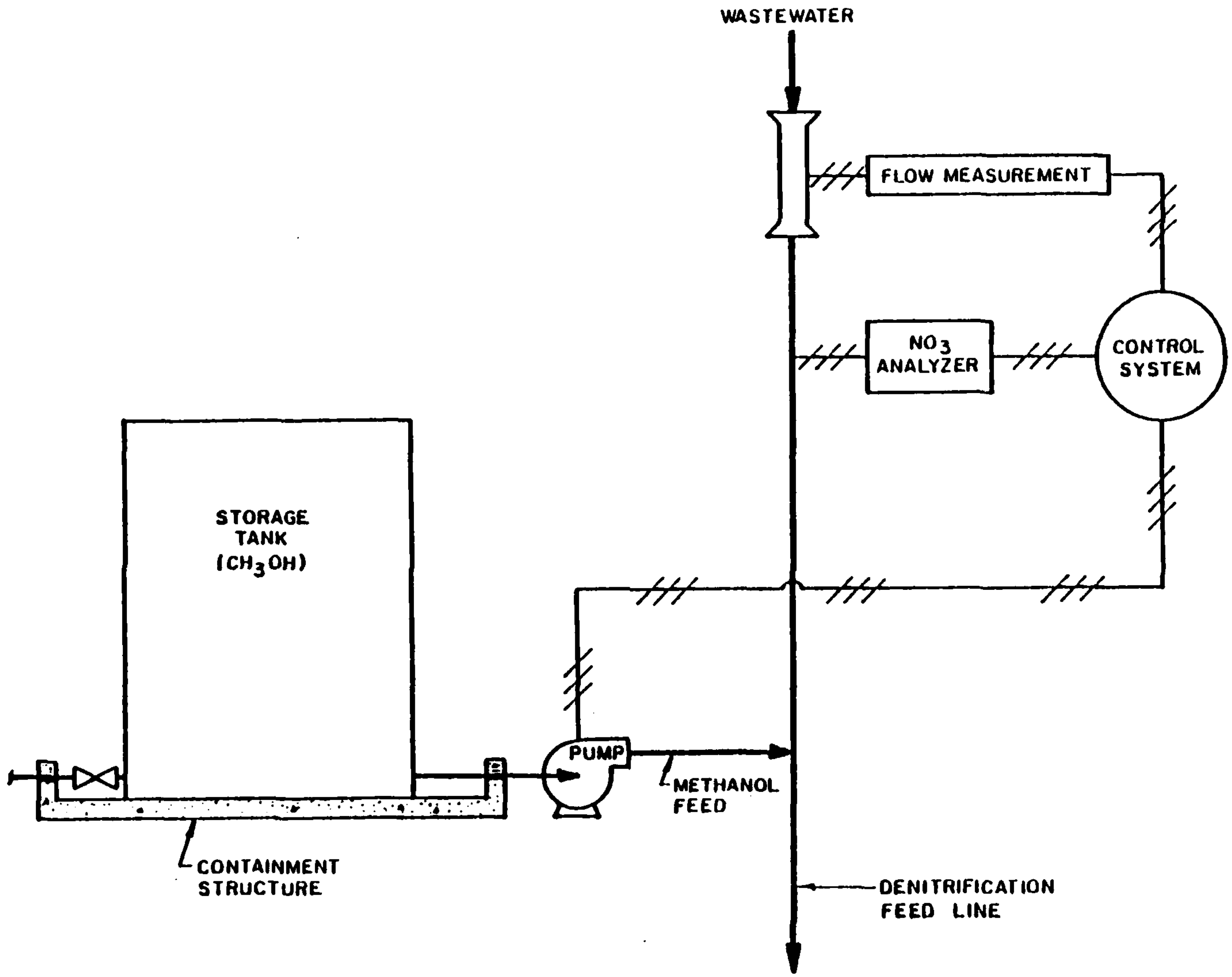

FIGURE 2.17-8. SCHEMATIC DIAGRAM OF METHANOL STORAGE AND FEED SYSTEM

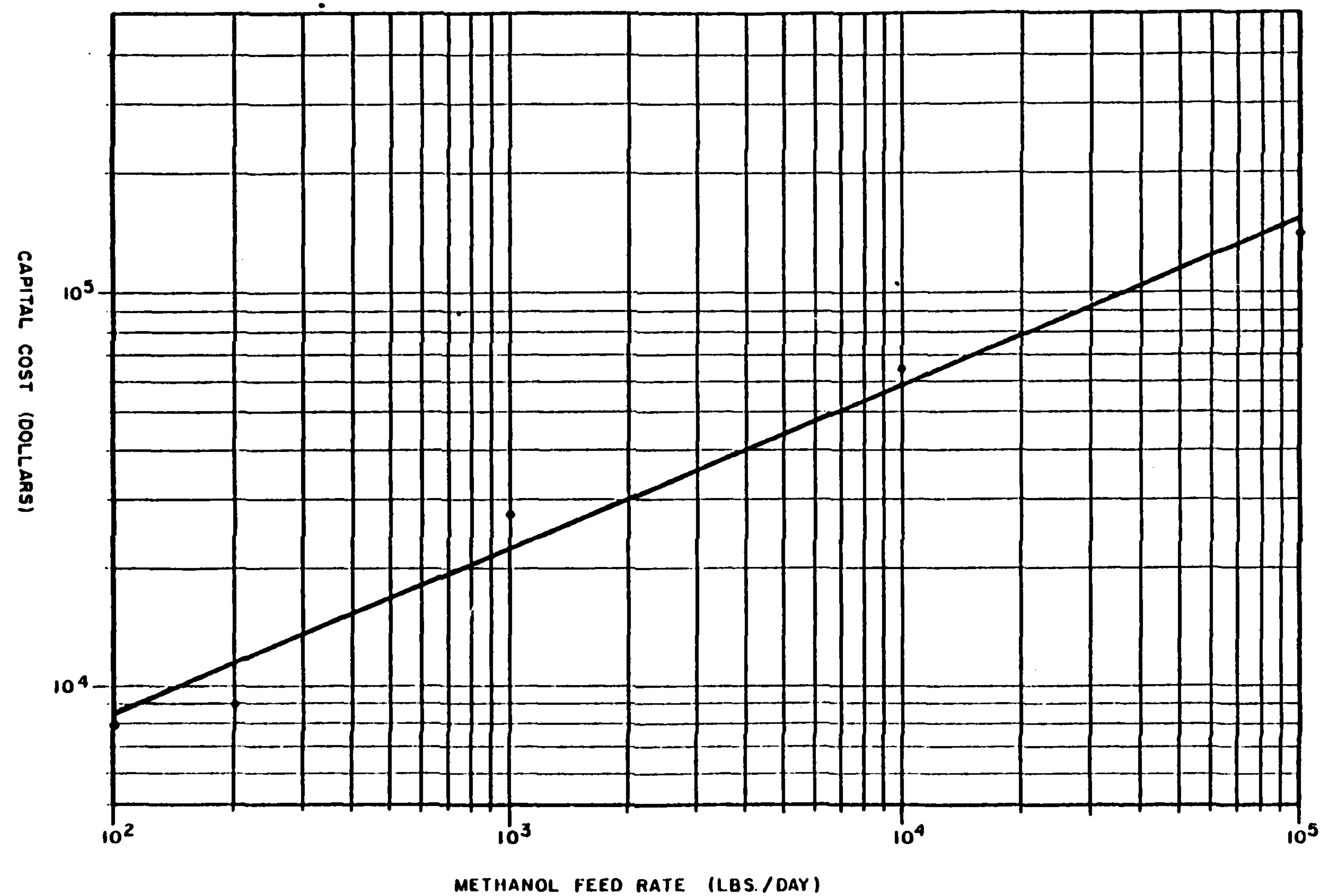

FIGURE 2.17-9. COST OF METHANOL STORAGE, TRANSFER, FEEDING AND CONTROL SYSTEM.

2.17.5.8.9 Total bare construction costs.

$$TBCC = (COSTE + COSTCW + COSTCS + IECA + IECM + CMAFS + COSTHR) \times CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

2.17.5.8.10 Operation and maintenance costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment cost, it can be calculated by:

$$OMMC = IECA \times \frac{OMMPA}{100} + IECM \times \frac{OMMPM}{100}$$

where

OMMC = operation and maintenance material and supply costs, \$/yr.

OMMPA = percent of the installed aerator cost as O&M material and supply expenses.

OMMPM = percent of the installed mixer cost as O&M material and supply expenses.

2.17.5.9 Cost Calculations Output Data.

2.17.5.9.1 Total bare construction cost of the denitrification and aerated stabilization process, TBCC, \$.

2.17.5.9.2 Operation and maintenance supply and material costs, OMMC, \$.

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2.19 DIGESTION

2.19.1 Background.

2.19.1.1 Sludge digestion primarily has a two fold objective, stabilization of the sludge and reduction of sludge quantities. Stabilization produces a less odorous and putrescible sludge and also reduces the number of pathogenic organisms in the sludge. The reduction in quantity of sludges is desirable because it decreases the quantity and thus the cost of ultimate disposal of sludges.

2.19.1.2 There are two very popular methods of digestion currently being used; aerobic and anaerobic. Aerobic digestion as its name suggests is carried out in an oxygen atmosphere while anaerobic digestion is accomplished in an oxygen free atmosphere.

2.19.1.3 Both the aerobic and anaerobic sludge digestion processes will be addressed in detail in this manual.

2.19.2 General Description Aerobic Digestion.

2.19.2.1 This method of digestion is capable of handling waste activated, trickling filter, or primary sludges as well as mixtures of the same. The aerobic digestor operates on the same principles as the activated sludge process. As food is depleted, the microbes enter the endogenous phase and the cell tissue is aerobically oxidized to CO_2, H_2O, NH_3, NO_2, and NO_3.

2.19.2.2 Up to 80 percent of the cell tissue may be oxidized in this manner; the remaining fractions contain inert and nonbiodegradable materials. Factors to be considered during the design process are characteristics (origin(s)) of the sludge, hydraulic residence, true solids loading criteria, energy requirement for mixing, environmental conditions, and process operation.

2.19.2.3 The aerobic combustion of the sludges is usually depicted as follows.

$$C_5H_7O_2N + 5O_2 = 5CO_2 + 2H_2O + NH_3$$

The NH_3 in the presence of oxygen can be further oxidized to NO_2 or NO_3.

2.19.2.4 Advantages claimed for aerobic as compared with anaerobic digestion are listed below.

2.19.2.4.1 VSS is reduced to 40-50 percent--nearly equivalent to that for anaerobic.

2.19.2.4.2 Supernatant has lower BOD.

2.19.2.4.3 A relatively stable humuslike end product is produced.

2.19.2.4.4 More basic fertilizer values are recovered.

2.19.2.4.5 Operation is relatively simple.

2.19.2.4.6 Capital cost is lower.

2.19.2.4.7 Odor is minimal.

2.19.2.4.8 Sludges dewater well (this is a controversial statement).

2.19.2.5 The major disadvantages appear to be:

2.19.2.5.1 A higher operating cost associated with O_2 supply.

2.19.2.5.2 The lack of a useful by-product (no CH_4).

2.19.3 General Description Anaerobic Digestion.

2.19.3.1 This type of digestion can be traced back to the 1850's. The mechanics involved in the process are summarized in Figure 2.19-1. Anaerobic digestion may be adapted to both stationary and mobile systems for handling solids from waste treatment systems, trailer dump stations, marine dump stations, and vault toilets.

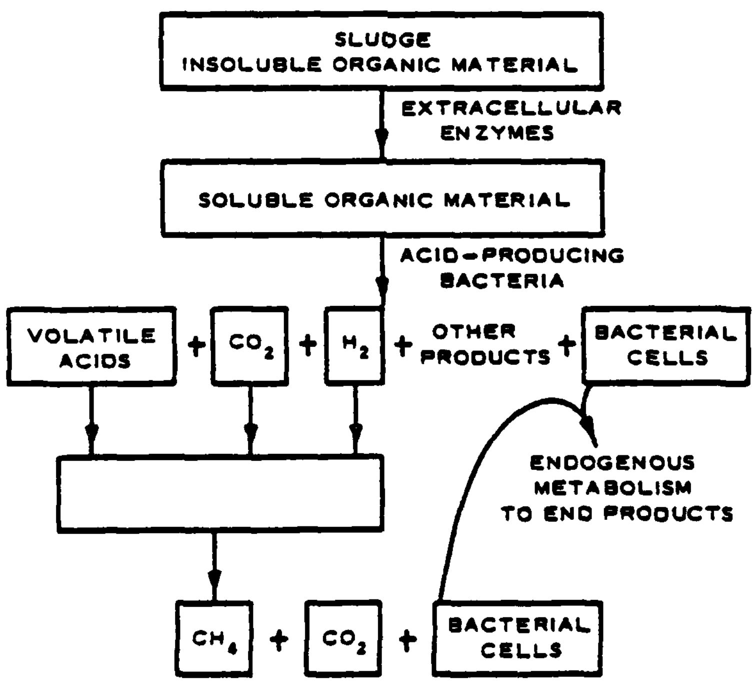


Figure 2.19-1 Mechanism of Anaerobic Sludge Digestion (Eckenfelder).

2.19.3.2 There are conventional and high-rate digesters; the conventional design uses the one- or two-stage process. In any of these systems, provisions are normally made for sludge heating. The principal difference in the one- and two-stage systems is that, in the two-stage systems, digestion is accomplished in the first tank. Figures 2.19-2 and 2.19-3 contain a pictorial explanation of the system.

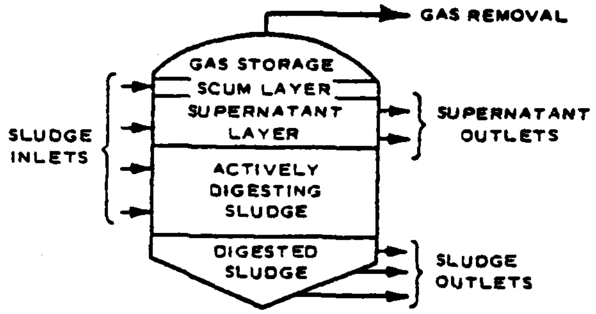


Figure 2.19-2 Schematic of Conventional Digester Used in the One-stage Process.

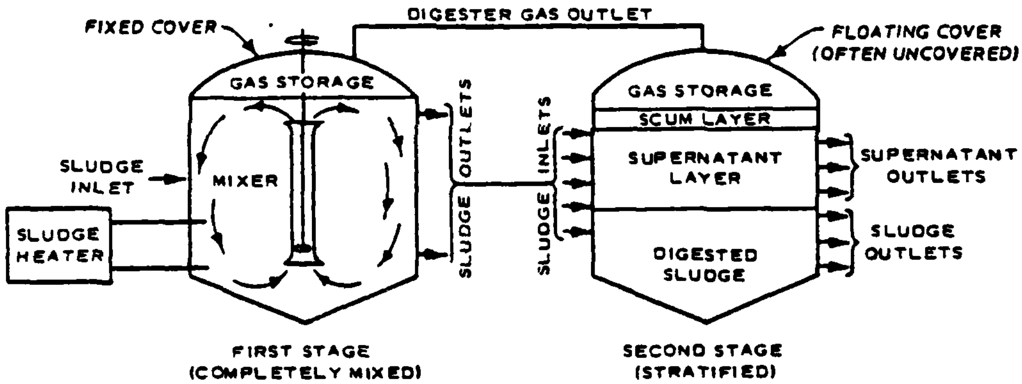


Figure 2.19-3. Schematic of Two-stage Digestion Process.

2.19.3.4 The advantages of anaerobic as compared with aerobic digestion include:

2.19.3.4.1 Higher organic loading, i.e. rates of treatment not limited by O_2 transfer.

2.19.3.4.2 Minimal need for biological nutrients (N and P) and for further treatment.

2.19.3.4.3 Lower energy requirement.

2.19.3.4.4 Production of a useful by-product (CH_4) which has a low heat value compared to natural gas.

2.19.3.5 The disadvantages are as follows:

2.19.3.5.1 Digesters must be heated to 85-90°F for optimum operation.

2.19.3.5.2 Molecular oxygen is toxic to the system and must be excluded.

2.19.3.5.3 Fundamental knowledge concerning the process is sparse.

2.19.3.5.4 Highly skilled operation is required.

2.19.3.5.5 Anaerobic digesters are easily upset by unusual conditions and are slow to recover.

2.19.3.5.6 Recycled supernatant liquor tends to "shock" wastewater treatment facilities.

2.19.3.5.7 The fact that a closed vessel is required complicates the inevitable cleaning necessary.

2.19.3.5.8 The recovered gas, while usefuly, increases initial costs because of the necessity of providing explosion-proof appurtenant equipment.

2.19.3.5.9 Digested sludge tends to be high in alkalinity. Where vacuum filtration preceded by chemical coagulation with the usual inorganic chemicals is practiced, increased chemical costs are inevitable.

2.19.4 Aerobic Digestion (Diffused Aeration).

2.19.4.1 Input Data.

2.19.4.1.1 Sludge production.

2.19.4.1.1.1 Primary, lb/day.

2.19.4.1.1.2 Secondary, lb/day.

2.19.4.1.2 Solids contents (percent solids).

2.19.4.1.2.1 Primary, percent.

2.19.4.1.2.2 Secondary, percent.

2.19.4.1.3 Specific gravity.

2.19.4.1.4 Volatile solids content, percent.

2.19.4.2 Design Parameters.

2.19.4.2.1 The design parameters are as shown in Table 2.19-1 below.

Table 2.19-1. Aerobic Digestion Design Parameters.

Parameter	Value
Hydraulic detention time, days at 20°C[a]	
Activated sludge only	12 to 16
Activated sludge from plant operated without primary settling	16 to 18
Primary plus activated or trickling-filter sludge	18 to 22
Solids loading, lb volatile solids/ft^3/day	0.1 to 0.2
Oxygen requirements	
BOD_5 in primary sludge, lb/lb cell tissue	2
Energy requirements for mixing	
Mechanical aerators, hp/1000 ft^3	0.5 to 1.0
Air mixing, cfm/1000 ft^3	20 to 30
Dissolved oxygen level in liquid, mg/l	1 to 2

[a]Detention times should be increased for temperatures below 20°C. If sludge cannot be withdrawn during certain periods (e.g. weekends, rainy weather), additional storage capacity should be provided. Ammonia produced during carbonaceous oxidation is oxidized to nitrate.

2.19.4.3 Process Design Calculations.

2.19.4.3.1 Calculate total quantity of raw sludge.

$$V_s = \frac{SP(100)}{(\text{specific gravity})(\text{percent solids})(8.34)}$$

where

V_s = volume of raw sludge to digester, gal/day.

SP = sludge produced, lb/day.

2.19.4.3.2 Select hydraulic detention time and calculate digesters' volume.

$$V = (t)(V_s)$$

where

V = volume of digester, gal.

t = hydraulic detention time, days.

V_s = volume of raw sludge to digester, gal/day.

2.19.4.3.3 Check volatile solids loading.

$$VS_L = \frac{\text{lb/day VS}}{V} (7.48) \quad (0.1\text{-}0.2)$$

where

VS_L = volatile solids loading, lb VS/ft^3/day.

V = volume of digester, gal.

2.19.4.3.4 Calculate solids retention time.

2.19.4.3.4.1 Assume percent destruction of volatile solids: 40 percent is common but it increases with temperature and retention time from approximately 33 to 70 percent.

2.19.4.3.4.2 Calculate solids accumulation per day.

$$S_{ac} = SP - SP \quad \frac{\%\ \text{volatile}}{100} \quad \frac{\%\ \text{destroyed}}{100} \quad (0.75)$$

where

S_{ac} = solids accumulated per day.

SP = sludge produced.

2.19.4.3.4.3 Assume MLSS in digester and calculate total capacity of sludge digester.

$$DC = (V)\ (MLSS)\ (8.34)\ (10^{-6})$$

where

DC = digester capacity, lb.

V = volume of digester, gal.

MLSS = mixed liquor SS in digester, mg/l.

2.19.4.3.4.4 Calculate solids retention time.

$$SRT = \frac{DC}{S_{ac}}$$

where

SRT = solids retention time, days.

DC = digester capacity, lb.

S_{ac} = total solids accumulated, lb/day.

2.19.4.3.5 Calculate sludge wasting schedule. Assume solids content in digested sludge is approximately 2.5 percent.

$$\text{Volume of sludge to be wasted} = \frac{\text{total sludge in digester (lb)(100)}}{\text{specific gravity)(\% solids)(8.34)}}$$

This volume must be wasted each SRT.

2.19.4.3.6 Calculate oxygen requirements.

2.19.4.3.6.1 O_2 required for bacterial growth. Assume O_2 required per pound of volatile solids destroyed.

$$O_2 = (O_T)SP \left(\frac{\%\ \text{volatile}}{100}\right)\left(\frac{\%\ \text{destroyed}}{100}\right)$$

where

O_2 = total oxygen required, lb/day.

O_T = oxygen required/lb of volatile solids destroyed $\approx$ 2.0 lb.

SP = sludge produced, lb/day.

2.19.4.3.6.2 Energy for mixing.

2.19.4.3.6.2.1 Assume standard transfer efficiency, percent.

2.19.4.3.6.2.2 Assume constants α, β , and p.

2.19.4.3.6.2.3 Select summer temperature T.

2.19.4.3.6.2.4 Calculate operating transfer efficiency.

$$OTE = STE \quad \frac{\left[{C_s}_T \beta p - C_L\right] \alpha\ (1.024)^{T-20}}{9.20}$$

where

OTE = operating transfer efficiency, percent.

STE = standard transfer efficiency, percent (5-8%).

$(C_s)_T$ = O_2 saturation at the summer temperature.

β = (C_s waste/C_s water) $\approx$ 0.9.

p = correction for altitude $\approx$ 1.0.

C_L = minimum oxygen to be maintained in the digester, mg/l.

= (K_{La} waste/K_{La} water) 0.9.

K_{La} = oxygen transfer coefficient.

T = temperature, $^{\circ}C$.

2.19.4.3.6.3.5 Calculate air supply; check against a minimum of 20 cfm/1000 ft^3.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_S = \frac{O_2(7.48)(10^5)}{(OTE\ \%)\ \ 0.0176\ \dfrac{lb\ O_2}{ft^3\ air}\ \ 1440\ \dfrac{min}{day}\ \ V}$$

where

R_s = air supply, cfm/1000 ft^3.

O_2 = oxygen supply, lb/day.

OTE = operating transfer efficiency, percent.

V = volume of the basin, gal.

2.19.4.3.7 Calculate volatile content of wasted sludge.

$$PSVOLAT = \frac{SLVS\ (1 - \frac{PSVD}{100})}{[1 - (\frac{PSVD}{100})\ (\frac{SLVS}{100})}$$

where

PSVOLAT = percent volatile solids in wasted sludge, %.

SLVS = percent volatile solids in influent sludge, %.

PSVD = percent of volatile solids destroyed, %.

2.19.4.3.8 Supernatant Return.

2.19.4.3.8.1 Quantity.

2.19.4.3.8.1.1 Activated Sludge and Oxidation Ditch.

$$QSUP = (Q_{avg})(0.02)$$

where

$QSUP$ = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.19.4.3.8.1.2 Trickling Filter and Rotating Biological Contactor.

$$QSUP = (Q_{avg})(0.007)$$

where

$QSUP$ = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.19.4.3.8.2 Supernatant Quality.

$$\begin{aligned} TSS &= 3400 \\ BOD &= 500 \\ COD &= 2600 \\ TKN &= 17.0 \\ PH &= 7.0 \end{aligned}$$

where

TSS = total suspended solids concentration, mg/l.

BOD = BOD_5 concentration, mg/l.

COD = COD concentration, mg/l.

PH = pH.

2.19.4.4 Process Design Output Data.

2.19.4.4.1 Raw sludge specific gravity.

2.19.4.4.2 Detention time, days.

2.19.4.4.3 Volatile solids destroyed, percent.

2.19.4.4.4 Mixed liquor solids, mg/l.

2.19.4.4.5 Solids in digested sludge, percent.

2.19.4.4.6 Rate constant, BOD_5 applied to filter.

2.19.4.4.7 Coefficient of O_2 saturation in waste/O_2 saturation in water.

2.19.4.4.8 Standard transfer efficiency, percent.

2.19.4.4.9 Digester volume, gal.

2.19.4.4.10 Volatile solids loading, lb VS/ft^3/day.

2.19.4.4.11 Solids accumulated, lb/day.

2.19.4.4.12 Volume of wasted sludge, gal.

2.19.4.4.13 Solids retention time, day.

2.19.4.4.14 Oxygen requirement, lb/day.

2.19.4.4.15 Air supply, cfm/1000 ft^3.

2.19.4.4.16 Percent volatile solids in wasted sludge, %.

2.19.4.5 Quantities Calculations.

2.19.4.5.1 Design values for activated sludge system.

$$V_d = \frac{V}{7.48}$$

$$CFM_d = (R_s)\ (V)\ (1.34 \times 10^{-4})$$

where

V = volume of digester, gal.

2.19.4.5.2 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

V_s (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

When V_s is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.19.4.5.3 Selection of number of tanks and number of batteries of tanks when V_s is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.19.4.5.3.1 When $V_s \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.19.4.5.3.2 When $100 < V_s \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.19.4.5.3 by using half the design flow as V_s. Thus

$$NB = 2$$

2.19.4.5.3.3 When $V_s > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.19.4.5.4 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10-15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

2.19.4.5.5 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.19.4.5.6 Design of aeration tanks.

2.19.4.5.6.1 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.19.4.5.6.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft and 30 ft, respectively.

2.19.4.5.6.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft, then recalculate VN using NT = NT + 1, then recalculate L.

2.19.4.5.7 Aeration tank arrangements.

2.19.4.5.7.1 Figure 2.19-4 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\ (\frac{Q_{avg}}{NB})$$

where

PGW = pipe gallery width, ft.

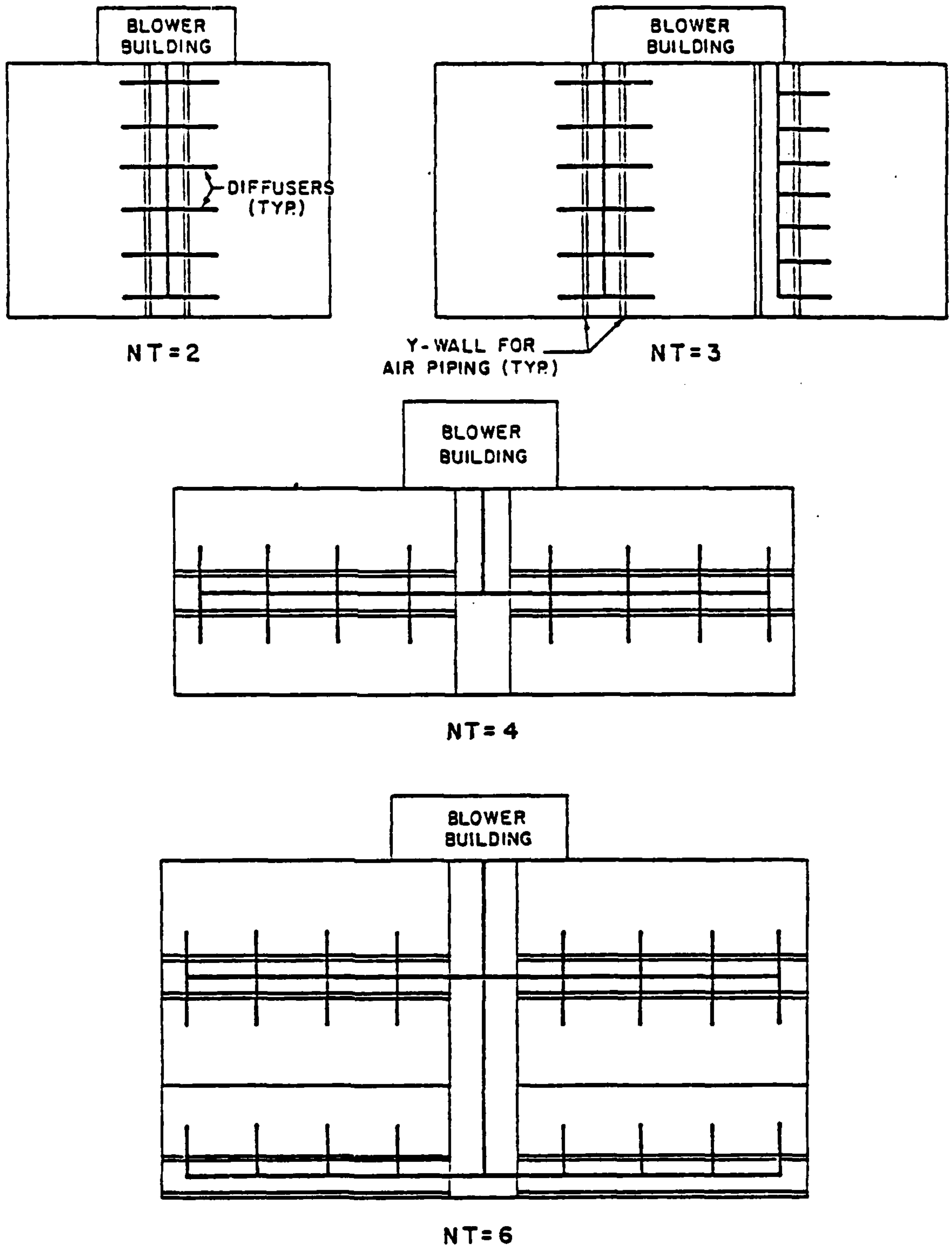


FIGURE 2.19-4. AERATION TANK ARRANGEMENT
DIFFUSED AERATION

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.19.4.5.8 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.19.4.5.8.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ \left[\frac{(NT(31.5) + 15.5)\ (L + 17) + (NT(31.5) + 23.5)\ (L + 25)}{2}\right]$$

where

V_{ew} = volume of earthwork required, cu ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.19.4.5.8.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ \left[\frac{(15.75(NT)+15.5)\ (2L+PGW+20) + (15.75(NT)+2.5)\ (2L+PGW+28)}{2}\right]$$

2.19.4.5.9 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

2.19.4.5.9.1 For NT less than 4:

$$V_{cs} = 1.5\ NB\ [(NT(31.5) + 15.5)\ (L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu ft.

2.19.4.5.9.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5\ NB\ [(15.75(NT) + 15.5)\ (2L + PGW + 200)]$$

2.19.4.5.10 Reinforced concrete wall quantities.

2.19.4.5.10.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.19-5. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

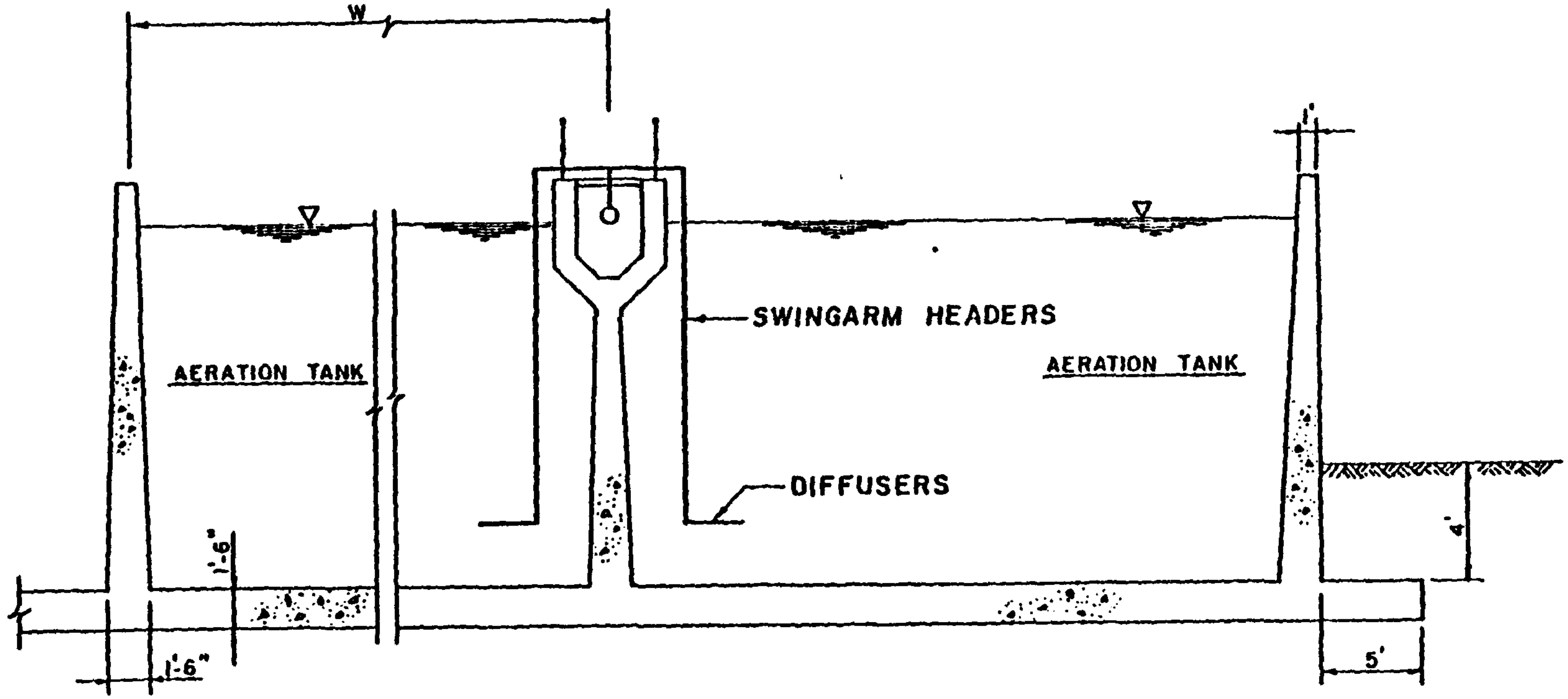


FIGURE 2.19-5. TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

2.19.4.5.11.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.19.4.5.11.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.19.4.5.11.4 When NT 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

where

V_{cw} = R.C. wall quantity required, cu ft.

L = length of aeration tanks, ft.

2.19.4.5.11 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.19.4.5.11.1 If NT is less than 4:

$$LHR = [2(NT)\ (L) + 2(L) + 61.5(NT) + 1.5]\ NB$$

2.19.4.5.11.2 If NT is greater than or equal to 4:

$$LHR = [2(NT)\ (L) + (4L) + 36.5(NT) + 2\ PGW + 13]\ NB$$

where

LHR = handrail length, ft.

2.19.4.5.12 Calculate operation manpower requirements.

2.19.4.5.12.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36\ (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, MH/yr.

2.19.4.5.12.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.19.4.5.13 Calculate maintenance manpower requirements.

2.19.4.5.13.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.19.4.5.13.2 If CFM_d 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance manpower required, MH/yr.

2.19.4.5.14 Energy requirement for operation. The electrical energy required for operation is related to the air requirement by the following equation:

$$KWH = (CFM_d)\ (241.6)$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.19.4.5.15 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.19.4.5.16 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.19.4.6 <u>Quantities Calculation Output Data.</u>

2.19.4.6.1 Number of aeration tanks, NT.

2.19.4.6.2 Number of diffusers per tank, ND_t.

2.19.4.6.3 Number of process batteries, NB.

2.19.4.6.4 Number of swing arm headers per tank, NSA_t.

2.19.4.6.5 Length of aeration tanks, L, ft.

2.19.4.6.6 Width of pipe gallery, PGW, ft.

2.19.4.6.7 Earthwork required for construction, V_{ew}, cu ft.

2.19.4.6.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.19.4.6.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.19.4.6.10 Quantity of handrail, LHR, ft.

2.19.4.6.11 Operation manpower requirement, OMH, MH/yr.

2.19.4.6.12 Maintenance manpower requirement, MMH, MH/yr.

2.19.4.6.13 Electrical energy for operation, KWH, kwhr/yr.

2.19.4.6.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.19.4.6.15 Correction factor for minor construction costs, CF.

2.19.4.7 Unit Price Input Required.

2.19.4.7.1 Cost of earthwork, UPIEX, \$/cu yd.

2.19.4.7.2 Unit price input R.C. wall in-place, UPICW, \$/cu yd.

2.19.4.7.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.19.4.7.4 Unit price input for handrails in-place, UPIHR, \$/ft.

2.19.4.7.5 Cost per diffuser, COSTPD, \$, (optional).

2.19.4.7.6 Cost per swing arm header, COSTPH, \$, (optional).

2.19.4.7.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.19.4.7.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.19.4.7.9 Equipment installation labor rate, LABRI, \$/MH.

2.19.4.7.10 Unit price input for crane rental, UPICR, \$/hr.

2.19.4.8 Cost Calculations.

2.19.4.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.19.4.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.19.4.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of concrete slab, cu yd.

UPICS = unit price R.C. slab in-place, \$/cu yd.

2.19.4.8.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.19.4.8.5 Cost of diffusers.

2.19.4.8.5.1 The oxygen transfer values given indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$\text{COSTPD} = 6.50 \frac{\text{MSECI}}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.19.4.8.5.2 Calculate COSTD.

$$\text{COSTD} = \text{COSTPD} \times \text{ND}_t \times \text{NT} \times \text{NB}$$

where

COSTD = cost of diffusers for system, \$.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.19.4.8.6 Cost of swing arm diffuser headers.

2.19.4.8.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$\text{COSTPH} = \$5{,}000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$\text{COSTPH} = \$5{,}000 \frac{\text{MSECI}}{491.6}$$

where

COSTPH = cost per swing arm header, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.19.4.8.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, \$.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.19.4.8.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, MH.

2.19.4.8.8 C-ane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.19.4.8.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.19.4.8.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air p ing can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.19.4.8.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.19.4.8.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.19.4.8.10 Other costs associated with the installed equipment. This category includes the cost for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.19.4.8.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.19.4.8.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR + COSTAP)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.19.4.8.13 Operation and maintenance material costs.

$$OMCC = TBCC \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/ yr.

OMMP = operation and maintenance material supply costs, as percent of total bare construction cost, percent.

2.19.4.9 Cost Calculations Output Data.

2.19.4.9.1 Total bare construction cost of diffused aeration aerobic digestion system, TBCC, \$.

2.19.4.9.2 Operation and maintenance material and supply costs, OMMC, \$.

2.19.5 Aerobic Digestion (Mechanical Aeration).

2.19.5.1 Input Data.

2.19.5.1.1 Sludge production.

2.19.5.1.1.1 Primary, lb/day.

2.19.5.1.1.2 Secondary, lb/day.

2.19.5.1.2 Solids contents (percent solids).

2.19.5.1.2.1 Primary, percent.

2.19.5.1.2.2 Secondary, percent.

2.19.5.1.3 Specific gravity.

2.19.5.1.4 Volatile solids content, percent.

2.19.5.2 Design Parameters.

2.19.5.2.1 The design parameters are as shown in Table 2.19-2 below.

Table 2.19-2. Aerobic Digestion Design Parameters

Parameter	Value
Hydraulic detention time, days at 20°C[a]	
Activated sludge only	12 to 16
Activated sludge from plant operated without primary settling	16 to 18
Primary plus activated or trickling-filter sludge	18 to 22
Solids loading, lb volatile solids/ft^3/day	0.1 to 0.2
Oxygen requirements	
BOD_5 in primary sludge, lb/lb cell tissue	2
Energy requirements for mixing	
Mechanical aerators, hp/1000 ft^3	0.5 to 1.0
Air mixing, cfm/1000 ft^3	20 to 30
Dissolved oxygen level in liquid, mg/l	1 to 2

[a]Detention times should be increased for temperature below 20°C. If sludge cannot be withdrawn during certain periods (e.g. weekends, rainy weather), additional storage capacity should be provided. Ammonia produced during carbonaceous oxidation is oxidized to nitrate.

2.19.5.3 Process Design Calculations.

2.19.5.3.1 Calculate total quantity of raw sludge.

$$V_s = \frac{SP(100)}{(\text{specific gravity})(\text{percent solids})(8.34)}$$

where

V_s = volume of raw sludge to digester, gal/day.

SP = sludge produced, lb/day.

2.19.5.3.2 Select hydraulic detention time and calculate digesters' volume.

$$V = (t)(V_s)$$

where

V = volume of digester, gal.

t = hydraulic detention time, days.

V_s = volume of raw sludge to digester, gal/day.

2.19.5.3.3 Check volatile solids loading.

$$VS_1 = \frac{\text{lb/day VS}}{V} (7.48) \quad (0.1\text{-}0.2)$$

where

VS_1 = volatile solids loading, lb VS/ft^3/day.

V = v lume of digester, gal.

2.19.5.3.4. Calculate solids retention time.

2.19.5.3.4.1 Assume percent destruction of volatile solids: 40 percent is common but it increases with temperature and retention time from approximately 33 to 70 percent.

2.19.5.3.4.2 Calculate solids accumulation per day.

$$S_{ac} = SP - SP\left(\frac{\%\ \text{volatile}}{100}\right)\left(\frac{\%\ \text{destroyed}}{100}\right) (0.75)$$

where

S_{ac} = solids accumulated per day.

SP = sludge produced.

2.19.5.3.4.3 Assume MLSS in digester and calculate total capacity of sludge digester.

$$DC = (V)(MLSS)(8.34)(10^{-6})$$

where

DC = digester capacity, lb.

V = volume of digester, gal.

MLSS = mixed liquor SS in digester, mg/l.

2.19.5.3.4.4 Calculate solids retention time.

$$SRT = \frac{DC}{S_{ac}}$$

where

SRT = solids retention time, days.

DC = digester capacity, lb.

S_{ac} = total solids accumulated, lb/day.

2.19.5.3.5 Calculate sludge wasting schedule. Assume solids content in digested sludge is approximately 2.5 percent.

$$\text{Volume of sludge to be wasted} = \frac{\text{total sludge in digester (lb)(100)}}{\text{(specific gravity)(\% solids)(8.34)}}$$

This volume must be wasted each SRT.

2.19.5.3.6 Calculate oxygen requirements.

2.19.5.3.6.1 O_2 required for bacterial growth. Assume O_2 required per pound of volatile solids destroyed.

$$O_2 = (O_T)SP\left(\frac{\%\ \text{volatile}}{100}\right)\left(\frac{\%\ \text{destroyed}}{100}\right)$$

where

O_2 = total oxygen required, lb/day.

O_T = oxygen required/lb of volatile solids destroyed $\approx$ 2.0 lb.

SP = sludge produced, lb/day.

2.19.5.3.7 Design Aeration System.

2.19.5.3.7.1 Assume the following design parameters and design aeration system and check horsepower supply for mixing against horsepower required for complete mixing 0.1 hp/1000 gal.

2.19.5.3.7.1.1 Standard transfer efficiency, lb/hp-hr (0 dissolved oxygen, $20^{o}C$, and tap water)(3-5 lb/hp-hr).

2.19.5.3.7.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.19.5.3.7.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.19.5.3.7.1.4 Correction factor for pressure $\approx$ 1.0.

2.19.5.3.7.2 Select summer operating temperature ($25-30^{o}C$) and determine (from standard tables) O_2 saturation.

2.19.5.3.7.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{[(C_s)_T(\beta)(p) - C_L]}{9.17} \alpha (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature T, ^{o}C mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, ^{o}C.

2.19.5.3.7.4 Calculate horsepower requirement.

$$hp = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)\ (V)} \times 1000$$

where

hp = horsepower required/1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of basin, gal.

2.19.5.3.8 Supernatant Return.

2.19.5.3.8.1 Quantity.

2.19.5.3.8.1.1 Activated Sludge and Oxidation Ditch.

$$QSUP = (Q_{avg})(0.02)$$

where

QSUP = quantity of supernatent returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.19.5.3.8.1.2 Trickling Filter and Rotating Biological Contactor.

$$QSUP = (Q_{avg})(0.007)$$

where

QSUP = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.19.5.3.8.2 Supernatant Quality.

TSS = 3400
BOD = 500
COD = 2600
TKN = 170
PH = 7.0

where

TSS = total suspended solids concentration, mg/l.

BOD = BOD_5 concentration, mg/l.

COD = COD concentration, mg/l.

TKN = total Kjeldahl nitrogen concentration, mg/l.

PH = pH.

2.19.5.4 Process Design Output Data.

2.19.5.4.1 Raw sludge specific gravity.

2.19.5.4.2 Detention time, days.

2.19.5.4.3 Volatile solids destroyed, percent.

2.19.5.4.4 Mixed liquor solids, mg/l.

2.19.5.4.5 Solids in digested sludge, percent.

2.19.5.4.6 Rate constant, BOD_5 applied to filter.

2.19.5.4.7 Coefficient of O_2 saturation in waste/O_2 saturation in water.

2.19.5.4.8 Standard transfer efficiency, percent.

2.19.5.4.9 Digester volume, gal.

2.19.5.4.10 Volatile solids loading, lb VS/ft^3/day.

2.19.5.4.11 Solids accumulated, lb/day.

2.19.5.4.12 Volume of wasted sludge, gal.

2.19.5.4.13 Solids retention time, days.

2.19.5.4.14 Oxygen requirement, lb/day.

2.19.5.4.15 Horsepower required, hp.

2.19.5.5 Quantities Calculations.

2.19.5.5.1 Determine volume of digester.

$$V_d = \frac{V}{7.48}$$

where

V = volume of digester, gal.

V_d = volume of digester, cu ft.

2.19.5.5.2 Selection of number of aeration tanks and mechanical aerators per tank. The following rule will be utilized in the selection of number of aeration tanks and mechanical aerators per tank.

V_s (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NT
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When V_s is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.19.5.5.3 Selection of number of tanks and number of batteries of tanks when V_s is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.19.5.5.3.1 When $V_s \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.19.5.5.3.2 When $100 < V_s \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsection 2.19.5.5.2 by using half the design flow as V_s. Thus

$$NB = 2$$

2.19.5.5.3.3 When $V_s > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.19.5.5.4 Mechanical aeration equipment design.

2.19.5.5.4.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.19.5.5.4.2 Horsepower for each individual aerator:

$$HPN = \frac{HP}{(NB)(NT)(NA)}$$

If HPN > 150 HP and NT $= 2$ or 3, then repeat the calculation with NT $=$ NT $+ 1$.

If HPN > 150 HP and NT ≥ 4, then repeat the calculation with NT $=$ NT $+ 2$.

where

HPN = horsepower of each unit, horsepower.

HP = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.19.5.5.4.3 Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.19.5.5.5 Design of aeration tanks.

2.19.5.5.5.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.19.5.5.5.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would occur at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When HPSN < 100 HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

When HPSN ≥ 100 HP

$$DW = 15 \text{ ft}$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.19.5.5.5.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tank, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \frac{VN}{(DW)\ (NA)}$$

The length of the aeration tank would be

$$L = (NA)\ (W)$$

2.19.5.5.6 Aeration tank arrangements.

2.19.5.5.6.1 Figure 2.19-6 shows the schematic diagram of the arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.19.5.5.6.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\ \left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

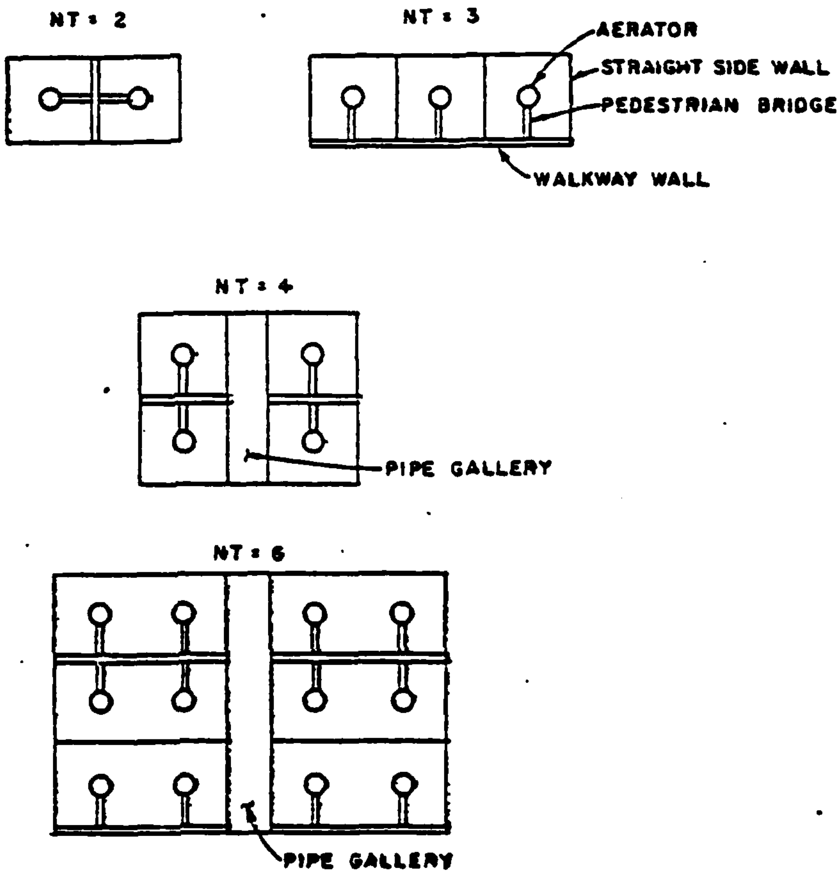
NB = number of batteries.

2.19.5.5.7 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.19.5.5.7.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\ [(2\ W + 18.5)\ (W + 17) + (2\ W + 26.5)\ (W + 25)]$$

where



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.19-6. AERATION TANK ARRANGEMENT
MECHANICAL AERATION

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.19.5.5.7.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\ [(3\ W + 28)\ (W + 25) + (3\ W + 20)\ (W + 17)]$$

2.19.5.5.7.3 When NT ≥ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\ L + PGW + 16$$

$$W_s = \tfrac{1}{2}\ (NT)\ (W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB)\ [(L_s + 4)\ (W_s + 4) + (L_s + 12)\ (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.19.5.5.8 Reinforced concrete slab quantity.

2.19.5.5.8.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.19.5.5.8.2 For NT = 2,

$$V_{cs} = 1.5\ (2\ W + 14.5)\ (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.19.5.5.8.3 NT = 3,

$$V_{cs} = 1.5\ (3\ W + 16)\ (W + 13)$$

2.19.5.5.8.4 When NT $\geq$ 4,

$$V_{cs} = 1.5\ (L_s)\ (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.19.5.5.9 Reinforced Concrete Wall Quantity.

2.19.5.5.9.1 In using the plug flow system, influent to the aeration basin will be piped to one end of the tank and discharged at the other end. Thus it does not require such an elaborate wall construction. Two typical wall sections are required, as shown in Figure 2.19-7. One would be simple straight side wall and the other would be enlarged on top so that walkways can be provided.

2.19.5.5.9.2 When NT = 2:

$$V_{cw} = W\ (1.25\ DW + 11) + (6\ W + 9)\ (1.25\ DW + 3.75)$$

2.19.5.5.9.3 When NT = 3:

$$V_{cw} = (1.25\ DW + 11)\ (3\ W + 6) + (1.25\ DW + 3.75)\ (7\ W + 6)$$

2.19.5.5.9.4 When NT $\geq$ 4:

$$V_{cw} = \frac{NT}{2}\ (L + 3)\ (1.25\ DW + 11) + [(0.5\ NT + 2)\ (L + 3) + 2\ (NT)\ (W)] \cdot (1.25\ DW + 3.75) \cdot (NB)$$

2.19.5.5.10 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.19.5.5.10.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.19.5.5.10.2 When NT $\geq$ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

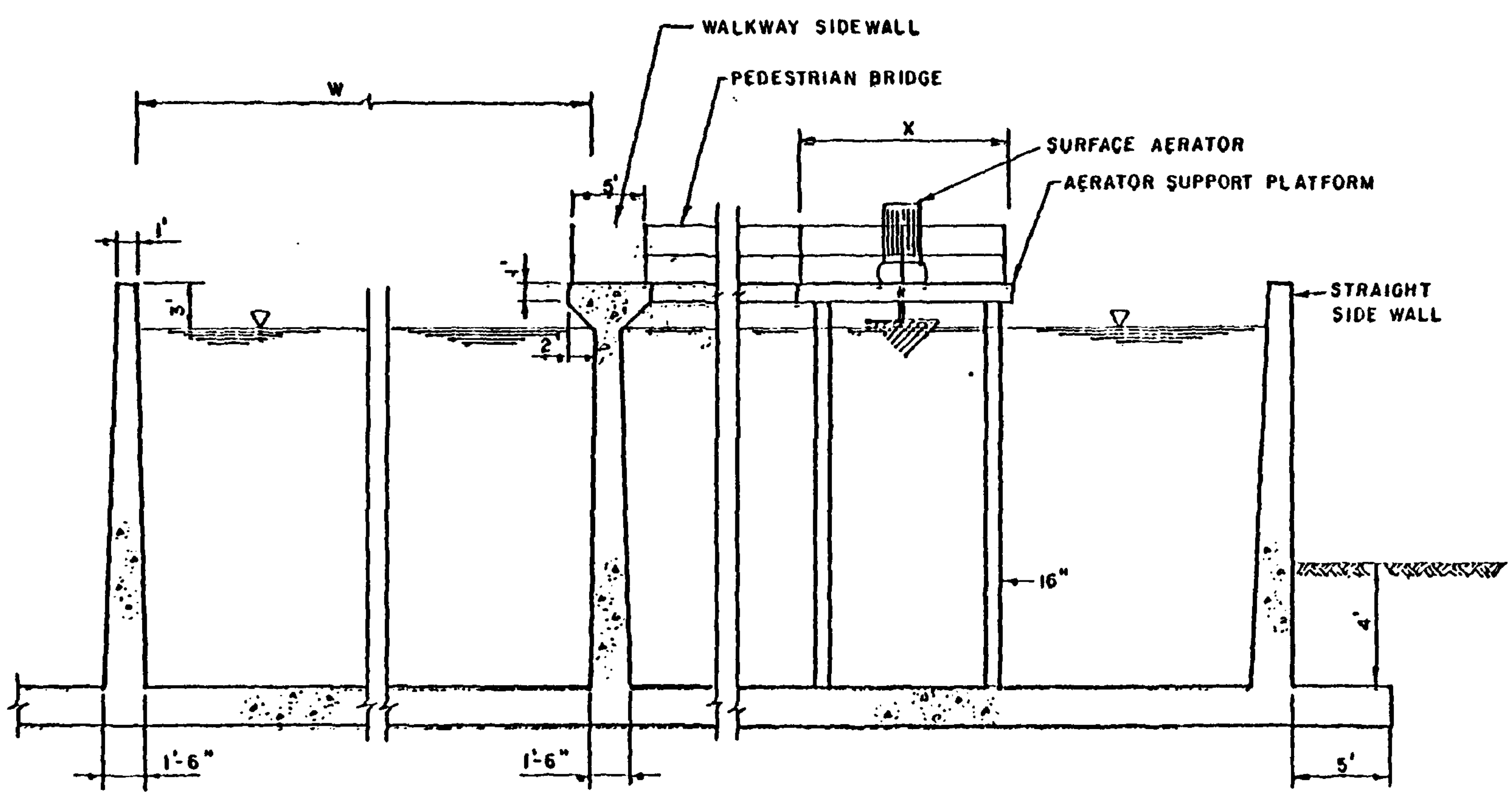
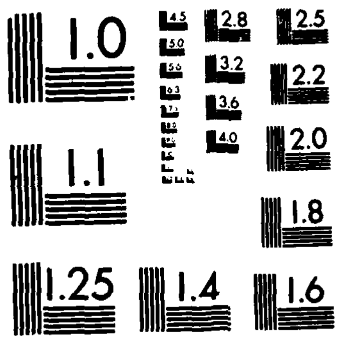


FIGURE 2.19-7. TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE 8/18
COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS
EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL.
UNCLASSIFIED JAN 82 F/G 5/3 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

$$V_{cgc} = (NB) \cdot (1.5) (PGW) [\frac{(NT) (W)}{2} + 0.75 (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2 (PGW) (NB) (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.19.5.5.11 Reinforced concrete quantity for aerator supporting platform construction.

2.19.5.5.11.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.19.5.5.11.2 Figure 2.19-8 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experienced formula is given to approximate this relationship.

$$X = 5 + 0.078 (HPSN)$$

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.19.5.5.11.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6 (DW + 2)] (NT) (NA) (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

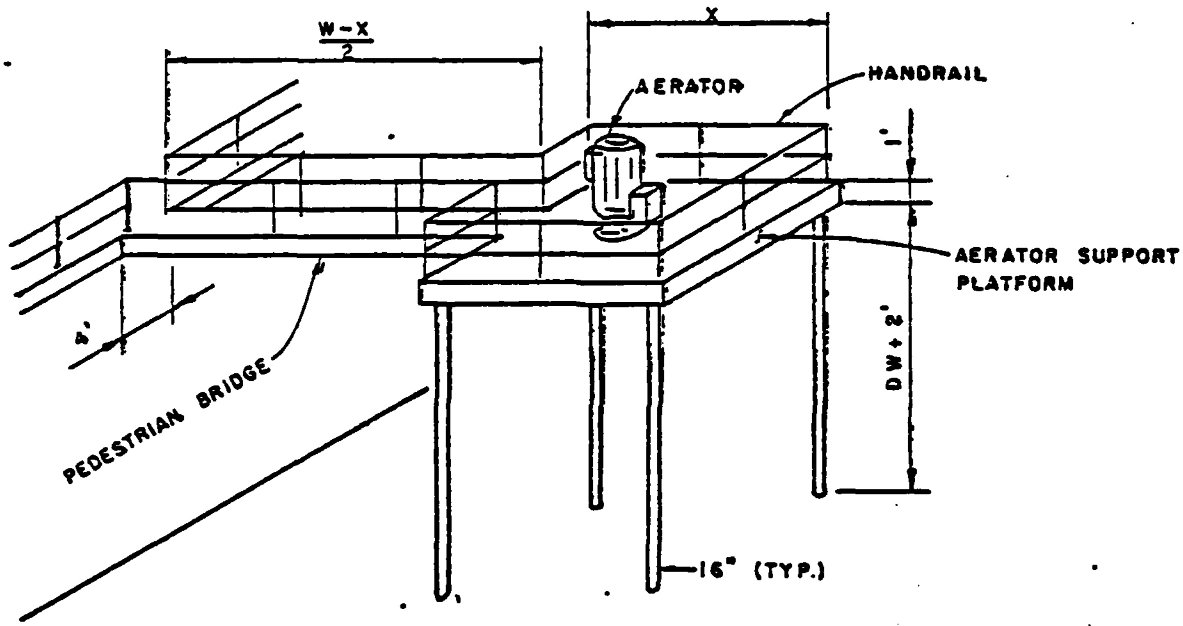


FIGURE 2.19-8. AERATOR SUPPORT PLATFORM

2.19.5.5.11.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2 (W - X)] (NB) (NT) (NA)$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.19.5.5.12 Summary of reinforced concrete structures.

2.19.5.5.12.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.19.5.5.12.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of aerator-supporting platforms, cu ft.

V_{cwb} = quantity of R.C. for the construction of pedestrian bridges.

2.19.5.5.13 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.19.5.5.13.1 When NT = 2,

$$LHR = 4\,W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.19.5.5.13.2 When NT = 3,

$$LHR = 6\,W + 10 + 3 \cdot (3X + W - 4)$$

2.19.5.5.13.3 When NT $\geq$ 4,

If $\frac{NT}{2}$ is an even number,

$$LHR = \left\{PGW + (NT)\,(W) + [L + 3 - 4\,(NA)]\,(NT) + (NA) \cdot (NT) \cdot (3X + W - 4)\right\} \cdot (NB)$$

If $\frac{NT}{2}$ is an odd number,

$$LHR = \left\{PGW + (NT)\,(W) + [L + 3 - 4\,(NA)]\,(NT + 2) + (NA)\,(NT)\,(3X + w - 4)\right\} \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.19.5.5.14 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.19.5.5.14.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)\,(NT)\,(NA)\,(HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.19.5.5.14.2 The operation manpower requirement can be estimated as follows:

$$\text{When TICA} < 200 \text{ hp}$$

$$OMH = 242.4\ (TICA)^{0.3731}$$

$$\text{When TICA} \geq 200 \text{ hp}$$

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.19.5.5.14.3 The maintenance manpower requirement can be estimated as follows:

$$\text{When TICA} \leq 100 \text{ hp}$$

$$MMH = 106.3\ (TICA)^{0.4031}$$

$$\text{When TICA} > 100 \text{ hp}$$

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.19.5.5.15 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.19.5.5.16 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint, and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.19.5.5.17 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate. Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.19.5.6 <u>Quantities Calculations Output Data.</u>

2.19.5.6.1 Number of aeration tanks, NT.

2.19.5.6.2 Number of aerators per tank, NA.

2.19.5.6.3 Number of process batteries, NB.

2.19.5.6.4 Capacity of each individual aerator, HPSN, hp.

2.19.5.6.5 Depth of aeration tanks, DW, ft.

2.19.5.6.6 Length of aeration tanks, L, ft.

2.19.5.6.7 Width of aeration tanks, W, ft.

2.19.5.6.8 Width of pipe gallery, PGW, ft.

2.19.5.6.9 Earthwork required for construction, V_{ew}, cu ft.

2.19.5.6.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.19.5.6.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.19.5.6.12 Quantity of handrail, LHR, ft.

2.19.5.6.13 Operation manpower requirement, OMH, MH/yr.

2.19.5.6.14 Maintenance manpower requirement, MMH, MH/yr.

2.19.5.6.15 Electrical energy for operation, KWH, kwhr/yr.

2.19.5.6.16 Percentage for O&M material and supply cost, OMMP, percent.

2.19.5.6.17 Correction factor for minor capital cost items, CF.

2.19.5.7 Unit Price Input Required.

2.19.5.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.19.5.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.19.5.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.19.5.7.4 Standard size low speed surface aerator cost (20 hp), SSXSA, $, optional.

2.19.5.7.5 Marshall & Swift Equipment Cost Index, MSECI.

2.19.5.7.6 Equipment installation labor rate, $/MH.

2.19.5.7.7 Crane rental rate, UPICR, $/hr.

2.19.5.7.8 Unit price of handrail, UPIHR, $/L.F.

2.19.5.8 Cost Calculations.

2.19.5.8.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.19.5.8.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in place, \$.

V_{cwt} = quantity of R.C. wall, cu yd.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.19.5.8.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete slab, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.19.5.8.4 Cost of installed aeration equipment.

2.19.5.8.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 hp.

RSXSA = ratio of cost of aerators with capacity of HPSN hp to that of the standard size aerator.

2.19.5.8.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, hp.

2.19.5.8.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16,300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16,300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, first quarter 1977.

2.19.5.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When $HPSN \leq 60$ hp

$$IMH = 39 + 0.55\ (HPSN)$$

When $HPSN > 60$ hp

$$IMH = 61.3 + 0.18\ (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.19.5.8.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requirement for installation, hr.

2.19.5.8.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchase equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.19.5.8.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA\ (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/man-hour.

UPICR = crane rental rate, \$/hr.

2.19.5.8.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, \$ per lineal foot. A value of \$25.20 per foot for the first quarter of 1977 is suggested.

2.19.5.8.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.19.5.8.7 Total bare construction costs, TBCC.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

2.19.5.8.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, \$/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.19.5.9 Cost Calculations Output Data.

2.19.5.9.1 Total bare construction cost of the mechanical aerated aerobic digestion process, TBCC, \$.

2.19.5.9.2 Operation and maintenance supply and material costs, OMMC, \$/yr.

2.19.6 Anaerobic Digestion.

2.19.6.1 Input Data.

2.19.6.1.1 Wastewater flow, average daily, Q_{avg}, mgd.

2.19.6.1.2 Sludge flow, gpd.

2.19.6.1.2.1 Primary.

2.19.6.1.2.2 Secondary.

2.19.6.1.3 Volatile solids in raw sludge, percent.

2.19.6.1.4 Solids content in raw sludge, percent.

2.19.6.1.5 Volatile solids destroyed during digestion, percent (usually 40-60 percent).

2.19.6.2 Design Parameters.

2.19.6.2.1 Digestion time, days.

2.19.6.2.2 Number of digesters.

2.19.6.2.3 Heat requirement, BTU/hr.

2.19.6.2.4 Sludge gas generation, cu ft/day.

2.19.6.2.5 Primary digester, volume, cu ft.

2.19.6.2.6 Secondary digester, volume, cu ft.

2.19.6.3 Process Design Calculations.

2.19.6.3.1 Calculate sludge production.

$$SP = \frac{(P_s)\ (SF)\ (Sp.gr.)\ (8.34)}{100}$$

where

SP = sludge produced, lb/day.

P_s = solids content in raw sludge, %.

SF = sludge flow, gpd.

$Sp.gr.$ = specific gravity of sludge.

2.19.6.3.2 Calculate total volume of raw sludge to digester.

$$V_f = \frac{SP(100)}{(\%\ solids)(62.4)(specific\ gravity)}$$

where

V_f = volume of raw sludge to digester, ft^3/day.

SP = sludge produced, lb/day.

2.19.6.3.3 Number and type of digesters to be used.

2.19.6.3.3.1 For small treatment works, usually only one digester is provided. In this case, the conventional digester will be utilized due to the fact that digestion, storage, and clarification are to be carried out in one tank. When more than two tanks are provided, the digestion and clarification processes will be carried out separately in the primary and secondary digesters. Thus the following assumption will be used in selecting the number of digesters. (The number of secondary digesters is determined by the storage time required for the digested sludge. Usually, 30 days storage time is provided. But since there is no general rule governing the length of the storage period, the following assumption is based on general field practice).

Q_{avg}, mgd	Total Number Digesters, NT	Number of Primary, NP	Number of Secondary, NS
0.5 - 2	1	-	-
2 - 10	2	1	1
10 - 20	3	2	1
20 - 30	4	3	1
30 - 40	6	4	2
40 - 50	8	5	3
50 - 60	9	6	3
60 - 80	12	8	4
80 - 100	16	10	6

When Q_{avg} is larger than 100 mgd, the system will be designed as several identical batteries of units.

When 100 Q_{avg}, only one battery of units will be utilized, thus, NB = 1.

2.19.6.3.3.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of digesters. Each can handle half the sludge flow. The number of digesters in each battery would be selected from the above table by using half the design flow. For instance, if the incoming wastewater flow is 120 mgd, the system will have two batteries of digesters. Each battery contains 9 digesters (using Q_{avg} = 60 mgd and from the above table). Total number of digesters would be 9 x 2 = 18. The design procedures would be based on one battery with Q_{avg} = 60 mgd and the total material and costs would be twice as much as those for one battery, NB = 2.

2.19.6.3.3.3 When $Q_{avg} > 200$ mgd, the design will be performed as three identical batteries of digesters. The Q_{avg} would be equal to $Q_{avg}/3$. Only one battery will be designed. The total material and costs would be three times as high, NB = 3.

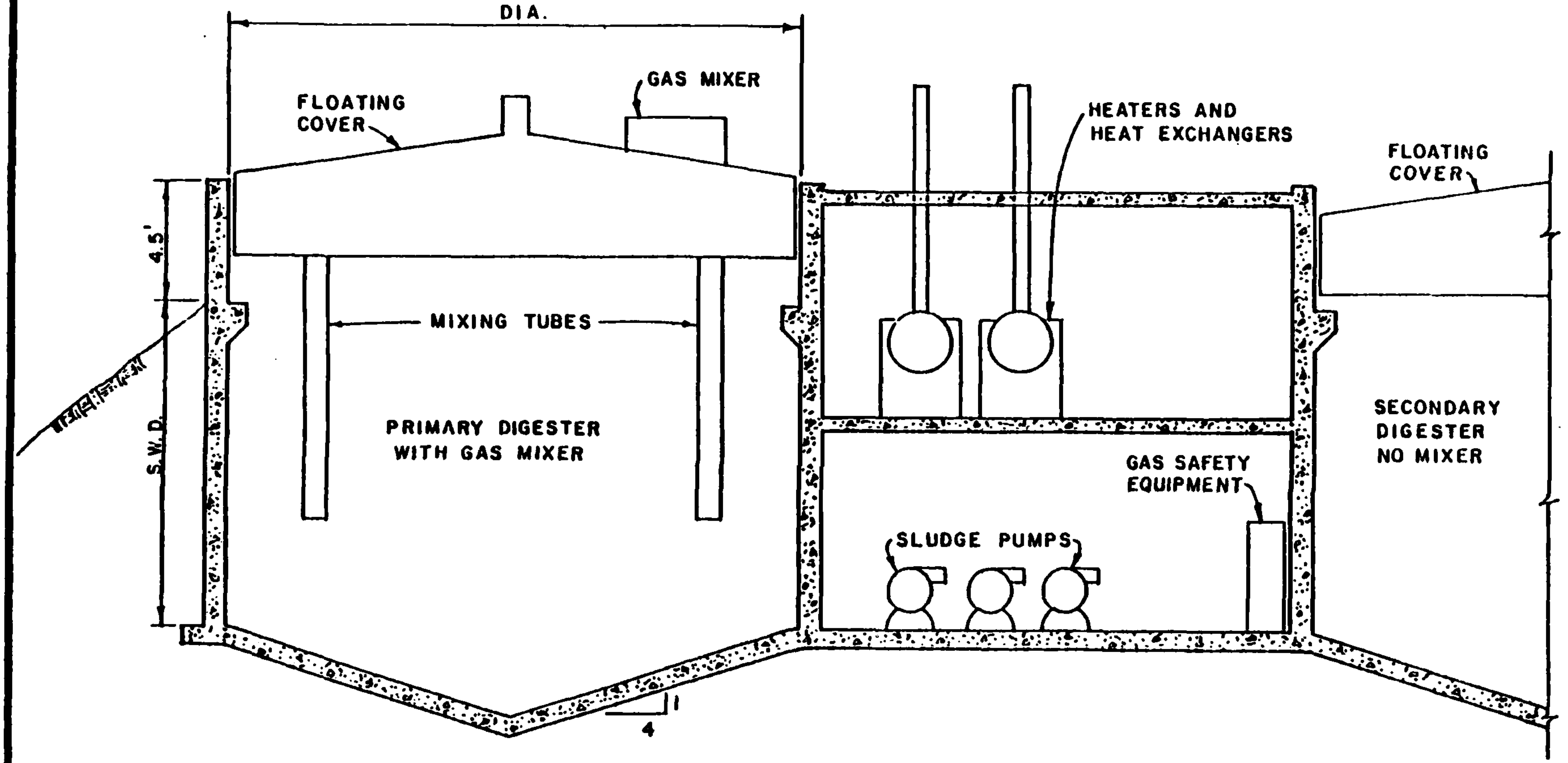


FIGURE 2.19-9. ANAEROBIC DIGESTER SYSTEM LAYOUT AND CONFIGURATION

2.19.6.3.4 Sizing individual units. Digestion time required for volatile solids reduction can be predicted by the following equation:

2.19.6.3.4.1 Conventional digesters: heated but no mixing.

2.19.6.3.4.1.1 Calculate digestion time.

$$t_d = (PVR - 30) \times 2$$

where

t_d = time of digestion, days.

PVR = percent volatile solid reduction, percent (usually use 50 percent).

2.19.6.3.4.1.2 Calculate quantity of digested solids.

The quantity of solids remaining in the digester at the end of digestion period can be calculated by:

$$SD = SP\ [1 - \frac{(PV)\ (PVR)}{(100)\ (100)}]$$

where

SD = quantity of digested solids, lb/day.

PV = percent volatile of fresh sludge, percent.

2.19.6.3.4.1.3 Calculate volume of digested solids.

By assuming that the digested sludge will be 7 percent solids, the volume of digested sludge would be expressed:

$$V_d = \frac{SD}{(1.04)\ (62.4)\ (0.07)}$$

where

V_d = digested sludge volume, cu ft/day.

2.19.6.3.4.1.4 Calculate digester volume.

By using the volume reduction method for the design of the standard rate digester, the volume of the digester may be calculated by:

$$V_T = [V_f - \frac{2}{3} (V_f - V_d)]\ t_d$$

where

V_T = volume of standard rate digester, cu ft.

Compare V_T with the available off-the-shelf sizes and select the appropriate size.

2.19.6.3.4.2 High rate digesters: Primary digesters are heated and completly mixed.

2.19.6.3.4.2.1 Calculate digestion time required, t_d.

$$t_d = e^{\left(\frac{PVR - 18.94}{13.7}\right)}$$

2.19.6.3.4.2.2 Calculate total primary digester volume:

$$V_p = V_f \cdot t_d$$

where

V_p = total primary digester volume requirement, cu ft.

2.19.6.3.4.2.3 Calculate individual digester volume.

$$V_{PU} = \frac{V_p}{(NP)\ (NB)}$$

where

V_{PU} = volume of each individual tank, cu ft.

Compare V_{PU} with the available off-the-shelf sizes and select the proper size.

2.19.6.3.4.3 Digester size and configuration. The diameter of digester, DIA, is determined by the available sizes of floating cover and side water depth of the digester. The side water depth is a function of diameter and can be expressed by:

$$SWD = 15.6 + (0.1765)\ (DIA)$$

where

SWD = side water depth of digester, ft.

DIA = diameter of digester, ft.

Figure 2.19-9 shows typical tank configurations of an anaerobic digester.

Volumes of available digester diameters are listed below:

Diameter ft	Side Water Depth ft	Volume cu ft
25	20.0	10,330
30	21.0	15,727
35	21.5	22,087
40	22.5	30,367
45	23.5	40,355
50	24.5	52,193
55	25.0	64,836
60	26.0	80,577
65	27.0	98,575
70	28.0	118,980
75	28.5	139,704
80	29.5	148,493
85	30.0	190,317
90	31.5	224,233
95	32.0	254,810
100	33.0	291,882
110	35.0	376,141

2.19.6.3.4.4 Total volume of digester.

2.19.6.3.4.4.1 For conventional digester:

$$V_{TOT} = V_T$$

2.19.6.3.4.4.2 For high-rate digester:

$$V_{TOT} = V_T$$

2.19.6.3.4.4.3 For high-rate digester:

$$V_{TOT} = V_{PU} \times NT \times NB$$

where

V_{TOT} = total digester volume, cu ft.

2.19.6.3.5 Calculate heat requirements.

2.19.6.3.5.1 Heat supplied must be sufficient to raise the temperature of the incoming sludge to 95°F.

$$Q_S = \left(\frac{SP}{P_s}\right) (C) \ (T_d - T_s) \left(\frac{1}{24}\right) (100)$$

where

Q_S = BTU/hr required for sludge heating.

SP = sludge added per day, lb.

P_S = solids in fresh sludge, percent.

C = 1.0 BTU/lb.

T_d = temperature of sludge in digester, 95°F.

T_s = temperature of incoming sludge, °F.

2.19.6.3.5.2 Heat losses through digester wall, floor and cover, etc. Use empirical formulae to estimate heat loss:

In Northern U.S. and Canada:

$$Q_L = (1.25) \ (46.3) \ (DIA)^{2.24}$$

In Middle U.S.:

$$Q_L = (1.25) \ (31.8) \ (DIA)^{2.24}$$

In Southern U.S.:

$$Q_L = (1.25) \ (25.9) \ (DIA)^{2.24}$$

where

Q_L = heat loss, BTU/hr.

1.25 = with safety factor of 25%.

DIA = diameter of digester.

2.19.6.3.5.3 Total heat requirement, Q_T, BTU/hr.

$$Q_T = Q_S + (NP)\ (Q_L)\ (NB)$$

2.19.6.3.6 Calculate daily gas production. Assuming 15 ft^3 of sludge gas is produced per pound of volatile solids destroyed, the gas production rate would be:

$$DGP = \frac{(SP)\ (\%\ volatile)\ (PVR)\ (10^{-4})\ x\ 15}{24}$$

where

DGP = daily gas produced, cu ft/hr.

SP = sludge produced, lb/day.

2.19.6.3.7 Wasted sludge characteristics.

2.19.6.3.7.1 Sludge flow out.

$$SFO = \frac{(SD)(100)}{PSD\ (8.34)\ (Sp.gr.)}$$

where

SFO = sludge flow out, gpd.

SD = quantity of sludge digested, lb/day.

PSD = percent solids in digested sludge, (input by user), %.

SP.gr. = specific gravity of sludge.

2.19.6.3.7.2 Volatile solids in wasted sludge.

$$PVDS = \frac{(PV)\ (SP)\ (1 - \frac{PVR}{100})\ (100)}{(SFO)\ (8.34)\ (Sp.gr.)\ (PSD)}$$

where

PVDS = percent volatile solids in digested sludge, %.

PV = percent volatile solids in influent sludge, %.

SP = input sludge quantity, lb/day.

PVR = percent volatile solid reduction, %.

SFO = sludge flow out, gpd.

Sp.gr. = specific gravity of sludge.

PSD = percent solids in digested sludge, (input by user), %.

2.19.6.3.8 Supernatant Return.

2.19.6.3.8.1 Supernatant Quantity.

2.19.6.3.8.1.1 Activated Sludge Processes and Oxidation Ditch.

$$QSUP = (Q_{avg})(0.02)$$

where

QSUP = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.19.6.3.8.1.2 Trickling Filter and Rotating Biological Contactor.

$$QSUP = (Q_{avg})(0.007)$$

where

QSUP = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.19.6.3.8.2 Supernatant Quality.

TSS = 6250
BOD = 1000
COD = 2150
TKN = 950
NHS = 650
PH = 7.4

where

TSS = total suspended solids concentration, mg/l.

BOD = BOD_5 concentration, mg/l.

COD = COD concentration, mg/l.

TKN = total Kjeldahl nitrogen concentration, mg/l.

NH3 = ammonia nitrogen concentration, mg/l.

PH = pH.

2.19.6.4 Process Design Output Data.

2.19.6.4.1 Volatile solids destroyed, PVR, percent.

2.19.6.4.2 Digester volume, V_{TOT}, cu ft.

2.19.6.4.3 Number of digesters per battery, NT.

2.19.6.4.4 Number of batteries, NB.

2.19.6.4.5 Number of primary digesters per battery, NP.

2.19.6.4.6 Number of secondary digesters per battery, NS.

2.19.6.4.7 Gas production, DGP, cu ft/day.

2.19.6.4.8 Heat requirement, Q_T, BTU/hr.

2.19.6.5 Quantities Calculations.

2.19.6.5.1 Quantity of earthwork. Assume the earthwork required would be the same as the digester volume. Thus, for one tank with diameter of DIA feet, the earthwork required would be, V_{ewn}.

$$V_{ewn} = (0.1713)\ (DIA)^3 + (12.174)\ (DIA)^2$$

where

V_{ewn} = earthwork required per tank, cu ft.

For NT number of tanks, the earthwork would be:

$$V_{ew} = (NT)\ (V_{ewn})\ (NB)$$

where

V_{ew} = volume of earthwork, cu ft.

2.19.6.5.2 Quantity of reinforced concrete wall, V_{cw}.

For one tank:

$$V_{cwn} = (0.275)\ (SWD + 4.5)\ (DIA)\ (t_w)$$

where

V_{cwn} = quantity of R.C. Wall, cu ft.

t_w = wall thickness, in.

and

$$t_w = 7.5 + (0.5)\ (SWD)$$

Thus the total wall quantity:

$$V_{cw} = (NT)\ (V_{cwn})\ (NB)$$

where

V_{cw} = total R.C. in place for wall, cu ft.

2.19.6.5.3 Quantity of reinforced concrete slab.

For one tank:

$$V_{csn} = (0.0675)\ (DIA)^2\ (t_s) + (3.1416)\ (DIA)$$

where

V_{csn} = quantity of R.C. slab, cu ft.

t_s = slab thickness, in.

and

$$t_s = 8 + 0.24\ (SWD - 15.5)$$

Thus the total slab quantity would be:

$$V_{cs} = (NT)\ (V_{csn})\ (NB)$$

where

V_{cs} = total R.C. slab in place, cu ft.

2.19.6.5.4 Dimension of control buildings. The control buildings are of two-story construction. Digester walls are utilized as common wall. The quantity, which is the floor space of a single floor, can be expressed as:

2.19.6.5.4.1 For NT < 6

$$A_{dcb} = [0.1 + 0.097\ (NT)]\ (DIA)^2 \cdot (NB)$$

2.19.6.5.4.2 For NT ≥ 6

$$A_{dcb} = [0.13\ (NT) - 0.08]\ (DIA)^2 \cdot (NB)$$

where

A_{dcb} = surface area of single floor of two-story control building, sq ft.

NT = total number of digesters per battery.

NB = number of process batteries.

2.19.6.5.5 Heater and heat exchanger selection.

2.9.6.5.5.1 Number of sludge heating systems, NH. The following assumption will be utilized in design of the heating system. This is based on field experience.

Number of Primary Digesters, NP	Number of Heating Systems Per Battery, NH
1	1
2	1
3	1
4	2
5	2
6	3
8	4
10	5

2.19.6.5.5.2 Thus each heating system would handle:

$$Q_H = \frac{Q_T}{(NH)(NB)}$$

where

Q_H = capacity of a single heating system, BTU/hr.

2.19.6.5.5.3 Select number and capacity of heater and heat exchanger. Available off-the-shelf sizes (sludge heating capacity in BTU/hr) are listed below:

140,000	1,125,000
175,000	1,250,000
250,000	1,500,000
375,000	2,000,000
500,000	2,800,000
750,000	3,500,000
1,000,000	

2.19.6.5.5.3.1 Calculate heating capacity of single unit.

If $Q_H \geq 7,000,000$ BTU/hr, number of heaters, NHE = 3.

If $3,500,000 \leq Q_H < 7,000,000$, number of heaters, NHE = 2.

If $Q_H < 3,500,000$, number of heaters, NHE = 1.

Thus capacity of one heating unit, Q_{HU}, would be

$$Q_{HU} = \frac{Q_H}{NHE}$$

where

Q_{HU} = capacity of one heating unit, BTU/hr.

NHE = number of heating units per heating system.

Compare Q_{HU} with the available off-the-shelf sizes and select a proper unit with capacity of Q_{HUS}.

2.19.6.5.5.4 Calculate additional units for backup. The number of additional units for emergency backup is dependent upon the type and number of digesters. The following rule-of-thumb is to be used:

Number of Digesters, NT	Number of Backup Heating Units, NHEB
1	0
2 - 4	1
6 - 9	2
9 - 16	3

2.19.6.5.5.5 Calculate total number of heating units and their capacity.

Total number of heating units, N_{HT}.

$$N_{HT} = [(NH)(NHE) + NHEB](NB)$$

where

N_{HT} = total number of heating units.

2.19.6.5.5.6 Calculate excess fuel to be used for heating. For daily sludge gas requirement (DGR) for heating, assume heat value of gas to be 600 BTU/ft^3 for digester gas, 1000 BTU/ft^3 for natural gas.

$$DGR = \frac{Q_t}{(0.80)(0.9)(600\ BTU/ft^3)}$$

where

DGR = daily gas requirement, cu ft/hr.

0.80 = boiler efficiency.

0.90 = heat exchanger efficiency.

Natural gas to be added, EGR:

$$EGR = (DGR - DGP)\left(\frac{600}{1000}\right)$$

If EGR > 0, EGR = EGR

If EGR ≤ 0, EGR = 0

where

EGR = natural gas requirement in cu ft/hr.

2.19.6.5.5.7 Calculate total natural gas requirement. The total natural gas required per year depends on the geographical location of the digester system and the temperature variations throughout the year. The following assumptions will be used in design calculations.

Total gas requirement per year, TNG:

2.19.6.5.5.7.1 In Northern U.S. - four months of a year require supplementary fuel:

$$TNG = 4 \times 30 \times 24 \times EGR$$

2.19.6.5.5.7.2 In Middle U.S. - two and one-half months of a year:

$$TNG = 2.5 \times 30 \times 24 \times EGR$$

2.19.5.5.5.7.3 In Southern U.S. - one month of a year:

$$TNG = 1.0 \times 30 \times 24 \times EGR$$

where

TNG = total annual natural gas requirement, cu ft/yr.

2.19.6.5.6 Gas safety equipment. This category includes accumulator pressure relief valve, flame trap, gas meter, and waste gas burner. This system is usually sized by the diameter of gas piping and can be related to the diameter of the digester.

Diameter ft	Gas Piping Size Ø", GPS
≤ 30	2"
30 - 50	3"
50 - 75	4"
75 - 100	6"

The number of gas safety equipment sets, NGSE, is dependent upon the number of digesters. It is assumed that one set of safety equipment will be utilized by two digesters.

Total Number of Digesters, NT	NGSE
1	1
>1	$\frac{NT}{2}$ or the next integer

2.19.6.5.7 Sludge recirculation pumps. Pumps are needed for sludge recirculation from the digester to the heating system, sludge wastage, and sludge transfer.

2.19.6.5.7.1 Type of pumps. Usually the positive displacement types are used for this purpose.

2.19.6.5.7.2 Number of pumps, NSTP, depends on the number of digesters. Backup units are included.

Number of Tanks, NT	NSTP
1	2
>1	$\frac{NT}{2} \times 3$ (NSTP must be an integer).

2.19.6.5.7.3 Size of pump depends on the sizes of the digester. The following rules will be used in the sizing.

Diameter of Digester DIA (ft)	Pump Capacity DPUMP (gpm)
25 - 30	75
30 - 70	150
>70	350

2.19.6.5.7.4 Pumping head. Assume 70 feet.

2.19.6.5.8 Piping System. The piping system within anaerobic digesters can be grouped into the following subcategories: raw sludge piping, sludge recirculation piping, supernatant piping, digested sludge piping, plant water supply piping, gas piping, etc. Only the sludge piping systems will be designed and quantities estimated. Other minor piping will be estimated as a percent of the total pipe system.

2.19.6.5.8.1 Size of piping system. Usually the sludge piping system in the anaerobic digesters utilizes cast iron pipes with sizes ranging from 4 Ø" to 12 Ø'. Size of pipe can be approximated by the following figures:

Diameter of Digester DIA (ft)	Pipe Size PIPES (in)
< 30	4
30 - 50	6
50 - 70	8
70 - 90	10
>90	12

2.19.6.5.8.2 Length of pipe.

$$PIPEL = NT \ [SWD + (2.17)\ (DIA) + 36] + \frac{NT}{2}$$
$$[2.25\ (DIA) + 70] \quad (NB)$$

where

PIPEL = length of total piping system, ft.

2.19.6.5.8.3 Quantity of fittings and valves.

Number of 90° elbows, PIRN.

$$PIRN = [(6)\ (NT) + 14\ (\frac{NT}{2})\ ]\ (NB)$$

PIRN must be an integer.

where

PIRN = number of 90° elbows.

Number of tees, PITN.

$$PITN = [16\ (NT) + 19\ (\frac{NT}{2})]\ (NB)$$

PITN must be an integer.

where

PITN = number of tees.

Number of plug valves, PIVN.

$$PIVN = [7\ (NT) + 23\ (\frac{NT}{2})\ ]\ (NB)$$

where

PIVN = number of plug valves.

2.19.6.5.8.4 Other piping. This includes gas piping, water supply piping and other minor pipes. The cost of this category can be estimated as a percentage of the major piping. The percentage is:

$$PENPIP = 25\%$$

where

PENPIP = percentage of the minor piping costs compared with major piping costs, percent.

2.19.6.5.9 Electrical energy required for operation. Electrical energy is required for sludge pumping, gas circulation for mixing purposes, and for certain electrical motors within the heating system. This item can be estimated by using the following equations:

2.19.6.5.9.1 First, calculate total sludge dry solids to be treated in the digester.

$$TDS = \frac{SP}{2000}$$

where

TDS = total dry solids to be treated per day, ton/day.

SP = sludge produced, lbs/day.

2.19.6.5.9.2 For TDS $\leq$ 8.5 tons/day:

$$KWH = (46720)\ (TDS)^{0.596}$$

2.19.6.5.9.3 For TDS > 8.5 tons/day:

$$KWH = (30691)\ (TDS)^{0.80}$$

where

KWH = electrical energy consumed per year, kwhr/yr.

2.19.6.5.10 Operation and maintenance manpower requirement.

2.19.6.5.10.1 Operation man-hour requirement, OMH.

2.19.6.5.10.1.1 For TDS $\leq$ 0.1 tons/day.

OMH = 608 man-hours/yr.

2.19.6.5.10.1.2 For 0.1 < TDS $\leq$ 1.0 tons/day.

$$OMH = 720\ (TDS)^{0.0734}$$

2.19.6.5.10.1.3 For 1.0 < TDS $\leq$ 10.0 tons/day.

$$OMH = 720\ (TDS)^{0.4437}$$

2.19.6.5.10.1.4 For TDS > 10.0 tons/day.

$$OMH = 280\ (TDS)^{0.8405}$$

where

OMH = operation man-hour requirement, man-hour/yr.

2.19.6.5.10.2 Maintenance man-hour requirement, MMH.

2.19.6.5.10.2.1 For $TDS \leq 0.1$ tons/day.

$$MMH = 352$$

2.19.6.5.10.2.2 For $0.1 < TDS \leq 1.0$ tons/day.

$$MMH = 448\ (TDS)^{0.105}$$

2.19.6.5.10.2.3 For $1.0 < TDS \leq 10.0$ tons/day.

$$MMH = 448\ (TDS)^{0.47}$$

2.19.6.5.10.2.4 For $TDS > 10.0$ tons/day.

$$MMH = 200\ (TDS)^{0.804}$$

where

MMH = maintenance man-hour requirement, man-hour/yr.

2.19.6.5.11 Other operation and maintenance material costs. This item includes repair and replacement material costs. It is expressed as a percentage of total installed equipment costs of the anaerobic digester system.

$$OMMP = 1.5\%$$

where

OMMP = O&M material costs as percent of the installed anaerobic digester equipment costs.

2.19.6.5.12 Other construction cost items.

2.19.6.5.12.1 From the above estimation, approximately 90 percent of the construction costs have been accounted for.

2.19.6.5.12.2 CF correction factor would be $\frac{1}{0.9} = 1.11$.

2.19.6.6 <u>Quantities Calculations Output Data.</u>

2.19.6.6.1 Volume of earthwork, V_{ew}, cu ft.

2.19.6.6.2 Volume of R.C. wall, V_{cw}, cu ft.

2.19.6.6.3 Volume of R.C. slab, V_{cs}, cu ft.

2.19.6.6.4 Surface area of single floor of two-story control building, A_{dcb}, sq ft.

2.19.6.6.5 Heater and heat exchanger number, N_{TH}.

2.19.6.6.6 Heater and heat exchanger capacity, Q_{HUS}, BTU/hr.

2.19.6.6.7 Excess fuel for heating, TNG, cu ft of natural gas/yr.

2.19.6.6.8 Gas safety equipment, number/battery, NGSE.

2.19.6.6.9 Gas safety equipment size, GPS.

2.19.6.6.10 Number of sludge pumps/battery, NSTP.

2.19.6.6.11 Sludge pump capacity, DPUMP, gpm.

2.19.6.6.12 Piping size, PIPES, in.

2.19.6.6.13 Length of pipes, PIPEL, ft.

2.19.6.6.14 Number of 90° elbows, PIRN.

2.19.6.6.15 Number of tees, PITN.

2.19.6.6.16 Number of plug valves, PIVN.

2.19.6.6.17 Other minor piping costs, PENPIP, percentage.

2.19.6.6.18 Electrical energy requirement, KWH, kwhr/yr.

2.19.6.6.19 Operational manpower requirement, OMH, man-hour/yr.

2.19.6.6.20 Maintenance manpower requirement, MMH, man-hour/ yr.

2.19.6.6.21 Other O&M material costs as percent of installed equipment cost, OMMP, %.

2.19.6.6.22 Correction factor for minor construction costs, CF.

2.19.6.6.23 Diameter of digester, ft.

2.19.6.7 Unit Price Input Required.

2.19.6.7.1 Cost of earthwork, UPIEX, $/cu yd.

2.19.6.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.19.6.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.19.6.7.4 Equipment installation labor rate, LABRI, $/MH.

2.19.6.7.5 Crane rental rate, UPICR, $/hr.

2.19.6.7.6 Piping installation labor rate, LABRI, $/MH.

2.19.6.7.7 Seventy foot diameter floating cover cost, SFLOCO, $, (optional).

2.19.6.7.8 Cost of hearth gas circulation equipment for 60-foot diameter digester, CGCUS, $ (optional).

2.19.6.7.9 Cost of external heater and heat exchanger unit with 1,000,000 BTU/hr capacity, HRHXS, $ (optional).

2.19.6.7.10 Cost of digester gas safety equipment with size of 2" Ø CGSES, $ (optional).

2.19.6.7.11 Cost of positive displacement pump with capacity of 80 gpm and 70-foot TDH, CSPUMP, $ (optional).

2.19.6.7.12 Purchase cost of 8" Ø cast iron pipe, CCIPS, $/L.F.

2.19.6.7.13 Purchase cost of 8" Ø plug valve, CCPUS, $/unit.

2.19.6.7.14 Purchase cost of 8" Ø 90° bend, CCRS, $/unit.

2.19.6.7.15 Purchase cost of 8" Ø tee, COTS, $/unit.

2.19.6.7.16 Marshall and Swift Equipment Cost Index, MSECI.

2.19.6.8 Cost Calculations.

2.19.6.8.1 Cost of earthwork.

$$COSTE = \frac{(V_{ew})\ (UPIEX)}{27}$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.19.6.8.2 Cost of reinforced concrete wall in-place.

$$COSTCW = \frac{(V_{cw})\ (UPICW)}{27}$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall, cu ft.

UPICW = unit price input R.C. wall in-place, \$/cu yd.

2.19.6.8.3 Cost of concrete slab in-place.

$$COSTCS = \frac{(V_{cs})\ (UPICS)}{27}$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab, cu ft.

UPICS = unit price input R.C. slab in-place, \$/cu yd.

2.19.6.8.4 Cost of Control Building.

2.19.6.8.4.1 Due to the fact that digester control building utilizes digester wall as common wall, the cost per unit sq ft would be lowered to approximately half the standard building cost.

2.19.6.8.4.2 Cost of control building.

$$COSTCB = (A_{dcb})\ (2)\ (0.5)\ (UPIBC)$$

where

COSTCB = cost of control building, \$.

A_{dcb} = surface area of single floor of two-story control building, sq ft.

UPIBC = unit price input for building cost, \$/sq ft.

2.19.6.8.5 Floating cover cost.

2.19.6.8.5.1 It is assumed that only primary digesters will require gas recirculation devices and both primary and secondary digesters will utilize floating covers.

2.19.6.8.5.2 Purchase cost of floating cover.

$$CFLOCO = (SFLOCO)\ (FLOCOR)$$

where

CFLOCO = purchase cost of floating cover, \$.

SFLOCO = cost of standard size floating cover (70 ft diameter), \$71,000 (first quarter, 1977).

FLOCOR = cost ratio between cover with diameter of DIA and the standard size (70 ft diameter).

For a better estimate, SFLOCO should be obtained from equipment vendors and treated as a unit price input. Otherwise, the price should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SFLOCO = 71{,}000 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter, 1977.

2.19.6.8.5.3 Determine FLOCOR.

2.19.6.8.5.3.1 When $30 \leq DIA < 70$

$$FLOCOR = (0.14)\ [10^{(0.0122)\ (DIA)}]$$

2.19.6.8.5.3.2 When $DIA \geq 70$

$$FLOCOR = (0.209)\ [10^{(0.00973)\ (DIA)}]$$

where

DIA = diameter of digester, ft.

FLOCOR = cost ratio between cover with diameter DIA and standard size (70 ft diameter).

2.19.6.8.5.4 Installation man-hour requirement.

$$IMH = 0.374\ (DIA)^{1.966}$$

where

IMH = installation man-hour requirement, MH.

DIA = diameter of digester, ft.

2.19.6.8.5.5 Installation crane hour.

$$ICH = 0.037\ (DIA)^{1.966}$$

where

ICH = installation crane requirement, hr.

DIA = diameter of digester, ft.

2.19.6.8.5.6 Other installation costs such as painting, electric work, etc., can be figured as a percentage of the purchase equipment cost.

2.19.6.8.5.6.1 When DIA < 70.

$$PMINC = 23\%$$

2.19.6.8.5.6.2 When DIA $\geq$ 70.

$$PMINC = 29\%$$

where

PMINC = minor cost as percentage of purchase cost of equipment, percent.

2.19.6.8.5.7 Installed floating cover cost.

$$IFLOCO = (CFLOCO)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (ICH)\ (UPICR)$$

where

LABRI = unit price of labor rate, classification I, \$/man-hour.

UPICR = crane rental rate, \$/hr.

IFLOCO = installed floating cover cost, \$.

PMINC = minor costs as percentage of purchase cost of equipment, percent.

IMH = installation man-hour requirement, MH.

ICH = installation crane requirement, hr.

2.19.6.8.5.8 Total floating cover cost.

$$IFLOCT = (NT)\ (IFLOCO)\ (NB)$$

where

IFLOCT = total cost of floating cover, \$.

NT = number of digesters per battery.

NB = number of batteries.

IFLOCO = installed floating cover cost, \$.

2.19.6.8.6 Gas circulation equipment costs.

2.19.6.8.6.1 The hearth gas recirculation system will be used as a digester mixing device. Only primary digesters will require mixing.

2.19.6.8.6.2 Purchase cost of gas circulation unit.

$$CGCUP = (CGCUS)\ (CGCUR)$$

where

CGCUP = purchase cost of gas circulation unit, \$.

CGCUS = purchase cost of standard size gas circulation unit. (Can handle 60-foot diameter digester). \$32,000 1st quarter 1977 price.

CGCUR = cost ratio between gas system for digester with diameter, DIA, to standard size digester.

For a better estimate, CGCUS should be obtained from equipment vendors and treated as a unit price input. Otherwise, the following equation will be utilized for cost escalation purposes:

$$CGCUS = 32{,}000 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = MSECI value, 1st quarter 1977.

2.19.6.8.6.3 Determine CGCUR.

2.19.6.8.6.3.1 When DIA $\leq$ 60.

$$CGCUR = 0.678 + (0.00537)\ (DIA)$$

2.19.6.8.6.3.2 When DIA > 60.

$$CGCUR = 0.3715 + (0.01048)\ (DIA)$$

where

CGCUR = cost ratio between gas system for digester with diameter DIA, to standard size digester.

DIA = diameter of digester, ft.

2.19.6.8.6.4 Installation costs. The installation costs and other items can be estimated as a percentage of the purchased equipment costs.

$$PMIC = 75\%$$

where

PMIC = percentage of purchase costs of equipment as installation costs, %.

2.19.6.8.6.5 Total installed gas circulation units cost, ICGCUP, $.

$$ICGCUP = (NP)\ [CGCUP\ (1 + \frac{PMIC}{100})] \cdot (NB)$$

where

ICGCUP = total installed cost of gas circulation units, $.

NP = number of primary digesters per battery.

CGCUP = purchase cost of gas recirculation unit, $.

PMIC = percentage of purchase cost of equipment as installation costs, %.

NB = number of batteries.

2.19.6.8.7 Cost of heater and heat exchanger.

2.19.6.8.7.1 Purchase cost of the combined heater and heat exchanger cost.

$$HRHXC = (HRHXS)\ (HRHXR)$$

where

HRHXC = purchase cost of the combined heater and heat exchanger, $.

HRHXS = purchase cost of the standard size heating unit (1,000,000 BTU/hr sludge heating capacity), $.

HRHXR = cost ratio between heating unit with capacity of Q_{HUS}, BTU/hr the standard size unit.

For a better estimate, HRHXS should be obtained from equipment vendors and treated as a unit price input. Otherwise, the following equation will be utilized for cost escalation purposes:

$$HRHXS = 49{,}000 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = MSECI value, 1st quarter 1977.

2.19.6.8.7.2 Determine HRHXR.

2.19.6.8.7.2.1 When $Q_{HUS} \leq 1{,}000{,}000$ BTU/hr.

$$HRHXR = 0.6778 + 3.222 \times 10^{-7}\ (Q_{HUS})$$

2.91.6.8.7.2.2 $Q_{HUS} > 1{,}000{,}000$ BTU/hr:

$$HRHXR = 0.256 + 7.44 \times 10^{-7}\ (Q_{HUS})$$

where

Q_{HUS} = heat exchanger capacity, BTH/hr.

HRHXR = cost ratio between heating unit with capacity Q_{HUS}, and the standard size unit.

2.19.6.8.7.3 Installation costs. Using a percentage of purchase equipment costs as the installation costs, it is estimated to be approximately 51%.

2.19.6.8.7.4 Total installed heating units cost.

$$IHRHX = (N_{HT}) [(HRHXC) (1.51)]$$

where

IHRHX = total installed cost of heating units, \$.

N_{HT} = total number of heaters and heat exchangers.

HRHXC = purchase cost of the combined heater and heat exchanger, \$.

2.19.6.8.8 Cost of gas safety equipment.

2.19.6.8.8.1 This item includes the following equipment: accumulator with drip trap, low pressure check valve, pressure relief and flame trap valve, flame trap, six drip traps, gas pressure gage, waste gas burner, and gas meter.

2.19.6.8.8.2 Purchase cost of the gas safety equipment can be expressed as:

$$CGSE = (CGSES) (CGSER)$$

where

CGSE = purchase cost of gas safety equipment, \$.

CGSES = purchase cost of the standard size gas safety equipment, 2-inch Ø, \$7,100 1st quarter 1977.

CGSER = cost ratio of purchase cost of gas safety equipment with pipe size GPS to standard size equipment.

For a better estimate, CGSES should be obtained from equipment vendors and treated as a unit price input. Otherwise, the following equation will be utilized for cost escalation purposes:

$$CGSES = 7,100 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = MSECI value, 1st quarter 1977.

2.19.6.8.8.3 Determine CGSER.

$$CGSER = 0.675 + (0.1625) (GPS)$$

where

CGSER = cost ratio of purchase cost of gas safety equipment with pipe size GPS to standard size equipment.

GPS = gas safety equipment size.

2.19.6.8.8.4 Installation cost. This cost can be estimated as a percent of the purchasing cost, approximately 90 percent.

2.19.6.8.8.5 Total installed gas safety equipment cost. This cost can be expressed as follows:

$$IGSEC = (1.90)\ (CGSE)\ (NGSE)\ (NB)$$

where

IGSEC = total installed cost of gas safety equipment, \$.

CGSE = purchase cost of gas safety equipment, \$.

NGSE = number of gas safety equipment sets per battery.

NB = number of batteries.

2.19.6.8.9 Cost of sludge recirculation pumps. Only the positive displacement type will be utilized.

2.19.6.8.9.1 Purchase cost of the sludge recirculation pumps can be expressed as follows:

$$CRPUMP = (CSPUMP)\ (CRPUMR)$$

where

CRPUMP = purchase cost of sludge recirculation pumps, \$.

CSPUMP = purchase cost of the standard size sludge pump, 80 gpm and 70 feet of total dynamic head, \$2,500 1st quarter 1977 cost.

CRPUMR = cost ratio of sludge pump with capacity of DPUMP to the standard size pump (80 gpm).

For a better estimate, CSPUMP should be obtained from equipment vendors and treated as a unit price input. Otherwise, the following equation will be utilized for cost escalation purposes:

$$CSPUMP = 2{,}500 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = MSECI value, 1st quarter 1977.

2.19.6.8.9.2 Determine CRPUMR.

$$CRPUMR = 0.379 + 0.00766 (DPUMP)$$

where

CRPUMR = cost ratio of sludge pump with capacity of DPUMP to the standard size pump (80 gpm).

DPUMP = sludge pump capacity, gpm.

2.19.6.8.9.3 Installation costs. This item includes electrical work, piping, concrete pad, instrumentation, painting, etc., and can be estimated as a percentage of the purchase equipment costs. Generally it adds 150 percent to the pump costs for the installed pump price.

2.19.6.8.9.4 Total installed pump costs. These costs can be expressed as follows:

$$IPUMPC = (2.50) (CRPUMP) (NSTP) (NB)$$

where

IPUMPC = total installed cost of pump, $.

CRPUMP = purchase cost of sludge recirculation pumps, $.

NSTP = number of sludge pumps per battery.

NB = number of batteries.

2.19.6.8.10 Costs of piping system.

2.19.6.8.10.1 Piping system costs can be divided into pipe and fitting costs, valve costs, and installation man-hour costs.

2.19.6.8.10.2 Cost of cast iron pipe. This cost can be expressed as follows:

$$CCIP = (CCIPS) (PCRIO)$$

where

CCIP = cost of cast iron pipe, $.

CCIPS = cost of standard size pipe per L.F., 8" Ø, price is \$7.41/L.F. 1st quarter 1977.

PCRIO = cost ratio of pipe of size PIPES to standard size pipe (8" Ø).

2.19.6.8.10.3 Determine PCRIO.

$$PCRIO = 0.083\ (PIPES)^{1.197}$$

where

PIPES = pipe diameter, inches.

PCRIO = cost ratio of pipe of size PIPES to standard size pipe (8" Ø).

2.19.6.8.10.4 Cost of plug valve. This cost can be expressed as follows:

$$CCPV = (CCPVS)\ (VCRIO)$$

where

CCPV = cost of plug valve, \$.

CCPVS = cost of 8" Ø plug valve. \$1,099.00/unit, 1st quarter 1977.

VCRIO = cost ratio of plug valve size PIPES to standard size valve (8" Ø).

2.19.6.8.10.5 Determine VCRIO.

$$VCRIO = 0.044\ (PIPES)^{1.507}$$

where

VCRIO = cost ratio of plug valve size PIPES to standard size valve (8" Ø).

PIPES = pipe diameter, inches.

2.19.6.8.10.6 Cost of 90° bend. This cost can be expressed as follows:

$$CCRB = (CCRS)\ (BENRIO)$$

where

CCRB = cost of 90° bend, \$.

CCRS = standard size (8" Ø) 90° bend cost. \$70.88/unit, 1st quarter 1977.

BENRIO = cost ratio of 90° bend of size PIPES to standard size 90° bend (8" Ø).

2.19.6.8.10.7 Determine BENRIO.

$$\text{BENRIO} = 0.078\ (\text{PIPES})^{1.233}$$

where

BENRIO = cost ratio of 90° bend of size PIPES to standard size 90° bend (8" Ø).

PIPES = pipe diameter, inches.

2.19.6.8.10.8 Cost of tee. This cost can be expressed as follows:

$$\text{COT} = (\text{COTS})\ (\text{TRIO})$$

where

COT = cost of tee, $.

COTS = standard size (8" Ø) tee cost $104.90/unit 1st quarter 1977.

TRIO = cost ratio of tee of size PIPES to standard size tee (8" Ø).

2.19.6.8.10.9 Determine TRIO.

2.19.6.8.10.9.1 For PIPES < 8" Ø.

$$\text{TRIO} = 0.074\ (\text{PIPES})^{1.263}$$

2.19.6.8.10.9.2 For PIPES ≥ 8" Ø.

$$\text{TRIO} = 0.011\ (\text{PIPES})^{2.16}$$

where

TRIO = cost ratio of tee of size PIPES to standard size tee (8" Ø).

PIPES = pipe diameter, inches.

2.19.6.8.10.10 Man-hours for erection of bolt-ups per joint, Man-hour requirements can be expressed as follows:

2.19.6.8.10.10.1 For erection of valve flange:

$$\text{MHVJ} = 0.25\ (\text{PIPES})$$

where

MHVJ = man-hour/joint.

2.19.6.8.10.10.2 For erection of pipe and fitting flanges:

$$MHPJ = 1.5 + 0.125\ (PIPES)$$

where

MHPJ = man-hour/joint.

2.19.6.8.10.11 Man-hours for material handling. Man-hour requirements can be estimated as follows:

2.19.6.8.10.11.1 For pipe spool pieces and fittings:

$$MHPH = 0.0633\ (PIPES)$$

where

MHPH = man-hours/joint.

2.19.6.8.10.11.2 For valve handling:

$$MHVH = 0.2 + 0.288\ (PIPES)$$

where

MHVH = man-hours/joint.

2.19.6.8.10.12 Other labor requirements. Add 21 percent to the total man-hours for field supervision, cleanup, minor labor, etc.

2.19.6.8.10.13 Piping system material cost. This cost can be expressed as follows:

$$PSMC = (PIPEL)\ (CCIP) + (PIRN)\ (CCRB) + (PITN)\ (COT) + (PIVN)\ (CCPV)$$

where

PSMC = piping system material cost, \$.

2.19.6.8.10.14 Piping system labor cost. This cost can be expressed as follows:

$$PSLC = (LABRII)\quad (PIVN)\ [2\ (MHVJ) + MHVH] + (MHPJ)\ [2(PIRN) + 3\ PITN)] + (PIPEL)\ (MHPH) \qquad (1.21)$$

where

PSLC = piping system labor cost, \$.

LABRII = composite piping labor rate, \$/man-hour.

2.19.6.8.10.15 Total piping system cost. This cost can be expressed as follows:

$$TPIPEC = (PSMC + PSLC)\left(1 + \frac{PENPIP}{100}\right)$$

where

TPIPEC = total cost of piping system, \$.

PENPIP = percent cost for other piping.

2.19.6.8.11 Total bare construction costs. These costs can be expressed as follows:

$$TBCC = CF\,(COSTE + COSTCW + COSTCS + COSTCB + IFLOCT + ICGCUP + IHRHX + IGSEC + IPUMPC + TRIPEC)$$

where

TBCC = total bare construction costs, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

COSTCB = cost of control building, \$.

IFLOCT = total cost of floating cover, \$.

ICGCUP = total installed cost of gas recirculation units, \$.

IHRHX = total installed cost of heating units, \$.

IGSEC = total installed cost of gas safety equipment, \$.

IPUMPC = total installed cost of pumps, \$.

TPIPEC = total cost of piping system, \$.

2.19.6.8.12 Operation and maintenance material costs. These costs can be expressed as follows:

$$OMMC = \frac{OMMP}{100}(IFLOCT + ICGCUP + IHRHX + IGSEC + IPUMPC)$$

where

OMMC = operation and maintenance material and supply costs.

OMMP = other O&M material cost as percent of installed equipment cost, %.

IFLOCT = total cost of floating cover, $.

ICGCUP = total installed cost of gas recirculation units, $.

IHRHX = total installed cost of heating units, $.

IGSEC = total installed cost of gas safety equipment, $.

IPUMPC = total installed cost of pumps, $.

2.19.6.9 Cost Calculations Output Data.

2.19.6.9.1 Total bare construction costs, TBCC, $.

2.19.6.9.2 Operation and maintenance material costs, OMMC, $.

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2.21 EQUALIZATION

2.21.1 Background.

2.21.1.1 Equalization is used for highly variable waste flows to dampen the variations so that the treatment facility receives a relatively constant flow. It has been shown that many treatment processes operate better if extreme fluctuations in hydraulic and organic loadings are eliminated.

2.21.1.2 Equalization basins are usually aerated to prevent the settling of solids and the development of anaerobic conditions.

2.21.1.3 The volume of equalization basins required is based on the magnitude and frequency of the variations of hydraulic and organic load. The volume of a basin required for equalizing dry weather diurnal flows will be calculated if the hourly flows for 24 consecutive hours are input. However, if the hourly flow data is not available the desired volume of the basin must be input by the user. Also the program can be used for equalization of flows other than dry weather diurnal flows simply by inputting the required basin volume.

2.21.2 Input Data.

2.21.2.1 Wastewater flow

2.21.2.1.1 Average daily flow, mgd.

2.21.2.1.2 Hourly flow for 24 consecutive hours, gph.

2.21.2.2 Basin volume, million gal. This is optional. If the user wishes to equalize wet weather flows or hourly flow data is not available the basin volume may be specified.

2.21.2.3 Influent BOD, S_o, mg/1.

2.21.3 Design Parameters.

2.21.3.1 Aerator mixing requirements (0.02 to 0.04 hp/1000 gal) (Default 0.03)

2.21.3.2 Oxygen requirements (15 mg/1/hr).

2.21.3.3 Standard transfer efficiency, STE, lbO_2/hp-hr, $\approx$ 5.0.

2.21.3.4 O_2 transfer in waste/O_2 transfer in water, α, $\approx$ 0.9.

2.21.3.5 O_2 saturation in waste/O_2 saturation in water, β, $\approx$ 0.9.

2.21.3.6 Correction for pressure, P, $\approx$ 1.0.

2.21.3.7 O_2 saturation at summer temperature, C_s, mg/l.

2.21.3.8 Minimum dissolved oxygen to be maintained in basin, C_L, mg/l 2.0.

2.21.3.9 Depth of basin, ft.

2.21.4 <u>Process Design Calculations</u>.

2.21.4.1 Determine equalization basin volume.

2.21.4.1.1 Determine average hourly flow.

$$FLOWA = \frac{FLOW\ 1 + FLOW2 + \cdots\cdots FLOW24}{24}$$

where

FLOWA = average hourly flow, gph.

FLOW1-24 = hourly flows for 24 consecutive hours.

2.21.4.1.2 Calculate equalization volume. Subtract each hourly flow rate from the calculated average hourly flow.

$$FLOWD = FLOW\ A - FLOW1$$

Some of these numbers will be negative and some will be positive. The sum of the positive numbers will be the equalization volume required.

$$V = \sum_{1}^{24} FLOWD$$

where

V = equalization volume required, gal.

FLOWD = The sum of the positive differences between the average hourly flow and actual hourly flow, gph.

2.21.4.2 Calculate dimensions for earthen basins. It is assumed the minimum depth the water will be 5 ft. due to operation of aerators. The lagoon side slopes will be 3 to 1. The basin will be square.

2.21.4.2.1 Calculate the length and width of the basin at water level.

$$Lw = \frac{\left[\frac{4V}{(D-5)(7.48)} - 54(D-5)^2 + 18(D-5)^2\right]^{0.5} + 3(D-5)}{2}$$

where

Lw = length of earthen basin at water level, ft.

V = equalization volume required, gal.

D = depth of basin, ft.

2.21.4.2.2 Calculate total volume of water for earthen basin.

$$V_T = D\,(Lw)^2 - 6(D)(Lw) + (12)(D)^2$$

where

V_T = total volume of water in earthen basin, gal.

D = depth of basin, ft.

Lw = length of basin at water level.

2.21.4.3 Calculate dimensions if concrete basin is used.

2.21.4.3.1 Calculate length of basin.

$$L = 2.73\left(\frac{V}{D\text{-}5}\right)^{0.5}$$

where

L = length of basin

D = depth of basin

V = equalization volume required.

2.21.4.3.2 Calculate total volume of water in concrete basin.

$$V_T = (L)^2(D)\ (7.48)$$

where

V_T = total volume of water in the basin, gal.

L = length of basin, ft.

D = depth of basin, ft.

2.21.4.4 Calculate horsepower required for aerators.

2.21.4.4.1 Determine horsepower based on mixing. The horsepower required to keep the solids suspended is between 0.02 and 0.04 hp per 1000 gal.

$$HPM = (V_T)(MR)(1000)$$

where

HPM = horsepower of aerators required for mixing, hp.

V_T = total volume of water in lagoon, gal.

MR = mixing requirement, hp/1000 gal.

2.21.4.4.2 Determine horsepower required for oxygen requirements.

2.21.4.4.2.1 Calculate oxygen required for aerobic conditions.

$$O_2 = \frac{(OR)(V_T)(8.34)}{10^6}$$

where

O_2 = oxygen required, lb/hr.

V_T = total volume of water in basin, gal.

OR = oxygen requirement for aerobic conditions, mg/l/hr.

2.21.4.4.2.2 Calculate O_2 transfer efficiency of operating conditions.

$$OTE = STE \; \frac{(C_s)(\beta)(P) - C_L \quad (\alpha)(1.02)_{T-20}}{9.17}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

C_s = O_2 saturation at summer temperature, mg/l.

β = O_2 saturation in waste/O_2 saturation in water.

p = correction factor for pressure.

C_L = minimum dissolved oxygen to be maintained in basin, mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, ^oC.

2.21.4.4.2.3 Calculate horsepower required for oxygen transfer.

$$HPO = \frac{(O_2)}{(OTE)}$$

where

HPO = horsepower required for oxygen transfer, hp.

O_2 = oxygen required, lb/hr.

OTE = operating transfer efficiency, lb O_2/hp-hr.

Compare horsepower for mixing (HPM) with horsepower for oxygen transfer (HPO) and use the largest for the horsepower of the aerators (HP).

2.21.4.5 Calculate equalized flow

$$Q_E = \frac{(FLOWA)(24)}{10^6}$$

where

Q_E = equalized flow, mgd.

FLOWA = average hourly flow, gph.

2.21.4.6 Effluent Characteristics.

2.21.4.6.1 BOD_5.

$$BODE = (.9)(S_o)$$

$$BODSE = (.9)(BODS)$$

If BODE < BODSE set BODE = BODSE

where

BODE = effluent BOD_5 concentration, mg/l.

S_o = influent BOD_5 concentration, mg/l.

BODSE = effluent soluble BOD_5 concentration, mg/l.

BOD_5 = influent soluble BOD_5 concentration, mg/l.

2.21.4.6.2 COD

$$CODE = COD - (.1)(S_o)$$

$$CODSE = CODS - (.1)\ (BODS)$$

If CODE < CODSE set CODE = CODSE

where

CODE = effluent COD concentration, mg/l.

COD = influent COD concentration, mg/l.

S_o = influent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

CODS = influent soluble COD concentration, mg/l.

BOD_5 = influent soluble BOD_5 concentration, mg/l.

2.21.5 <u>Process Design Output Data.</u>

2.21.5.1 Equalization volume required, V, gal.

2.21.5.2 Average hourly flow, FLOWA, gph.

2.21.5.3 Length of earthen basin at water level, Lw, ft.

2.21.5.4 Length of concrete basin, L, ft.

2.21.5.5 Total volume of water in basin, V_T, gal.

2.21.5.6 Horsepower of aerators required, HP, hp.

2.21.5.7 Effluent BOD_5, BODE, mg/l.

2.21.5.8 Depth of basin, D, ft.

2.21.5.9 Equalized flow, Q_E, mgd.

2.21.5.10 Effluent soluble BOD_5 concentration, BODSE, mg/l.

2.21.5.11 Effluent COD concentration, CODE, mg/l.

2.21.5.12 Effluent soluble COD concentration, CODSE, mg/l.

2.21.6 <u>Quantities Calculations.</u>

2.21.6.1 Calculate volume of earthwork for earthen basin. The following assumptions are made on construction of basin.

The basin will be constructed using equal cut and fill.
The side slopes will be 3 to 1.
The basin will have 2 ft. of freeboard.
The levees will be 10 ft. wide across the top.
There will be only one basin.

2.21.6.1.1 The volume of earthwork must be calculated by trial and error. Assume a depth of cut (DC) of 1 ft.

2.21.6.1.1.1 Calculate the length of basin at original ground level.

$$Lc = Lw - 6(D-DC)$$

where

Lc = length of basin at original ground level, ft.

Lw = length of earthen basin at water level, ft.

D = depth of basin, ft.

DC = depth of cut, ft.

2.21.6.1.1.2 Calculate the volume of cut.

$$VC = (1.3)(DC)\ (L_c)^2 - (6)(D_c)(L_c) + 12\ (D_c)^2$$

where

VC = volume of cut, cu. ft.

1.3 = 30% shrinkage factor.

DC = depth of cut, ft.

L_c = length of basin at original ground level, ft.

2.21.6.1.1.3 Calculate length of basin at top of levee.

$$L_T = Lw + 12$$

where

L_T = length of basin at top of levee, ft.

Lw = length of basin at water level, ft

2.21.6.1.1.4 Calculate the depth of fill.

$$DF = D + 2 - DC$$

where

DF = depth of fill, ft.

D = depth of basin, ft.

DC = depth of cut, ft.

2 = freeboard, ft.

2.21.6.1.1.5 Calculate the volume of fill.

$$VF = (4)(L_T) \quad (10)(DF) + (3)(DF)^2$$

where

VF = volume of fill, cu. ft.

L_T = length of basin at top of levee, ft.

DF = depth of fill, ft.

4 = number of levees.

2.21.6.1.1.6 Compare VF and VC

If VC < VF then assume DC > 1 ft. and recalculate VC and VF. If VC > VF then assume DC < 1 ft. and recalculate VC and VF. Repeat this procedure until VC = VF. This is the volume of earthwork required.

$$VC = VF = VLEW$$

where

VC = volume of cut, cu. ft.

VF = volume of fill, cu. ft.

$VLEW$ = volume of earthwork for earthen lagoon, cu. ft.

2.21.6.2 Calculate quantity of basin liner. In some areas the soils are such that basins must be lined to prevent loss of water. User should designate whether liner is required.

2.21.6.2.1 Calculate area of the bottom of the basin.

$$SA_B = (L_w - 6D)^2$$

where

SA_B = area of the bottom of basin, ft^2.

L_w = length of basin at water level, ft.

2.21.6.2.2 Calculate area of basin embankment.

$$SA_E = 4\ (L_T)\ \left(\frac{D+2}{.3159} + 5\right)$$

where

SA_E = area of basin embankment, ft ^{2}.

L_T = length of basin at top of levee, ft.

D = depth of basin, ft.

2.21.6.2.3 Calculate total surface area required for liner.

$$SA_L = SA_B + SA_E$$

where

SA_L = Total area of basin liner, ft ^{2}.

SA_B = area of the bottom of basin, ft^2.

SA_E = area of basin embankment, ft^2.

2.21.6.3 Calculate earthwork for concrete basin. Assume 2 ft. overexcavation.

$$VEW = (L+5.5)^2(D+2.75)$$

where

VEW = volume of earthwork for concrete basin, cu. ft.

L = length of basin, ft.

D = depth of basin, ft.

2 = freeboard, ft.

2.21.6.4 Calculate reinforced concrete required for concrete basin. Assume concrete to be 9" thick.

2.21.6.4.1 Calculate R.C. wall required.

$$V_{cw} = (4)(L)(D+2)(0.75)$$

where

V_{cw} = volume of R.C. wall required, cu. ft.

L = length of basin, ft.

D = depth of basin, ft.

2.21.6.4.2 Calculate R.C. slab required.

$$V_{cs} = (L)^2(0.75)$$

where

V_{cs} = volume of R.C. slab required, cu. ft.

L = length of basin, ft.

2.21.6.5 Determine number of aerators per basin. The number of aerators per basin must be one of the following 2, 3, 4, 6, 8, etc. Also the aerators must be one of the following sizes, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100. The selection process will be trial and error.

Assume number of aerator per basin (K) is 2. If $\frac{HP}{K} > 100$, go to next trials K=3, 4, 6, 8 until $\frac{HP}{K} \leq 100$, then compare $\frac{HP}{K}$ with values for individual aerators (HP_a) given above. Select the smallest value of HP_a that is greater than $\frac{HP}{K}$. Compare HP_a x K with HP. If HP_a x K is larger than HP by 10% or more, go to next trial using the next larger K, until HP_a x K is within 10% of HP.

2.21.6.6 Calculate total installed horsepower.

$$HP_T = (K)(HPa)$$

where

HP_T = total installed horsepower, hp.

K = number of aerators.

HP_a = horsepower of individual aerators, hp.

2.21.6.7 Calculate electric energy required.

$$KWH = (HP_T)\ (365)\ (24)\ (.9)\ (.85)\ (.877)$$

where

KWH = electrical energy required, kwhr/yr.

HP_T = total installed horsepower, hp.

2.21.6.8 Calculate Operation and Maintenance Manpower.

2.21.6.8.1 Operation manpower.

$$OMH = 500\ (Q_E)^{0.5129}$$

where

OMH = operation manpower required, MH/yr.

Q_E = equalized flow, mgd.

2.21.6.8.2 Maintenance manpower.

$$MMH = 380\ (Q_E)^{0.6781}$$

where

MMH = maintenance manpower required, MH/yr.

Q_E = equalized flow, mgd.

2.21.6.9 Other operation and maintenance material and supply costs. This item includes such items as lubrication oil, paint, repair and replacement parts, etc. These costs are expressed as a percent of the installed aeration equipment cost.

$$OMMP = 4.93\ (HP_T)^{-0.1827}$$

where

$OMMP$ = O&M material and supply costs as percent of installed aeration equipment costs, %.

HP_T = totaled installed horsepower, hp.

2.21.6.10 Other minor construction cost items.

2.21.6.10.1 If an earthen basin is used the calculations account for approximately 90% of costs.

$$CF = \frac{1}{0.9} = 1.11$$

2.21.6.10.2 If a concrete basin is used the calculations account for approximately 95% of the costs.

$$CF = \frac{1}{0.95} = 1.05$$

where

CF = correction factor for other minor costs.

2.21.7 Quantities Calculations Output Data.

2.21.7.1 Volume of earthwork for earthen lagoon, VLEW, cu. ft.

2.21.7.2 Total area of basin liner, SA_L, ft^2.

2.21.7.3 Volume of earthwork for concrete basin, VEW, cu. ft.

2.21.7.4 Volume of R.C. wall required, Vcw, Cu. ft.

2.21.7.5 Volume of R.C. slab required, V_{cs}, cu. ft.

2.21.7.6 Number of aerators, k.

2.21.7.7 Horsepower of individual aerators, HP_a, hp.

2.21.7.8 Total installed horsepower, HP_T, hp.

2.21.7.9 Electrical energy required, KWH, kwhr/yr.

2.21.7.10 Operation manpower required, OMH, MH/yr.

2.21.7.11 Maintenance manpower required, MMH, MH/yr.

2.21.7.12 O & M material and supply costs as percent of installed aeration equipment cost, OMMP, %.

2.21.7.13 Correction factor for other minor costs, CF.

2.21.8 Unit Price Input Required.

2.21.8.1 Unit price input for earthwork, UPIEX, $/cu yd.

2.21.8.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

2.21.8.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

2.21.8.4 Unit price input for lagoon liner, UPILL, \$/ft^2.

2.21.8.5 Cost of standard size aerator (50 hp), COSTSA, \$ (optional).

2.21.8.6 Marshall and Swift Equipment Cost Index, MSECI.

2.21.8.7 Installation labor rate, LABRI, \$/MH.

2.21.9 <u>Cost Calculations</u>.

2.21.9.1 Calculate cost of earthwork for earthen basin.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

VLEW = volume of earthwork required for earthen basin, cu. ft.

COSTE = cost of earthwork for earthen basin, \$.

UPIEX = unit price input for earthwork, \$/cu.yd.

2.21.9.2 Calculate cost of basin liner.

$$COSTLL = (SA_L)\ (UPILL)$$

where

COSTLL = cost of basin liner, \$.

SA_L = total area of basin liner, ft^2.

UPILL = unit price input for lagoon liner, \$/ft^2.

2.21.9.3 Calculate cost of earthwork for concrete basin.

$$COSTEC = \frac{VEW}{27}\ (UPIEX)$$

where

VEW = volume of earthwork for concrete basin, cu. ft.

COSTEC = cost of earthwork for concrete basin, \$.

UPIEX = unit price input for earthwork, \$/cu.yd.

2.21.9.4 Cost of R.C. wall in-place for concrete basin.

$$COSTCW = \frac{Vcw}{27} \text{ (UPICW)}$$

where

COSTCW = cost of R.C. wall in-place for concrete basin, \$.

Vcw = volume of R.C. wall required, cu. ft.

UPICW = unit price input for R.C. wall in-place \$/cu. yd.

2.21.9.5 Cost of R.C. slab in-place for concrete basin.

$$COSTCS = \frac{Vcs}{27} \text{ UPICS}$$

where

COSTCS = cost of R.C. slab in-place for concrete basin, \$.

Vcs = volume of R.C. slab required, cu. ft.

UPICS = unit price input for R.C. slab in-place, \$/cu. yd.

2.21.9.6 Calculate purchase cost of aerators.

$$COSTA = \frac{(COSTSA)\ (COSTR)\ (K)}{100}$$

where

COSTA = purchase cost of aerators, \$.

COSTSA = cost of standard size aerator (50 hp), \$.

COSTR = cost of aerator of horsepower HP_a as a percent of the cost of the standard size aerators, %.

K = number of aerators per basin.

2.21.9.6.1 Calculate COSTR.

If $HP_a \leq 25$ hp; COSTR is calculated by:

$$COSTR = 20.7\ (HP_a)^{0.2686}$$

If $HP_a > 25$ hp, COSTR is calculated by:

$$COSTR = 4.12\ (HP_a)^{0.7878}$$

2.21.9.6.2 Purchase cost of standard size aerator. The standard size aerator is a 50 hp, high-speed floating aerator. The cost of the 50 hp aerator in the first quarter of 1977 is:

$$COSTSA = \$13{,}960$$

For better cost estimation, COSTSA should be obtained from the equipment vendor and treated as a unit price input. However, if COSTSA is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSA = \$13{,}960\ \frac{MSECI}{491.6}$$

where

COSTSA = cost of standard size aerator (50 hp), $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

2.21.9.7 Calculate total installed equipment cost.

2.21.9.7.1 Calculate aerator installation labor.

$$IMH = 0.633\ (HP_a) + 40$$

where

IMH = aerator installation labor, MH.

HP_a = horsepower of individual aerators, hp.

2.21.9.7.2 Calculate aerator installation cost.

$$AIC = (IMH)\ (K)\ (LABRI)$$

where

AIC = aerator installation cost, $.

IMH = aerator installation labor, MH.

K = number of aerators per basin.

LABRI = installation labor rate, $/MH.

2.21.9.7.3 Calculate installed cost for electrical/mechanical.

$$EMC = 0.589\ (HP_a)^{-0.1465}\ (COSTA)$$

where

EMC = installed cost for electrical/mechanical, \$.

HP_a = horsepower of individual aerators, hp.

COSTA = purchase cost of aerators, \$.

2.21.9.7.4 Calculate total installed equipment cost.

$$IEC = COSTA + AIC + EMC$$

where

IEC = total installed equipment cost, \$.

AIC = aerator installation cost, \$.

EMC = installed cost of electrical/mechanical, \$.

2.21.9.8 Calculate total bare construction cost for earthen basin.

$$TBCC = (COSTE + COSTLL + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork for earthen basin, \$.

COSTLL = cost of basin liner, \$.

IEC = installed equipment cost, \$.

CF = correction factor for other minor costs, \$.

2.21.9.9 Calculate total bare construction cost for concrete basin.

$$TBCC = (COSTEC + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

COSTEC = cost of earthwork for concrete basin, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

IEC = installed equipment cost, \$.

CF = correction factor for other minor construction costs.

2.21.9.10 Calculate O & M material and supply costs.

$$OMMC = \frac{OMMP}{100}\ (IEC)$$

where

OMMC = O & M material and supply costs, \$/yr.

OMMP = O & M material and supply costs as percent of installed equipment cost, %.

IEC = installed equipment cost, \$.

2.21.10 <u>Cost Calculations Output Data</u>.

2.21.10.1 Total bare construction cost, TBCC, \$.

2.21.10.2 O & M material and supply costs, OMMC, \$/yr.

2.21.11 <u>Bibliography</u>.

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2.23 FILTRATION

2.23.1 Background.

2.23.1.1 Filtration is the removal of suspended solids through a porous medium. Until recently, filtration was used mainly in water treatment to remove suspended solids and bacteria. However, the increasing concern for abatement of water pollution and the requirements for high quality effluents from wastewater treatment facilities have resulted in the rapid, wide acceptance of filtration in wastewater treatment. Filtration is being used for the removal of biological floc from secondary effluents, phosphate precipitates from phosphate removal processes, and as a tertiary wastewater treatment operation to prepare effluents for reuse in industry, agriculture, and recreation.

2.23.1.2 Granular media used in filtration include sand, coal, crushed anthracite, diatomaceous earth, perlite, and powdered activated carbon. Sand filters have been mostly used in water treatment. These filters are classified into slow sand filters and rapid sand filters.

2.23.1.3 Slow sand filters are normally 12-30 in. deep. The sand rests on a layer of gravel which, in turn, rests on an underdrain system. The filter is usually operated at a rate of 3 gal/ft^2/hr. When the filter becomes clogged, it is normally deactivated, drained, allowed to partially dry, and the surface layer of sludge is manually removed. Since slow sand filters require large space, have high maintenance costs, and clog rapidly, their application to water treatment has been abondoned; their application to wastewater treatment has been very limited.

2.23.1.4 Rapid sand filters consist of a layer of sand 18-30 in thick supported by a layer of gravel 6-18 in thick and an underdrain system. The underdrain system not only supports the sand, but also collects the filtered water and distributes the backwash water. The gravel aids in the distribution of the wash water while preventing the loss of filter media to the underdrain. Rapid sand filters are usually operated at a rate of 2 gal/ft^2/min. The filters are cleaned by backwashing. Normally, the filter is backwashed when the head loss increases to a value approaching the actual head available or when the effluent quality begins to deteriorate. The length of time of filter runs normally depends on the quality of the feed water. The common rate of backwashing is 24-30 in/min (about 15 ga/lft^2/min) for 3-10 min. This rate results in about 30 percent expansion of the sand.

2.23.1.5 Rapid sand filters are usually constructed in duplicate. The filters are commonly arranged in rows along one or both sides of a pipe gallery. Total depths of filters from water surface to underdrains range from 8 to 10 ft. A length-to-width ratio of the filter box of 3 to 6 has been found most economical. Wash water gutters must be located so that they limit the horizontal travel of dirty water during backwashing to 3 ft. As a safety factor, the top edge of the backwash gutter is normally located 6-12 in above the allowed expansion of the sand (usually 50 percent). Backwash gutters must be designed to carry all backwash water with a minimum 3-in free fall at the upper end. The underdrain system must be designed to carry the backwash water and to provide uniform distribution of backwash water.

2.23.1.6 The removal of surface material by sand filters leaves a heavy residue of removed solids on the raw water side of the filter medium. Surface filtration is extremely sensitive to suspended solids concentration in the feed water. Sand filters may quickly become clogged at the surface, resulting in an extremely short filter run, thereby limiting the practical application of sand filters in wastewater treatment.

2.23.1.7 One method used to increase the effective depth of filtration involves the use of dual-media or multimedia beds. The filters are composed of two or three materials of different specific gravities and sizes. The coarsest and lightest materials are placed on top; the finest and heaviest materials are on the bottom. An anthracite/sand filter is an example of a dual-media filter. Typical anthracite/ sand filters may include from 12 to 24 in of anthracite (specific gravity, 1.4-1.6) and 6 to 16 in of sand (specific gravity, 2.65). Multimedia filters normally consist of anthracite placed on top of sand which is placed on top of garnet.

2.23.1.8 Another innovation introduced into filtration is the mixed media concept. For this process, the size distribution of the different media is selected to ensure intermixing between the various media at the interfaces. This mixing will prevent the formation of an impervious layer of the interface during filtration. A typical mixed dual-media filter may consist of 12 in of sand with an effective size of 0.5-0.55 mm and a uniformity coefficient of less than 1.65, and 12 in of crushed anthracite coal with an effective size of 0.9-1.0 mm and a uniformity coefficient of less than 1.8. A typical mixed multimedia filter has a particle size gradation that decreases from about 2-mm anthracite at the top to about 0.15-mm garnet at the bottom.

2.23.1.9 A mixed-media design typically used for removal of moderate quantities of chemical floc requires a backwash rate of about 15 gpm/ft^2. The head loss through the expanded filter is 2-4 ft. The required duration for backwash water is typically 2-5 percent of the plant throughout. Surface wash is also necessary to break up the clumps. Normally, surface wash is initiated 1 min before the main backwash starts and is stopped about 1 min prior to the end of the backwash.

2.23.1.10 In conclusion, the design of filters depends on the influent wastewater characteristics, process and hydraulic loadings, method and intensity of cleaning, nature, size, and depth of the filtering material, and the required quality of the final effluent. In general, mixed dualmedia and multimedia filters are more effective and easier and less expensive to operate than sand filters for the treatment of wastewaters; therefore, they are more widely accepted in wastewater treatment.

2.23.1.11 However, in reality, the design of a tertiary filtration system involves using pilot study data or rules-of-thumb to calculate the required filter surface area and select the appropriate off-the-shelf units available from various manufacturers. The filtration manufacturers usually provide customers with all necessary equipment except pumps, concrete structure, and housing. The filter media, backwash trough, and underdrain system are specified by the manufacturers. The consulting engineers only have to select the appropriate model from the manufacturers and design the concrete structures.

2.23.1.12 There are hundreds of various sizes and types of filtration units available in the market. For smaller installations, the package units usually are selected. They are completely self-contained filter units to be shipped from the factories preassembled. The general contractors have to provide only the concrete slabs and influent-effluent piping systems. For larger installations, concrete wall constructions are used for containing the filter units. The filter manufacturers supply the media, backwashing troughs, control systems, underdrain, etc., which are installed in the field by the general contractor. Thus, the design of of these two types of filters are different and should be treated separately.

2.23.1.13 Each manufacturer specifies his own filter configurations, dimensions and other requirements. These requirements are often different and proprietary information is involved. These diversifications make the computer modeling extremely difficult. This is especially true when package units are used. It is thus decided that a parametric cost curve will be provided for the package type filtration units. The construction cost for the larger concrete wall constructed, four-rectangular cell filtration systems, will be estimated based on equipment and material costs.

2.23.1.14 Due to the fact that tertiary filters are usually at the end of treatment processing trains and large head losses are expected, intemediate pumping is always required to deliver the main stream to the filter units. However, information on the main stream pumping facility is not included in this section; rather, design and cost data are presented in the section entitled "Intermediate Pumping Station".

2.23.2 Input Data.

2.23.2.1 Wastewater characteristics.

2.23.2.1.1 Average daily flow, Q_{avg}, mgd.

2.23.2.1.2 Peak flow, Q_p, mgd.

2.23.2.1.3 Suspended solids concentration, SS_{inf}, mg/l.

2.23.2.2 Loading rate, gpm/sq ft of filter surface area, if available.

2.23.3 Design Parameters.

2.23.3.1 Backwash rate, gpm/ft^2.

2.23.3.2 Backwash period, min.

2.23.3.3 Loading rate, gpm/ft^2.

2.23.4 Process Design Calculations.

2.23.4.1 Filter surface area required.

2.23.4.1.1 If a loading rate is not available from the input, a value of 3.5 gpm/sq ft will be used. The surface area required would be:

$$SA = \frac{Q_{avg}\ (10^6)}{(1440)\ LR}$$

where

SA = surface area of filter required, sq ft.

LR = loading rate, gpm/sq ft.

2.23.4.1.2 The minimum number of filter cells should be two. Usually three to four cells are provided in package units and four-cell construction is common among the concrete filtration facilities. It is generally true that package filters are more economically feasible when the surface area required is less than approximately 400 sq ft. For applications with surface area requirements larger than 400 sq ft, filters with concrete wall construction are used. The maximum size filter of this type is approximately 2200 sq ft (with four cells). When larger sizes are required, multiple filtration units will be selected. The general rules are summarized in the following table.

Q_p, mgd	Number of Filtration Units, NU	Number of Cells per Units, NC
1 - 10	1	4
10 - 20	2	4
20 - 40	4	4
40 - 60	6	4
60 - 80	8	4
80 - 100	10	4

When Q_{avg} is larger than 100 mgd, the system will be designed as several batteries of units.

2.23.4.1.3 When $Q_p \leq 100$ mgd, only one battery of filtration units will be used. NB = 1.

2.23.4.1.4 When $100 < Q_p \leq 200$ mgd, the number of process batteries, NB, would be two. The system would be design as two identical batteries with flow to each battery at $Q_p/2$.

where

NU = number of filtration units per battery.

NB = number of batteries.

NC = number of cells per filter units.

2.23.4.1.6 It is stated in Addendum No. 6 of the Ten State Standards that "Filtration rate should not exceed 5 gpm/sq ft based on the maximum hydraulic flow rate with one of the filter units out of service". Thus, the filter surface area based on this requirement is:

$$SAP = \frac{Q_p \cdot 10^6}{(5)\ (1440)} \cdot \frac{NU \cdot NC}{(NU)\ (NC-1)}$$

$$= 138.9\ (Q_p)\ \left(\frac{NU \cdot NC}{(NU)\ (NC-1)}\right)$$

where

SAP = surface area requirement based on Ten-State Standard, sq ft.

Q_p = peak flow, mgd.

NU = number of filtration units.

NC = number of cells per filter.

2.23.4.1.7 Compare SA with SAP and use the larger value as the design surface area, SAD, sq ft.

2.23.4.1.8 The surface area of individual filtration unit can be calculated by:

$$SAU = \frac{SAD}{(NB)(NU)}$$

where

SAU = surface area of individual filtration units, sq ft.

SAD = designed total surface area with filter required, sq ft.

2.23.4.1.9 The size of each cell within a unit can be calculated by:

$$SAC = \frac{SAU}{NC}$$

where

SAC = surface area of each cell, sq ft.

2.23.4.2 Backwash requirements.

2.23.4.2.1 It is stated in Ten-State Standards that "Provision should be made for a minimum of 20 gpm/sq ft and a minimum backwash period of 10 minutes". Usually, filtered water is used for backwash purposes. In certain cases, product water from other filters cannot meet the demand for backwashing one unit. In these cases a clearwell is provided to store filtered water for this purpose. It is suggested that storage capacity be provided which equals twice the volume of required backwash water. In this model, however, it is assumed that chlorine contact tanks will follow the tertiary filtration units and the contact tanks would provide more than enough capacity for backwash water storage.

2.23.3.2.2 Pumps are usually used to provide pressure for backwash in package filter units. For the concrete construction types, enough head (3 ft) is provided in the effluent clearwell; thus no pumping is necessary.

2.23.4.2.3 It is stated in Ten-State Standards that the rate of return of waste filter backwash water to treatment units should be controlled so that the rate does not exceed 15 percent of the design flow rate to the treatment units. Surge tanks should have a minimum capacity of twice the volume of backwash water required.

2.23.4.2.4 The following table gives the design backwash surge tank capacity, based on design experience.

Number of Filter Units, NU	Number of Backwash Volumes To Be Stored, N_{bws}
1	2
2	2
4	3
6	4
8	5
10	6

2.23.4.2.5 The volume of the backwash surge tank can be calculated by:

$$V_{bws} = \frac{(NB)(N_{bws}) \cdot (SAC) \cdot (20) \cdot (10)}{7.48}$$

where

V_{bws} = capacity of surge tank, cu ft.

20 = backwash rate, gpm/sq ft.

10 = backwash period, minutes.

2.23.4.3 Effluent Characteristics.

2.23.4.3.1 Suspended solids.

$$TSSE = (.4)(TSS)$$

where

TSSE = total suspended solids in effluent, mg/l.

TSS = total suspended solids in influent, mg/l.

2.23.4.3.2 BOD.

$$BODE = BODS$$

where

BODE = total effluent BOD_5 concentration, mg/l.

BODS = influent soluble BOD_5 concentration, mg/l.

2.23.4.3.3 COD.

$$CODE = CODS$$

where

CODE = total effluent COD concentration, mg/l.

CODS = influent soluble COD concentration, mg/l.

2.23.4.3.4 Oil and grease.

$$OAG = 0$$

where

OAG = effluent oil and grease concentration, mg/l.

2.23.4.3.5 Nitrogen.

$$TKNE = NH3E$$

where

TKNE = effluent total Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

2.23.4.3.6 Settleable solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.23.5 <u>Process Design Output Data.</u>

2.23.5.1 Design filter surface area, SAD, sq ft.

2.23.5.2 Number of process batteries, NB.

2.23.5.3 Number of filtration units per battery, NU.

2.23.5.4 Number of cells per filtration units, NC.

2.23.5.5 Surface area of individual filtration unit, SAU, sq ft.

2.23.5.6 Volume of backwash surge tank, V_{bws}, cu ft.

2.23.5.7 Total suspended solids in effluent, TSSE, mg/l.

2.23.5.8 Total effluent BOD_5 concentration, BODE, mg/l.

2.23.5.9 Total effluent COD concentration, CODE, mg/l.

2.23.5.10 Effluent oil and grease concentration, OAG, mg/l.

2.23.6 <u>Quantities Calculations</u>.

2.23.6.1 When SAD $\leq$ 400 sq ft, parametric cost estimating procedures will be utilized. The output required for cost estimates is surface area.

$$SAPF = SAD$$

where

SAPF = surface area of package filter, sq ft.

The quantities of earthwork, concrete and others would be zero.

When SAD > 400 sq ft, concrete construction is required. Thus, SAPF should be zero and other material quantities could be calculated by using the formulae provided in the following sections. Figure 2.23-1 shows the cross-sections of one of the commercially available units.

2.23.6.2 Earthwork required for construction of the filter unit.

2.23.6.2.1 A wide range of filter sizes are available from various manufacturers. In this model the sizes are limited from 400 sq ft to 2200 sq ft.

2.23.6.2.2 The following equation provides the estimated earthwork quantity for the construction of a single filter unit with surface area of SAU sq ft.

$$V_{ewf} = 8.875\ (SAU) + 5300$$

where

V_{ewf} = earthwork quantity for the construction of a single filter unit, cu ft.

SAU = surface area of filter unit, sq ft.

2.23.6.2.3 For multiple filter units, the earthwork required would be:

$$V_{ewm} = V_{ewf}\ (0.1667 + 0.8333 \cdot NU) \cdot (NB)$$

where

V_{ewm} = quantity of earthwork for multiple filter units, cu ft.

NU = number of filter units.

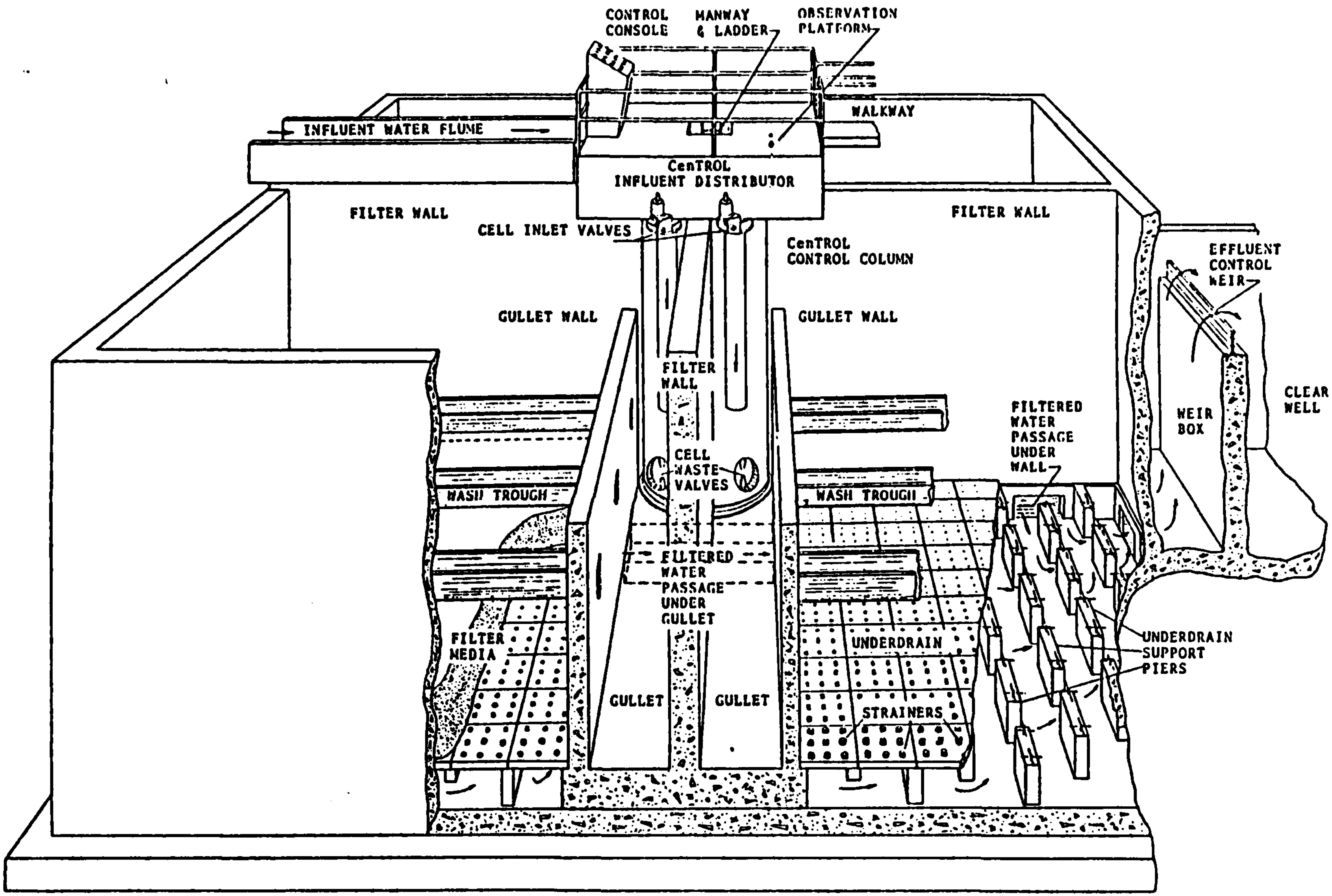

FIGURE 2.23-1. TYPICAL FOUR-CELL FILTRATION UNIT

NB = number of batteries.

(0.1667 + 0.8333 $\cdot$ NU) = Experience equation to account for the common wall construction and requirements when multiple filter units are used.

2.23.6.3 Concrete works required for construction of filtration units.

2.23.6.3.1 The relationship between the concrete works required for the construction of filter units and their sizes is given by:

$$V_{cwf} = 4.4 \text{ (SAU)} + 2850$$

where

V_{cwf} = volume of reinforced concrete requirement, cu ft.

2.23.6.3.2 When multiple units are used, the following equation gives the required quantities:

$$V_{cwm} = V_{cwf} \text{ (0.056 + 0.9444} \cdot \text{NU)} \cdot \text{(NB)}$$

where

V_{cwm} = quantity of R.C. for multiple filter units, cu ft.

(0.056 + 0.9444 $\cdot$ NU) = experience equation to account for the common wall construction and other requirements when multiple units are used.

2.23.6.4 Backwash waste surge control tank design. The backwash waste would be drained from the bottom of the filtration units into a surge control tank by gravity flow. It is assumed that this surge tank would be a concrete structure built below grade. The depth of this tank is assumed to be 7 feet and the length to width ratio is to be 2:1.

2.23.6.4.1 Calculate the dimension of the surge tank.

$$W_{st} = \frac{V_{bws}}{14}^{1/2}$$

where

W_{st} = width of surge tank, ft.

V_{bws} = volume of backwash surge tank, cu ft.

The length of surge tank would be:

$$L_{st} = 2 \cdot W_{st}$$

where

L_{st} = length of surge tank, ft.

2.23.6.4.2 Quantities of earthwork and reinforced concrete calculations. Figure 2.23-2 illustrates the configuration of a typical surge tank. The quantities of reinforced concrete and earthwork for its construction can be estimated.

Earthwork required:

$$V_{ewst} = 1/3\ (A_1 + A_2 + \quad A_1 \cdot A_2) \cdot 10$$

where

V_{ewst} = volume of earthwork, cu ft.

$A_1 = (L_{st} + 10) \cdot (W_{st} + 10)$

$A_2 = (L_{st} + 20) \quad (W_{st} + 10)$

Reinforced concrete required:

$$V_{cwst} = 20\ (L_{st} + W_{st}) + (L_{st} + 6)\ (W_{st} + 6)$$

where

V_{cwst} = volume of reinforced concrete for the construction of the backwash surge tank, cu ft.

2.23.6.5 Electrical energy required for operation. In the multimedia filtration system the process which requires the most electrical energy is pumping of the main stream of wastewater to the filters. Minor but significant energy requirements include power for backwash, surface spray and air blowers. In this section only the energy for the minor requirements will be calculated. Energy required for influent pumping is estimated in the section entitled "Intermediate Pumping Station". Electrical energy for filter backwash, surface spray and air blowers is given by the following equation:

$$KWH = 8213 \cdot (Q_{avg})^{0.972}$$

where

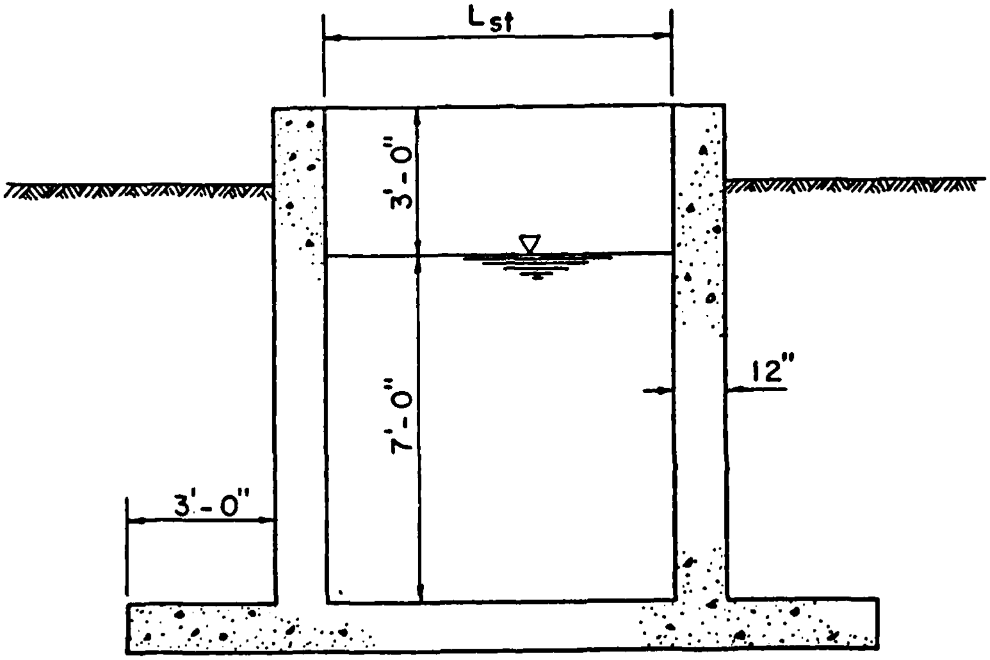

FIGURE 2.23-2. TYPICAL BACKWASH SURGE TANK

KWH = electrical energy requirement, kwhr/yr.

Q_{avg} = averaged design flow, mgd.

2.23.6.6 Operation and maintenance manpower requirements.

2.23.6.6.1 Operation man-hour requirement, OMH.

$$OMH = 80.4 \cdot (Q_{avg})^{0.572}$$

where

OMH = operational man-hour/yr.

2.23.6.6.2 Maintenance man-hour requirements, MMH.

$$MMH = 54 \cdot (Q_{avg})^{0.585}$$

where

MMH = maintenance man-hour/yr.

2.23.6.7 Other operation and maintenance material cost. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of total installed costs of the multimedia filtration unit.

$$OMMP = 5\%$$

where

OMMP = percent of the installed filtration system costs for operation and maintenance material costs.

2.23.6.8 Other construction cost items. In the above estimations approximately 80 percent of the construction costs have been accounted for. The remaining 20 percent would include minor costs such as backwash auxillary supply pumps, piping, housing, etc.

$$\text{CF, correction factor would be } \frac{1}{0.8} = 1.25$$

2.23.7 <u>Quantities Calculations Output Data.</u>

2.23.7.1 Surface area of package filters, SAPF, sq ft.

2.23.7.2 Design filter surface area, SAD, sq ft.

2.23.7.3 Volume of earthwork required for a single filter unit, V_{ewf}, cu ft.

2.23.7.4 Volume of earthwork required for multiple filter units, V_{ewm}, cu ft.

2.23.7.5 Volume of R.C. required for a single filter unit, V_{cwf}, cu ft.

2.23.7.6 Volume of R.C. required for multiple filter units, V_{cwm}, cu ft.

2.23.7.7 Volume of earthwork required for backwash surge tank, V_{ewst}, cu ft.

2.23.7.8 Volume of R.C. required for backwash surge tank, V_{cwst}, cu ft.

2.23.7.9 Electrical energy requirement for operation, KWH, kwhr/yr.

2.23.7.10 Operation manpower requirements, OMH, man-hours/yr.

2.23.7.11 Maintenance manpower requirements, MMH, man-hours/yr.

2.23.7.12 Other operation and maintenance material costs as percent of total bare construction cost, OMMP, percent.

2.23.7.13 Correction factor for minor construction costs.

2.23.8 Unit Price Input Required.

2.23.8.1 Unit price input for earthwork, UPIEX, $/cu yd.

2.23.8.2 Unit price input for concrete wall in-place, UPICW, $/cu yd.

2.23.8.3 Cost of standard size filter equipment (784 sq ft), COSF, $, (optional).

2.23.8.4 EPA construction cost index for municipal wastewater treatment facilities, EPACI.

2.23.9 Cost Calculations.

2.23.9.1 Cost of earthwork for filtration units.

2.23.9.1.1 When NU = 1

$$COSTEF = \frac{V_{ewf}}{27} \, UPIEX$$

2.23.9.1.2 When NU > 1

$$COSTEF = \frac{V_{ewm}}{27} \, UPIEX$$

where

COSTEF = cost of earthwork for filtration units, $.

V_{ewf} = volume of earthwork for a single filtration unit, cu ft.

V_{ewm} = volume of earthwork for multiple filtration units, cu ft.

UPIEX = unit price input for earthwork, $/cu yd.

2.23.9.2 Cost of earthwork for backwash surge tank.

$$COSTEST = \frac{V_{ewst}}{27} \, UPIEX$$

where

COSTEST = cost of earthwork for backwash surge tank, $.

V_{ewst} = volume of earthwork for backwash surge tank, cu ft.

UPIEX = unit price input for earthwork, $/cu yd.

2.23.9.3 Cost of reinforced concrete in-place for filtration units.

2.23.9.3.1 When NU = 1

$$COSTCF = \frac{V_{csf}}{27} UPICW$$

2.23.9.3.2 When NU > 1

$$COSTCF = \frac{V_{cwm}}{27} UPICW$$

where

COSTCF = cost of reinforced concrete in-place for filtration units, \$.

V_{cwf} = volume of R.C. required for a single filtration unit, cu ft.

V_{cwm} = volume of R.C. required for multiple filtration units, cu ft.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.23.9.4 Cost of reinforced concrete in-place for backwash surge tank:

$$COSTCST = \frac{V_{cwst}}{27} UPICW$$

where

V_{cwst} = volume of R.C. required for backwash surge tank, cu ft.

UPICW = unit price input of concrete wall in-place, \$/cu yd.

2.23.9.5 Installed cost of package filter units (SAD $\leq$ 400 sq ft). As stated previously, because of the wide differences in package plants supplied by different manufacturers, parametric cost estimation will be used for these plants.

$$COSTPF = 33{,}149.9 \ (SAPF)^{0.3203} \frac{EPACI}{131.52}$$

where

COSTPF = installed cost of package filter equipment, \$.

SAPF = surface area of package filter, sq ft.

EPACI = current Environmental Protection Agency construction cost index for municipal wastewater treatment facilities (Index 100 3d Quarter 1973).

131.52 = Environmental Protection Agency construction cost index for municipal wastewater treatment facilities, first quarter, 1977.

2.23.9.6 Filter equipment cost for concrete construction (SAD > 400 sq ft).

2.23.9.6.1 Cost of standard size unit. A unit with 784 sq ft of filter area was selected as the standard unit.

$$COSF = \$165,000$$

For a better cost estimation COSF should be obtained from the equipment vendor and treated as a unit price input. However, if this is not done, the cost should be adjusted by using the EPA construction cost index.

$$COSF = \$165,000 \frac{EPACI}{131.52}$$

where

COSF = cost of standard size filter equipment (784 sq ft), \$.

2.23.9.6.2 Calculate COSTR.

When SAD ≤ 784 sq ft:

$$COSTR = 8.99 \ (SAD)^{0.3615}$$

When SAD > 784 sq ft:

$$COSTR = 1.04 \ (SAD)^{0.6853}$$

where

COSTR = purchase cost of filter equipment for filter of size SAD as percent of standard size filter cost, percent.

SAD = area of filter unit, sq ft.

2.23.9.6.3 Calculate equipment purchase cost.

$$COSTFE = \frac{COSTR}{100} \times COSTES$$

where

COSTFE = purchase cost of filter equipment for filter of SAD sq ft, \$.

2.23.9.7 Installation cost. The installation cost is best related to the concrete cost. The installation cost is estimated to be 53 percent of the concrete cost.

$$EIC = .53\ (COSTCF + COSTCST)$$

where

EIC = equipment installation cost, \$.

2.23.9.8 Total bare construction cost.

2.23.9.8.1 For package filter units.

$$TBCC = (COSTPF + COSTCST + COSTEST)\ CF$$

where

TBCC = total bare construction cost for filter unit, \$.

CF = correction factor for minor construction costs.

2.23.9.8.2 For concrete units.

$$TBCC = (COSTFE + COSTEF + COSTCF + COSTSCST + COSTEST + EIC)\ CF$$

where

TBCC = total bare construction cost for filter units, \$.

2.23.9.9 Operation and maintenance material costs.

2.23.9.9.1 For package filter units.

$$OMMC = (COSTPF)\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$.

OMMP = operation and maintenance material costs as percent of installed filtration equipment cost.

2.23.9.9.2 For filter units of concrete construction.

$$OMMC = (COSTFE + EIC)\ OMMP$$

2.23.10 Cost Calculations Output Data.

2.23.10.1 Total bare construction cost, TBCC, $.

2.23.10.2 Operation and maintenance material costs, OMMC, $.

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2.25 FILTER PRESS

2.25.1 Background.

2.25.1 The filter press is a type of pressure filtration and differs from vacuum filtration in that the liquid is forced through the filter medium by a positive pressure instead of a vacuum. The press finds application primarily in the field of water and wastewater treatment as a means for dewatering of sludges. A number of different kinds of filter presses are currently available. Older models apply pressure by pumping the solution into chambers lined with filter cloth, whereas the most recent designs actually press the water out by applying direct pressure to the sludge.

2.25.1.2 Filter presses are available in varying sizes according to the amount of sludge to be dewatered. Optimum operation conditions are usually provided by conditioning, thickening, and other pretreatment to produce a sludge with an initial concentration of 3-5% and specific resistances of 10^9-10^{10} cm/g.

2.25.1.3 The word press is somewhat misleading when applied to the earliest designs which originated in the early twentieth century. These early filter presses did not actually press the water out by consolidating the sludge, but instead the solution was pumped between plates that were covered with a fiber filter cloth where the liquid seeped through the filter cloth leaving the solids behind between the plates. When the void spaces became filled, the operator separated the plates to remove the solids. Figure 2.25-1 is a schematic for this type of press.

2.25.1.4 This type of filter pressing is a cyclic operation which has been the most objectionable characteristic of the process in the United States. However, this cyclic operation is considered to be a great advantage in Europe, since in the smaller towns, the wastewater treatment plant operator can start the press in the afternoon or late evening, let the operation proceed through the night and upon returning to work the following day, empty the solid materials.

2.25.1.5 The unpopularity of the earlier filter presses in the United States has prompted new design approaches by several manufacturers toward continuous operating presses. Pretreatment of the sludge is required, however, for either a batch or continuous flow operation. Usually a chemical addition (alum, lime, or ferric chloride) is followed by mixing, flocculation, and gravity thickening. Conditioning of the filter media may also prove to be beneficial. Extensive laboratory work is required to determine chemical requirements for plant optimization.

2.25.1.6 Filter presses are superior to vacuum filters in both cake solids concentration and suspended solids in the filtrate. Incineration and landfill costs may be reduced if filter presses are used because they produce a cake having 40-50% solids, while centrifugation and vacuum filtration produce only 20-25% cake solids. The filtrate from presses may contain as little as 75 mg/l suspended solids while the other two methods typically discharge 800-1000 mg/l suspended solids.

2.25.2 <u>Input Data</u>.

2.25.2.1 Average wastewater flow, mgd.

2.25.2.2 Sludge volume, gallons/million gallons.

2.25.2.3 Raw sludge solids concentration, %.

2.25.2.4 Sludge produced by conditioning chemicals.

2.25.2.5 Type and amount of conditioning material required, %.

2.25.2.6 Cake thickness, inches.

2.25.2.7 Cake solids content, %.

2.25.2.8 Filter cake density, lb/ft^3.

2.25.2.9 Volume of filter chamber, ft^3.

2.25.2.10 Filter cycle time, hours.

2.25.2.11 Operating schedule, hr/day.

2.25.3 <u>Design Parameters</u>.

2.25.3.1 Sludge volume per million gallons treated.

2.25.3.2 Raw sludge solids concentration, %. 1.5-15%.

2.25.3.3 Conditioning solids, from laboratory studies. Use 0.2 of sludge solids as an estimate.

2.25.3.4 Type and amount of conditioning chemicals, %, from laboratory studies.

2.25.3.5 Cake thickness (0.5-1.5 inches). Use 1.2 inches.

2.25.3.6 Cake solids content (35-50%). Use 45%.

2.25.3.7 Filter chamber volume, ft^3. From manufacturer's literature (If unknown, use 1-2 cu ft).

2.25.3.8 Filter cycle time, hr. Use 2.0 hr.

2.25.3.9 Operating schedule, hours per day (8-16 hr).

2.25.4 <u>Process Design Calculations</u>.

2.25.4.1 Calculate the pounds of dry solids in sludge flow per day.

$$DSS = \frac{(Q_{avg})(SF)(SS)(8.34)}{100}$$

where

DSS = pounds of dry sludge solids per day.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gallons/million gallons.

SS = suspended solids flow to filter press, %.

2.25.4.2 Calculate the total pounds of dry solids produced per day.

$$PDSPD = DSS + (CS)(DSS)$$

where

PDSPD = pounds of dry solids produced per day.

CS = conditioning solids, expressed as a fraction of sludge solids. (0.2).

2.25.4.3 Calculate the weight of filter cake produced per day.

$$PFC = \frac{PDSPD}{CSC} \times 100$$

where

PFC = pounds of filter cake produced per day, net weight.

CSC = cake solids content, %.

2.25.4.4 Calculate the cake volume.

$$CV = \frac{PFC}{CD}$$

where

CV = cake volume, ft^3/day.

CD = cake density, lb/ft^3.

2.25.4.5 Calculate the number of chambers.

$$CPD = \frac{CV}{FCV}$$

where

CPD = chambers per day.

FCV = filter's chamber volume, ft^3.

2.25.4.6 Calculate the number of filter cycles per day.

$$NFC = \frac{HPD}{FCT}$$

where

NFC = number of filter cycles per day.

HPD = opeerating schedule for filter, hours per day.

FCT = filter cycle time, hours per cycle.

2.25.4.7 Calculate the number of filter chambers required in the filter press.

$$FCR = \frac{CPD}{NFC}$$

where

FCR = number of filter chambers required in the filter press.

2.25.4.8 Detemine sludge flow out.

$$SFO = \frac{PDSPD\ (100)}{(CSC)\ (8.34)\ (Sp.gr.)}$$

where

SFO = sludge flow out, gpd.

PDSPD = pounds of dry solids produced per day, lb/day.

CSC = cake solids content, %.

Sp.gr. = specific gravity of sludge.

2.25.5 <u>Process Design Output Data</u>.

2.25.5.1 Total dry solids produced, lb/day.

2.25.5.2 Weight of filter cake produced, lb/day.

2.25.5.3 Cake moisture content, %.

2.25.5.4 Cake density, lb/ft^3.

2.25.5.5 Cake volume, ft^3.

2.25.5.6 Filter chamber volume, ft^3.

2.25.5.7 Number of chambers per day.

2.25.5.8 Operating schedule, hours/day.

2.25.5.9 Number of filter cycles per day.

2.25.5.10 Number of filter chambers required in the filter press.

2.25.6 <u>Quantities Calculations</u>. Not Used.

2.25.7 <u>Quantities Calculations Output Data</u>. Not Used.

2.25.8 <u>Unit Price Input Required</u>. Not Used.

2.25.9 <u>Cost Calculations</u>.

2.25.9.1 Unit costing is not available for this treatment process, therefore parametric costing will be used.

2.25.9.2 Calculate total bare construction cost.

$$X = \log (Q_{avg})$$

$$Y = -0.69698 - 0.12594(X) + 0.095578(X)^2$$

$$TBCC = e^Y$$

where

TBCC = total bare construction cost, \$.

2.25.9.3 Operation and maintenance cost.

$$Z = 1.3906 - 0.73944(X) + 0.081625(X)^2$$

$$O\&M = e^Z \frac{Q_{avg} \times 10^3}{100}$$

where

O&M = operation and maintenance cost, \$/yr.

2.25.10 <u>Cost Calculations Output Data</u>.

2.25.10.1 Total bare construction cost, TBCC, \$.

2.25.10.2 Operation and maintenance cost, O&M, \$/yr.

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2.27 FLOTATION.

2.27.1 Background.

2.27.1.1 Flotation is a solid-liquid separation process. Separation is artifically induced by introducing fine gas bubbles (usually air) into the system. The gas-solid aggregate with an overall bulk density less than the density of the liquid; thus, these aggregates rise to the surface of the fluid. Once the solid particles have been floated to the surface, they can be collected by a skimming operation.

2.27.1.2 In wastewater treatment, flotation is used as a clarification process to remove suspended solids and as a thickening process to concentrate various types of sludges. However, high operating costs of the process generally limit its use to clarification of certain industrial wastes and for concentration of waste-activated sludge.

2.27.1.3 Air flotation systems may be classified as dispersed air flotation or dissolved air flotation. In dispersed air flotation, air bubbles are generated by introducing air through a revolving impeller or porous media. This type of flotation system is ineffective and finds very limited application in wastewater treatment. Dissolved air flotation may be subclassified as pressure flotation or vacuum flotation. Pressure flotation involves air being dissolved in the wastewater under elevated pressures and later released at atmospheric pressure. Vacuum flotation, however, consists of applying a vacuum to wastewater aerated at atmospheric pressure. Dissolved air-pressure flotation, considered herein is the most commonly used in waterwater treatment.

2.27.1.4 The principal components of a dissolved air-pressure flotation system as shown in Figure 2.27-1 are a pressurizing pump, air injection facilities, a retention tank, a back pressure regulating device, and a flotation unit. The primary variables for flotation design are pressure, recycle ratio, feed solid concentration, detention period, air-to-solids ratio, use of polymers and solids and hydraulic loadings. Optimum design parameters must be obtained from bench scale or pilot plant studies. Typical design parameters are listed in Table 2.27-1.

Table 2.27-1. Air Flotation Parameters

Parameter	Typical Value Thickening	Typical Value Clarification
Air Pressure, psig	40 to 70	40 to 70
Effluent recycle, %	130 to 150	30 to 120
Detention time, hr	3	0.25 to 0.5
Air-to-solids ratio (lb air/lb solids)	(0.005 to 0.06)	
Solid loading, lb/ft^2/day		
Activated sludge (mixed liquor)	5 to 15	
Activated sludge (settled)	10 to 20	
50% primary +50% activated	20 to 40	
Primary only	to 55	
Hydraulic loading, gpm/ft^2	0.2 to 4	1 to 4
Detention time, min (pressurizing tank)	1 to 3	1 to 3

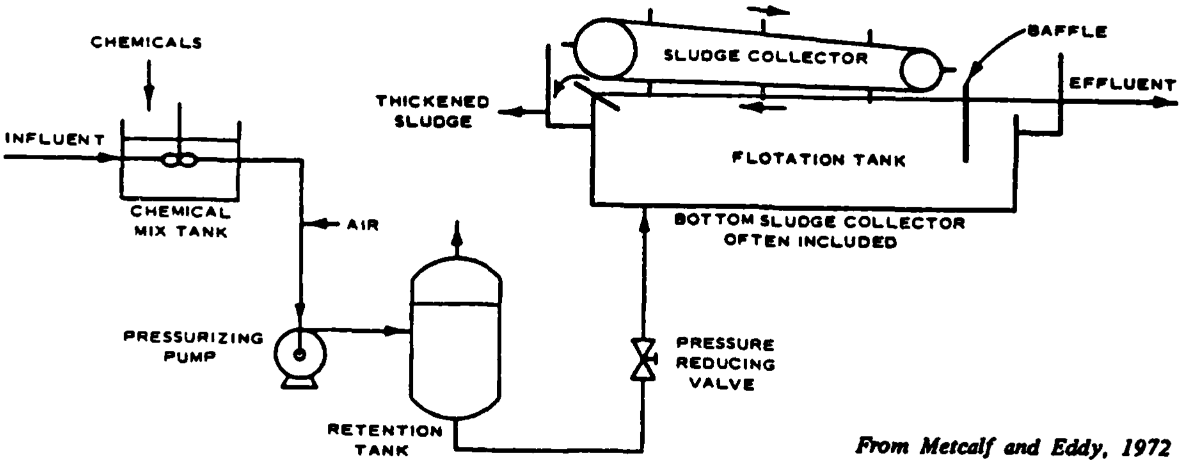

Figure 2.27-1. Schematic of Dissolved-Air Flotation Tank

2.27.2 Input Data.

2.27.2.1 Wastewater flow, mgd.

2.27.2.2 Suspended solids concentration in the feed, mg/l.

2.27.2.2.1 Average concentration.

2.27.2.2.2 Variation in concentration.

2.27.2.3 Polymer dosage, lb/ton dry solids.

2.27.3 Design Parameters. From laboratory or pilot plant studies.

2.27.3.1 Air-to-solid ratio, A/S.

2.27.3.2 Air pressure, P, psig.

2.27.3.3 Detention time in flotation tank, DTFT, hr.

2.27.3.4 Solids loading, ML, $lb/ft^2/day$.

2.27.3.5 Hydraulic loading, HL, gpm/ft^2.

2.27.3.6 Detention time in pressure tank, DTPT, min.

2.27.3.7 Float concentration, C_F, percent.

2.27.4 <u>Process Design Calculations</u>.

2.27.4.1 Select air-to-solids ratio.

2.27.4.2 Assume air pressure (40 to 60 psig).

2.27.4.3 Calculate P in atmospheres $= \dfrac{\text{psig} + 14.7}{14.7}$

2.27.4.4 Calculate recycle flow.

$$R = \frac{(A/S)(Q)(Co)}{1.3\ S_a(0.5P-1)}$$

where

A/S = air-to-solid ratio.

S_a = air solubility at standard conditions, cc/l.

P = absolute pressure, atmospheres.

R = recycle flow, mgd.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

2.27.4.5 Calculate surface area required.

2.27.4.5.1 Select a solids loading rate and calculate surface area. If no pilot data is available, use the following mass loadings:

With polymer addition: 30 lb/sq ft/day

Without polymer addition: 10 lb/sq ft/day

$$SA = \frac{(Q)(C_o)(8.34)}{ML}$$

where

SA = surface area, ft^2.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

ML = solids loading rate, $lb/ft^2/day$.

2.27.4.5.2 Select a hydraulic loading rate and calculate the surface area.

$$SA = \frac{(Q+R)(10^6)}{(HL)(60)(24)}$$

where

SA = surface area, ft^2.

Q = feed flow, mgd.

R = recycle flow, mgd.

HL = hydraulic loading rate, gpm/ft^2.

2.27.4.5.3 Compare the surface areas calculated and use the larger of the two.

2.27.4.6 Select detention time in the flotation tank and calculate the volume.

$$VOLFT = (Q + R) \times \frac{1}{7.48} \ \frac{1}{24} \ (DTFT)\ (10^6)$$

where

$VOLFT$ = volume of flotation tank, ft^3.

Q = total flow, mgd.

R = recycle flow, mgd.

$DTFT$ = detention time in flotation tank, hr.

2.27.4.7 Select pressure tank detention time and calculate volume of pressure tank.

$$VOLPT = (R)\ \left(\frac{1}{7.48}\right)\ \left(\frac{1}{24}\right)\ \left(\frac{1}{60}\right)\ (DTPT)(10^6)$$

where

$VOLPT$ = volume of pressure tank, ft^3.

R = recycle flow, mgd.

$DTPT$ = detention time in pressure tank, min.

2.27.4.8 Calculate volume of sludge.

$$VS = \frac{(Q)(C_o)(\%\ \text{removal})}{(C_F)(\text{specific gravity})}$$

where

VS = volume of sludge, gpd.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

C_F = solids concentration in float, percent.

2.27.4.9 Calculate polymer usage (if applicable).

$$PU = \frac{(PD)(Q)(C_o)\ 8.34}{2000}$$

where

PU = polymer usage, lb/day.

PD = polymer dosage, lb/ton dry solids (if polymers are used and no dosage rates are given in input, use 10 lb/ton dry solids).

Q = sludge flow, mgd.

C_o = suspended solids concentration in the feed, mg/l.

2.27.4.10 Effluent Characteristics.

2.27.4.10.1 Suspended solids.

$$SSE = (C_o)\ (1.0 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

C_o = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

2.27.4.10.2 BOD.

$$BODE = (BOD)\ (1 - \frac{BODR}{100})$$

If BODE < BODSE set BODE = BODSE

where

$BODE$ = effluent BOD_5 concentration, mg/l.

BOD = influent BOD_5 concentration, mg/l.

$BODSE$ = effluent soluble BOD_5 concentration, mg/l.

$BODR$ = BOD_5 removal rate, %.

2.27.4.10.3 COD.

$$CODE = (COD)\ (1.0 - \frac{CODR}{100})$$

If CODE < CODSE, set CODE = CODSE

where

CODE = effluent COD concentration, mg/l.

COD = influent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

CODR = COD removal rate, %.

2.27.4.10.4 Nitrogen.

$$TKNE = (TKN)\ (1.0 - \frac{TKNR}{100})$$

$$NH3E = TKNE$$

where

TKNE = effluent Kjeldahl nitrogen concentration, mg/l.

TKN = influent Kjeldahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

TKNR = Kjeldahl nitrogen removal rate, %.

2.27.4.10.4 Oil and grease.

$$OAGE = (OAG)\ (0.05)$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

2.27.5 Process Design Output Data.

2.27.5.1 Suspended solids concentration, C_o, mg/l.

2.27.5.2 Air-to-solid ratio, A/S.

2.27.5.3 Air pressure, P, psig.

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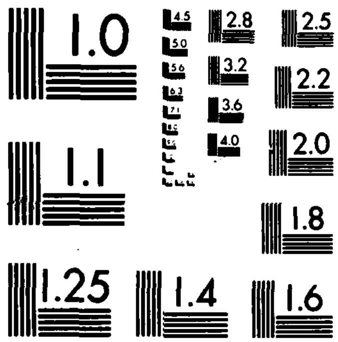

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

2.27.5.4 Solids loading, ML, lb/ft^2/day.

2.27.5.5 Hydraulic loading, HL, gpm/ft^2.

2.27.5.6 Recycle flow, R, mgd.

2.27.5.7 Surface area, SA, ft^2.

2.27.5.8 Volume of pressure tank, VOLPT, ft^3.

2.27.5.9 Volume of flotation tank, VOLFT, ft^3.

2.27.5.10 Pressure tank detention time, DTPT, min.

2.27.5.11 Flotation tank detention time, DTFT, hr.

2.27.5.12 Polymer usage, PU, lb/day.

2.27.5.13 Effluent suspended solids concentration, SSE, mg/l.

2.27.5.14 Effluent BOD_5 concentration, BODE, mg/l.

2.27.5.15 Effluent COD concentration, CODE, mg/l.

2.27.5.16 Effluent Kjeldahl nitrogen concentration, TKNE, mg/l.

2.27.5.17 Effluent oil and grease concentration, OAGE, mg/l.

2.27.6 Quantities Calculations.

2.27.6.1 Select number and size of flotation units. The standard size units available commercially are 40, 50, 70, 100, 140, 200, 280, 350, 450, 570, 750, 960, and 1250 sq ft.

2.27.6.1.1 If SA is less than 1250 sq ft, then NU is one. Compare SA to the commercially available units and select the smallest unit that is larger than SA.

2.27.6.1.2 If SA is greater than 1250 sq ft, then NU must be two or greater. Try NU = 2 first, if SA/NU is greater than 1250, then NU = NU + 1. Repeat the procedure until SA/NU is less than 1250, then compare SA/NU to the commercially available units and select the smallest unit that is larger than SA/NU.

where

SA = calculated surface area, sq ft.

NU = number of units.

SAS = surface area of unit selected.

2.27.6.2 Calculate building area. In area where freezing weather may be expected, flotation units would nomally be enclosed in buildings.

2.27.6.2.1 Calculate diameter of unit.

$$DIA = \left(\frac{4\ SAS}{\pi}\right)^{0.5}$$

where

DIA = diameter of unit selected, ft.

2.27.6.2.2 Calculate building area.

$$A_B = (DIA + 2)\ (DIA + 5)\ (NU)$$

where

A_B = area of building, sq ft.

2.27.6.3 Earthwork required for construction. The procedure to estimate the earthwork requirement is the same as that for circular clarifier.

$$V_{ew} = (1.15)\ NU[0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = earthwork required for construction, cu ft.

1.15 = 15 percent excess volume as safety factor.

NU = number of units.

2.27.6.4 Reinforced concrete quantities.

2.27.6.4.1 Calculate side water depth. The side water depth can be related to the diameter by the following equation:

$$SWD = 6.72 + 0.0476\ (DIA)$$

where

SWD = side water depth, ft.

2.27.6.4.2 Calculate the thickness of the slab. The thickness of the slab can be related to the side water depth by the following equation:

$$t_s = 7.9 + 0.25\ SWD$$

where

t_s = thickness of the slab, inches.

2.27.6.4.3 Calculate the wall thickness. The wall thickness can be related to the side water depth by the following:

$$t_w = 7 + (0.5)\ SWD$$

where

t_w = wall thickness, inches.

2.27.6.4.4 Calculate reinforced concrete slab quantity.

$$V_{cs} = 0.825\ (DIA + 4)^2\ \left(\frac{t_s}{12}\right)\ (NU)$$

where

V_{cs} = quantity of R.C. slab in-place, cu ft.

2.27.6.4.5 Calculate reinforced concrete wall quantity.

$$V_{cw} = (3.14)\ (SWD + 1.0)\ (DIA)\ \left(\frac{t_w}{12}\right)\ (NU)$$

where

V_{cw} = quantity of R.C. wall in-place, cu ft.

2.27.6.4.6 Quantity of concrete for splitter box.

$$V_{cb} = 100\ (NU)^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter box, cu ft.

NU = number of units.

2.27.6.4.7 Total quantity of R.C.

$$\text{wall:}\quad V_{cwt} = V_{cw} + V_{cb}$$

$$\text{slab:}\quad V_{cst} = V_{cs}$$

where

V_{cwt} = total quantity of R.C. wall in-place, cu ft.

V_{cst} = total quantity of R.C. slab in-place, cu ft.

2.27.6.5 Calculate dry solids produced.

$$DSTPD = \frac{(Q)\ (C_o)\ (8.34)}{2000}$$

where

DSTPD = dry solids produced, tpd.

Q = sludge flow, mgd.

C_o = suspended solids concentration in the feed, mg/l.

2.27.6.6 Calculate operational labor.

2.27.6.6.1 If DSTPD is between 0 and 2.3 tpd, the operational labor is calculated by:

$$OMH = 560\ (DSTPD)^{0.4973}$$

2.27.6.6.2 If DSTPD is greater than 2.3 tpd, the operational labor is calculated by:

$$OMH = 496\ (DSTPD)^{0.5092}$$

where

OMH = operation labor, man-hour/yr.

2.27.6.7 Calculate maintenance labor.

2.27.6.7.1 If DSTPD is between 0 and 3.0 tpd, the maintenance labor is calculated by:

$$MMH = 156.0\ (DSTPD)^{0.4176}$$

2.27.6.7.2 If DSTPD is greater than 3.0 tpd, the maintenance labor is calculated by:

$$MMH = 124.0\ (DSTPD)^{0.6429}$$

where

MMH = maintenance labor, man-hour/yr.

2.27.6.8 Calculate electrical energy requirements for operation.

$$KWH = 63{,}000\ (DSTPD)^{0.9422}$$

KWH = electrical energy requirement for operation, kwhr/yr.

2.27.6.9 Operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of total bare construction cost.

$$OMMP = 1\%$$

where

OMMP = percent of air flotation total bare construction cost as operation and maintenance materials costs, percent.

2.27.6.10 Other construction cost items.

2.27.6.10.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

2.27.6.10.2 Other minor cost items such as piping, electrical wiring and conduit, concrete slab for pumps and pressure tanks, etc., would be 15 percent.

2.27.6.10.3 The correction factor would be

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for construction costs.

2.27.7 <u>Quantities Calculations Output Data.</u>

2.27.7.1 Surface area of unit selected, SAS, sq ft.

2.27.7.2 Number of units, NU.

2.27.7.3 Area of building, A_B, sq ft.

2.27.7.4 Earthwork required, V_{ew}, cu ft.

2.27.7.5 Total quantity of R.C. Wall in-place, V_{cwt}, cu ft.

2.27.7.6 Total quantity of R.C. slab in-place, V_{cst}, cu ft.

2.27.7.7 Operational labor, OMH, man-hour/yr.

2.27.7.8 Maintenance labor, MMH, man-hour/yr.

2.27.7.9 Electrical energy requirement for operation, KWH, kw-hr/yr.

2.27.7.10 Operation and maintenance material costs as percent of air flotation total bare construction cost, percent.

2.27.7.11 Correction factor for construction costs, CF.

2.27.8 Unit Price Input Required.

2.27.8.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.27.8.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

2.27.8.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

2.27.8.4 Cost of standard size flotation equipment, COSTFS, \$, (optional).

2.27.8.5 Marshall and Swift Equipment Cost Index, MSECI.

2.27.9 Cost Calculations.

2.27.9.1 Cost of building.

$$COSTB = A_B \times UPIBC\ (.75)$$

where

COSTB = cost of building, \$.

A_B = building area, sq ft.

where

COSTB = cost of building, \$.

A_B = building area, sq ft.

UPIBC = unit price input for building cost, \$/ft^2.

.75 = correction factor since slab is already accounted for in concrete costs.

2.27.9.2 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.27.9.3 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cwt}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cwt} = total quantity of R.C. wall in-place, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.27.9.4 Purchase cost of flotation equipment. The costs given include the basic mechanism to be mounted in the concrete tank, air pressurization tank, pressurization pump, pressure release valve, air injection system, and electrical panel.

$$COSTF = COSTFS \times \frac{COSTRO}{100}$$

where

COSTF = purchase cost of flotation equipment of SAS surface area, \$.

COSTFS = cost of standard size air flotation unit of 350 sq ft, \$.

COSTRO = cost of unit of SAS sq ft expressed as percent of cost of standard size unit.

2.27.9.5 Calculate COSTRO.

2.27.9.5.1 If SAS is less than 240 sq ft, COSTRO is calculated by:

$$COSTRO = 0.3\,(SAS) + 25$$

2.27.9.5.2 If SAS is between 240 sq ft and 480 sq ft, COSTRO is calculated by:

$$COSTRO = 0.092\,(SAS) + 75$$

2.27.9.5.3 If SAS is greater than 480 sq ft, COSTRO is calculated by:

$$COSTRO = 0.161\,(SAS) + 43$$

2.27.9.6 Cost of standard size unit. The cost of a dissolved air flotation unit with 350 sq ft of surface area for the first quarter of 1977 is:

$$COSTFS = \$44,200$$

For better cost estimation, COSTFS should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTFS = \$44,200\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Cost Index.

491.6 = MSECI first quarter 1977.

2.27.9.7 Equipment Installation Costs. These costs would include mounting of a flotation mechanism in the flotation tank, setting pumps and tanks, interconnecting piping, electrical installation, etc. These costs are estimated as 75 percent of the purchase cost of the equipment.

$$EIC = .75\ COSTF$$

where

EIC = equipment installation costs, dollars.

2.27.9.8 Installed equipment cost.

$$IEC = (COSTF + EIC)\ NU$$

where

IEC = installed equipment cost, \$.

NU = number of units of area SAS sq ft.

2.27.9.9 Total bare construction costs.

$$TBCC = (COSTB + COSTE + COSTCW + COSTCS + IEC)\ CF$$

where

TBCC = total bare construction costs, \$.

CF = construction cost correction factor,

2.27.9.10 Operation and maintenance material costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance costs, \$/yr.

2.27.10 Cost Calculations Output Data.

2.27.10.1 Total bare construction cost for dissolved air flotation unit, TBCC, \$.

2.27.10.2 Operation and maintenance material costs, OMMC, \$/yr.

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2.29 INCINERATION

2.29.1 Background.

2.29.1.1 High temperature processes have been used for combustion of municipal wastewater solids since the early 1900's. Popularity of these processes has fluctuated greatly since their adaptation from the industrial combustion field. In the past, combustion of wastewater solids was both practical and inexpensive. Solids were easily dewatered and the fuel required for combustion was cheap and plentiful. In addition, air emission standards were virtually non-existent.

2.29.1.2 In today's environment, wastewater solids are more complex and include sludges from secondary and advanced waste treatment (AWT) processes. These sludges are more difficult to dewater and thereby increase fuel requirements for combustion. Due to environmental concerns with air quality and the energy crisis, the use of high temperature processes for combustion of municipal solids is being scrutinized.

2.29.1.3 However, recent developments in more efficient solids dewatering processes and advances in combustion technology have renewed an interest in the use of high temperature processes for specific applications. High temperature processes should be considered where available land is scarce, stringent requirements for land disposal exist, destruction of toxic materials is required, or the potential exists for recovery of energy, either with wastewater solids alone or combined with municipal refuse.

2.29.1.4 High temperature processes have several potential advantages over other methods:

2.29.1.4.1 Maximum volume reduction. Reduces the volume and weight of wet sludge cake by approximately 95 percent, thereby reducing disposal requirements.

2.29.1.4.2 Detoxification. Destroys or reduces toxics that may otherwise create adverse environmental impacts.

2.29.1.4.3 Energy recovery. Potentially recovers energy through the combustion of waste products, thereby reducing the overall expenditure of energy.

2.29.1.5 Disadvantages of high temperature processes include:

2.29.1.5.1 Cost. Both capital and operation and maintenance costs, including costs for supplemental fuel, are generally higher than for other disposal alternatives.

2.29.1.5.2 Operating problems. High temperature operations create high maintenance requirements and can reduce equipment reliability.

2.29.1.5.3 Staffings. Highly skilled and experienced operators are required for high temperature processes. Municipal salaries and operator status may have to be raised in many locations to attract the proper personnel.

2.29.1.5.4 Environmental impacts. Discharges to atmosphere (particulates and other toxic or noxious emissions), surface waters (scrubbing water), and land (furnace residues) may require extensive treatment to assure protection of the environment.

2.29.1.6 Multiple-hearth and fluid bed furnaces are the most commonly used sludge combustion equipment in the United States, Europe and Great Britain. These two processes will be considered in this section.

2.29.2 General Description Fluidized Bed Incineration.

2.29.2.1 Fluidized bed incineration for wastewater sludges involves the destruction of wastewater solids through combustion. Basically, dewatered sludge is pumped into the incineration vessel containing a heated catalytic bed. This bed is fluidized by a controlled upward airflow at pressures of 2.0 to 5.0 psig; this air also supplies oxygen for combustion. Temperatures for combustion range from 1200° to 1600° F. Supplemental fuel may be added by burners to keep temperatures at optimum levels if the sludge characteristics do not allow for autogenous combustion. Burning of wastewater sludge produces ash and several gases which are carried upward by the flow of air through an exhaust stack. Figure 2.29-1 describes a typical material balance for incineration of 1 lb of dry sludge. Normally, some type of air pollution equipment such as scrubbers, electrostatic precipitators, and cyclones are connected to process the incinerator byproducts. This exhaust may also pass through other control devices if noxious odors are expected to result from combustion.

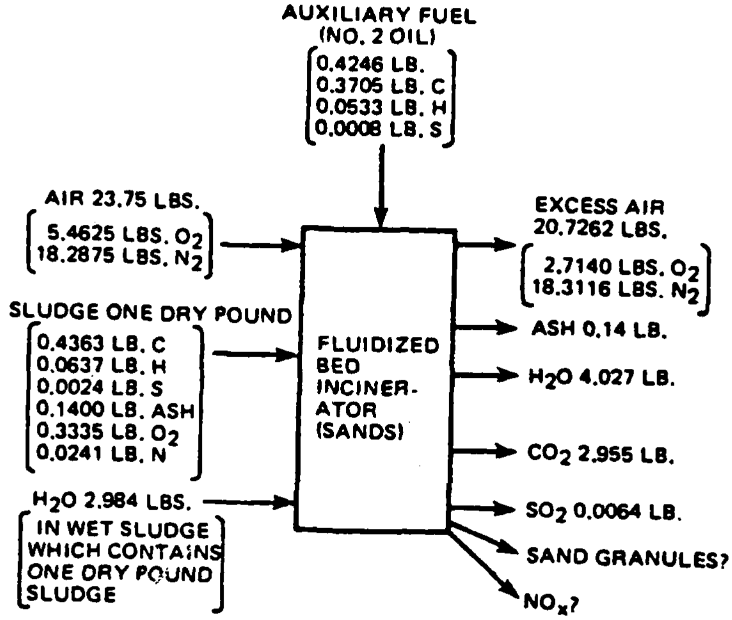

Figure 2.29-1. Material balance for fluidized bed sewage sludge incineration

2.29.2.2 The fluidized bed incinerator used for combustion of wastewater sludge is a vertical cylinder with an air distributor plate containing small openings near the bottom as seen in Figure 2.29-2. The base plant serves two functions: (1) allows air to pass into the media and (2) supports the media. An external air source forces the air into the bottom of the vessel where it is distributed in such a manner as to fluidize the bed and supply oxygen for combustion.

2.29.2.3 The bed material is composed of graded silica sand with size varying from ASTM No. 8 to No. 20. The nomal operating temperatures for these fluidized sand beds are between 1200° and 1600° F, the maximum being 2000° F. At this temperature, the sand approaches its melting point which is detrimental to the incinerator process. Also, damage to the incinerator vessel would be experienced in the heat exchanger and flue piping.

2.29.2.4 There are two possible locations for the sludge feed inlet to be placed on a fluidized bed incinerator vessel. One is positioned so that the sludge is pumped (screw-type) directly into the fluidized bed. The advantage of this configuration lies in the fact that complete combustion is realized in a short time. Yet, problems can be incurred due to clogging from dried sludge. The second location is above the fluidized bed or the freeboard zone.

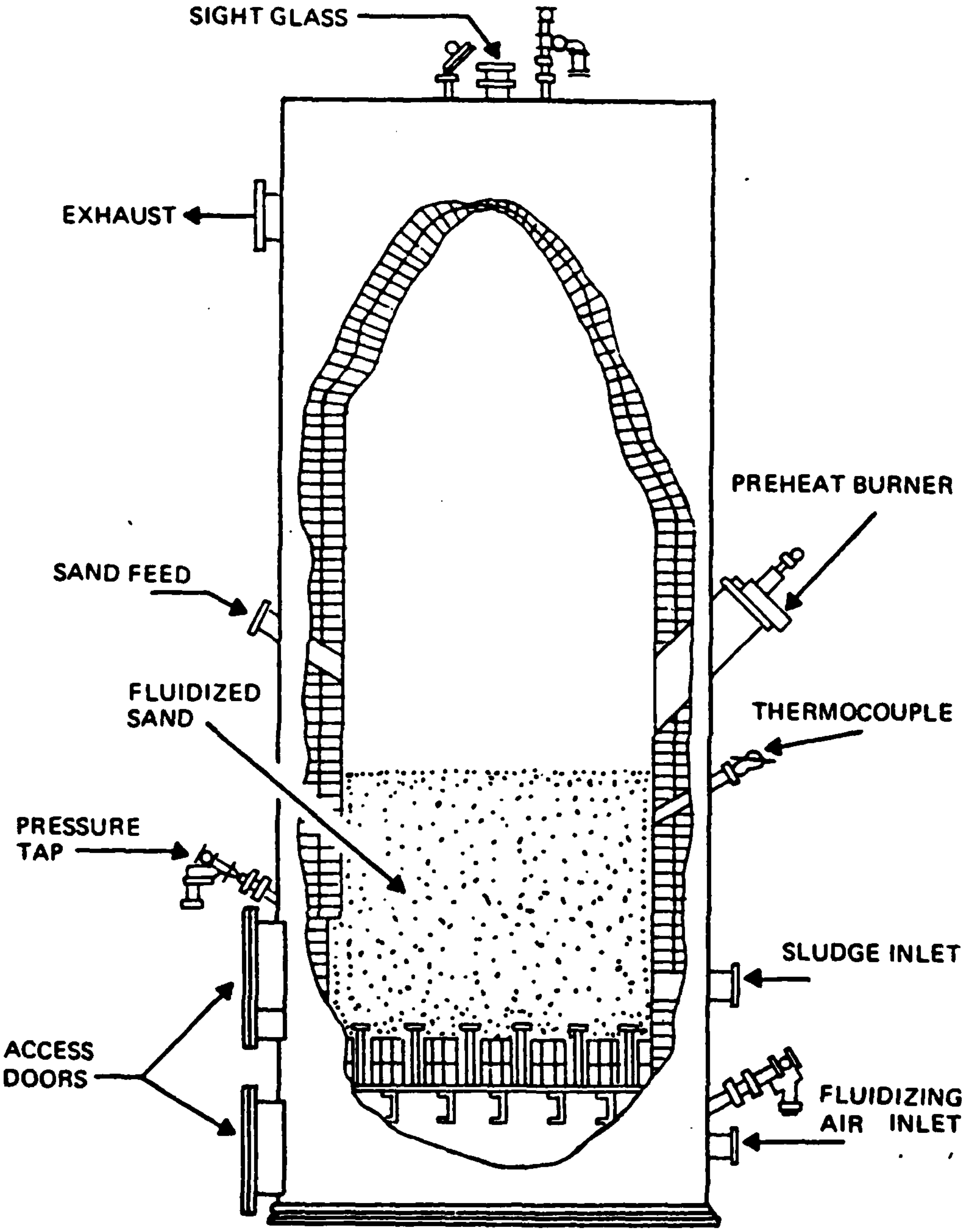

FIGURE 2.29-2. CROSS-SECTION OF A FLUID BED REACTOR

Hot gases evaporate the water in the sludge as the solids enter the vessel. This operation is more amenable to combustion of solids with high moisture contents. However, combustion time for the elevated configuration is increased.

2.29.2.5 Sludge combustion in an incinerator occurs in two zones: Zone 1 (bed) where the principal processes are pyrolysis and combustion and Zone 2 (freeboard) where the principal processes are flame holding and final burnup. Combustible elements contained in sludge are carbon, hydrogen, nitrogen, and sulfur, which when completely burned with oxygen, form the combustion products CO_2, H_2O, NO_x, and SO_2, respectively. Ash is also generated in the process of incinerating sludge. The NO_x, SO_2, and particulate ash can be classified as major air pollutants. In order to prevent the release of these by-products into the atmosphere, all fluidized bed incinerators are equipped with scrubbers of varying efficiency. These units have been found to be quite effective.

2.29.2.6 In some cases, an air preheater or heat exchanger can be used in conjunction with a fluidized bed as seen in Figure 2.29-2. The function of the preheater is to raise the temperature of the incoming air to 1000° F by mixing the cool air at 70° F with the exhaust gas at 1500° F.

2.29.2.7 It should be noted that fluidized bed incinerators are very specialized equipment and are not usually designed by general consultants and not installed by the general contractor for the sewage treatment facility. The incinerator is usually obtained on a turnkey basis from the manufacturer; that is, the manufacturer designs and installs all the equipment required for incineration of the sludge. The only work done by the general contractor would be construction of the foundation and building to house the incinerator. Normally the dewatering equipment is furnished and installed by the incinerator manufacturer, but it is not included here as it has been provided for in a separate section of this report.

2.29.2.8 Much of the information concerning sizing and design of fluidized bed incinerators is confidential since it has been developed by the manufacturers. It is difficult to relate any one design parameter to the cost, because of the difference in design from manufacturer to manufacturer. We have chosen to use the diameter of the unit to relate to the cost.

2.29.3 <u>General Description Multiple-Hearth Incineration.</u>

2.29.3.1 The multiple-hearth furnace is the most widely used wastewater sludge incinerator in use today because it is simple to operate, durable and capable of burning a wide variety of materials. Reasonable fluctuations in the feed rate may be accommodated without interruption of the incineration process. Figure 2.29-3 represents a typical cross section of a multiple-hearth incinerator. Sludge from

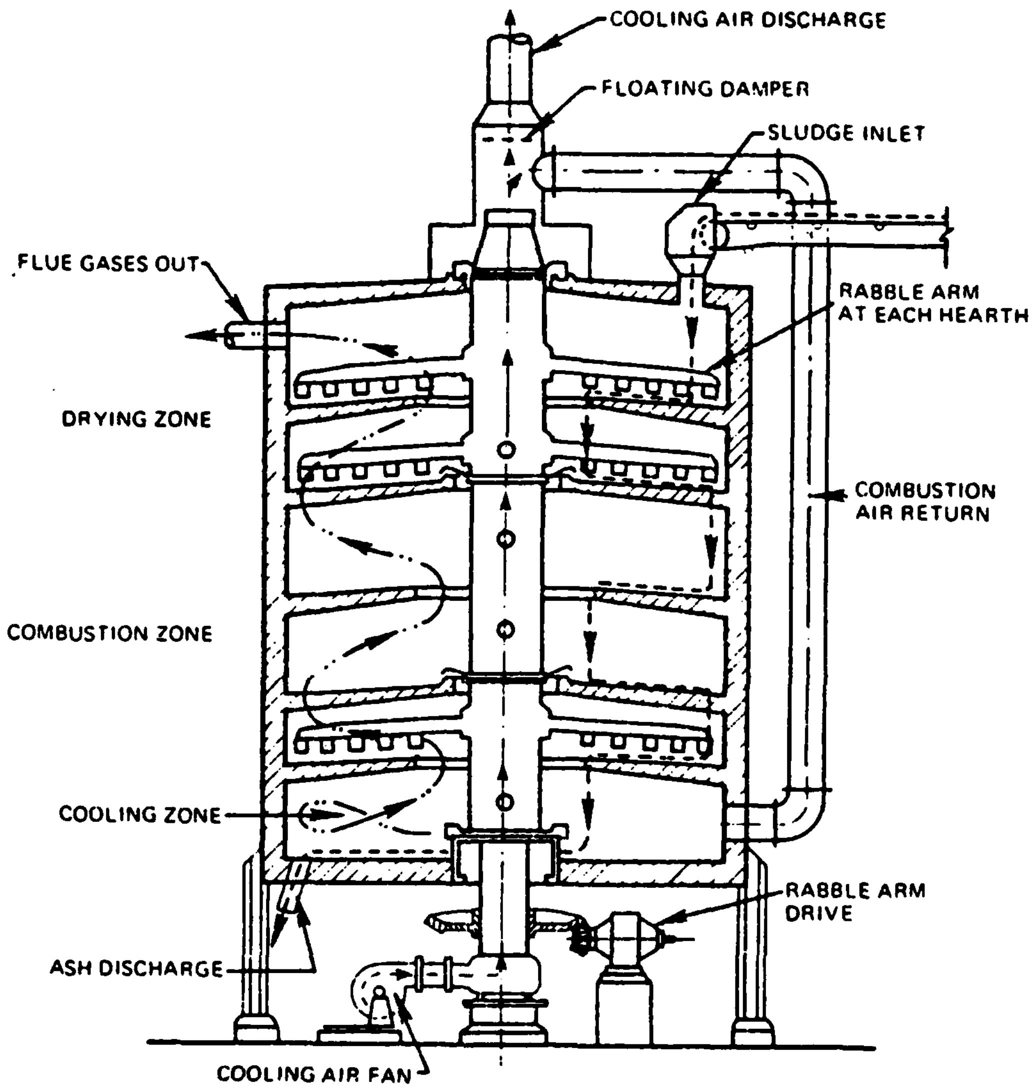

FIGURE 2.29-3. TYPICAL MULTIPLE-HEARTH FURNACE

water or wastewater treatment is normally thickened and dewatered by vacuum filtration and/or centrifugation. The dewatered sludge enters the multiple-hearth furnace at the top and is held first on the top hearth. The sludge is stirred constantly to promote drying and burning by rabble arms. These slow-moving arms move the sludge across the hearths to the inner or outer edge where it drops to the hearth beneath. This process continues until the sludge reaches the bottom of the furnace as ash.

2.29.3.2 The multiple-hearth incinerator, like the fluidized bed incinerator, is a special item of equipment. It is generally provided by the manufacturer on a turnkey basis. All necessary equipment, installation, and start-up are provided by the manufacturer. The only thing provided by the general contractor are the concrete foudation and a building, if required. Normally dewatering equipment is supplied by the manufacturer but it has been omitted here, since it is provided for in another part of this report.

2.29.4 <u>Fluidized Bed Incinerator</u>.

2.29.4.1 <u>Input Data</u>.

2.29.4.1.1 Average flow, Q_{avg}, mgd.

2.29.4.1.2 Sludge volume, gal/million gal.

2.29.4.1.3 Sludge solids concentration, percent.

2.29.4.1.4 Moisture content of dewatered sludge, percent.

2.29.4.1.5 Work schedule, hr/day.

2.29.4.1.6 Sludge analysis.

2.29.4.1.6.1 Carbon content, percent.

2.29.4.1.6.2 Hydrogen content, percent.

2.29.4.1.6.3 Oxygen content, percent.

2.29.4.1.6.4 Sulfur content, percent.

2.29.4.1.7 Heat value of fuel.

2.29.4.1.8 Fuel analysis, for fuel oil.

2.29.4.1.8.1 Carbon content, percent.

2.29.4.1.8.2 Hydrogen content, percent.

2.29.4.1.8.3 Sulfur content, percent.

2.29.4.1.9 Operating temperature of preheater, °F.

2.29.4.1.10 Ambient air temperature, °F.

2.29.4.1.11 Sand-to-sludge ratio.

2.29.4.1.12 Specific weight of sand, lb/ft^3.

2.29.4.1.13 Volatile solid content of sludge, percent.

2.29.4.1.14 Fuel cost, $ per million BTU's.

2.29.4.2 <u>Design Parameters</u>.

2.29.4.2.1 Sludge solids concentration, 1.5 to 5 percent.

2.29.4.2.2 Moisture content of dewatered sludge, 40 to 96 percent.

2.29.4.2.3 Sludge analysis. Use laboratory values if known; otherwise use the following.

2.29.4.2.3.1 Carbon content, 43.6 percent.

2.29.4.2.3.2 Hydrogen content, 6.4 percent.

2.29.4.2.3.3 Oxygen content, 33.4 percent.

2.29.4.2.3.4 Sulfur content, 0.3 percent.

2.29.4.2.4 Heat value of fuel oil, 18,000 BTU/lb.

2.29.4.2.5 Fuel analysis. Use reported values, or for fuel oil use the following.

2.29.4.2.5.1 Carbon content, 87.3 percent.

2.29.4.2.5.2 Hydrogen content, 12.6 percent.

2.29.4.2.5.3 Oxygen content, 0 percent.

2.29.4.2.5.4 Sulfur content, 1.0 percent.

2.29.4.2.6 Operating temperature of preheater, 1000° to 1200° F.

2.29.4.2.7 Incinerator retention time, 10 to 50 sec.

2.29.4.2.8 Ratio of incinerator height:diameter, 4:1 to 6:1.

2.29.4.2.9 Heat release rate, < 50,000 BTU/hr/ft.

2.29.4.2.10 Sand-to-sludge ratio, 3 to 8 lb/lb/hr.

2.29.4.2.11 Specific weight of sand, 110 lb/ft^3.

2.29.4.2.12 Grid jet velocity, > 300 fps.

2.29.4.2.13 Volatile solids content of sludge, percent. Use reported values, or 40 to 60 percent volatile.

2.29.4.2.14 Fuel cost, $ per million BTU.

2.29.4.3 <u>Process Design Calculations</u>.

2.29.4.3.1 Determine the amount of sludge to be incinerated, lb/day.

$$SP = \frac{(Q_{avg})(SF)(SS)(SCAP)(8.34)}{(100)(100)}$$

where

SP = dry sludge produced per day, lb.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gal/million gal.

SS = suspended solids in sludge, percent.

SCAP = solids capture, percent.

2.29.4.3.2 Calculate the sludge hourly loading rate.

$$LR = \frac{SP}{HPD}$$

where

LR = dry sludge loading rate, lb/hr.

HPD = work schedule, hr/day.

2.29.4.3.3 Calculate sludge heat value.

$$BS = 145C_1 + 620 \quad H_1 - \frac{O_1}{7.94} \quad + 45S_1$$

where

BS = sludge heat value, BTU/lb.

C_1 = carbon in sludge, percent (if unknown, use 43.6).

H_1 = hydrogen in sludge, percent (if unknown, use 6.4).

O_1 = oxygen in sludge, percent (if unknown, use 33.4).

S_1 = sulfur in sludge, percent (if unknown, use 0.3).

2.29.4.3.4 Calculate sludge loading rate.

$$SL = 10^{(2.7-0.0222M)}$$

where

SL = sludge loading rate, lb/ft^2/hr.

M = moisture content of dewatered sludge, percent.

2.29.4.3.5 Calculate cross-sectional area of incinerator.

$$A = \frac{LR}{SL}$$

where

A = area of incinerator, ft^2.

2.29.4.3.6 Calculate diameter of incinerator.

$$D = (1.273A)^{0.5}$$

where

D = diameter, ft.

2.29.4.3.7 Compute auxiliary fuel supply.

2.29.4.3.7.1 Calculate burning rate.

$$BR = 10^{(5.947-0.0096M)}$$

where

BR = burning rate, BTU/ft^2/hr.

2.29.4.3.7.2 Compute total heat input rate.

$$HIR = (BR)(A)$$

where

HIR = total heat input rate, BTU/hr.

2.29.4.3.7.3 Calculate heat input from sludge.

$$HIS = (BS)(LR)$$

where

HIS = heat input from sludge, BTU/hr.

2.29.4.3.7.4 Calculate auxiliary fuel supply.

$$AFS = HIR - HIS$$

where

AFS = auxiliary fuel supply, BTU/hr.

2.29.4.3.7.5 Calculate fuel oil required.

$$FO = \frac{AFS}{HV}$$

where

FO = fuel oil required, lb/hr.

HV = heat value of fuel.

2.29.4.3.8 Compute air supply rate (20 percent excess air).

2.29.4.3.8.1 Calculate air supply rate for sludge.

$$q_1 = 0.0127[(LR)(2.67C_1 + 7.94H_1 + S_1 - O_1)]$$

where

q_1 = air supply rate for sludge, scfm.

2.29.4.3.8.2 Calculate air supply rate for fuel.

$$q_2 = 0.0127[(FO)(2.67C_2 + 7.94H_2 + S_2 - O_2)]$$

where

q_2 = air supply rate for fuel, scfm.

C_2 = carbon in fuel, percent (if unknown, use 87.3).

H_2 = hydrogen in fuel, percent (if unknown, use 12.6).

S_2 = sulfur in fuel, percent (if unknown, use 1.0).

O_2 = oxygen in fuel, percent (if unknown, use 0.0).

2.29.4.3.8.3 Calculate air supply rate.

$$q = q_1 + q_2$$

where

q = total dry air supply, scfm.

2.29.4.3.9 Calculate total gas flow (air plus water).

$$q_t = q + \frac{LR}{100 - M} \quad \frac{M}{3.01}$$

where

q_t = total gas flow, scfm.

2.29.4.3.10 Calculate air preheater capacity.

2.29.4.3.10.1 Calculate the air-to-sludge ratio.

$$ASR = \frac{4.50q}{LR}$$

where

ASR = air-to-sludge ratio, lb air/lb sludge.

2.29.4.3.10.2 Calculate air preheater capacity.

$$APHS = (0.24)(ASR)(T_2 - T_1)(LR)$$

where

APHS = air preheater capacity, BTU/hr.

T_2 = operating temperature, °F (1000° to 1200° F).

T_1 = incoming air temperature, °F.

2.29.4.3.11 Assume a retention time for the incinerator.

$$10 \leq dt \leq 50 \text{ sec}$$

where

dt = retention time, sec.

2.29.4.3.12 Determine volume of reactor.

$$V = q_t \frac{dt}{60}$$

where

V = volume of reactor, ft^3.

2.29.4.3.13 Calculate the height of the reactor. (4 H/D 6).

$$H = \frac{V}{A}$$

where

H = height of reactor, ft.

(If H/D does not fall between 4 and 6, adjust dt and make a new determination of the volume, V).

2.29.4.3.14 Check heat release rate. (hr $\leq$ 50,000 BTU/hr/ft).

$$Hr = \frac{HIR}{V}$$

where

Hr = heat release rate, BTU/hr/ft^3.

2.29.4.3.15 Assume a sand-to-sludge ratio. Use 3 to 8 lb sand/lb sludge/hr for ASTM No. 8 sand.

$$3 \leq Rss \leq 8 \text{ lb/lb/hr}$$

where

Rss = ratio of sand to sludge.

2.29.4.3.16 Calculate the depth of ASTM No. 8 silica sand required.

$$L = \frac{(12)(LR)(Rss)}{(\ _s)(A)}$$

where

L = depth of sand, in.

$_s$ = specific weight of sand, lb/cu ft (approx 110).

2.29.4.3.17 Calculate the grid jet velocity.

$$V_j = 160 + 160 \log L$$

where

V_j = grid jet velocity, fps.

2.29.4.4 Process Design Output Data.

2.29.4.4.1 Dry sludge loading rate, lb/hr.

2.29.4.4.2 Sludge heat value, BTU/lb.

2.29.4.4.3 Sludge loading rate, lb/ft^2/hr.

2.29.4.4.4 Cross-sectional area of incinerator, ft^2.

2.29.4.4.5 Diameter of incinerator, ft.

2.29.4.4.6 Burning rate, BTU/ft^2/hr.

2.29.4.4.7 Total heat input rate, BTU/hr.

2.29.4.4.8 Heat input from sludge, BTU/hr.

2.29.4.4.9 Auxiliary fuel supply, BTU/hr.

2.29.4.4.10 Fuel oil required, lb/hr.

2.29.4.4.11 Total dry air supply, scfm.

2.29.4.4.12 Total gas flow, scfm.

2.29.4.4.13 Air preheater capacity, BTU/hr.

2.29.4.4.14 Volume of reactor, ft^3.

2.29.4.4.15 Height of reactor, ft.

2.29.4.4.16 Heat release rate, BTU/ft/hr.

2.29.4.4.17 Depth of sand, in.

2.29.4.4.18 Grid jet velocity, fps.

2.29.4.5 Quantities Calculations.

2.29.4.5.1 Determine size and number of incinerators to be used.

2.29.4.5.1.1 It was determined from manufacturers that units less than 6 ft in diameter are not normally built. For this reason, if the diameter calculated is less than 6 ft, use a 6-ft diameter unit.

2.29.4.5.1.2 The size of commercially available units begins at 6-ft diameter and increases in 1-ft increments to the largest diameter of 25 ft.

Table 2.29-1. Normal Quantities of Sludge Produced by Different Treatment Processes

Wastewater Treatment Process	Gallons Sludge/ mg Treated	Solids Percent	Sludge Specific Gravity
Primary sedimentation			
Undigested	2,950	5.0	1.02
Digested in separate tanks	1,450	6.0	1.03
Trickling filter	745	7.5	1.025
Chemical precipitation	5,120	7.5	1.03
Primary sedimentation and activated sludge			
Undigested	6,900	4.0	1.02
Digested in separate tanks	2,700	6.0	1.03
Activated sludge			
Waste sludge	19,400	1.5	1.005
Septic tanks, digested	900	10.0	1.04
Imhoff tanks, digested	500	15.0	1.04

Table 2.29-2. Process Efficiencies for Dewatering of Wastewater Sludge

Unit Process	Solids Capture, Percent	Cake Solids, Percents
Centrifugation		
Solid bowl	80-90	5-13
Disc-nozzle	80-97	5-7
Basket	70-90	9-10
Dissolved air flotation	95	4-6
Dry beds	85-99	8-25
Filter press	99	40-60
Gravity thickener	90-95	5-12
Vacuum filter	90+	28-35

Table 2.29-3. Recommended Fluidized Bed Sludge
Incinerator Design Criteria

Parameter	Magnitude
Sludge loading rate*	5 to 40 lb dry sludge/hr/ft^2
Burning rate	Up to 300,000 BTU/hr/ft^2
Heat release rate	Up to 50,000 BTU/hr/ft^3
Air:dry sludge ratio	6 to 50 lb/lb
Fuel: dry sludge ratio*	0 to 30,000 BTU/lb
Combustion temperature	1200 to 1400°F
Retention time (at standard conditions)	10 time 50 sec
Pressure drop	50 to 100 in of water
Height:diameter ratio	4 to 6:1
Grid jet velocity	Greater than 300 fps
Freeboard gas velocity	5 to 10 fps
Stack gas velocity	20 to 50 fps
Ratio of freeboard diameter:grid diameter	1.5
Ratio of No. 8 sand to sludge rate**	3 to 8 lb/hr/lb

*Sludge loading rates and air and fuel requirements depend on sludge moisture content and the ratio of air and sludge.
**Sand to sludge ratios depend on sludge moisture content.

2.29.4.5.1.3 If the diameter calculated is between 6 ft and 25 ft, the number of units will be one. If the diameter is not an integer, the diameter will be equal to the next larger integer.

2.29.4.5.1.4 If the diameter is greater than 25 ft, then multiple units must be used and the diameter is calculated by:

$$D = 1.273\ (A/N)^{.5}$$

Try N = 2 first, if A/N is greater than 490 sq ft; then try N = N + 1 and repeat until A/N is less than 490 sq ft.

where

D = diameter of incinerator, ft.

A = area of incinerator, sq ft.

N = number of incinerators.

2.29.4.5.2 Calculate the area of incinerator building. The area of the building to house the incinerator and associated equipment was taken from manufacturer's literature and recommendations.

2.29.4.5.2.1 If the incinerator diameter (D) is between 6 ft and 12 ft, the area of the building is calculated by:

$$A_B = [2140 + 28\ D]\ N$$

2.29.4.5.2.2 If the incinerator diameter (D) is between 13 ft and 15 ft, the area of the building is calculated by:

$$A_B = [1665 + 90\ D]\ N$$

2.29.4.5.2.3 If the incinerator diameter (D) is between 16 ft and 25 ft, the area of the building is calculated by:

$$A_B = [2930 + 40.3\ D]\ N$$

where

A_B = area of incinerator building, sq ft.

D = diameter of incinerator, ft.

N = number of incinerators.

2.29.4.5.3 Calculate volume of concrete foundations.

$$V_{cs} = A_B \times 1.5$$

where

V_{cs} = volume of reinforced concrete required for foundations, cu ft.

2.29.4.5.4 Calculate maintenance labor per year.

2.29.4.5.4.1 If PDSPD is between 0 lb/day and 5400 lb/day, the maintenance labor per year is calculated by:

$$MMH = 38.34\ (PDSPD)^{0.3653}$$

2.29.4.5.4.2 If PDSPD is between 5400 lb/day and 44,000 lb/day, the maintenance labor per year is calculated by:

$$MMH = 6.02\ (PDSPD)^{0.5783}$$

2.29.4.5.4.3 If PDSPD is greater than 44,000 lb/day, the maintenance labor is calculated by:

$$MMH = 0.194\ (PDSPD)^{0.9021}$$

where

PDSPD = pounds of dry sludge produced per day, lb/day.

MMH = maintenance labor required per year, man-hours/yr.

2.29.4.5.5 Calculate operational labor per year.

2.29.4.5.5.1 If PDSPD is between 0 lb/day and 6200 lb/day, the operational labor is calculated by:

$$OMH = 62.4\ (PDSPD)^{0.4014}$$

2.29.4.5.5.2 If PDSPD is between 6200 lb/day and 46,000 lb/day, the operational labor is calculated by:

$$OMH = 17.968\ (PDSPD)^{0.5483}$$

2.29.4.5.5.3 If PDSPD is greater than 46,000 lb/day, the operational labor is calculated by:

$$OMH = .24\ (PDSPD)^{0.9431}$$

where

PDSPD = pounds of dry sludge produced per day, lb/day.

OMH = operational labor required per year, man-hours/yr.

2.29.4.5.6 Calculate electrical energy required per year.

2.29.4.5.6.1 Electrical energy is required for motors which power fluidizing air blower, sludge pumps, scrubber recycle pump, air blower for preheater burner, ash thickener driver, and ash dewatering driver. From data furnished by manufacturers, the total horsepower required can be related to the incinerator diameter by the following:

$$HP = 1.165\ (D)^{1.9115}$$

where

HP = operating horsepower required, hp.

2.29.4.5.6.2 The operating horsepower is related to electrical energy usage by the following equation:

$$KWH = (N)\ (260)\ (.85)\ (HPD)\ (HP)$$

where

KWH = electrical energy requirement per year, kw hr/yr.

N = number of incinerators.

260 = days per year of operation, days/yr.

.877 = conversion from horsepower to kilowatts.

HPD = work schedule, hr/day.

2.29.4.5.7 Operation and maintenance material costs. This item covers the cost of replacement parts, replacing sand lost from the bed, and replacement of insulation. It is expressed as a percent of the total bare construction cost of the incinerator:

$$OMMP = 0.45\%$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.29.4.6 Quantities Calculations Output Data.

2.29.4.6.1 Diameter of incinerator, D, ft.

2.29.4.6.2 Number of incinerators, N.

2.29.4.6.3 Area of incinerator building, A_B, sq ft.

2.29.4.6.4 Volume of reinforced concrete required for foundation, V_{cs}, cu ft.

2.29.4.6.5 Maintenance labor required per year, MMH, man-hours/yr.

2.29.4.6.6 Operational labor required per year, OMH, man-hours/yr.

2.29.4.6.7 Electrical energy required per year, KWH, kw hr/yr.

2.29.4.6.8 Operation and maintenance material costs as percent of total bare construction cost, OMMP, percent.

2.29.4.7 Unit Price Input Required.

2.29.4.7.1 Unit price input for concrete slab, UPICS, $/cu yd.

2.29.4.7.2 Unit price input for building costs, UPIBC, $/sq ft.

2.29.4.7.3 Current Marshall and Swift Equipment Cost Index, MSECI.

2.29.4.7.4 Standard size incinerator, COSTFI, $ (optional).

2.29.4.8 Cost Calculations.

2.29.4.8.1 Calculate installed equipment costs. Fluidized bed incinerators are usually procured on a turnkey basis from the manufacturer. The costs used here include all equipment required except dewatering equipment. The costs also include installation and start-up. This is the usual way in which manufacturers quote costs for fluidized bed incinerators.

2.29.4.8.1.1 If N is equal to 1, the equipment cost is:

$$COSTFB = .122\ (D)^{0.7788}\ (COSTFI)$$

2.29.4.8.1.2 If N is greater than 1, the equipment cost is:

$$COSTFB = .122\ (D)^{0.7788}\ (.9)\ (N)\ (COSTFI)$$

where

COSTFB = cost of fluidized bed incinerator, $.

D = diameter of incinerator, ft.

N = number of incinerators.

COSTFI = cost of standard size (15-ft diameter) fluidized bed incinerator, $.

0.9 = discount when more than one unit is purchased at the same time.

2.29.4.8.1.3 Cost of standard size fluidized bed incinerator. The approximate cost of a 15-ft diameter incinerator for the first quarter of 1977 is:

$$COSTFI = \$1,100,000$$

For a better estimation, COSTFI should be obtained from the equipment manufacturer and treated as a unit price input. If COSTFI is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTFI = \$1,100,000 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index First quarter 1967.

2.29.4.8.2 Calculate cost of foundation.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of reinforced concrete for foundation, \$.

V_{cs} = volume of reinforced concrete required for foundation, cu ft.

UPICS = unit price input for reinforced concrete slab, \$/cu yd.

2.29.4.8.3 Cost of incinerator building.

$$COSTIB = A_B \times UPIBC$$

where

COSTIB = cost of incinerator building, \$.

A_B = area of incinerator building, sq ft.

UPIBC = unit price input building cost, \$/sq ft.

2.29.4.8.4 Calculate total bare construction cost.

$$TBCC = COSTFB + COSTCS + COSTIB$$

where

TBCC = total bare construction cost, \$.

2.29.4.8.5 Calculate operation and maintenance material costs.

$$OMMC = (TBCC) \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

2.29.4.9 Cost Calculations Output Data.

2.29.4.9.1 Total bare construction cost, \$.

2.29.4.9.2 Operation and maintenance material costs, \$/yr.

2.29.5 Multiple-Hearth Incineration.

2.29.5.1 Input Data.

2.29.5.1.1 Average wastewater flow, mgd.

2.29.5.1.2 Sludge volume, gal/million gal.

2.29.5.1.3 Raw sludge concentration, percent solids.

2.29.5.1.4 Dewatered sludge concentration, percent solids.

2.29.5.1.5 Wet sludge loading rate, $lb/hr/ft^2$.

2.29.5.2 Design Parameters.

2.29.5.2.1 Sludge volume per million gal treated.

2.29.5.2.2 Raw sludge concentration, percent solids, 1.5-15 percent.

2.29.5.2.3 Dewatered sludge concentration, percent solids, 4-60 percent.

2.29.5.2.4 Design multiple-hearth furnace wet sludge loading rate, 7-12 $lb/hr/ft^2$ at 20-25 percent total solids.

2.29.5.3 Process Design Calculations.

2.29.5.3.1 Calculate the pounds of dry solids in sludge flow per day.

$$SP = \frac{(Q_{avg})(SF)(SS)(SCAP)(8.34)}{(100)(100)}$$

where

SP = dry solids produced per day, lb.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gal/million gal (Table 2.29-4).

SS = suspended solids flow to dewatering process, percent.

SCAP = solids capture, percent (Table 2.29-5).

2.29.5.3.2 Calculate the dry solids loading rate.

$$LR = \frac{(WSLR)(PSTMHF)(24)}{100}$$

Table 2.29-4. Nomal Quantities of Sludge Produced by Different Treatment Processes

Wastewater Treatment Process	Gallons Sludge/ mg Treated	Solids Percent	Sludge Specific Gravity
Primary sedimentation			
Undigested	2,950	5.0	1.02
Digested in separate tanks	1,450	6.0	1.03
Trickling filter	745	7.5	1.025
Chemical precipitation	5,120	7.5	1.03
Primary sedimentation and activated sludge			
Undigested	6,900	4.0	1.02
Digested in separate tanks	2,700	6.0	1.03
Activated sludge			
Waste sludge	19,400	1.5	1.005
Septic tanks, digested	900	10.0	1.04
Imhoff tanks, digested	500	15.0	1.04

Table 2.29-5. Process Efficiencies for Dewatering of Wastewater Sludge

Unit Process	Solids Capture, percent	Cake Solids, percent
Centrifugation		
Solid bowl	80-90	5-13
Disc-nozzle	80-97	5-7
Basket	70-90	9-10
Dissolved air flotation	95	4-6
Drying beds	85-99	8-25
Filter press	99	40-60
Gravity thickener	90-95	5-12
Vacuum filter	90+	28-35

where

LR = dry solids loading rate, lb/day/ft^2.

$WSLR$ = wet sludge loading rate, lb/hr/ft^2.

$PSTMHF$ = percent solids in sludge to multiple-hearth furnace.

2.29.5.3.3 Calculate total hearth area requirement.

$$SA = \frac{SP}{LR}$$

where

SA = required total hearth area, ft^2.

2.29.5.3.4 From Table 2.29-6, select the next larger standard size of multiple-hearth furnace and enter data for final design.

$$THA = xxxx.x\ ft^2$$

$$O.D. = xx.xx\ ft$$

$$\text{No. of hearths} = xx$$

$$\text{No of furnaces} = x$$

where

THA = total hearth area furnished.

2.29.5.3.5 Calculate combustion air blower horsepower required.

$$CBHP = \frac{(0.15)(WSLR)(THA)24}{2000}$$

where

$CBHP$ = combustion air blower horsepower required.

0.15 = horsepower required per ton of wet sludge per day at 1-psig pressure.

2.29.5.3.6 Calculate combustion air blower SCFM supplied.

$$CBSCFM = \frac{(22.5)(WSLR)(THA)24}{2000}$$

where

$CBSCFM$ = combustion air blower SCFM supplied.

Table 2.29-6. Standard Sizes of Multiple-Hearth Furnace Units

No. of Hearths	6 to 12
Wall Thickness, in.	13.5
Outer Diameter (O.D.), ft	6.75 to 22.25
Effective Hearth Area, ft^2	85 to 3120

THA Ft^2	O.D. ft	No. Hearths	THA Ft^2	O.D. ft	No. Hearths
85	6.75	6	988	16.75	7
98	6.75	7	1041	14.25	11
112	6.75	8	1068	18.75	6
125	7.75	6	1117	16.75	8
126	6.75	9	1128	14.25	12
140	6.75	10	1249	18.75	7
145	7.75	7	1260	16.75	9
166	7.75	8	1268	20.25	6
187	7.75	9	1400	16.75	10
193	9.25	6	1410	18.75	8
208	7.75	10	1483	20.25	7
225	9.25	7	1540	16.75	11
256	9.25	8	1580	22.25	6
276	10.75	6	1591	18.75	9
288	9.25	9	1660	20.25	8
319	9.25	10	1675	16.75	12
323	10.75	7	1752	18.75	10
351	9.25	11	1849	22.25	7
364	10.75	8	1875	20.25	9
383	9.25	12	1933	18.75	11
411	10.75	9	2060	20.25	10
452	10.75	10	2084	22.25	8
510	10.75	11	2090	18.75	12
560	10.75	12	2275	20.25	11
575	14.25	6	2350	22.25	9
672	14.25	7	2464	20.25	12
760	14.25	8	2600	22.25	10
845	16.75	6	2860	22.25	11
857	14.25	9	3120	22.25	12
944	14.25	10			

22.5 = SCFM required per ton of wet sludge per day at 1-psig pressure.

2.29.5.3.7 Calculate cooling air fan horsepower requirements.

$$CLHP = \frac{(0.08)(WSLR)(THA)24}{2000}$$

where

CLHP = cooling air fan horsepower required.

0.08 = horsepower required per ton wet sludge per day at 8-in. water static pressure.

2.29.5.3.8 Calculate cooling air fan SCFM supplied.

$$CLSCFM = \frac{(36.0)(WSLR)(THA)24}{2000}$$

where

CLSCFM = cooling air fan SCFM supplied.

36.0 = SCFM required per wet ton sludge per day at 8-in. water static pressure.

2.29.5.4 Process Design Output Data.

2.29.5.4.1 Raw sludge concentration, percent solids.

2.29.5.4.2 Dewatered sludge concentration, percent solids.

2.29.5.4.3 Dry solids produced per day, lb.

2.29.5.4.4 Dry solids loading rate, lb/hr/ft^2.

2.29.5.4.5 Required total hearth area, ft^2.

2.29.5.4.6 Total hearth area furnished, ft^2.

2.29.5.4.7 Outside diameter, ft.

2.29.5.4.8 Number of hearths.

2.29.5.4.9 Number of furnaces.

2.29.5.4.10 Combustion air blower horsepower required, hp.

2.29.5.4.11 Combustion air blower SCFM supplied, SCFM.

2.29.5.4.12 Cooling air fan horsepower required, hp.

2.29.5.4.13 Cooling air fan SCFM supplied, SCFM.

2.29.5.5 Quantities Calculations

2.29.5.5.1 Calculate building area. In the colder climates the incinerators are usually housed in a building for operator comfort and to reduce heat loss. The floor area of this housing can be estimated by the following equation:

$$A_B = (2 \cdot OD)^2 \ (N)$$

where

A_B = incinerator building area, sq ft.

OD = outside diameter of incinerator selected, ft.

N = number of incinerators.

2.29.5.5.2 Calculate reinforced concrete for foundation.

$$V_{cs} = 2 \quad (OD + 4)^2 \cdot N$$

where

V_{cs} = volume of reinforced concrete required for slab, cu ft.

2 = thickness of concrete slab, ft.

2.29.5.5.3 Calculate dry solids produced.

$$DSTPD = \frac{PDSPD}{2000}$$

where

DSTPD = dry solids produced, tpd.

PDSPD = dry solids produced, lb/day.

2.29.5.5.4 Calculate operational labor.

2.29.5.5.4.1 If DSTPD is between 0 and 3.1 tpd, then operational labor is calculated by:

$$OMH = 1320 \ (DSTPD)^{0.4014}$$

2.29.5.5.4.2 If DSTPD is between 3.1 and 23.0 tpd, then operational labor is calculated by:

$$OMH = 1160 \ (DSTPD)^{0.5483}$$

2.29.5.5.4.3 If DSTPD is greater than 23.0 tpd, then operational labor is calculated by:

$$OMH = 319\ (DSTPD)^{0.9431}$$

where

OMH = operational labor, man-hours/yr.

2.29.5.5.5 Calculate maintenance labor.

2.29.5.5.5.1 If DSTPD is between 0 and 2.7 tpd, then maintenance labor is calculated by:

$$MMH = 616\ (DSTPD)^{0.3653}$$

2.29.5.5.5.2 If DSTPD is between 2.7 and 22.0 tpd, then maintenance labor is calculated by:

$$MMH = 488.0\ (DSTPD)^{0.5783}$$

2.29.5.5.5.3 If DSTPD is greater than 22.0 tpd, then maintenance labor is calculated by:

$$MMH = 184.0\ (DSTPD)^{0.9021}$$

where

MMH = maintenance labor, man-hours/yr.

2.29.5.5.6 Calculate electrical energy required for operation.

2.29.5.5.6.1 If DSTPD is between 0 and 10.0 tpd, then electrical energy requirement is calculated by:

$$KWH = 30{,}000\ (DSTPD)^{0.6843}$$

2.29.5.5.6.2 If DSTPD is greater than 10.0 tpd, then electrical energy requirement is calculated by:

$$KWH = 15{,}000\ (DSTPD)^{0.9847}$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.29.5.5.7 Calculate auxiliary fuel required.

$$AF = 3.7 \times 10^{9}\ (DSTPD)^{0.9544}$$

where

AF = auxiliary fuel required, BTU/yr.

2.29.5.5.8 Other operation and maintenance material costs. This item includes repair and replacement material costs such as refractory or casting. It is expressed as a percentage of the total bare construction cost of the incinerator system.

$$OMMP = 5\%$$

where

OMMP = percent of total bare construction costs as O&M material costs.

2.29.5.6 <u>Quantities Calculations Output Data.</u>

2.29.5.6.1 Incinerator building area, A_B, sq ft.

2.29.5.6.2 Volume of reinforced concrete required for slab, V_{cs}, cu ft.

2.29.5.6.3 Operational labor, OMH, man-hours/yr.

2.29.5.6.4 Maintenance labor, MMH, man-hours/yr.

2.29.5.6.5 Electrical energy requirement, KWH, kwhr/yr.

2.29.5.6.6 Auxiliary fuel requirement, AF, BTU/yr.

2.29.5.6.7 Other operation and maintenance material costs, OMMP, percent.

2.29.5.7 <u>Unit Price Input Required.</u>

2.29.5.7.1 Standard size multiple-hearth incinerator, COSTIS, $ (optional).

2.29.5.7.2 Unit price input for building construction cost, UPIBC, $/sq ft.

2.29.5.7.3 Unit price input for R.C. slab, UPICS, \$/cu yd.

2.29.5.7.4 Current Marshall and Swift Equipment Cost Index, MSECI.

2.29.5.8 Cost Calculations.

2.29.5.8.1 Cost of incinerator building.

$$COSTB = A_B \times UPICS$$

where

COSTB = cost of incinerator building, \$.

UPIBC = unit price input for building cost, \$/sq ft.

2.29.5.8.2 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.29.5.8.3 Cost of installed equipment.

2.29.5.8.3.1 Calculate COSTR.

$$COSTR = .0375\ (THA) + 43.0$$

where

COSTR = cost of incinerator expressed as percent of standard size incinerator cost.

THA = total hearth area of incinerator selected, sq ft.

2.29.5.8.3.2 Purchase cost of standard size incinerator. The unit selected as standard is a unit with a total hearth area of 1580 sq ft. The cost of the 1580 sq ft multiple-hearth incinerator for the 1st quarter of 1977 is:

$$COSTIS = \$1,190,000$$

where

COSTIS = cost of standard size incinerator, \$.

For better cost estimation, COSTIS should be obtained from the manufacturer and treated as a unit price input. If COSTIS is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTIS = \$1,190,000 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Index 1st quarter, 1977.

2.29.5.8.3.3 Purchase cost of multiple-hearth incinerator selected.

$$COSTIE = \frac{COSTR}{100} \times COSTIS \times N$$

where

COSTIE = purchase cost of multiple-hearth incinerator selected, \$.

N = number of incinerators.

2.29.5.8.4 Calculate total bare construction cost.

$$TBCC + COSTB + COSTCS + COSTIE$$

where

TBCC = total bare construction cost, \$.

2.29.5.8.5 Calculate operation and maintenance material and supply costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs, \$/yr.

2.29.5.9 Cost Calculations Output Data.

2.29.5.9.1 Total bare construction cost, TBCC, \$.

2.29.5.9.2 Operation and maintenance material and supply costs, OMMC, \$/yr.

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2.31 ION EXCHANGE

2.31.1 Background.

2.31.1.1 Ion exchange is the reversible interchange of ions between a solid ion-exchange medium and a solution. Ion exchange has been used extensively in the removal of hardness and iron and manganese from groundwater supplies. In wastewater treatment, ion exchange has been used mainly for the treatment of industrial wastes. Recently, however, ion exchange was evaluted for the removal of nitrogen and phosphorus from municipal wastes.

2.31.1.2 An ion-exchange system usually consists of the exchange resin (cation or anion natural or synthetic), with provisions made for regeneration and rinsing. The most commonly used regenerants include sulfuric acid and caustic soda. Prior to application to the ion-exchange bed, wastewater may be subjected to pretreatment to remove certain contaminants which may hinder the perfomance of the exchange bed. Common pretreatment requirements are listed in Table 2.31-1.

2.31.2 Input Data.

2.31.2.1 Wastewater flow.

2.31.2.1.1 Average flow, mgd.

2.31.2.1.2 Minimum and maximum flows, mgd.

2.31.2.2 Cation and anion concentrations, mg/l.

2.31.2.3 Allowable effluent concentration, mg/l.

2.31.3 Design Parameters.

2.31.3.1 Type of resin.

2.31.3.2 Resin exchange capacity, lb/ft^3 (manufacturer's specifications).

2.31.3.3 Regenerant dosage, lb/ft^3 (consult resin manufacturer's specifications).

2.31.3.4 Flow rates.

2.31.3.4.1 Treatment flow rate (2-5 gpm/ft^3).

2.31.3.4.2 Regenerant flow rate (1-2 gpm/ft^3).

2.31.3.4.3 Rinsing flow rate (0.5-1.5 gpm/ft^3).

2.31.3.5 Amount of rinse water (30-120 gal/ft^3).

2.31.3.6 Column depth (24-30 in. minimum).

2.31.3.7 Operation per day, hr.

2.31.3.8 Amount of backwash water, gal/ft^3.

2.31.3.9 Regenerant level, lb/ft^3.

2.31.3.10 Regenerant level, lb/ft^3.

2.31.3.11 Regenerant specific gravity.

2.31.3.12 Backwash water rate, gpm/ft^3.

2.31.4 <u>Process Design Calculations</u>.

2.31.4.1 Select an ion-exchange system (consult manufacturer's specifications).

2.31.4.2 Select leakage, regenerant dosage, exchange capacities, and flow rates (consult manufacturer's specifications).

2.31.4.3 Compute service volume and time.

$$SV = \frac{REC}{IRC}$$

where

$$IRC = (\text{influent concentration} - \text{allowable effluent concentration}) \times \frac{0.624(10^{-4})}{7.48}$$

SV = service volume, gal/ft^3.

REC = resin exchange capacity, lb/ft^3.

IRC = ion removal concentration, lb/gal.

$$ST = \frac{SV}{SFR}$$

where

ST = service time, min.

SV = service volume, gal/ft^3.

SFR = service flow rate, gpm/ft^3.

2.31.4.4 Compute volume of resins.

$$Vol = \left[\frac{IRC(Q)(10^6)}{REC}\right] \times \left[\frac{ST}{(60)(24)}\right]$$

where

Vol = volume of resin, ft^3.

IRC = ion removal concentration, lb/gal.

Q = total flow, mgd.

REC = resin exchange capacity, lb/ft^3.

ST = service time min.

2.31.4.5 Calculate volume of regenerant.

$$V_R = \frac{RL \times Vol \times (100)}{RC(8.34)(\text{Specific Gravity})}$$

where

V_R = volume of regenerant, gal.

RL = regenerant level, lb/ft^3.

Vol = volume of resin, ft^3.

RC = regenerant concentration, percent.

2.31.4.6 Compute rinse requirements.

$$VRW = RW \times Vol$$

where

VRW = volume of rinse water, gal.

RW = rinse water, gal/ft^3.

Vol = volume of resin, ft^3.

2.31.4.7 Compute backwash water requirements.

$$VBW = (BW)(Vol)$$

where

VBW = backwater requirements, gal.

BW = backwash water, gal/ft^3.

Vol = volume of resin, ft^3.

2.31.4.8 Compute exchange cycle time.

$$EC = ST + BT + RT + WT + OMT$$

where

EC = exchange cycle, min.

ST = service time, min.

BT = backwash time, min.

RT = regeneration time, min.

WT = rinse time, min.

OMT = operation and maintenance time, hr.

ST = service time, min.

2.31.4.8.1 Calculate backwash time.

$$BT = \frac{BW}{BR}$$

where

BW = backwash water, gal/ft^3.

BR = rate of backwashing, gpm/ft^3.

2.31.4.8.2 Calculate regeneration time.

$$RT = \frac{V_R}{RRF} \times \frac{1}{Vol}$$

where

V_R = volume of regenerant.

RRF = rate of regenerant flow, gpm/ft^3.

Vol = volume of resin, ft^3.

2.31.4.8.3 Calculate rinse time.

$$WT = \frac{RW}{RRW}$$

where

RW = rinse water, gal/ft^3.

RRW = rate of rinse water flow, gpm/ft^3.

2.31.4.8.4 Calculate O&M time, hr.

$$OMT = 0.10(ST + BT + RT + WT) .$$

where

OMT = O&M time, hr.

2.31.4.9 Calculate number of cycles per day.

$$\text{Cycles/day} = \frac{HPD(60)}{EC}$$

where

HPD = operation per day, hr.

2.31.5 Process Design Output Data.

2.31.5.1 Average waste flow, mgd.

2.31.5.2 Effluent concentration, mg/l.

2.31.5.3 Resin exchange capacity, lb/ft^3.

2.31.5.4 Regenerant level, lb/ft^3.

2.31.5.5 Treatment flow rate, gpm/ft^3.

2.31.5.6 Regenerant flow rate, gpm/ft^3.

2.31.5.7 Rinse flow rate, gpm/ft^3.

2.31.5.8 Backwash flow rate, gpm/ft^3.

2.31.5.9 Volume of regenerant, gal.

2.31.5.10 Volume of rinse water, gal.

2.31.5.11 Volume of backwash water, gal.

2.31.5.12 Volume of resin, ft^3.

2.31.5.13 Service time, min.

2.31.5.14 Exchange cycle, min.

2.31.5.15 Cycles per day.

2.31.5.16 Service volume, gal/ft^3.

2.31.5.17 Backwash water, gal/ft^3.

2.31.5.18 Rinse water, gal/ft^3.

2.31.6 Quantities Calculations. Not used.

2.31.7 Quantities Calculations Output Data. Not used.

2.31.8 Unit Price Input Required. Not used.

2.31.9 Cost Calculations.

2.31.9.1 Unit costing is not available for this treatment process. Parametric costing will be used.

2.31.9.2 Calculate total bare construction cost.

$$TBCC = 146{,}943\ (Q_{avg})^{0.88}$$

where

TBCC = total bare construction cost, \$.

Q_{avg} = average daily flow, mgd.

2.31.9.3 Calculate O&M material and supply cost.

$$OMMC = 3{,}371\ (Q_{avg})^{0.72}$$

where

OMMC = O&M material and supply costs, \$/yr.

Q_{avg} = average daily flow, mgd.

2.31.10 Cost Calculations Output Data.

2.31.10.1 Total bare construction cost, TBCC, \$.

2.31.10.2 O&M material and supply cost, OMMC, \$/yr.

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Table 2.31-1. Common Pretreatment Requirements

Contaminant	Effect	Removal
Suspended solids	Bind resin particles	Coagulation and filtration
Organic	Large molecules foul strong base resins	Carbon absorption or weak base resins only
Oxidants	Slowly oxidize resins	Avoid prechlorination
Iron and mangenese	Coat resin particles	Aeration

2.33 LAGOONS

2.33.1 Background. Lagoons have been extensively used for municipal and industrial waste where sufficient land area is available. Some of the reasons for the popularity of lagoons are operational stability with fluctuating loads, require relatively unskilled operators, low O&M costs, and low construction costs.

Six different types of lagoons will be addressed; aerated aerobic lagoons, aerated facultative lagoons, anaerobic lagoons, facultative lagoons, oxidation lagoons, and sludge lagoons.

2.33.2 General Description Aerated Aerobic Lagoons. The contents of an aerobic aerated lagoon must be completely mixed so that the incoming solids and the biological solids produced in the lagoon do not settle. Effluent quality is a function of detention time and will normally have a BOD ranging from one-third to one-half of the influent value. This BOD is due to the endogenous respiration of the biological solids escaping in the effluent. Before the effluent is discharged, the solids may be removed by settling.

2.33.3 General Description Aerated Facultative Lagoon.

2.33.3.1 The contents of this type of lagoon are not completely mixed. Thus, portions of the incoming solids and the biologically produced solids settle out and undergo anaerobic decomposition. As a result, the effluent from the facultative lagoon would contain higher soluble BOD concentration than from the aerobic one.

2.33.3.2 Algal growth is possible, due to the non-complete mixing. The contribution of effluent suspended solids can be very high, dependent on the season, temperature and mixing intensity in the lagoon.

2.33.4 General Description Anaerobic Lagoon. As the name suggests these lagoons are anaerobic throughout their depth except for a very shallow upper layer. These lagoons are constructed deep in order to insure anaerobic conditions and to conserve heat. Typically they are from 8 to 20 feet deep. Reductions of ,0% of the influent BOD_5 are common with anaerobic lagoons and under ideal condition reductions of 85% are possible.

2.33.5 General Description Facultative Lagoon. In a facultative lagoon three zones exist; aerobic, facultative and anaerobic. The aerobic zone is near the surface and is like the aerobic or oxidation pond with aerobic bacteria and algae existing in a symbiotic relationship. The anaerobic zone is at the bottom of the lagoon where accumulated solids are decomposed by anaerobic bacteria. The facultative zone is an intermediate zone between the surface and bottom of the lagoon which is partly aerobic and partly anaerobic. Decomposition of the waste in this zone is accomplished by facultative bacteria. Normal depths for these lagoons is 3 to 8 feet.

2.33.6 <u>General Description Oxidation Lagoon</u>. The aerobic or oxidation lagoon is one in which aerobic bacteria and algae coexist in an aerobic environment. The oxygen required for reduction of the organic waste by the aerobic bacteria is supplied by algae production of oxygen through photosynthesis and atmospheric reaertion. The pond depths are very shallow, usually not greater than 4 feet, because of the dependency of algae photosynthesis upon sunlight. Soluble BOD_5 removal is high, but this is misleading because the high concentration of algae in the effluent.

2.33.7 <u>General Description Sludge Lagoons</u>. Lagoons have been used extensively in small systems for the dewatering of sludge. Drying lagoons are similar to sandbeds in that they are both designed for the dewatered sludge to be removed periodically and the lagoon refilled. However, they differ from sandbeds in that they use earthern levees and are built on the natural ground. therefore they are inherently cheaper to build.

Several factors must be considered in designing drying lagoons. The major factors include climate, subsoil permeability, lagoon depth, solids loading rates, and sludge characteristics.

2.33.8 <u>Aerated Aerobic Lagoon</u>.

2.33.8.1 <u>Input Data</u>.

2.33.8.1.1 Waste flow.

2.33.8.1.1.1 Average daily flow, mgd.

2.33.8.1.1.2 Peak (hourly) flow, mgd.

2.33.8.1.2 Wastewater characteristics.

2.33.8.1.2.1 BOD influent, mg/l.

2.33.8.1.2.3 Influent suspended solids, mg/l.

2.33.8.1.2.3 Influent volatile suspended solids, mg/l.

2.33.8.1.2.4 Nitrogen, mg/l as N.

2.33.8.1.2.5 Phosphorus, mg/l as P.

2.33.8.1.2.6 Non-biodegradable fraction of VSS.

2.33.8.1.3 Desired degree of treatment.

2.33.8.1.4 Temperature, ^{o}C (summer and winter).

2.33.8.2 <u>Design Parameters</u>.

2.33.8.2.1 Reaction rate constant, 0.0007-0.002 1/mg-hr.

2.33.8.2.2 MLSS, 200-500 mg/1.

2.33.8.2.3 MLVSS, 140-350 mg/1.

2.33.8.2.4 Fraction of BOD synthesized, 0.73.

2.33.8.2.5 Fraction of BOD oxidized for energy, 0.52.

2.33.8.2.6 Endogenous respiration rate per day, (b = 0.075/day, b' = 0.15/day).

2.33.8.2.7 Temperature coefficient, 1.035.

2.33.8.2.8 Hydraulic detention time, 2-4 days.

2.33.8.2.9 Depth, 6-12 ft.

2.33.8.3 <u>Process Design Calculations</u>.

2.33.8.3.1 Calculate volume of lagoon.

2.33.8.3.1.1 Adjust reaction rate constant for winter and summer temperatures.

$$K_T = K_{20}\,\theta^{(T-20)}$$

where

K_T = adjusted reaction rate constant, 1/mg hr.

K_{20} = reaction rate constant at 20° C, 1/mg hr.

θ = temperature coefficient, (1.035).

T = temperature, °C.

2.33.8.3.1.2 Calculate detention time.

$$t = \frac{1}{24aK_TS_e - b}$$

where

t = detention time, days.

a = fraction of BOD synthesized.

K_T = adjusted reaction rate constant, 1/mg hr.

S_e = effluent soluble BOD, mg/1.

b = endogenous respiration rate, 1/day.

Calculate the detention time based on summer temperature and winter temperature and desired treatment, then select the larger detention time.

2.33.8.3.1.3 Calculate lagoon volume.

$$V = Q_{avg}\ t$$

where

V = volume of lagoon, million gallons.

Q_{avg} = average daily flow, mgd.

t = detention time, days.

2.33.8.3.2 Calculate mixed liquor volatile suspended solids.

$$X_V = \frac{X_o + a\ (S_o - S_e)}{1 + bt}$$

where

X_V = mixed liquor volatile suspended solids, mg/l.

a → BOD → CELLS
b → CELLS → CO_2

X_o = influent suspended solids, mg/l.

a = fraction of BOD synthesized.

S_o = influent BOD, mg/l.

S_e = effluent soluble BOD, mg/l.

b = endogenous respiration rate, 1/day.

t = detention time, days.

2.33.8.3.3 Calculate oxygen requirement.

$$O_2 = (a')\ (S_o - S_e)\ (Q_{avg})\ (8.34) + (b')\ (X_V)\ (V)\ (8.34)$$

where

O_2 = oxygen required, lb/day.

a' = fraction of BOD oxidized for energy (0.52).

S_o = influent BOD, mg/l.

S_e = effluent soluble BOD, mg/l.

Q_{avg} = average daily flow, mgd.

b' = endogenous respiration rate, 1/day (0.15/day).

X_V = mixed liquor volatile suspended solids, mg/l.

V = volume of lagoon, million gallons.

2.33.8.3.4 Design mechanical aeration system.

2.33.8.3.4.1 Assume the following design parameters.

2.33.8.3.4.1.1 Standard transfer efficiency, STE, lb/hp-hr $\approx$ 5.

2.33.8.3.4.1.2 O_2 transfer in waste/O_2 transfer in water, α, $\approx$ 0.9.

2.33.8.3.4.1.3 O_2 saturation in waste/O_2 saturation in water, β, $\approx$ 0.9.

2.33.8.3.4.1.4 Correction factor for pressure, P, $\approx$ 1.0.

2.33.8.3.4.1.5 O_2 saturation at selected summer temperature, $(C_s)_T$, mg/l.

2.33.8.3.4.1.6 Minimum dissolved oxygen to be maintained in the basin, 2.0 mg/l.

2.33.8.3.4.2 Adjust standard transfer efficiency to operating efficiency.

$$OTE = STE \frac{[(C_s)_T\ (\beta)\ (P) - C_L]}{9.17} \alpha\ (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature, mg/l.

β = O_2 saturation in waste/O_2 saturation in water.

P = correction factor for pressure.

C_L = minimum dissolved oxygen to be maintained in basin, mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, $^\circ$C.

2.33.8.3.4.3 Calculate horsepower for aerators.

$$HP = \frac{(O_2)\ (1000)}{(OTE)\ (24)\ (V)\ (10^6)}$$

where

HP = aerator horsepower required per 1000 gallons, hp/1000 gallons.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of lagoon, million gal.

2.33.8.3.4.4 Check the calculated horsepower versus minimum required for complete mixing.

If HP 0.06 hp/1000 gallons set hp = 0.06 hp/1000 gal.

2.33.8.3.4.5 Calculate total horsepower required.

$$THP = \frac{(HP)\ (V)\ (10^6)}{1000}$$

where

THP = total aerator horsepower required, hp.

HP = aerator horsepower required per 1000 gal., hp/1000 gal.

2.33.8.3.5 Calculate nutrient requirements.

BOD: Nitrogen: Phosphorus = 100:5:1

2.33.8.3.6 Effluent Characteristics.

2.33.8.3.6.1 Calculate total BOD of the effluent.

$$BODE = S_e + 0.3\ X_V$$

where

$BODE$ = total BOD of effluent, mg/l.

S_e = effluent soluble BOD_5, mg/l.

X_V = mixed liquor volatile suspended solids, mg/l.

2.33.8.3.6.2 Suspended Solids.

$$SSE = \frac{X_v}{.8}$$

where

SSE = effluent suspended solids concentration, mg/l.

X_v = mixed liquor volatile suspended solids, mg/l.

2.33.8.3.6.3 COD.

$$CODE = 1.5\ BODE$$

$$CODSE = 1.5\ Se$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration.

CODSE = effluent COD soluble concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.33.8.3.6.4 Nitrogen.

$$TKNE = TKN$$

$$NH3E = .25\ TKN$$

$$NO2E = 0.0$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

2.33.8.3.6.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.33.8.4 Process Design Output Data

2.33.8.4.1 Average daily flow, Q_{avg}, mgd.

2.33.8.4.2 Peak flow, Q_p, mgd.

2.33.8.4.3 Detention time, t, days.

2.33.8.4.4 Effluent soluble BOD, S_e, mg/l.

2.33.8.4.5 Volume of lagoon, V, million gal.

2.33.8.4.6 Oxygen required, O_2 lb/day.

2.33.8.4.7 Total aerator horsepower required, THP, hp.

2.33.8.4.8 Total BOD of effluent, BOD_{eff}, mg/l.

2.33.8.5 Quantities Calculations.

2.33.8.5.1 Calculate number of basins required, the Number of basins may be designated, if not, it will be selected from the following, based on flow.

Flow Range (Q_{avg})	Number of Basins (N)
Less than 0.5 mgd	1
From 0.5 to 2.0 mgd	2
From 2.0 to 5.0 mgd	3
Greater than 5.0 mgd	4

where

W_{avg} = average daily flow, mgd.

N = number of basins.

2.33.8.5.2 Select size and number of aerators.

2.33.8.5.2.1 Calculate aerator horsepower per basin.

$$HP_b = \frac{THP}{N}$$

where

HP_b = aerator horsepower required per basin, hp.

THP = total aerator horsepower required, hp.

N = number of basins.

2.33.8.5.2.2 Determine number of aerators per basin. The number of aerators per basin must be one of the following 2, 3, 4, 6, 8. Also the aerators must be one of the following sizes, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100 or 150. The selection process will be trial and error.

Assume number of aerator per basin (K) is 2. If $\frac{HP_b}{K} > 150$, go to next trials K = 3, 4, 6, 8 until $\frac{HP_b}{K} \leq 150$, then compare $\frac{HP_b}{K}$ with values for individual aerators (HP_a) given above. Select the smallest value of HP_a that is greater than $\frac{HP_b}{K}$. Compare HP_a x K with HP_b. If HP_a x K is larger than HP_b by 5% or more, go to next trial using the next larger K, until HP_a x K is within 5% of HP_b.

2.33.8.5.3 Calculate basin depth. The basin water depth (Dw) is controlled by the aerator sizes used. This is true because each size aerator has a maximum depth at which it can be used and still achieve mixing without the use of a draft tube.

2.33.8.5.3.1 If $HP_a < 100$ HP

$$DW = 4.82\ (HP_a)^{0.2467}$$

2.33.8.5.3.2 If $100 \leq HP_a \leq 150$

$$DW = 15 \text{ ft}$$

where

DW = basin water depth, ft.

HP_a = horsepower of individual aerators, hp.

2.33.8.5.4 Calculate basin dimensions.

2.33.8.5.4.1 Calculate length to width ratio.

$$\text{If } K \leq 4,\ r = K$$

$$\text{If } K > 4,\ r = k/2$$

where

K = number of aerators per basin.

r = basin length to width ratio.

2.33.8.5.4.2 Calculate the length of the basin at water level.

$$L_w = \frac{r\left[4/r\ A_a - 6/r\ (3DW)^2 + 2\ (3DW)^2\right]^{0.5} + 3DW\ (1+r)}{2}$$

$$A_a = \frac{V \times 10^6}{(N)\ (DW)\ (7.48)}$$

where

L_w = length of basin at water level, ft.

r = length to width ratio.

s = side slope = 3 to 1 for all basins.

A_a = average lagoon surface area, ft^2.

DW = basin water depth, ft.

2.33.8.5.4.3 Calculate basin width at water level.

$$W_w = \frac{Lw}{r}$$

where

W_w = width of basin at water level, ft.

L_w = length of basin at water level, ft.

r = length to width ratio.

2.33.8.5.5 Calculate volume of earthwork required. The following assumptions were made concerning basin construction.

Basins will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
A 2 ft. freeboard will be used on all lagoons.
Common levee construction will be used where practical.

2.33.8.5.5.1 The volume of earthwork must be determined by trial and error. Assume a depth of cut of 1 ft.

2.33.8.5.5.1.1 Calculate the length and width at original ground level.

$$L_c = L_w - 6\ (DW - DC)$$

$$W_c = \frac{L_c}{r}$$

where

L_c = length of lagoon at original ground level, ft.

W_c = width of lagoon at original ground level, ft.

r = length to width ratio.

DW = basin water depth, ft.

DC = depth of cut, ft.

2.33.8.5.5.1.2 Calculate the volume of cut.

$$V_c = (1.3)(N)(DC)\left[W_c L_c - 3(DC)(W_c) - 3(DC)(L_c) + 12(DC)^2\right]$$

where

V_c = volume of cut, cu ft.

N = number of lagoons.

DC = depth of cut, ft.

W_c = width of lagoon at original ground level, ft.

L_c = length of lagoon at original ground level, ft.

2.33.8.5.5.1.3 Calculate length and width at top of levee.

$$L_T = L_W + 12$$

$$W_T = \frac{L_T}{r}$$

where

L_T = length of lagoon at top of levee, ft.

W_T = width of lagoon at top of levee, ft.

L_W = length of lagoon of water level, ft.

r = length to width ratio.

2.33.8.5.5.1.4 Calculate the depth of fill.

$$DF = DW + 2 - DC$$

where

DF = depth of fill, ft.

DW = basin water depth, ft.

DC = depth of cut, ft.

2.33.8.5.5.1.5 Calculate number of levees.

$$N_L = N + 1$$

$$N_W = 2\ N$$

where

N_L = number of levees of length, L_T.

N_W = number of levees of width, W_T.

N = number of lagoons.

2.33.8.5.5.1.6 Calculate volume of fill.

$$V_F = [10DF + 3(DF)^2]\ (L_T)\ (N_L) + (W_T)\ (N_W)$$

where

V_F = volume of fill, cu ft.

DF = depths of fill, ft.

L_T = length of lagoon at the top of the levee, ft.

W_T = width of the lagoon at the top of the levee, ft.

N_L = number of levees of length, L_T.

N_W = number of levees of width, W_T.

2.33.8.5.5.1.7 Compare V_c and V_f.

If $V_c < V_F$ then assume DC > 1 ft and recalculate V_c and V_F.

If $V_c > V_F$ then assume DC < 1 ft and recalculate V_c and V_F.

Repeat this procedure until $V_c = V_F$. This is the volume of earthwork required.

$$V_c = V_F = VLEW$$

where

DC = depth of cut, ft.

V_c = volume of cut, cu ft.

V_F = volume of fill, cu ft.

VLEW = volume of earthwork required, cu ft.

2.33.8.5.6 Calculate reinforced concrete requirement.

2.33.8.5.6.1 Influent structure. The influent structure would be a flow splitter box. The size of the structure would be determined by the number of basins, since it would have to accommodate weirs for each basin. The following quantities will be used.

N = 1, or 2 V_{cwi} = 81 cu ft
33 cu ft

N = 3 V_{cwi} = 97 cu ft
V_{csi} = 43 cu ft

N = 4 V_{cwi} = 135 cu ft
V_{csi} = .66 cu ft

where

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

N = number of basins.

2.33.8.5.6.2 Effluent structure. Each basin would have a separate effluent structure. The size of the structure would be approximately the same regardless of flow.

$$V_{cwe} = (81)\ (N)$$

$$V_{cse} = (32)\ (N)$$

where

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

N = number of basins.

2.33.8.5.6.3 Total reinforced concrete required.

$$V_{cw} = V_{cwi} + V_{cwe}$$

$$V_{cs} = V_{csi} + V_{cse}$$

where

V_{cw} = total volume of R.C. wall required, cu ft.

V_{cs} = total volume of R.C. slab required, cu ft.

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

2.33.8.5.7 Calculate volume of concrete required for embankment protection. In large lagoons and in aerated lagoons the earthern levees require protection from the wave action of the water. For this purpose concrete is placed around the interior of the levee at the water line. The concrete would extend from the top of the levee to about 1.5 ft below the water surface. Using a 3 to 1 side slope, the width of the slab would be 11 ft. The slab would be 8 inches thick.

$$V_{cep} = (2L_w + 2W_w)\ (11)\ (0.67)\ (N)$$

where

V_{cep} = volume of concrete for embankment protection, cu ft.

L_w = length of levee at water level, ft.

W_w = width of levee at water level, ft.

N = number of basins.

2.33.8.5.8 Calculate lagoon surface area.

$$A_s = \frac{(N)\ (Lw)\ (Ww)}{43560}$$

where

A_s = lagoon water surface area, acres.

N = number of basins.

Lw = length of levee at water surface, ft.

Ww = width of levee at water surface, ft.

2.33.8.5.9 Calculate operation manpower required.

2.33.8.5.9.1 If $A_S \leq 4$ acres, the operation manpower is calculated by:

$$OMH = 700 \ (A_S)^{0.2571}$$

where

OMH = operation manpower required, MH/yr.

A_s = lagoon water surface area, acres.

2.33.8.5.9.2 If $A_S > 4$ acres, the operation manpower is calculated by:

$$OMH = 539.3 \ (A_S)^{0.4453}$$

2.33.8.5.10 Calculate maintenance manpower required.

2.33.5.10.1 If $A_S \leq 4$ acres, the maintenance manpower is calculated by:

$$MMH = 130.0 \ (A_S)^{0.2925}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

2.33.8.5.10.2 If $A_S > 4$ acres, the maintenance manpower is calculated by:

$$MMH = 105 \ (A_S)^{0.4466}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

2.33.8.5.11 Calculate electrical energy required for operation. Assuming that all the aerators operate 90% of the time and the efficiency of the electric motors is 0.877.

2.33.8.5.11.1 Calculate total installed horsepower.

$$HP_T = (HP_a)\ (K)\ (N)$$

where

HP_a = horsepower of individual aerators, hp.

HP_T = total installed horsepower, hp.

K = number of aerators per basin.

N = number of basins.

2.33.8.5.11.2 Calculate electric energy required.

$$KWH = (HP_T)\ (365)\ (24)\ (.9)\ (.85)\ (.877)$$

where

KWH = electrical energy required, kwhr/yr.

HP_T = total installed horsepower, hp.

2.33.8.5.12 Calculate quantity for lagoon liner. In some areas the soils are such that lagoons must be lined to prevent loss of water. User should designated whether liner is required.

2.33.8.5.12.1 Calculate area of the bottom of the lagoon.

$$SA_B = (L_w - 6DW)\ (W_w - 6DW)$$

where

SA_B = area of the bottom of lagoon, ft^2.

L_w = length of basin at water level, ft.

W_w = width of basin at water level, ft.

DW = basin water depth, ft.

2.33.8.5.12.2 Calculate area of lagoon embankment.

$$SA_E = 2(L_T + W_T)\left(\frac{DW+2}{.3159} + 5\right)$$

where

SA_E = area of lagoon embankment, ft^2.

L_T = length of basin at top of levee, ft.

W_T = width of basin at top of levee, ft.

DW = basin water depth, ft.

2.33.8.5.12.3 Calculate total surface area required for liner.

$$SA_L = N\ (SA_B + SA_E)$$

where

SA_L = total area of lagoon liner, ft^2.

N = number of basins.

SA_B = area of bottom of lagoon, ft^2.

SA_E = area of lagoon embankment, ft^2.

2.33.8.5.13 Other operation and maintenance material and supply costs. This item includes such items as lubrication oil, paint, repair and replacement parts. These costs are estimated as a percent of the installed equipment cost.

$$OMMP = 4.93\ (HP_T)^{-0.1827}$$

where

OMMP = O&M maintenance and supply costs as percent of the installed equipment cost, %.

2.33.8.5.14 Other minor construction cost items. From the above calculations approximately 90% of the construction costs have been accounted for. Other minor items such as piping, grass seeding, etc., would be 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs items.

2.33.8.6 <u>Quantities Calculations Output Data.</u>

2.33.8.6.1 Number of basins, N.

2.33.8.6.2 Basin water depth, DW, ft.

2.33.8.6.3 Number of aerators per basin, K.

2.33.8.6.4 Basin length to width ratio, r.

2.33.8.6.5 Horsepower of individual aerators, HP_a, hp.

2.33.8.6.6 Length of basin at water level, L_w, ft.

2.33.8.6.7 Width of basin at water level, W_w, ft.

2.33.8.6.8 Volume of earthwork required, VLEW, cu ft.

2.33.8.6.9 Total volume of R.C. wall required, V_{cw}, cu ft.

2.33.8.6.10 Total volume of R.C. slab required, V_{cs}, cu ft.

2.33.8.6.11 Volume of concrete for embankment protection, V_{cep}, cu ft.

2.33.8.6.12 Operation manpower required, OMH, MH/yr.

2.33.8.6.13 Maintenance manpower required, MMH, MH/yr.

2.33.8.6.14 Electrical energy required, KWH, Kwhr/yr.

2.33.8.6.15 Total area of lagoon liner, SA_L, ft^2.

2.33.8.6.16 O&M maintenance and supply costs as percent of the installed equipment cost, OMMP, %.

2.33.8.6.17 Correction factor for other minor construction costs, CF.

2.33.8.7 Unit Price Input Required.

2.33.8.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.33.8.7.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

2.33.8.7.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

2.33.8.7.4 Cost of standard size aerator (50 hp), COSTA, \$, (optional).

2.33.8.7.5 Current Marshall and Swift Equipment Cost Index, MSECI.

2.33.8.7.6 Installation labor rate, LABRI, \$/MH.

2.33.8.7.7 Unit price input for lagoon liner, UPILL, $\$/ft^2$.

2.33.8.8 Cost Calculations.

2.33.8.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.33.8.8.2 Cost of reinforced concrete wall.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, cu yd.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.33.8.8.3 Cost of reinforced concrete slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.33.8.8.4 Cost of concrete embankment protection.

$$COSTEP = \frac{V_{cep}}{27} (0.5)(UPICS)$$

where

COSTEP = cost of concrete embankment protection in-place, \$.

V_{cep} = volume of concrete for embankment protection, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

0.5 = factor to adjust R.C. concrete unit price to nonreinforced.

2.33.8.8.5 Calculate purchase cost of aerators.

$$COSTA = \frac{(COSTSA)\ (COSTR)\ (K)\ (N)}{100}$$

where

COSTA = purchase cost of aerators, \$.

COSTSA = cost of standard size aerator (50 hp), \$.

COSTR = cost of aerator of horsepower HP_a as a percent of the cost of the standard size aerators, %.

K = number of aerators per basin.

N = number of basins.

2.33.8.8.5.1 Calculate COSTR.

If $HP_a \leq 25$ hp; COSTR is calculated by:

$$COSTR = 20.7\ (HP_a)^{0.2686}$$

If $HP_a > 25$ hp; COSTR is calculated by:

$$COSTR = 4.12\ (HP_a)^{0.7878}$$

2.33.8.8.5.2 Purchase cost of standard size aerator. The standard size aerator is a 50 hp, high-speed floating aerator. The cost of the 50 hp aerator in the first quarter of 1977 is:

$$COSTSA = \$13,960$$

For better cost estimation, COSTSA should be obtained from the equipment vendor and treated as a unit price input. However, if COSTA is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTA = \$13,960\ \frac{MSECI}{491.6}$$

where

COSTA = cost of standard size aerator (50 hp), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

2.33.8.8.6 Calculate total installed equipment cost.

2.33.8.8.6.1 Calculate aerator installation labor.

$$IMH = 0.633\ (HP_a) + 40$$

where

IMH = aerator installation labor, MH.

HP_a = horsepower of individual aerators, hp.

2.33.8.8.6.2 Calculate aerator installation cost.

$$AIC = (IMH)\ (K)\ (N)\ (LABRI)$$

where

AIC = aerator installation cost, $.

IMH = aerator installation labor, MH.

K = number of aerators per basin.

N = number of basins.

LABRI = installation labor rate, $/MH.

2.33.8.8.6.3 Calculate installed cost for electrical/mechanical.

$$EMC = 0.589\ (HP_a)^{-0.1465}\ (COSTA)$$

where

EMC = installed cost for electrical/mechanical, $.

HP_a = horsepower of individual aerators, hp.

COSTA = purchase cost of aerators, $.

2.33.8.8.6.4 Calculate total installed equipment cost.

$$IEC = COSTA + AIC + EMC$$

where

IEC = total installed equipment cost, $.

AIC = aerator installation cost, $.

EMC = installed cost of electrical/mechanical, $.

2.33.8.8.7 Calculate cost of lagoon liner.

$$COSTLL = (SA_L)\ (UPILL)$$

COSTLL = installed cost of lagoon liner, $.

SA_L = total area of lagoon liner, ft^2.

UPILL = unit price input for lagoon liner, $/$ft^2$.

2.33.8.8.8 Calculate total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + COSTEP + COSTLL + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

COSTEP = cost of concrete embankment protection in-place, \$.

COSTLL = installed cost of lagoon liner, \$.

IEC = total installed equipment cost, \$.

2.33.8.8.9 Calculate O&M maintenance and supply cost.

$$OMMC = \frac{OMMP}{100} IEC$$

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as percent of installed equipment cost, %.

IEC = total installed equipment cost, \$.

2.33.8.9 <u>Cost Calculations Output Data</u>.

2.33.8.9.1 Total bare construction cost, TBCC, \$.

2.33.8.9.2 O&M material and supply costs, \$/yr.

2.33.9 **Aerated Facultative Lagoon.**

2.33.9.1 Input Data.

2.33.9.1.1 Wastewater flow.

2.33.9.1.1.1 Average daily flow, mgd.

2.33.9.1.1.2 Peak (hourly) flow, mgd.

2.33.9.1.2 Wastewater characteristics.

2.33.9.1.2.1 BOD influent, mg/l.

2.33.9.1.2.2 Influent suspended solids, mg/l.

2.33.9.1.2.3 Influent volatile suspended solids, mg/l.

2.33.9.1.2.4 Nitrogen, mg/l as N.

2.33.9.1.2.5 Phosphorus, mg/l as P.

2.33.9.1.2.6 Non-biodegradable fraction of VSS.

2.33.9.1.3 Desired degree of treatment.

2.33.9.1.4 Temperature, ^{o}C (summer and winter).

2.33.9.2 Design Parameters.

2.33.9.2.1 Reaction rate constant/day (0.5-1.0, avg 0.75).

2.33.9.2.2 Temperature correction coefficient ≈ 1.075.

2.33.9.2.3 Fraction BOD removed for respiration (0.9-1.4).

2.33.9.2.4 BOD feedback from bottom or sediment (summer = 20 percent; winter = 5 percent).

2.33.9.2.5 MLVSS, mg/l, (50-150) average 100.

2.33.9.3 Process Design Calculations.

2.33.9.3.1 Select the rate constant, K. Adjust K for summer and winter temperatures.

$$K_T = K_{20}\theta^{(T-20)}$$

where

K_T = rate constant for desired temperature, ^{o}C.

K_{20} = rate constant at 20°C.

θ = temperature correction coefficient.

T = temperature, °C.

2.33.9.3.2 For summer and winter efficiencies, calculate detention times to meet winter efficiency.

$$t = \frac{1.05\ So}{S_e K} - \frac{1}{K}$$

where

S_e = effluent soluble BOD_5, mg/l.

S_o = influent BOD_5, mg/l.

K = reaction rate constant.

t = detention time, days.

and summer efficiency:

$$t = \frac{1.2\ So}{S_c K} - \frac{1}{K}$$

Select the larger of the detention times.

2.33.9.3.3 Calculate volume.

$$V = Q_{avg} t$$

where

V = volume, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, days.

2.33.9.3.4 Detemine oxygen requirements, assume a'.

$$O_2 = a'S_r Q(8.34)\ (1.2)$$

where

O_2 = oxygen required, lb/day.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed, mg/l.

Q = flow, mgd.

2.33.9.3.5 Design mechanical aeration system and check horsepower supply to allow solids to settle: horsepower required to allow solids to settle (0.01-0.02 hp/1000 gal).

2.33.9.3.5.1 Assume the following design parameters.

2.33.9.3.5.1.1 Standard transfer efficiency, lb/hp-hr (O_2 dissolved oxygen, 20°C, and tap water).

2.33.9.3.5.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

2.33.9.3.5.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

2.33.9.3.5.1.4 Correction factor for pressure $\approx$ 1.0.

2.33.9.3.5.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

2.33.9.3.5.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \; \frac{(C_s)_T(\beta)\,(p) - C_L}{9.17} \; (\alpha) \; (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin 2.0 mg/l.

α = O_2 transfer in waste/O_2 transfer in water
$\approx$ 0.9.

T = temperature, ^{o}C.

2.33.9.3.5.4 Calculate horsepower requirement.

$$HP = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)(V)(10^6)} \times 1000$$

where

HP = horsepower required per 1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of the basin, million gal.

2.33.9.3.5.5 Calculate total horsepower requirement.

$$THP = \frac{(HP)(V)(10^6)}{1000}$$

where

THP = total horsepower required, hp.

V = volume of lagoon, million gal.

HP = horsepower required per 1000 gal, hp/1000 gal.

2.33.9.3.6 Effluent Characteristics.

2.33.9.3.6.1 Effluent suspended solids concentration. The suspended solids from an aerated facultative lagoon can be divided into three categories; inert solids, bacteria cells and algal cells. The growth of algae in sewage lagoons is a very complex process. Geographic location, temperature, season, organic loading, nutrient concentration and light penetration are the factors that govern the ecological system in a sewage lagoon. However, White and Rich have presented the following which relates the algal concentration with mixing level in aerated lagoon systems.

$$SS_{AL} = \frac{0.407}{HP} - 10.7$$

where

SS_{AL} = algal concentration of suspended solids, mg/l.

HP = horsepower required/1000 gal.

The suspended solids due to inert material and bacteria cells is more or less independent of the mixing level. It is usually in the range of 25 to 30 mg/l and approximately 85% volatile.

$$SS_{oi} = 25 \text{ mg/l}$$

Thus, the total suspended solids concentration would be

$$(SS)_{eff} = SS_{AL} + SS_{oi}$$

where

$(SS)_{eff}$ = total suspended solids in effluent, mg/l.

SS_{AL} = algal concentration of suspended solids, mg/l.

SS_{oi} = suspended solids due to inert material and bacteria cells, mg/l.

2.33.9.3.6.2 Calculate BOD in the final effluent.

$$BODE = Se + 0.3\ (0.85 \times SS_{oi}) + 0.12\ (SS_{AL})$$

where

BODE = BOD in effluent in mg/l.

Se = soluble effluent BOD, mg/l.

0.12 = BOD concentration contributed by algae in the effluent.

2.33.9.3.6.3 COD

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.33.9.3.6.4 Nitrogen

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

2.33.9.3.6.5 Phosphorus

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.33.9.3.6.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

2.33.9.3.8 Determine nutrient requirements.

BOD:Nitrogen:Phosphorus = 100:5:1

2.33.9.4 <u>Process Design Output Data.</u>

2.33.9.4.1 Average daily flow, Q_{avg}, mgd.

2.33.9.4.2 Peak flow, Q_p, mgd.

2.33.9.4.3 Detention time, t, days.

2.33.9.4.4 Effluent soluble BOD, S_e, mg/1.

2.33.9.4.5 Volume of lagoon, V, million gal.

2.33.9.4.6 Oxygen required, O_2, lb/day.

2.33.9.4.7 Total aerator horsepower required, THP, hp.

2.33.9.4.8 Total BOD of effluent, BODE, mg/1.

2.33.9.5 Quantities Calculations.

2.33.9.5.1 Calculate number of basins required. The number of basins may be designated, if not, it will be selected from the following, based on flow.

Flow Range (Q_{avg})	Number of Basins (N)
Less than 0.5 mgd	1
From 0.5 to 2.0 mgd	2
From 2.0 to 5.0 mgd	3
Greater than 5.0 mgd	4

where

Q_{avg} = average daily flow, mgd.

N = number of basins.

2.33.9.5.2 Select size and number of aerators.

2.33.9.5.2.1 Calculate aerator horsepower per basin.

$$HP_b = \frac{THP}{N}$$

where

HP_b = aerator horsepower required per basin, hp.

THP = total aerator horsepower required, hp.

N = number of basins.

2.33.9.5.2.2 Determine number of aerators per basin. The number of aerators per basin must be one of the following 2, 3, 4, 6, 8. Also the aerators must be one of the following sizes, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100, or 150. The selection process will be trial and error.

Assume number of aerator per basin (K) is 2. If $\frac{HP_b}{K} > 150$, go to next trials K=3, 4, 6, 8 until $\frac{HP_b}{K} \leq 150$, then compare $\frac{HP_b}{K}$ with values for individual aerators (HP_a) given above. Select the smallest value of HP_a that is greater than $\frac{HP_b}{K}$.

2.33.9.5.3 Calculate basin depth. The basin water depth (Dw) is controlled by the aerator sizes used. This is true because each size aerator has a maximum depth at which it can be used and still achieve mixing without the use of a draft tube.

2.33.9.5.3.1 If $HP_a \leq 100$ HP

$$DW = 4.82\ (HP_a)^{0.2467}$$

2.33.9.5.3.2 If $100 \leq HP_a \leq 150$

$$DW = 15 \text{ ft}$$

where

DW = basin water depth, ft.

HP_a = horsepower of individual aerators, hp.

2.33.9.5.4 Calculate basin dimensions.

2.33.9.5.4.1 Calculate length to width ratio.

If $K \leq 4$, $r = K$

If $K > 4$, $r = K/2$

where

K = number of aerators per basin.

r = basin length to width ratio.

2.33.9.5.4.2 Calculate the length of the basin at water level.

$$L_w = \frac{r\ 4/r\ A_a - 6/r\ (3DW)^2 + 2\ (3DW)^2 \quad ^{0.5} + 3DW\ (1+r)}{2}$$

$$A_a = \frac{V \times 10^6}{(N)\ (DW)\ (7.48)}$$

where

L_w = length of basin at water level, ft.

r = length to width ratio.

s = side slope = 3 to 1 for all basins.

A_a = average lagoon surface area, ft^2.

DW = basin water depth, ft.

2.33.9.5.4.3 Calculate basin width at water level.

$$W_w = \frac{L_w}{r}$$

where

W_w = width of basin at water level, ft.

L_w = length of basin at water level, ft.

r = length to width ratio.

2.33.9.5.5 Calculate volume of earthwork required. The following assumptions were made concerning basin construction.

Basins will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
A 2 ft freeboard will be used on all lagoons.
Common levee construction will be used where practical.

2.33.9.5.5.1 The volume of earthwork must be determined by trial and error. Assume a depth of cut of 1 ft.

2.33.9.5.5.1.1 Calculate the length and width at original ground level.

$$L_c = L_w - 6\ (DW - DC)$$

$$W_c = \frac{L_c}{r}$$

where

L_c = length of lagoon at original ground level, ft.

W_c = width of lagoon at original ground level, ft.

r = length to width ratio.

DW = basin water depth, ft.

DC = depth of cut, ft.

2.33.9.5.5.1.2 Calculate the volume of cut.

$$V_c = (1.3)(N)(DC)\ [W_c L_c - 3(DC)(W_c) - 3(DC)(L_c) + 12(DC)^2]$$

where

V_c = volume of cut, cu ft.

N = number of lagoons.

DC = depth of cut, ft.

W_c = width of lagoon at original ground level, ft.

L_c = length of lagoon at original ground level, ft.

2.33.9.5.5.1.3 Calculate length and width at top of levee.

$$L_T = L_w + 12$$

$$W_T = \frac{L_T}{r}$$

where

L_T = length of lagoon at top of levee, ft.

W_T = width of lagoon at top of levee, ft.

L_w = length of lagoon at water level, ft.

r = length to width ratio.

2.33.9.5.5.1.4 Calculate the depth of fill.

$$DF = DW + 2 - DC$$

where

DF = depth of fill, ft.

DW = basin water depth, ft.

DC = depth of cut, ft.

2.33.9.5.5.1.5 Calculate number of levees.

$$N_L = N + 1$$

$$N_w = 2\ N$$

where

N_L = number of levees of length, L_T.

N_W = number of levees of width, W_T.

N = number of lagoons.

2.33.9.5.5.1.6 Calculate volume of fill.

$$V_F = [10DF + 3(DF)^2] (L_T) (N_L) + (W_T) (N_w)$$

where

V_F = volume of fill, cu ft.

DF = depths of fill, ft.

L_T = length of lagoon at the top of the levee, ft.

W_T = width of the lagoon at the top of the levee, ft.

N_L = number of levees of length, L_T.

N_W = number of levees of width, W_T.

2.33.9.5.5.1.7 Compare V_c and V_f.

If $V_c < V_F$ then assume DC > 1 ft and recalculate V_c and V_F.

If $V_c > V_F$ then assume DC < 1 ft and recalculate V_c and V_F.

Repeat this procedure until $V_c = V_F$. This is the volume of earthwork required.

$$V_c = V_F = VLEW$$

where

DC = depth of cut. ft.

V_c = volume of cut, cu ft.

V_F = volume of fill, cu ft.

VLEW = volume of earthwork required, cu ft.

2.33.9.5.6 Calculate reinforced concrete requirement.

2.33.9.5.6.1 Influent structure. The influent structure would be a flow splitter box. The size of the structure would be determined by the number of basins, since it would have to accommodate weirs for each basin. The following quantities will be used.

$$N = 1, \text{ or } 2 \qquad V_{cwi} = 81 \text{ cu ft.}$$
$$33 \text{ cu ft.}$$

$$N = 3 \qquad V_{cwi} = 97 \text{ cu ft.}$$
$$V_{csi} = 43 \text{ cu ft.}$$
$$N = 4 \qquad V_{cwi} = 135 \text{ cu ft.}$$
$$V_{csi} = 66 \text{ cu ft.}$$

where

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

N = number of basins.

2.33.9.5.6.2 Effluent Structure. Each basin would have a separate effluent structure. The size of the structure would be approximately the same regardless of flow.

$$V_{cwe} = (81)\ (N)$$

$$V_{cse} = (32)\ (N)$$

where

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

N = number of basins.

2.33.9.5.6.3 Total reinforced concrete required.

$$V_{cw} = V_{cwi} + V_{cwe}$$

$$V_{cs} = V_{csi} + V_{cse}$$

where

V_{cw} = total volume of R.C. wall required, cu ft.

V_{cs} = total volume of R.C. slab required, cu ft.

V_{cwi} = volume of R.C. wall required for influent sturcture, cu ft.

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

2.33.9.5.7 Calculate volume of concrete required for embankment protection. In large lagoons and in aerated lagoons the earthern levees require protection from the wave action of the water. For this purpose concrete is placed around the interior of the levee at the water line. The concrete would extend from the top of the levee to about 1.5 ft below the water surface. Using a 3 to 1 side slope, the width of the slab would be 11 ft. The slab would be 8 inches thick.

$$V_{cep} = (2L_w + 2W_w)\ (11)\ (0.67)\ (N)$$

where

V_{cep} = volume of concrete for embankment protection, cu ft.

L_w = length of levee at water level, ft.

W_w = width of levee at water level, ft.

N = number of basins.

2.33.9.5.8 Calculate lagoon surface area.

$$A_s = \frac{(N)(Lw)(Ww)}{43560}$$

where

A_s = lagoon water surface area, acres.

N = number of basins.

Lw = length of levee at water surface, ft.

Ww = width of levee at water surface, ft.

2.33.9.5.9 Calculate operation manpower required.

2.33.9.5.9.1 If $A_S \leq 4$ acres, the operation manpower is calculated by:

$$OMH = 700\ (A_S)^{0.2571}$$

where

OMH = operation manpower required, MH/yr.

A_S = lagoon water surface area, acres.

2.33.9.5.9.2 If $A_S > 4$ acres, the operation manpower is calculated by:

$$OMH = 539.3\ (A_S)^{0.4453}$$

2.33.9.5.10 Calculate maintenance manpower required.

2.33.9.5.10.1 If $A_S \leq 4$ acres, the maintenance manpower is calculated by:

$$MMH = 130.0\ (A_S)^{0.2925}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

2.33.9.5.10.2 If $A_S > 4$ acres, the maintenance manpower is calculated by:

$$MMH = 105\ (A_S)^{0.4466}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

2.33.9.5.11 Calculate electrical energy required for operation. Assuming that all the aerators operate 90% of the time and the efficiency of the electric motors is 0.877, the power required would be.

2.33.9.5.5.11.1 Calculate total installed horsepower.

$$HP_T = (HP_a)\ (K)\ (N)$$

where

HP_a = horsepower of individual aerators, hp.

HP_T = total installed horsepower, hp.

K = number of aerators per basin.

N = number of basins.

2.33.9.5.11.2 Calculate electric energy required.

$$KWH = (HP_T)\ (365)\ (24)\ (.9)\ (.85)\ (.877)$$

where

KWH = electrical energy required, kwhr/yr.

HP_T = total installed horsepower, hp.

2.33.9.5.12 Calculate quantity for lagoon liner. In some areas the soils are such that lagoons must be lined to prevent loss of water. User should designate whether liner is required.

2.33.9.5.12.1 Calculate area of the bottom of the lagoon.

$$SA_B = (L_w - 6Dw)\ (W_w - 6Dw)$$

where

SA_B = area of the bottom of lagoon, ft^2.

L_w = length of basin at water level, ft.

W_w = width of basin at water level, ft.

DW = basin water depth, ft.

2.33.9.5.12.2 Calculate area of lagoon embankment.

$$SA_E = 2(L_T + W_T)\ (\frac{DW+2}{.3159} + 5)$$

where

SA_E = area of lagoon embankment, ft^2.

L_T = length of basin at top of levee, ft.

W_T = width of basin at top of levee, ft.

DW = basin water depth, ft.

2.33.9.5.12.3 Calculate total surface area required for liner.

$$SA_L = N\ (SA_B + SA_E)$$

where

SA_L = total area of lagoon liner, ft^2.

N = number of basins.

SA_B = area of bottom of lagoon, ft^2.

SA_E = area of lagoon embankment, ft^2.

2.33.9.5.13 Other operation and maintenance material and supply costs. This item includes such items as lubrication oil, paint, repair and replacement parts. These costs are estimated as a percent of the installed equipment cost.

$$OMMP = 4.93\ (HP_T)^{-0.1827}$$

where

OMMP = O&M maintenance and supply costs as percent of the installed equipment cost, %.

2.33.9.5.14 Other minor construction cost items. From the above calculations approximately 90% of the construction costs have been accounted for. Other minor items such as piping, grass seeding, etc., would be 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs items.

2.33.9.6 Quantities Calculations Output Data.

2.33.9.6.1 Number of basins, N.

2.33.9.6.2 Basin water depth, DW, ft.

2.33.9.6.3 Number of aerators per basin, K.

2.33.9.6.4 Basin length to width ratio, r.

2.33.9.6.5 Horsepower of individual aerators, HP_a, hp.

2.33.9.6.6 Length of basin at water level, L_w, ft.

2.33.9.6.7 Width of basin at water level, W_w, ft.

2.33.9.6.8 Volume of earthwork required, VLEW, cu ft.

2.33.9.6.9 Total volume of R.C. wall required, V_{cw}, cu ft.

2.33.9.6.10 Total volume of R.C. slab required, V_{cs}, cu ft.

2.33.9.6.11 Volume of concrete for embankment protection V_{cep}, cu ft.

2.33.9.6.12 Operation manpower required, OMH, MH/yr.

2.33.9.6.13 Maintenance manpower required, MMH, MH/yr.

2.33.9.6.14 Electrical energy required, KWH, kwhr/yr.

2.33.9.6.15 Total area of lagoon liner, SA_L, ft^2.

2.33.9.6.16 O&M maintenance and supply costs as percent of the installed equipment cost, OMMP, %.

2.33.9.6.17 Correction factor for other minor construction costs, CF.

2.33.9.7 Unit Price Input Required.

2.33.9.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.33.9.7.2 Unit price input for R.C. slab in-place, UPICW, \$/cu yd.

2.33.9.7.3 Unit price input for R.C. slab in-place UPICS, \$/cu yd.

2.33.9.7.4 Cost of standard size aerator (50 hp), COSTA, \$, (optional).

2.33.9.7.5 Current Márshall and Swift Equipment Cost Index, MSECI.

2.33.9.7.6 Installation labor rate, LABRI, \$/MH.

2.33.9.7.7 Unit price input for lagoon liner, UPILL, \$/ft^2.

2.33.9.8 Cost Calculations.

2.33.9.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.33.9.8.2 Cost of reinforced concrete wall.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, cu yd.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.33.9.8.3 Cost of reinforced concrete slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.33.9.8.4 Cost of concrete embankment protection.

$$COSTEP = \frac{V_{cep}}{27} (0.5)(UPICS)$$

where

COSTEP = cost of concrete embankment protection in-place, \$.

V_{cep} = volume of concrete for embankment protection, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

0.5 = factor to adjust R.C. concrete unit price to nonreinforced.

2.33.9.8.5 Calculate purchase cost of aerators.

$$COSTA = \frac{(COSTSA)\ (COSTR)\ (K)\ (N)}{100}$$

where

COSTA = purchase cost of aerators, \$.

COSTSA = cost of standard size aerator (50 hp), \$.

COSTR = cost of aerator of horsepower HP_a as a percent of the cost of the standard size aerators, %.

K = number of aerators per basin.

N = number of basins.

2.33.9.8.5.1 Calculate COSTR.

If $HP_a \leq 25$ hp; COSTR is calculated by:

$$COSTR = 20.7\ (HP_a)^{0.2686}$$

If $HP_a > 25$ hp; COSTR is calculated by:

$$COSTR = 4.12\ (HP_a)^{0.7878}$$

2.33.9.8.5.2 Purchase cost of standard size aerator. The standard size aerator is a 50 hp, high-speed floating aerator. The cost of the 50 hp aerator in the first quarter of 1977 is:

$$COSTSA = \$13,960$$

For better cost estimation, COSTSA should be obtained from the equipment vendor and treated as a unit price input. However, if COSTA is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTA = \$13,960 \frac{MSECI}{491.6}$$

where

COSTA = cost of standard size aerator (50 hp), $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

2.33.9.8.6 Calculate total installed equipment cost.

2.33.9.8.6.1 Calculate aerator installation labor.

$$IMH = 0.633\ (HP_a) + 40$$

where

IMH = aerator installation labor, MH.

HP_a = horsepower of individual aerators, hp.

2.33.9.8.6.2 Calculate aerator installation cost.

$$AIC = (IMH)\ (K)\ (N)\ (LABRI)$$

where

AIC = aerator installation cost, $.

IMH = aerator installation labor, MH.

K = number of aerators per basin.

N = number of basins.

LABRI = installation labor rate, $/MH.

2.33.9.8.6.3 Calculate installed cost for electrical/mechanical.

$$EMC = 0.589\ (HP_a)^{-0.1465}\ (COSTA)$$

where

EMC = installed cost for electrical/mechanical, \$.

HP_a = horsepower of individual aerators, hp.

$COSTA$ = purchase cost of aerators, \$.

2.33.9.8.6.4 Calculate total installed equipment cost.

$$IEC = COSTA + AIC + EMC$$

where

IEC = total installed equipment cost, \$.

AIC = aerator installation cost, \$.

EMC = installed cost of electrical/mechanical, \$.

2.33.9.8.7 Calculate cost of lagoon liner.

$$COSTLL = (SA_L)\ (UPILL)$$

$COSTLL$ = installed cost of lagoon liner, \$.

SA_L = total area of lagoon liner, ft^2.

$UPILL$ = unit price input for lagoon liner, \$/ft^2.

2.33.9.8.8 Calculate total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + COSTEP + COSTLL + IEC)\ (CF)$$

where

$TBCC$ = total bare construction cost, \$.

$COSTE$ = cost of earthwork, \$.

$COSTCW$ = cost of R.C. wall in-place, \$.

$COSTCS$ = cost of R.C. slab in-place, \$.

$COSTEP$ = cost of concrete embankment protection in-place, \$.

$COSTLL$ = installed cost of lagoon liner, \$.

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL. JAN 82 10/18

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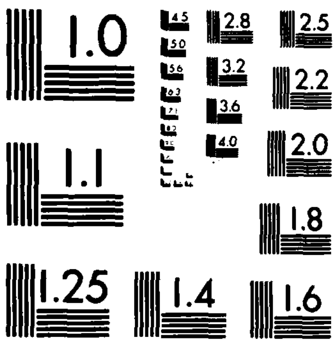

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

IEC = total installed equipment cost, \$.

2.33.9.8.9 Calculate O&M maintenance and supply cost.

$$OMMC = \frac{OMMP}{100} IEC$$

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as percent of installed equipment cost, %.

IEC = total installed equipment cost, \$.

2.33.9.9 Cost Calculations Output Data.

2.33.9.9.1 Total bare construction cost, TBCC, \$.

2.33.9.9.2 O&M material and supply costs, \$/yr.

2.33.10 Anaerobic Lagoons

2.33.10.1 Input Data.

2.33.10.1.1 Wastewater flow.

2.33.10.1.1.1 Average daily flow, mgd.

2.33.10.1.1.2 Peak hourly flow, mgd.

2.33.10.1.2 Wastewater strength, BOD_5, mg/l.

2.33.10.1.3 Other characteristics.

2.33.10.1.3.1 pH.

2.33.10.1.3.2 Temperature (maximum and minimum).

2.33.10.2 Design Parameters (See Table 2.33-1).

2.33.10.3 Process Design Calculations.

2.33.10.3.1 Calculate BOD_5 in the waste.

$$BOD = (Q_{avg}) (BODI) (8.34)$$

where

BOD = quantity of BOD_5 in waste, lb/day.

Q_{avg} = average daily flow, mgd.

$BODI$ = concentration of BOD_5 in influent, mg/l.

8.34 = conversion factor.

2.33.10.3.2 Determine lagoon surface area.

- Based on type of lagoon and climate select a loading rate (LBOD).

$$SA = \frac{BOD}{LBOD}$$

where

SA = lagoon surface area, acres.

BOD = quantity of BOD_5 in waste, lb/day.

$LBOD$ = lagoon loading rate, lb/day acre.

2.33.10.3.3 Determine volume of lagoon.

- Based on type of lagoon select an operating depth.

$$V = (SA)\ (D)\ (0.32585)$$

where

V = volume of lagoon, million gal.

SA = lagoon surface area, acres.

D = lagoon operating depth, ft.

0.32585 = conversion factor, acre ft to million gallons.

2.33.10.3.4 Determine detention time.

$$DT = \frac{V}{Q_{avg}}$$

where

DT = detention time, days.

V = volume of lagoon, million gal.

Q_{avg} = average daily flow, mgd.

Check detention time against minimum detention times in Table 2.33-1. If DT is less than minimum increase the surface area (SA) until the minimum detention time is obtained.

2.33.10.3.5 Effluent Characteristics.

2.33.10.3.5.1 Determine effluent BOD_5 concentration. The mechanisms in lagoons are complex and can not be accurately predicted, therefore effluent concentrations will be determined based on percent reduction from actual experience. The average soluble BOD_5 reduction for lagoons is 65%.

$$BODE = (1 - .65)\ (BODI)$$
$$S_e = 0.75\ BODE$$

where

BODE = concentration of BOD_5 in effluent, mg/l.

BODI = concentration of BOD_5 in influent, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.33-4A

TABLE 2.33-1
DESIGN PARAMETERS FOR STABILIZATION PONDS

Parameter	Type of Pond			
	Aerobic[a]	Facultative	Facultative	Anaerobic
Flow regime	Intermittently mixed	--	Mixed surface layer	--
Pond size, acres	10 multiples	2 to 10 multiples	2 to 10 multiples	0.5 to 2.0 multiples
Operation[b]	Series or parallel	Series or parallel	Series or parallel	Series
Detention time, days[b]	10 to 40	7 to 30	7 to 20	20 to 50
Depth, ft	3 to 4	3 to 6	3 to 8	8 to 15
pH	6.5 to 10.5	6.5 to 9.0	6.5 to 8.5	6.8 to 7.2
Temperature range, °C	0 to 40	0 to 50	0 to 50	6 to 50
Optimum temperature, °C	20	20	20	30
BOD_5 loading, lb/acre/day[c]	60 to 120[d]	15 to 50	30 to 100	200 to 500
BOD_5 conversion	60 to 70	60 to 70	60 to 70	50 to 70
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue
Algal concentration, mg/l	80 to 200	40 to 160	10 to 40	--
Effluent suspended solids, mg/l[e]	140 to 340	160 to 400	110 to 340	80 to 160

(a) Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produc
(b) Depends on climatic conditions.
(c) Typical values (much higher values have been applied at various loadings). Loading values are often specified by state control agencies.
(d) Some states limit this figure to 50 or less.
(e) Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/l and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/l.

2.33.10.3.5.2 Suspended Solids.

$$SSE = 100$$

where

SSE = effluent suspended solids concentration, mg/l.

2.33.10.3.5.3 COD

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.33.10.3.5.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

2.33.10.3.5.5 Phosphorus

$$PO4E = 0.7\ PO4$$

where

PO4 = influent phosphorus concentration, mg/l.

PO4E = effluent phosphorus concentration, mg/l.

2.33.10.3.5.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAG = influent oil and grease concentration, mg/l.

OAGE = effluent oil and grease concentration, mg/l.

2.33.10.3.5.7 pH.

$$PH = 6.8$$

where

PH = effluent pH.

2.33.10.4 Process Design Output Data.

2.33.10.4.1 Lagoon loading rate, LBOD, lb/day acre.

2.33.10.4.2 Lagoon surface area, SA, acres.

2.33.10.4.3 Lagoon operating depth, D, ft.

2.33.10.4.4 Volume of lagoon, V, million gal.

2.33.10.4.5 Concentration of BOD_5 in effluent, BODE, mg/l.

2.33.10.5 Quantities Calculations.

2.33.10.5.1 Determine quantity of earthwork.

The following assumptions are made concerning the construction of the lagoons.

A minimum of 2 cells will always be used.
Anaerobic lagoon cells will not be greater than 2 acres in surface area.
Lagoon cells will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
Lagoon cells will be square.

2.33.10.5.1.1 Determine the number and size of lagoon cells.

2.33.10.5.1.1.1 For Anaerobic lagoons.

$$\text{If } SA \leq 4, \ NLC = 2$$

$$\text{If } SA > 4, \ NLC = \frac{SA}{2}$$

NLC must be an even number.

$$CSA = \frac{SA}{NLC}$$

where

NLC = number of lagoon cells.

SA = lagoon surface area, acres.

CSA = lagoon cell surface area, acres.

2.33.10.5.1.2 Determine lagoon cell dimensions.

$$L = 208.7 \ (CSA)^{0.5} + 12$$

where

L = length of one side of lagoon cell, ft.

CSA = lagoon cell surface area, acres.

208.7 = conversion factor acres to sq ft.

12 = additional length required for 2 ft freeboard.

2.33.10.5.1.3 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = D + 2$$

$$VF = [3 \ (DF)^2 + 10 \ DF] \quad [\frac{5NLC}{2}] + 2 \quad (L)$$

$$VC = (1.3) \ (NLC) \ (DC) \quad [L^2 - 6(DF)(L) + 12 \ (DF)^2 + 120 \ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If $VC < VF$ then assume $DC > 1$ ft and recalculate VC and VF.
IF $VC > VF$ then assume $DC < 1$ ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = lagoon operating depth, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

2.33.10.5.2 Determine requirement for lagoon liner. In some areas due to soil conditions the lagoons must be lined with an impervious material to prevent percolation of the wastewater into the natural ground.

2.33.10.5.2.1 Calculate area to be lined.

$$ALL = NLC\ [(4)\ (3D)(L-3D-6) + (L-6D-12)^2]$$

where

ALL = area of lagoon liner, ft^2.

NLC = number of lagoon cells.

D = lagoon operating depth, ft.

L = length of one side of lagoon cell, ft.

2.33.10.5.3 Piping for lagoon cells.

Assume:

Pipes are flowing full.
Velocity is 3 fps.
Pipes going through the levee extend 10 ft past toe of the levee.

2.33.10.5.3.1 Determine pipe size.

$$DIA = 9.72\ (Q_{max})^{0.5}$$

The smallest pipe to be used will be 4 inches. If DIA < 4 inches set DIA = 4 inches. DIA must be one of the following 4, 6, 8, 10, 12, 14, etc. Always use next higher diameter.

where

DIA = pipe diameter, inches.

Q_{max} = peak hourly flow, mgd.

9.72 = combined conversion factors.

2.33.10.5.3.2 Determine length of pipe.

$$LDIA = (6D + 10)\ NLC$$

where

LDIA = length of pipe of diameter DIA, ft.

D = lagoon operating depth, ft.

NLC = number of lagoon cells.

2.33.10.5.4 Valve for lagoons. Each lagoon cell will be capable of being isolated by the use of valves. There will be one valve for each lagoon cell and the valves will be the same size as the pipe feeding the cell. The valves will be butterfly valves.

$$NBV = NLC + 1$$

$$DBV = DIA$$

where

NBV = number of valves.

NLC = number of lagoon cells.

DBV = diameter of valves, inches.

DIA = pipe diameter, inches.

2.33.10.5.5 Effluent structure. The effluent structure for all flows in this range is assumed to be a concrete structure 4 feet by 4 feet with 6" thick walls. The depth will be the same as the total depth of the lagoon.

$$V_{cw} = (8)(D+2)$$

$$V_{cs} = 8$$

where

V_{cw} = volume of concrete wall, ft^3.

D = lagoon operating depth.

V_{cs} = volume of concrete slab, ft^3.

2.33.10.5.6 Calculate operation and maintenance manpower.

$$\text{If } Q_{avg} \leq 0.1; \quad OMMH = 160$$

$$\text{If } Q_{avg} > 0.1; \quad OMMH = 313.8\ (Q_{avg})^{0.2925}$$

where

Q_{avg} = average daily flow, mgd.

$OMMH$ = operation and maintenance manhours, MH/yr.

2.33.10.5.7 Other miscellaneous construction costs. The item already calculated represents approximately 90% of the construction cost. The other 10% consists of items such as seeding, miscellaneous concrete pads, walkways, etc.

$$CF = \frac{1}{.9} = 1.11$$

where

CF = correction factor for miscellaneous construction.

2.33.10.6 Quantities Calculations Output Data.

2.33.10.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

2.33.10.6.2 Area of lagoon liner, ALL, ft^2.

2.33.10.6.3 Pipe diameter, DIA, inches.

2.33.10.6.4 Length of pipe of diameter DIA, LDIA, ft.

2.33.10.6.5 Number of valves, NBV.

2.33.10.6.6 Diameter of valves, DBV, inches.

2.33.10.6.7 Volume of concrete wall, V_{cw}, ft^3.

2.33.10.6.8 Volume of concrete slab, V_{cs}, ft^3.

2.33.10.6.9 Operation and maintenance manpower, OMMH, MH/yr.

2.33.10.6.10 Correction factor for miscellaneous construction, CF.

2.33.10.7 Unit Price Input Required.

2.33.10.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

2.33.10.7.2 Unit price input for concrete wall, UPICW, $/cu yd.

2.33.10.7.3 Unit price input for concrete slab, UPICS, $/cu yd.

2.33.10.7.4 Cost of standard size pipe (12" Ø), COSP, $/ft.

2.33.10.7.5 Cost of standard size valve (12" butterfly), COSTSV, $.

2.33.10.8 Cost Calculations.

2.33.10.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

27 = conversion from ft^3 to cu yd.

2.33.10.8.2 Calculate cost of piping.

2.33.10.8.2.1 Installed cost of pipe.

$$ICP = \frac{COSTP}{100} (COSP) (LDIA)$$

where

ICP = installed cost of pipe, \$.

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12"Ø), \$/ft.

LDIA = length of pipe of diameter DIA, ft.

2.33.10.8.2.2 Determine COSTP.

$$COSTP = 6.842 (DIA)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

DIA = pipe diameter, inches.

2.33.10.8.2.3 Determine COSP. COSP is the cost per foot of 12" Ø welded steel pipe. This cost is \$13.50 per foot in 4th quarter, 1977.

2.33.10.8.3 Calculate cost of concrete.

2.33.10.8.3.1 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})}{27} (UPICW)$$

where

COSTCW = cost of concrete wall, \$.

V_{cw} = volume of concrete wall, ft^3.

UPICW = unit price input for concrete wall, \$/cu yd.

27 = conversion factor ft^3 to cu yd.

2.33.10.8.3.2 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})}{27} UPICS$$

where

COSTCS = cost of concrete slab, \$.

V_{cs} = volume of concrete slab, ft^3.

UPICS = unit price input for concrete slab, \$/cu yd.

27 = conversion factor from ft^3 to cu yd.

2.33.10.8.4 Calculate cost of valves.

2.33.10.8.4.1 Installed cost of valves.

$$IBV = \frac{(COSTBV)\ (COSTSV)\ (NBV)}{100}$$

where

IBV = installed cost of valves, \$.

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

COSTSV = Cost of standard size valve, \$.

NBV = number of valves.

2.33.10.8.4.2 Determine COSTBV.

$$COSTBV = 3.99\ (DBV)^{1.395}$$

where

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

DBV = diameter of the valve, inches.

2.33.10.8.4.2 Determine COSTSV. COSTSV is the cost of a 12" ɓ butterfly valve suitable for water service. This cost if $1004 for 4th quarter, 1977.

2.33.10.8.5 Calculate total bare construction cost.

$$TBCC = (COSTE + ICP + COSTCW + COSTCS + IBV)\ CF$$

where

TBCC = total bare construction cost, $.

COSTE = cost of earthwork, $.

ICP = installed cost of pipe, $.

COSTCW = cost of concrete wall, $.

COSTCS = cost of concrete slab, $.

IBV = installed cost of valves, $.

CF = correction factor for miscellaneous construction.

2.33.10.9 Cost Calculations Output Data.

2.33.10.9.1 Total bare construction cost, TBCC, $.

2.33.11 Facultative Lagoon

2.33.11.1 Input Data.

2.33.11.1.1 Wastewater flow.

2.33.11.1.1.1 Average daily flow, mgd.

2.33.11.1.1.2 Peak hourly flow, mgd.

2.33.11.1.2 Wastewater strength, BOD_5, mg/l.

2.33.11.1.3 Other characteristics.

2.33.11.1.3.1 pH.

2.33.11.1.3.2 Temperature (maximum and minimum).

2.33.11.2 Design Parameters (See Table 2.33-2).

2.33.11.3 Process Design Calculations.

2.33.11.3.1 Calculate BOD_5 in the waste.

$$BOD = (Q_{avg})\ (BODI)\ (8.34)$$

where

BOD = quantity of BOD_5 in waste, lb/day.

Q_{avg} = average daily flow, mgd.

BODI = concentration of BOD_5 in influent, mg/l.

8.34 = conversion factor.

2.33.11.3.2 Determine lagoon surface area.

- Based on type of lagoon and climate select a loading rate (LBOD).

$$SA = \frac{BOD}{LBOD}$$

where

SA = lagoon surface area, acres.

BOD = quantity of BOD_5 in waste, lb/day.

LBOD = lagoon loading rate, lb/day acre.

2.33.11.3.3 Determine volume of lagoon.

- Based on type of lagoon select an operating depth.

$$V = (SA)\ (D)\ (0.32585)$$

where

V = volume of lagoon, million gal.

SA = lagoon surface area, acres.

D = lagoon operating depth, ft.

0.32585 = conversion factor, acre ft to million gallons.

2.33.11.3.4 Determine detention time.

$$DT = \frac{V}{Q_{avg}}$$

where

DT = detention time, days.

V = volume of lagoon, million gal.

Q_{avg} = average daily flow, mgd.

Check detention time against minimum detention times in Table 2.33-2. If DT is less than minimum increase the surface area (SA) until the minimum detention time is obtained.

2.33.11.3.5 Effluent Characteristics.

2.33.11.3.5.1 Determine effluent BOD_5 concentration. The mechanisms in lagoons are complex and can not be accurately predicted, therefore effluent concentrations will be determined based on percent reduction from actual experience. The average soluble BOD_5 reduction for lagoons is 65%.

$$BODE = (1 - .65)\ (BODI)$$
$$S_e = 0.75\ BODE$$

where

BODE = concentration of BOD_5 in effluent, mg/l.

BODI = concentration of BOD_5 in influent, mg/l.

S_e = effluent soluble BOD_5, mg/l.

2.33-61

TABLE 2.33-2
DESIGN PARAMETERS FOR STABILIZATION PONDS

	Type of Pond			
Parameter	Aerobic[(a)]	Facultative	Facultative	Anaerobic
Flow regime	Intermittently mixed	--	Mixed surface layer	--
Pond size, acres	10 multiples	2 to 10 multiples	2 to 10 multiples	0.5 to 2.0 multiples
Operation[(b)]	Series or parallel	Series or parallel	Series or parallel	Series
Detention time, days[(b)]	10 to 40	7 to 30	7 to 20	20 to 50
Depth, ft	3 to 4	3 to 6	3 to 8	8 to 15
pH	6.5 to 10.5	6.5 to 9.0	6.5 to 8.5	6.8 to 7.2
Temperature range, °C	0 to 40	0 to 50	0 to 50	6 to 50
Optimum temperature, °C	20	20	20	30
BOD_5 loading, lb/acre/day[(c)]	60 to 120[(d)]	15 to 50	30 to 100	200 to 500
BOD_5 conversion	60 to 70	60 to 70	60 to 70	50 to 70
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue
Algal concentration, mg/l	80 to 200	40 to 160	10 to 40	--
Effluent suspended solids, mg/l[(e)]	140 to 340	160 to 400	110 to 340	80 to 160

(a) Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produce
(b) Depends on climatic conditions.
(c) Typical values (much higher values have been applied at various loadings). Loading values are often specified by state control agencies.
(d) Some states limit this figure to 50 or less.
(e) Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/l and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/l.

2.33.11.3.5.2 Suspended Solids.

$$SSE = 100$$

where

SSE = effluent suspended solids concentration, mg/l.

2.33.11.3.5.3 COD

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.33.11.3.5.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

2.33.11.3.5.5 Phosphorus

$$PO4E = 0.7\ PO4$$

where

PO4 = influent phosphorus concentration, mg/l.

PO4E = effluent phosphorus concentration, mg/l.

2.33.11.3.5.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAG = influent oil and grease concentration, mg/l.

OAGE = effluent oil and grease concentration, mg/l.

2.33.11.3.5.7 pH.

$$PH = 6.8$$

where

PH = effluent pH.

2.33.11.4 Process Design Output Data.

2.33.11.4.1 Lagoon loading rate, LBOD, lb/day acre.

2.33.11.4.2 Lagoon surface area, SA, acres.

2.33.11.4.3 Lagoon operating depth, D, ft.

2.33.11.4.4 Volume of lagoon, V, million gal.

2.33.11.4.5 Concentration of BOD_5 in effluent, BODE, mg/l.

2.33.11.5 Quantities Calculations.

2.33.11.5.1 Determine quantity of earthwork.

The following assumptions are made concerning the construction of the lagoons.

A minimum of 2 cells will always be used.
Facultative lagoon cells will not be greater than 10 acres in surface area.
Lagoon cells will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
Lagoon cells will be square.

2.33.11.5.1.1 Determine the number and size of lagoon cells.

2.33.11.5.1.1.1 For facultative lagoons.

$$\text{If } SA \leq 20 \quad NLC = 2$$

$$\text{If } SA > 20, \; NLC = \frac{SA}{10}$$

$$CSA = \frac{SA}{NLC}$$

where

NLC = number of lagoon cells.

SA = lagoon surface area, acres.

CSA = lagoon cell surface area, acres.

2.33.11.5.1.2 Determine lagoon cell dimensions.

$$L = 208.7 \; (CSA)^{0.5} + 12$$

where

L = length of one side of lagoon cell, ft.

CSA = lagoon cell surface area, acres.

208.7 = conversion factor acres to sq ft.

12 = additional length required for 2 ft freeboard.

2.33.11.5.1.3 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = D + 2$$

$$VF = [\; 3 \; (DF)^2 + 10 \; DF \;] \quad [\frac{5NLC}{2} + 2 \;] \quad (L)$$

$$VC = (1.3) \; (NLC) \; (DC) \; [\; L^2 - 6(DF)(L) + 12 \; (DF)^2 + 120 \; DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC < VF then assume DC > 1 ft and recalculate VC and VF.
IF VC > VF then assume DC < 1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = lagoon operating depth, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

2.33.11.5.2 Detemine requirement for lagoon liner. In some areas due to soil conditions the lagoons must be lined with an impervious material to prevent percolation of the wastewater into the natural ground.

2.33.11.5.2.1 Calculate area to be lined.

$$ALL = NLC\ [(4)\ (3D)(L-3D-6) + (L-6D-12)^2]$$

where

ALL = area of lagoon liner, ft^2.

NLC = number of lagoon cells.

D = lagoon operating depth, ft.

L = length of one side of lagoon cell, ft.

2.33.11.5.3 Piping for lagoon cells.

Assume:

Pipes are flowing full.
Velocity is 3 fps.
Pipes going through the levee extend 10 ft past toe of the levee.

2.33.11.5.3.1 Detemine pipe size.

$$DIA = 9.72\ (Q_{max})^{0.5}$$

The smallest pipe to be used will be 4 inches. If DIA 4 inches set DIA = 4 inches. DIA must be one of the following 4, 6, 8, 10, 12, 14, etc. Always use next higher diameter.

where

DIA = pipe diameter, inches.

Q_{max} = peak hourly flow, mgd.

9.72 = combined conversion factors.

2.33.11.5.3.2 Detemine length of pipe.

$$LDIA = (6D + 10)\ NLC$$

where

LDIA = length of pipe of diameter DIA, ft.

D = lagoon operating depth, ft.

NLC = number of lagoon cells.

2.33.11.5.4 Valve for lagoons. Each lagoon cell will be capable of being isolated by the use of valves. There will be one valve for each lagoon cell and the valves will be the same size as the pipe feeding the cell. The valves will be butterfly valves.

$$NBV = NLC + 1$$

$$DBV = DIA$$

where

NBV = number of valves.

NLC = number of lagoon cells.

DBV = diameter of valves, inches.

DIA = pipe diameter, inches.

2.33.11.5.5 Effluent structure. The effluent structure for all flows in this range is assumed to be a concrete structure 4 feet by 4 feet with 6" thick walls. The depth will be the same as the total depth of the lagoon.

$$V_{cw} = (8)(D+2)$$

$$V_{cs} = 8$$

where

V_{cw} = volume of concrete wall, ft^3.

D = lagoon operating depth.

V_{cs} = volume of concrete slab, ft^3.

2.33.11.5.6 Calculate operation and maintenance manpower.

$$\text{If } Q_{avg} \leq 0.1 \quad OMMH = 160$$

$$\text{If } Q_{avg} > 0.1 \quad OMMH = 313.8\ (Q_{avg})^{0.2925}$$

where

Q_{avg} = average daily flow, mgd.

OMMH = operation and maintenance manhours, MH/yr.

2.33.11.5.7 Other miscellaneous construction costs. The item already calculated represents approximately 90% of the construction cost. The other 10% consists of items such as seeding, miscellaneous concrete pads, walkways, etc.

$$CF = \frac{1}{.9} = 1.11$$

where

CF = correction factor for miscellaneous construction.

2.33.11.6 <u>Quantities Calculations Output Data</u>.

2.33.11.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

2.33.11.6.2 Area of lagoon liner, ALL, ft^2.

2.33.11.6.3 Pipe diameter, DIA, inches.

2.33.11.6.4 Length of pipe of diameter DIA, LDIA, ft.

2.33.11.6.5 Number of valves, NBV.

2.33.11.6.6 Diameter of valves, DBV, inches.

2.33.11.6.7 Volume of concrete wall, V_{cw}, ft^3.

2.33.11.6.8 Volume of concrete slab, V_{cs}, ft^3.

2.33.11.6.9 Operation and maintenance manpower, OMMH, MH/yr.

2.33.11.6.10 Correction factor for miscellaneous construction, CF.

2.33.11.7 Unit Price Input Required.

2.33.11.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.33.11.7.2 Unit price input for concrete wall, UPICW, \$/cu yd.

2.33.11.7.3 Unit price input for concrete slab, UPICS, \$/cu yd.

2.33.11.7.4 Cost of standard size pipe (12" Ø), COSP, \$/ft.

2.33.11.7.5 Cost of standard size valve (12" butterfly), COSTSV, \$.

2.33.11.8 Cost Calculations.

2.33.11.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

27 = conversion from ft^3 to cu yd.

2.33.11.8.2 Calculate cost of piping.

2.33.11.8.2.1 Installed cost of pipe.

$$ICP = \frac{COSTP}{100} (COSP) (LDIA)$$

where

ICP = installed cost of pipe, \$.

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12"ϕ), \$/ft.

LDIA = length of pipe of diameter DIA, ft.

2.33.11.8.2.2 Determine COSTP.

$$COSTP = 6.842\ (DIA)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

DIA = pipe diameter, inches.

2.33.11.8.2.3 Determine COSP. COSP is the cost per foot of 12" ϕ welded steel pipe. This cost is \$13.50 per foot in 4th quarter, 1977.

2.33.11.8.3 Calculate cost of concrete.

2.33.11.8.3.1 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})}{27}\ (UPICW)$$

where

COSTCW = cost of concrete wall, \$.

V_{cw} = volume of concrete wall, ft^3.

UPICW = unit price input for concrete wall, \$/cu yd.

27 = conversion factor ft^3 to cu yd.

2.33.11.8.3.2 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})}{27}\ UPICS$$

where

COSTCS = cost of concrete slab, \$.

V_{cs} = volume of concrete slab, ft^3.

UPICS = unit price input for concrete slab, \$/cu yd.

27 = conversion factor from ft^3 to cu yd.

2.33.11.8.4 Calculate cost of valves.

2.33.11.8.4.1 Installed cost of valves.

$$IBV = \frac{(COSTBV)\ (COSTSV)\ (NBV)}{100}$$

where

IBV = installed cost of valves, $.

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

COSTSV = Cost of standard size valve, $.

NBV = number of valves.

2.33.11.8.4.2 Determine COSTBV.

$$COSTBV = 3.99\ (DBV)^{1.395}$$

where

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

DBV = diameter of the valve, inches.

2.33.11.8.4.3 Determine COSTSV. COSTSV is the cost of a 12" ∅ butterfly valve suitable for water service. This cost if $1004 for 4th quarter, 1977.

2.33.11.8.5 Calculate total bare construction cost.

$$TBCC = (COSTE + ICP + COSTCW + COSTCS + IBV)\ CF$$

where

TBCC = total bare construction cost, $.

COSTE = cost of earthwork, $.

ICP = installed cost of pipe, $.

COSTCW = cost of concrete wall, $.

COSTCS = cost of concrete slab, $.

IBV = installed cost of valves, $.

CF = correction factor for miscellaneous construction.

2.33.11.9 <u>Cost Calculations Output Data.</u>

2.33.11.9.1 Total bare construction cost, TBCC, $.

2.33.12 Oxidation Lagoon.

2.33.12.1 Input Data.

2.33.12.1.1 Wastewater flow.

2.33.12.1.1.1 Average daily flow, mgd.

2.33.12.1.1.2 Peak hourly flow, mgd.

2.33.12.1.2 Wastewater strength, BOD_5, mg/l.

2.33.12.1.3 Other characteristics.

2.33.12.1.3.1 pH.

2.33.12.1.3.2 Temperature (maximum and minimum).

2.33.12.2 Design Parameters. (See Table 2.33-3).

2.33.12.3 Process Design Calculations.

2.33.12.3.1 Calculate BOD_5 in the waste.

$$BOD = (Q_{avg})\ (BODI)\ (8.34)$$

where

BOD = quantity of BOD_5 in waste, lb/day.

Q_{avg} = average daily flow, mgd.

BODI = concentration of BOD_5 in influent, mg/l.

8.34 = conversion factor.

2.33.12.3.2 Determine lagoon surface area.

- Based on type of lagoon and climate select a loading rate (LBOD).

$$SA = \frac{BOD}{LBOD}$$

where

SA = lagoon surface area, acres.

BOD = quantity of BOD_5 in waste, lb/day.

LBOD = lagoon loading rate, lb/day acre.

2.33.12.3.3 Determine volume of lagoon.

- Based on type of lagoon select an operating depth.

$$V = (SA)\ (D)\ (0.32585)$$

where

V = volume of lagoon, million gal.

SA = lagoon surface area, acres.

D = lagoon operating depth, ft.

0.32585 = conversion factor, acre ft to million gallons.

2.33.12.3.4 Determine detention time.

$$DT = \frac{V}{Q_{avg}}$$

where

DT = detention time, days.

V = volume of lagoon, million gal.

Q_{avg} = average daily flow, mgd.

Check detention time against minimum detention times in Table 2.33-3. If DT is less than minimum increase the surface area (SA) until the minimum detention time is obtained.

2.33.12.3.5 Effluent Characteristics.

2.33.12.3.5.1 Determine effluent BOD_5 concentration. The mechanisms in lagoons are complex and can not be accurately predicted, therefore effluent concentrations will be determined based on percent reduction from actual experience. The average soluble BOD_5 reduction for lagoons is 65%.

$$BODE = (1 - .65)\ (BODI)$$

where

$BODE$ = concentration of BOD_5 in effluent, mg/l.

$BODI$ = concentration of BOD_5 in influent, mg/l.

TABLE 2.33-3
DESIGN PARAMETERS FOR STABILIZATION PONDS

Parameter	Aerobic[(a)]	Facultative	Facultative	Anaerobic
	Type of Pond			
Flow regime	Intermittently mixed	--	Mixed surface layer	--
Pond size, acres	10 multiples	2 to 10 multiples	2 to 10 multiples	0.5 to 2.0 multiples
Operation[(b)]	Series or parallel	Series or parallel	Series or parallel	Series
Detention time, days[(b)]	10 to 40	7 to 30	7 to 20	20 to 50
Depth, ft	3 to 4	3 to 6	3 to 8	8 to 15
pH	6.5 to 10.5	6.5 to 9.0	6.5 to 8.5	6.8 to 7.2
Temperature range, ^{o}C	0 to 40	0 to 50	0 to 50	6 to 50
Optimum temperature, ^{o}C	20	20	20	30
BOD_5 loading, lb/acre/day[(c)]	60 to 120[(d)]	15 to 50	30 to 100	200 to 500
BOD_5 conversion	60 to 70	60 to 70	60 to 70	50 to 70
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue
Algal concentration, mg/l	80 to 200	40 to 160	10 to 40	--
Effluent suspended solids, mg/l[(e)]	140 to 340	160 to 400	110 to 340	80 to 160

(a) Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produce
(b) Depends on climatic conditions.
(c) Typical values (much higher values have been applied at various loadings). Loading values are often specified by state control agencies.
(d) Some states limit this figure to 50 or less.
(e) Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/l and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/l.

2.33.12.3.5.2 Suspended Solids.

$$SSE = 100$$

where

SSE = effluent suspended solids concentration, mg/l.

2.33.12.3.5.3 COD

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.33.12.3.5.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

2.33.12.3.5.5 Phosphorus

$$PO4E = 0.7\ PO4$$

where

PO4 = influent phosphorus concentration, mg/l.

PO4E = effluent phosphorus concentration, mg/l.

2.33.12.3.5.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAG = influent oil and grease concentration, mg/l.

OAGE = effluent oil and grease concentration, mg/l.

2.33.12.3.5.7 pH.

$$PH = 6.8$$

where

PH = effluent pH.

2.33.12.4 Process Design Output Data.

2.33.12.4.1 Lagoon loading rate, LBOD, lb/day acre.

2.33.12.4.2 Lagoon surface area, SA, acres.

2.33.12.4.3 Lagoon operating depth, D, ft.

2.33.12.4.4 Volume of lagoon, V, million gal.

2.33.12.4.5 Concentration of BOD_5 in effluent, BODE, mg/l.

2.33.12.5 Quantities Calculations.

2.33.12.5.1 Determine quantity of earthwork.

The following assumptions are made concerning the construction of the lagoons.

A minimum of 2 cells will always be used.
Oxidation lagoon cells will not be greater than 10 acres in surface area.
Lagoon cells will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
Lagoon cells will be square.

2.33.12.5.1.1 Detemine the number and size of lagoon cells.

2.33.12.5.1.1.1 For oxidation lagoons.

$$\text{If } SA \leq 20 \quad NLC = 2$$

$$\text{If } SA > 20,\ NLC = \frac{SA}{10}$$

$$CSA = \frac{SA}{NLC}$$

where

NLC = number of lagoon cells.

SA = lagoon surface area, acres.

CSA = lagoon cell surface area, acres.

2.33.12.5.1.2 Detemine lagoon cell dimensions.

$$L = 208.7\ (CSA)^{0.5} + 12$$

where

L = length of one side of lagoon cell, ft.

CSA = lagoon cell surface area, acres.

208.7 = conversion factor acres to sq ft.

12 = additional length required for 2 ft freeboard.

2.33.12.5.1.3 Calculate volume of earthwork required for lagoons. The volume of earthwork must be detemined by trial and error using the following equations:

$$DC + DF = D + 2$$

$$VF = [\ 3\ (DF)^2 + 10\ DF\ ]\ [\frac{5NLC}{2} + 2\ ]\quad (L)$$

$$VC = (1.3)\ (NLC)\ (DC)\ [\ L^2 - 6(DF)(L) + 12\ (DF)^2 + 120\ DF - 60L + 1200\ ]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC < VF then assume DC > 1 ft and recalculate VC and VF.
IF VC > VF then assume DC < 1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = lagoon operating depth, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

2.33.12.5.2 Determine requirement for lagoon liner. In some areas due to soil conditions the lagoons must be lined with an impervious material to prevent percolation of the wastewater into the natural ground.

2.33.12.5.2.1 Calculate area to be lined.

$$ALL = NLC\ [(4)\ (3D)(L-3D-6) + (L-6D-12)^2]$$

where

ALL = area of lagoon liner, ft^2.

NLC = number of lagoon cells.

D = lagoon operating depth, ft.

L = length of one side of lagoon cell, ft.

2.33.12.5.3 Piping for lagoon cells.

Assume:

Pipes are flowing full.
Velocity is 3 fps.
Pipes going through the levee extend 10 ft past toe of the levee.

2.33.12.5.3.1 Determine pipe size.

$$DIA = 9.72\ (Q_{max})^{0.5}$$

The smallest pipe to be used will be 4 inches. If DIA < 4 inches set DIA = 4 inches. DIA must be one of the following 4, 6, 8, 10, 12, 14, etc. Always use next higher diameter.

where

DIA = pipe diameter, inches.

Q_{max} = peak hourly flow, mgd.

9.72 = combined conversion factors.

2.33.12.5.3.2 Determine length of pipe.

$$LDIA = (6D + 10)\ NLC$$

where

LDIA = length of pipe of diameter DIA, ft.

D = lagoon operating depth, ft.

NLC = number of lagoon cells.

2.33.12.5.4 Valve for lagoons. Each lagoon cell will be capable of being isolated by the use of valves. There will be one valve for each lagoon cell and the valves will be the same size as the pipe feeding the cell. The valves will be butterfly valves.

$$NBV = NLC + 1$$

$$DBV = DIA$$

where

NBV = number of valves.

NLC = number of lagoon cells.

DBV = diameter of valves, inches.

DIA = pipe diameter, inches.

2.33.12.5.5 Effluent structure. The effluent structure for all flows in this range is assumed to be a concrete structure 4 feet by 4 feet with 6" thick walls. The depth will be the same as the total depth of the lagoon.

$$V_{cw} = (8)(D+2)$$

$$V_{cs} = 8$$

where

V_{cw} = volume of concrete wall, ft^3.

D = lagoon operating depth.

V_{cs} = volume of concrete slab, ft^3.

2.33.12.5.6 Calculate operation and maintenance manpower.

$$\text{If } Q_{avg} \leq 0.1 \quad OMMH = 160$$

$$\text{If } Q_{avg} > 0.1 \quad OMMH = 313.8\ (Q_{avg})^{0.2925}$$

where

Q_{avg} = average daily flow, mgd.

OMMH = operation and maintenance manhours, MH/yr.

2.33.12.5.7 Other miscellaneous construction costs. The item already calculated represents approximately 90% of the construction cost. The other 10% consists of items such as seeding, miscellaneous concrete pads, walkways, etc.

$$CF = \frac{1}{.9} = 1.11$$

where

CF = correction factor for miscellaneous construction.

2.33.12.6 <u>Quantities Calculations Output Data.</u>

2.33.12.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

2.33.12.6.2 Area of lagoon liner, ALL, ft^2.

2.33.12.6.3 Pipe diameter, DIA, inches.

2.33.12.6.4 Length of pipe of diameter DIA, LDIA, ft.

2.33.12.6.5 Number of valves, NBV.

2.33.12.6.6 Diameter of valves, DBV, inches.

2.33.12.6.7 Volume of concrete wall, V_{cw}, ft^3.

2.33.12.6.8 Volume of concrete slab, V_{cs}, ft^3.

2.33.12.6.9 Operation and maintenance manpower, OMMH, MH/yr.

2.33.12.6.10 Correction factor for miscellaneous construction, CF.

2.33.12.7 Unit Price Input Required.

2.33.12.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.33.12.7.2 Unit price input for concrete wall, UPICW, \$/cu yd.

2.33.12.7.3 Unit price input for concrete slab, UPICS, \$/cu yd.

2.33.12.7.4 Cost of standard size pipe (12" Ø), COSP, \$/ft.

2.33.12.7.5 Cost of standard size valve (12" butterfly), COSTSV, \$.

2.33.12.8 Cost Calculations.

2.33.12.8.1 Calculate cost of earthwork.

$$\text{COSTE} = \frac{\text{VLEW}}{27}\ \text{UPIEX}$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

27 = conversion from ft^3 to cu yd.

2.33.12.8.2 Calculate cost of piping.

2.33.12.8.2.1 Installed cost of pipe.

$$\text{ICP} = \frac{\text{COSTP}}{100}\ (\text{COSP})\ (\text{LDIA})$$

where

ICP = installed cost of pipe, \$.

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12"ϕ), \$/ft.

LDIA = length of pipe of diameter DIA, ft.

2.33.12.8.2.2 Determine COSTP.

$$COSTP = 6.842\ (DIA)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

DIA = pipe diameter, inches.

2.33.12.8.2.3 Determine COSP. COSP is the cost per foot of 12" ϕ welded steel pipe. This cost is \$13.50 per foot in 4th quarter, 1977.

2.33.12.8.3 Calculate cost of concrete.

2.33.12.8.3.1 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})}{27}\ (UPICW)$$

where

COSTCW = cost of concrete wall, \$.

V_{cw} = volume of concrete wall, ft^3.

UPICW = unit price input for concrete wall, \$/cu yd.

27 = conversion factor ft^3 to cu yd.

2.33.12.8.3.2 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})}{27}\ UPICS$$

where

COSTCS = cost of concrete slab, \$.

V_{cs} = volume of concrete slab, ft^3.

UPICS = unit price input for concrete slab, \$/cu yd.

27 = conversion factor from ft^3 to cu yd.

2.33.12.8.4 Calculate cost of valves.

2.33.12.8.4.1 Installed cost of valves.

$$IBV = \frac{(COSTBV)\ (COSTSV)\ (NBV)}{100}$$

where

IBV = installed cost of valves, \$.

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

COSTSV = Cost of standard size valve, \$.

NBV = number of valves.

2.33.12.8.4.2 Determine COSTBV.

$$COSTBV = 3.99\ (DBV)^{1.395}$$

where

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

DBV = diameter of the valve, inches.

2.33.12.8.4.2 Determine COSTSV. COSTSV is the cost of a 12" ∅ butterfly valve suitable for water service. This cost if \$1004 for 4th quarter, 1977.

2.33.12.8.5 Calculate total bare construction cost.

$$TBCC = (COSTE + ICP + COSTCW + COSTCS + IBV)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

ICP = installed cost of pipe, \$.

COSTCW = cost of concrete wall, \$.

COSTCS = cost of concrete slab, \$.

IBV = installed cost of valves, $.

CF = correction factor for miscellaneous construction.

2.33.12.9 Cost Calculations Output Data.

2.33.12.9.1 Total bare construction cost, TBCC, $.

2.33.13 Sludge Lagoon.

2.33.13.1 Input Data.

2.33.13.1.1 Sludge flow, Q_s, gpd.

2.33.13.1.2 Initial solids content in sludge, S_o, %.

2.33.13.1.3 Solids loading rate, LRS, lb/yr cu ft (2.2 to 2.4 lb/yr cu ft.)

2.33.13.2 Design Parameters.

2.33.13.2.1 Solids loading rate, lb/yr cu ft.

2.33.13.2.2 Sludge characteristics.

2.33.13.2.3 Soil permeability.

2.33.13.3 Process Design Calculations.

2.33.13.3.1 Calculate dry solids produced.

$$DSP = \frac{(Q_s)(S_o)(8.34)(365)}{100}$$

where

DSP = dry solids produced, lb/yr.

S_o = initial solids content in sludge, %.

8.34 = conversion from gal to lb, lb/gal.

365 = conversion, days/yr.

2.33.13.3.2 Calculate volume of lagoons.

$$LV = \frac{DSP}{LRS}$$

where

LV = lagoon volume, cu ft.

DSP = dry solids produced, lb/yr.

LRS = solids loading rate, lb/yr cu ft.

2.33.13.3.3 Calculate lagoon surface area.

$$TLSA = \frac{LV}{D}$$

where

TLSA = total lagoon surface area, sq ft.

LV = lagoon volume, cu ft.

D = sludge depth in lagoon, ft.

2.33.13.3.4 Calculate number of lagoons. There should always be a minimum of 2 lagoons so that one is drying while the other is being filled. In this flow range no more than 2 lagoons should be required.

$$NL = 2$$

where

NL = number of lagoons.

2.33.13.3.5 Final sludge volume.

$$Q_f = \frac{(Q_s)(So)}{30}$$

where

Q_f final sludge volume, gpd.

Q_s = initial sludge volume, gpd.

So = initial solids content, %.

30 = final solids content, %.

2.33.13.4 Process Design Output Data.

2.33.13.4.1 Dry solids produced, DSP, lb/yr.

2.33.13.4.2 Sludge flow, Q_s, gpd.

2.33.13.4.3 Initial solid content in sludge, S_o, %.

2.33.13.4.4 Solids loading rate, LRS, lb/yr cu ft.

2.33.13.4.5 Sludge depth in lagoon, D, ft.

2.33.13.4.6 Lagoon volume, LV, cu ft.

2.33.13.4.7 Total lagoon surface area, TLSA, sq ft.

2.33.13.4.8 Number of lagoons, NL.

2.33.13.5 <u>Quantities Calculations</u>.

2.33.13.5.1 Assumptions. The following assumptions are made concerning the construction of the lagoons.

2.33.13.5.1.1 The levees will have a 3 to 1 side slope.

2.33.13.5.1.2 The levees will have a 10 ft wide flat top for access.

2.33.13.5.1.3 The lagoons will be constructed with equal cut and fill.

2.33.13.5.1.4 There will be a minimum of 2 lagoons for operational purposes.

2.33.13.5.1.5 The sludge depth in the lagoons will be a maximum of 2 ft with 2 ft of freeboard.

2.33.13.5.1.6 Common levee construction will be used.

2.33.13.5.1.7 Lagoons will be square.

2.33.13.5.2 Calculate surface area per lagoon.

$$SAL = \frac{TLSA}{2}$$

where

SAL = surface area per lagoon, sq ft.

TLSA = total lagoon surface area, sq ft.

2 = number of lagoons.

2.33.13.5.3 Calculate dimensions of lagoon.

$$L = (SAL)^{0.5} + 12$$

where

L = length of one side of lagoon at top of levee, ft.

SAL = surface area per lagoon, sq ft.

12 = additional length for 2 ft freeboard.

2.33.13.5.4 Calculate volume of earthwork required. The volume of earthwork must be determined by trial and error using the following equations:

$$DF + DC = D + 2$$

$$VF = [3(DF)^2 + 10\ DF]\ [7L]$$

$$VC = (2.6)(DC)\ [L^2 - 6(DF)(L) + 12(DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 foot. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

IF VC $<$ VF then assume DC $>$ 1 ft and recalculate VC and VF.
IF VC $>$ VF then assume DC $<$ 1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = sludge depth in lagoon, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

L = length of one side of lagoon at the to of levee, ft.

VLEW = volume of earthwork required, ft^3.

2.33.13.5.5 Calculate concrete required for overflow-decant structure. It is assumed that the same structure will be used for all flows. The structure will be 4' x 4' with 6" thick walls. The height of the structure will be 3 ft above the depth of sludge in the lagoon.

$$V_{cw} = [8 + (8)\ (D + 3)]\ 2$$

where

V_{cw} = volume of concrete wall required, ft^3.

D = sludge depth in lagoon, ft.

2 = number of lagoons.

2.33.13.5.6 Calculate operation manpower required.

2.33.13.5.6.1 If DSP $\leq$ 73,000 lb/yr.

$$OMH = 46$$

2.33.13.5.6.2 If DSP > 73,000 lb/yr.

$$OMH = 5.81\ (DSP)^{0.1847}$$

where

DSP = dry solids produced, lb/yr.

OMH = operation man-hour requirement, MH/yr.

2.33.13.5.7 Calculate maintenance manpower required.

2.33.13.5.7.1 If DSP $\leq$ 73,000 lb/yr.

$$MMH = 24$$

2.33.13.5.7.2 If DSP > 73,000 lb/yr.

$$MMH = 1.47\ (DSP)^{0.2491}$$

where

DSP = dry solids produced, lb/yr.

MMH = maintenance man-hours required, MH/yr.

2.33.13.5.8 Other construction cost items. The previous calculations account for approximately 80% of the cost of the drying lagoons. The other 20% includes influent piping, slide gates for decanting, grassing slopes, etc.

$$CF = \frac{1}{0.8} = 1.25$$

where

CF = correction factor for other construction cost items.

2.33.13.6 Quantities Calculations Output Data.

2.33.13.6.1 Volume of earthwork required, VLEW, ft^3.

2.33.13.6.2 Volume of concrete wall required, V_{cw}, ft^3.

2.33.13.6.3 Operation man-hour requirement, OMH, MH/yr.

2.33.13.6.4 Maintenance man-hour requirement, MMH, MH/yr.

2.33.13.6.5 Correction factor for other construction cost items, CF.

2.33.13.7 Unit Price Input Required.

2.33.13.7.1 Unit price input for excavation, UPIEX, \$/cu yd.

2.33.13.7.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

2.33.13.8 Cost Calculations.

2.33.13.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VLEW}{27} (UPIEX)$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required, ft^3.

UPIEX = unit price input for excavation, \$/cu yd.

2.33.13.8.2 Calculate cost of concrete.

$$COSTCW = \frac{V_{cw}}{27} (UPICW)$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, ft^3.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.33.13.8.3 Calculate total bare construct cost.

$$TBCC = (COSTE + COSTCW)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-Place, \$.

CF = correction factor for other construction cost.

2.33.13.9 Cost Calculations Output Data.

2.33.13.9.1 Total bare construction cost, TBCC, \$.

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2.35 LAND TREATMENT

2.35.1 Background.

2.35.1.1 Land treatment of wastewater involves the use of plants, the soil surface and the soil matrix for wastewater treatment. Although there are some differences in the use and definition of terms, there are three principal processes of land treatment. The "slow rate process" (also called crop irrigation) couples wastewater management with recycling of nutrients in crop production. "Rapid infiltration" (also known as infiltration/percolation) emphasizes water reclamation rather than direct nutrient recycling. The product water from rapid infiltration may be reused for crop production, returned to surface waters, or allowed to recharge groundwaters. "Overland flow" also emphasizes water reclamation. Unlike rapid infiltration, however, the product water from overland flow is almost always discharged directly to surface waters.

2.35.1.2 Selection of the appropriate land application method requires matching the management objectives and wastewater characteristics to the characteristics of potential sites, expected treatment efficiencies, and land requirements. Factors such as wastewater quality, climate, soil geology, topography, land availability and effluent quality requirements will determine which process is most suitable for a particular application.

2.35.1.3 Typical design features for the various land treatment processes are presented in Table 2.35-1. Site characteristics are compared in Table 2.35-2. Expected quality of treated water and removal efficiencies for the three principal land processes are summarized in Table 2.35-3 and 2.35-4, respectively.

TABLE 2.35-1

COMPARISON OF DESIGN FEATURES FOR LAND TREATMENT PROCESSES

Feature	Principal Processes			Other Processes	
	Slow Rate	Rapid Infiltration	Overland Flow	Wetlands	Subsurface
Application techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface	Sprinkler or Surface	Subsurface piping
Annual application rate, ft	2 to 20	20 to 560	10 to 70	4 to 100	8 to 87
Field area required, acres[b]	56 to 560	2 to 56	16 to 110	11 to 280	13 to 140
Typical weekly application rate, in.	0.5 to 4	4 to 120	2.5 to 6[c] 6 to 16[d]	1 to 25	2 to 20
Minimum preapplication treatment provided in United States	Primary Sedimentation[e]	Primary Sedimentation	Screening and Grit removal	Primary Sedimentation	Primary Sedimentation
Disposition of Applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evaporation with some percolation	Evapotranspiration, percolation, and runoff	Percolation with some evapotranspiration
Need for vegetation	Required	Optional	Required	Required	Optional

a. Includes ridge-and-furrow and border strip.
b. Field area in acres not including buffer area, roads, or ditches for 1 Mgal/d (43.8 L/S) flow.
c. Range for application of screened wastewater.
d. Range for application of lagoon and secondary effluent.
e. Depends on the use of the effluent and the type of crop.

1 in. = 2.54 cm
1 ft = 0.305 m
1 acre = 0.405 ha

TABLE 2.35-2

COMPARISON OF SITE CHARACTERISTICS FOR LAND TREATMENT PROCESSES

	Principal processes			Other processes	
Characteristics	Slow rate	Rapid infiltration	Overland flow	Wetlands	Subsurface
Slope	Less than 20% on cultivated land; less than 40% on noncultivated land	Not critical; excessive slopes require much earthwork	finish slopes 2 to 8%	Usually less than 5%	Not critical
Soil permeability	Moderately slow to moderately rapid	Rapid (sands, loamy sands)	Slow (clays silts, and soils with impermeable barriers)	Slow to moderate	Slow to rapid
Depth to groundwater	2 to 3 ft (minimum)	10 ft (less depths are acceptable where underdrainage is provided)	Not critical	Not critical	Not critical
Climatic restrictions	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often needed for cold weather	Storage may be needed for cold weather	None

1 ft = 0.305 m

TABLE 2.35-3

EXPECTED QUALITY OF TREATED WATER FROM LAND TREATMENT PROCESSES
MG/L

Constituent	Slow rate[a]		Rapid Infiltration[b]		Overland flow[c]	
	Average	Maximum	Average	Maximum	Average	Maximum
BOD	2	5	2	5	10	15
Suspended solids	1	5	2	5	10	20
Ammonia nitrogen as N	0.5	2	0.5	2	0.8	2
Total nitrogen as N	3	8	10	20	3	5
Total phosphorus as P	0.1	0.3	1	5	4	6

a. Percolation of primary or secondary effluent through 5 ft (1.5 m) of soil.

b. Percolation of primary or secondary effluent through 15 ft (4.5 m).

c. Runoff of comminuted municipal wastewater over about 150 ft (45 m) of slope.

TABLE 2.35-4

PERFORMANCE OF THE THREE PRINCIPAL LAND TREATMENT METHODS

Parameter	Removals (%)		
	Slow Rate	High Rate	Overland Flow
BOD_5	90-99+	95-99	80-95
TSS	90-99+	95-99	80-95
Total-N	50-95[a]	25-75	75-90
Total-P	80-99[b]	0-90[d]	30-60
Fecal Coliform	99.99[c]	99.9-99.99+	90-99.9

a. Depending on nitrogen uptake of vegetation.

b. Diminishes when P uptake exhausted.

c. When applied counts are more than 10^4 MPN/100 ml.

d. Until flooding exceeds adsorptive capacity.

2.35.2 General Description Overland Flow.

2.35.2.1 In overland flow land treatment, wastewater is applied over the upper reaches of sloped terraces and is treated as it flows across the vegetated surface to runoff collection ditches. The wastewater is renovated by physical, chemical and biological means as it flows in a thin film down the relatively impermeable slope. Typical hydraulic pathway is presented in Figure 2.35-1. As shown, there is relatively little percolation involved either because of an impermeable soil or a subsurface barrier to percolation.

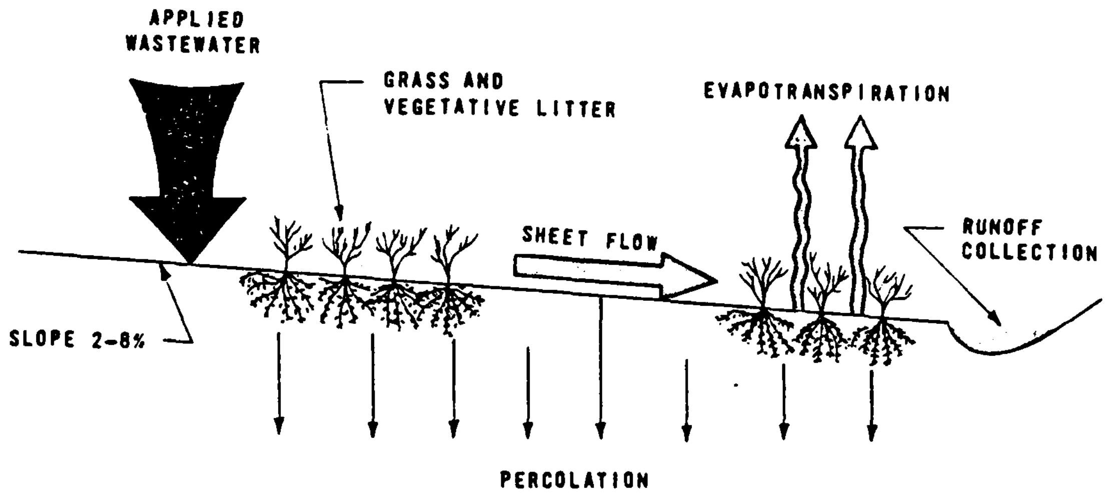
2.35.2.2 The primary objective of overland flow is wastewater treatment. A secondary objective of the system is for crop production. Perennial grasses (Reed, Canary, Bermuda, Red Top, Tall Fescue and Italian Rye) with long growing seasons, high moisture tolerance and extensive root formation are best suited to overland flow. Harvested grass is suitable for cattle feed.

2.35.2.3 Biological oxidation, sedimentation and grass filtration are the primary removal mechanisms for organics and suspended solids. Nitrogen removal is attributed mainly to nitrification/denitrification and plant uptake. Permanent nitrogen removal by plant uptake is only possible if the crop is harvested and removed from the field. Ammonia volatilization can be significant if the pH of the wastewater is above 7. Nitrogen removals normally range from 75-90% with runoff nitrogen being mostly in the nitrate form. Phosphorus is removed by adsorption, plant uptake and precipitation. Treatment efficiencies are somewhat limited because of the incomplete contact between the wastewater and the adsorption sites within the soil. Phosphorus removals usually range from 30-60%. Increased removals may be accomplished through preapplication treatment with aluminum or iron salts. Trace elements removal is relatively good with removal efficiencies ranging from 90-98%.

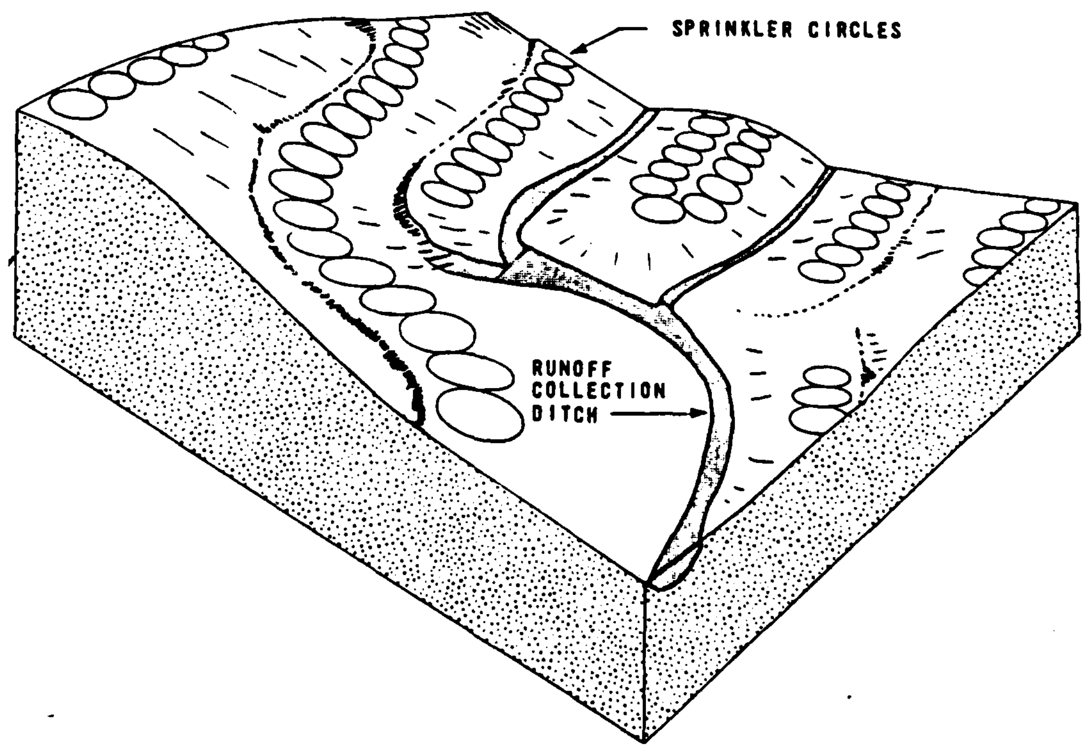
2.35.2.4 Loading rates and cycles are designed to maintain active microorganism growth on the soil surface. The operating principles are similar to a conventional trickling filter with intermittent dosing. The rate and length of application is controlled to minimize severe anaerobic conditions that result from overstressing the system. The resting period should be long enough to prevent surface ponding yet short enough to keep the microorganisms in an active state. Surface methods of distribution include the use of gated pipe or bubbling orifice. Slopes must be steep enough to prevent ponding of the runoff yet mild enough to prevent erosion and provide sufficient detention time for the wastewater on the slopes. Slopes must also have a uniform cross slope and be free from gullies to prevent channeling and allow distribution over the surfaces. Storage must be provided for nonoperating periods when air temperatures fall below freezing. Runoff is normally collected in open ditches.

FIGURE 2.35-1

OVERLAND FLOW



(a) HYDRAULIC PATHWAY



(b) PICTORIAL VIEW OF SPRINKLER APPLICATION

2.35.2.5 Typical design criteria include field area requirements of 35-100 acres/mgd; terraced slopes of 2 to 8%; terrace lengths of 120 to 140 ft; application rate of 11 to 32 ft/yr (2.5 to 16 in/wk); BOD_5 loading rate of 5 to 50 lb/acre-d; soil depth must be sufficient to form slopes that are uniform and to maintain a vegetative cover; soil permeability of 0.2 in/hr or less; hydraulic loading cycle of 6 to 8 hours application period followed by 16 to 18 hours resting period; operating period of 5 to 6 days/wk; soil texture of clay or clay loams.

2.35.2.6 Common preapplication treatment include screening or comminution for isolated sites with no public access, screening or comminution plus aeration to control odors during storage or application for urban locations with no public access. Wastewater high in metal content should be pretreated to avoid soil and plant contamination.

2.35.3 General Description Rapid Infiltration.

2.35.3.1 Rapid infiltration treats the wastewater with a minimum of land area. Wastewater is applied to deep and permeable deposits such as sand or sandy loam usually by distributing in basins or infrequently by sprinkling, and is treated as it travels through the soil matrix by filtration, adsorption, ion exchange precipitation and microbial action. Vegetation is not usually used, but crops may be grown to help maintain infiltration rates, however, harvest nomally would not be an objective. Typical hydraulic pathway for rapid infiltration is shown in Figure 2.35-2. A much greater portion of the applied wastewater percolates to the groundwater than with slow rate land treatment. An underdrain system may be incorporated into the system to recover the renovated water for reuse, to control groundwater mounding, or to minimize trespass of wastewater onto adjoining property by horizontal subsurface flow. There is little or no consumptive use by plants and less evaporation in proportion to a reduced surface area. A cycle of flooding and drying maintians the infiltration and treatment capacity of the soil material.

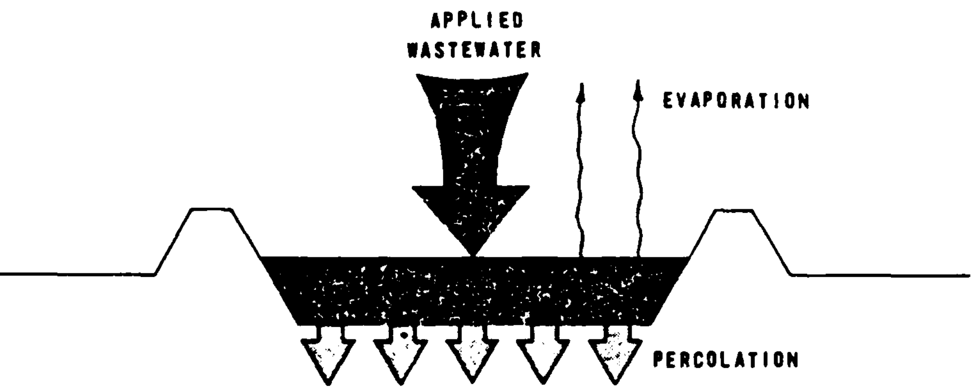
2.35.3.2 Removal of wastewater constituents by filtering and straining action of the soil are excellent. Suspended solids, BOD, and fecal colifoms are almost completely removed in most cases. Removal of BOD is primarily accomplished by aerobic bacteria that depend on the resting period to reaerate the soil. BOD loading rates have some effect on removals but too many other variables such as temperature, resting period, and soil type are involved to allow estimation of removals from loading rates alone.

2.35.3.3 Nitrogen removals are generally poor. The basic mechanisms for nitrogen removal include nitrification-denitrification and ammonium sorption. Denitrification may be enhanced through adjusting application cycles, supplying an additional carbon source, using vegetated basins, recycling of renovated water and reducing application rates. Rapid infiltration systems will produce a nitrified effluent at nitrogen loadings up to 60 lbs/acre-day. Nitrification below a temperature of 36°F and below a pH of 5 is minimal.

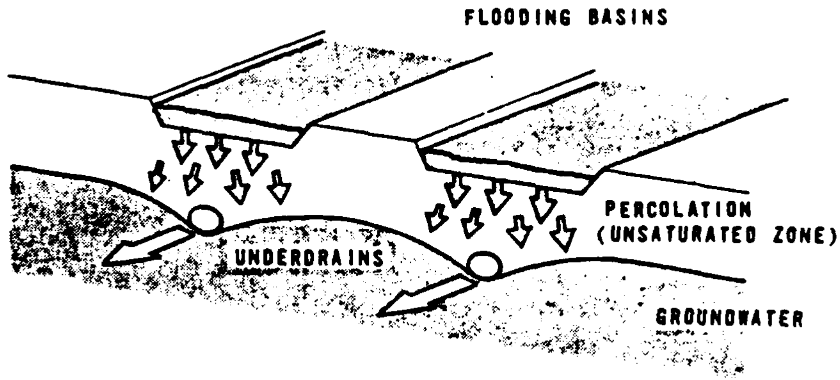
2.35.3.4 Phosphorus removals can range from 70-99% depending on the physical and chemical characteristics of the soil. The primary removal mechanism is adsorption with some chemical precipitation. Consequently, long-term capacity is limited by the mass of the soil in contact with the wastewater. Removals are also related to the residence time of the wastewater in the soil and travel distance.

FIGURE 2.35-2

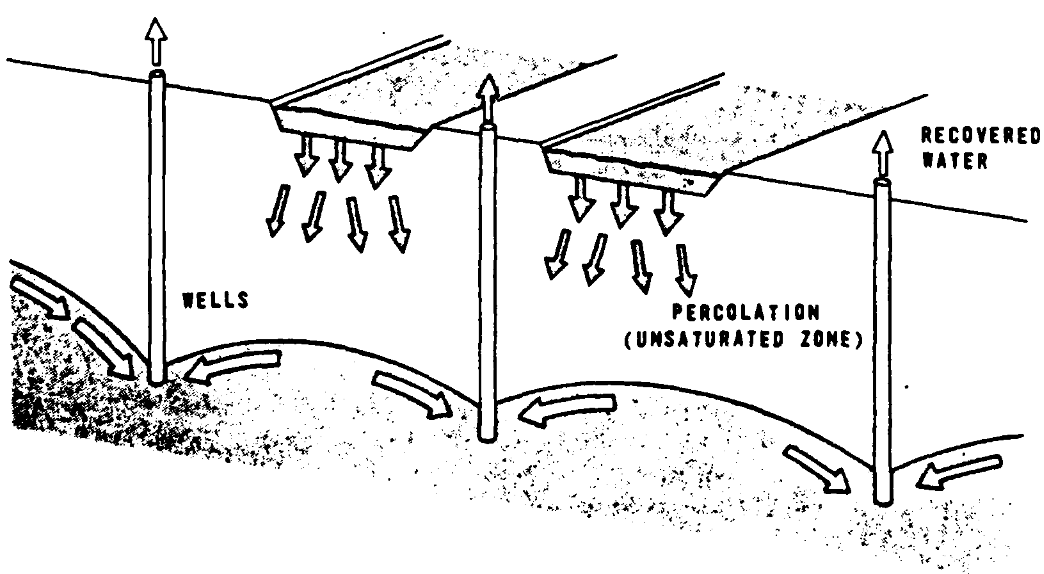
RAPID INFILTRATION



(a) HYDRAULIC PATHWAY



(b) RECOVERY OF RENOVATED WATER BY UNDERDRAINS



(c) RECOVERY OF RENOVATED WATER BY WELLS

2.35.3.5 Heavy metals are removed from solution by adsorptive process and by precipitation and ion exchange in the soil. The concern about heavy metals in rapid infiltration systems are related to the high rate of application and low adsorptive potential of the coarse soils. Microorganism removal is accomplished through sedimentation, predation and desiccation during preapplication treatment, desiccation and radiation during application; and straining, desiccation, radiation, predation and hostile enviromental factors upon application to the land.

2.35.3.6 Typical design criteria include field areas of 3 to 56 acre/mgd; application rate of 20-400 ft/yr (4-92 in/wk); BOD_5 loading rate of 20 to 100 lb/acre-d; soil depth of 10 to 15 ft or more; soil permeability of 0.6 in/hr or more; hydraulic loading cycle of 9 hours to 2 weeks application period followed by 15 hours to 2 weeks resting period; soil texture - sand, sandy loams; basin size of 1 to 10 acres with a minimum of 2 basins/site; height of dikes of 4 ft; underdrains of 6 or more ft deep, application techniques - flooding or sprinkling.

2.35.3.7 Common preapplication treatment practices include: primary treatment for isolated locations with restricted public access; biological treatment for urban locations with controlled public access. Storage is sometimes provided for flow equalization and for non-operating periods. Environmental impacts include potential for contamination of groundwater by nitrites and nitrates. Heavy metals may be eliminated by pretreatment techniques as necessary.

2.35.4 General Description Slow Infiltration.

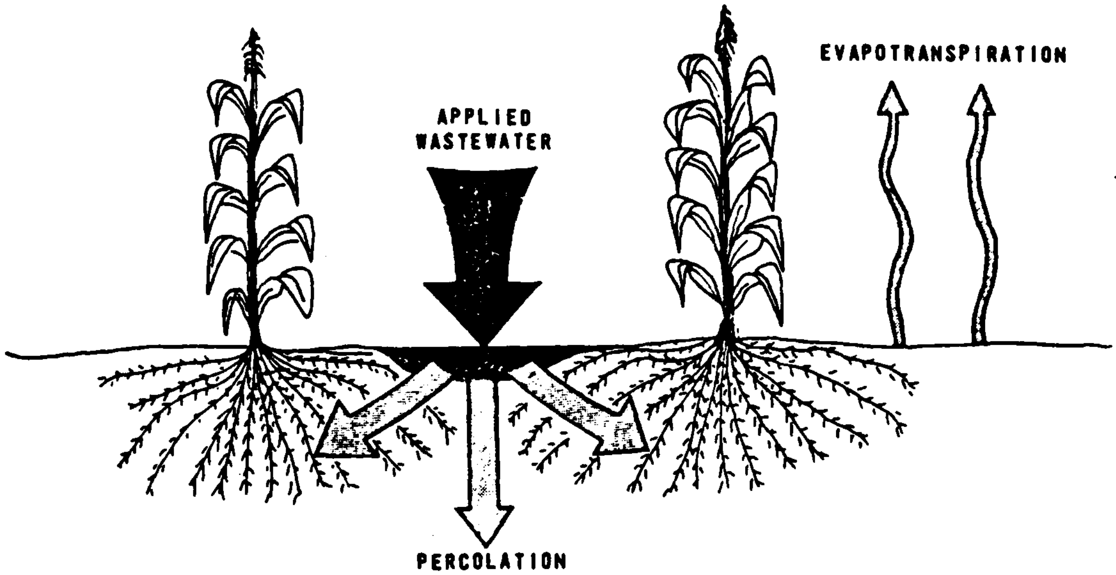
2.35.4.1 Slow rate infiltration is the most common method of treatment by land application. Wastewater is applied to vegetated soils that are slow to moderate in permeability (clay loams to sand loams) and is treated as it travels through the soil matrix by filtration, adsorption, ion exchange, precipitation, microbial action and by plant uptake. Part of the water is lost through evaporation and plant transpiration, part is stored in plant tissue, and the remainder either percolates to groundwater or is collected in an underdrainage system and reused (Figure 2.35-3a). Surface runoff is generally negligible.

2.35.4.2 Wastewater application techniques include sprinkling or surface distribution (Figure 2.35-3b and 2.35-3c). Sprinklers can be categorized as hand moved, mechanically moved and permanent set, the selection of which includes the following considerations: field conditions (shape, slope, vegetation and soil type), climate, operating conditions, and economics. Surface distribution employs gravity flow from piping systems or open ditches to flood the application area with several inches of water. Application techniques include ridge and furrow and surface flooding (border strip flooding). Ridge and furrow irrigation consists of running irrigation streams along small channels (furrows) bordered by raised beds (ridges) upon which crops are grown. Surface flooding irrigation consists of directing a sheet flow of water along border strips or cultivated strips of land bordered by small levees. The latter method is suited to close-growing crops such as grasses that can tolerate periodic inundation at the ground surface. A tail water return system for wastewater runoff from excess surface application is usually employed. Advantages of sprinkler application over surface distribution methods include: more uniform distribution of water and greater flexibility in range of application rates, applicability to most crops, less susceptibility to topographic constraints, and reduced operator skill and experience requirements. Limitations to sprinkling include adverse wind conditions and clogging of nozzles.

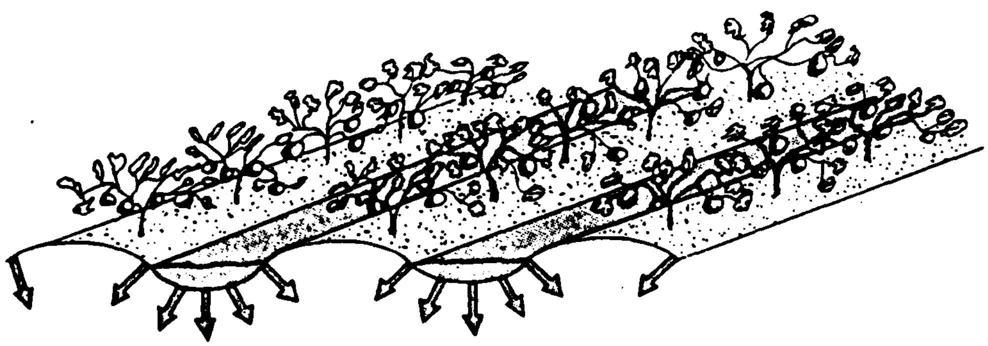
2.35.4.3 Slow rate treatment systems produce the best results of land treatment systems. Organics are removed substantially by biological oxidation within the top few inches of soil. Filtration and adsorption are the initial mechanisms in BOD removal but biological oxidation is the ultimate treatment mechanism. Suspended solids removals are not as well documented as BOD removals, but concentrations of 1 mg/l or less can generally be expected in the renovated water. Filtration is the major removal mechanism for suspended solids while volatile solids are biologically oxidized and fixed or mineral solids become part of the soil matrix.

FIGURE 2.35-3

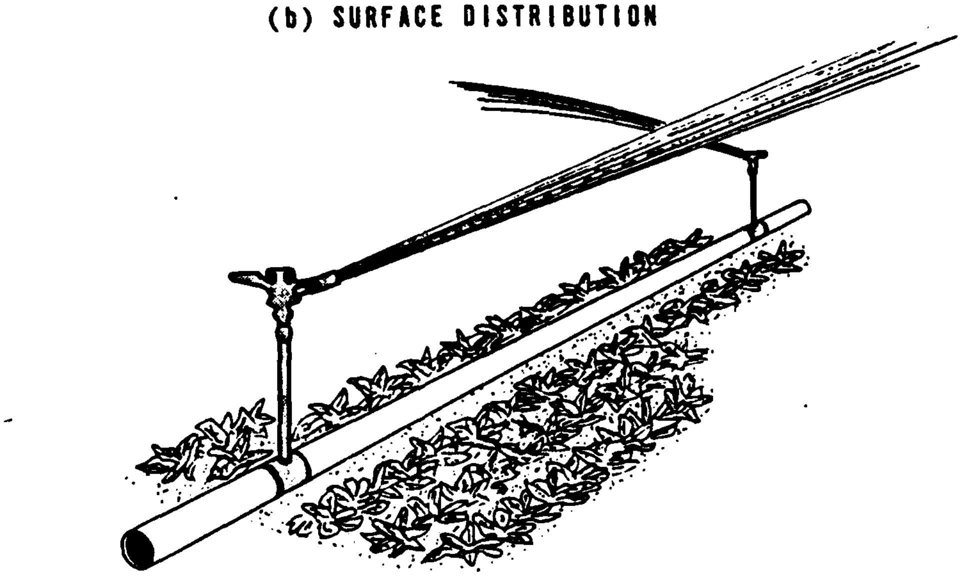
SLOW RATE LAND TREATMENT



(a) HYDRAULIC PATHWAY



(b) SURFACE DISTRIBUTION



(c) SPRINKLER DISTRIBUTION

2.35.4.4 Nitrogen is removed primarily by crop uptake, which varies with the type of crop grown and the crop yield. Crop nitrogen uptake values, based on typical yields under commercial fertilization, is presented in Table 2.35-1. Nitrogen removal can also be partially accomplished through biological denitrification and volatilization. Losses due to denitrification has been reported to range from 15% to 25% of the applied nitrogen. To protect groundwaters, the percolate nitrogen should be limited to 10 mg/l for design purposes.

2.35.4.5 Phosphorus removal is extremely effective as a result of soil adsoprtion, precipitation and crop uptake. Trace metals are removed from solution by adsorptive process and by precipitation and ion exchange in the soil.

2.35.4.6 Vegetation is a vital part of a slow rate system and serves to extract nutrients, reduce erosion and maintain soil permeability. Considerations for crop selection include: (1) Suitability to local climate and soil conditions, (2) consumptive water use and water tolerance, (3) nutrient uptake and sensitivity to wastewater constituents, (4) economic value and marketability, (5) length of growing season, (6) ease of management, and (7) public health regulations.

2.35.4.7 Typical design criteria include: field area 56 to 560 acres/mgd; application rate 2-20 ft/yr (0.5-4 in/wk); BOD_5 loading rate 0.2 to 5 lb/acre-d; soil depths 2-3 ft or more; soil permeability 0.06-2.0 in/hr; minimum preapplication treatment equivalent to primary; lower temperature limit 25°F.

2.35.4.8 Preapplication treatment include the following: primary treatment for isolated locations with restricted public access and when limited to crops not for direct human consumption; biological treatment plus control of coliform to 1000 MPN/100 ml for agricultural irrigation except for food crops to be eaten raw; secondary treatment plus disinfection for public access areas (parks). Wastewater high in metal content must also be pretreated to avoid soil and plant contamination.

2.35.5 Overland Flow.

2.35.5.1 Input Data.

2.35.5.1.1 Wastewater flow, Q, mgd.

2.35.5.1.1.1 Minimum flow, mgd.

2.35.5.1.1.2 Average flow, mgd.

2.35.5.1.1.3 Maximum flow, mgd.

2.35.5.1.2 Wastewater characteristics.

2.35.5.1.2.1 Suspended solids, mg/l.

2.35.5.1.2.2 Volatile suspended solids, % of suspended solids.

2.35.5.1.2.3 Settleable solids, mg/l.

2.35.5.1.2.4 BOD_5 (soluble and total), mg/l.

2.35.5.1.2.5 COD (soluble and total), mg/l.

2.35.5.1.2.6 Phosphorus (as PO_4), mg/l.

2.35.5.1.2.7 Total kjeldahl nitrogen (TKN), mg/l.

2.35.5.1.2.8 Ammonia-nitrogen, NH_3, mg/l.

2.35.5.1.2.9 Nitrite-nitrogen, NO_2, mg/l.

2.35.5.1.2.10 Nitrate-nitrogen, NO_3, mg/l.

2.35.5.1.2.11 Temperature, °C.

2.35.5.1.2.12 pH, units.

2.35.5.1.2.13 Oil and grease, mg/l.

2.35.5.1.2.14 Cations, mg/l.

2.35.5.1.2.15 Anions, mg/l.

2.35.5.2 Design Parameters.

2.35.5.2.1 Application rate.

2.35.5.2.1.1 Screened wastewater = 2.5 - 6 in/wk.

2.35.5.2.1.2 Lagoon or secondary effluent = 6 - 16 in/wk.

2.35.5.2.2 Precipitation rate, P_r, in/wk.

2.35.5.2.3 Evapotranspiration rate, ET, in/wk.

2.35.5.2.4 Runoff, R, in/wk (site dependent).

2.35.5.2.5 Wastewater generation period, WWGP, days/yr.

2.35.5.2.6 Field application period, FAP, wks/yr.

2.35.5.2.7 Spray evaporation (percent of application rate) = 2-8%.

2.35.5.2.8 Storage requirements (specify one).

2.35.5.2.8.1 No storage.

2.35.5.2.8.2 Minimum storage, days/yr.

2.35.5.2.9 Liner required (only used with storage).

2.35.5.2.10 Embankment protection (only used with storage).

2.35.5.2.11 Recovery system (specify one).

2.35.5.2.11.1 Gravity pipe.

2.35.5.2.11.2 Open channel recovery system.

2.35.5.2.12 Buffer zone width, ft (site dependent) = 0.0 - 500 ft.

2.35.5.2.13 Current ground cover.

2.35.5.2.13.1 Forest, %.

2.35.5.2.13.2 Brush, %.

2.35.5.2.13.3 Pasture, %.

2.35.5.2.14 Slope of land, % = 2.0 - 8.0%.

2.35.5.2.15 Monitoring wells.

2.35.5.2.15.1 Number.

2.35.5.2.15.2 Depth per well, ft.

2.35.5.2.16 Fraction denitrified, D, % = 75 - 90%.

2.35.5.2.17 Ammonia volatilization, AV, % = 0.0%.

2.35.5.2.18 Removal of phosphorus, % = 80%.

2.35.5.2.19 Hours per day operation, hrs/day.

2.35.5.2.20 Days per week operation, days/wk.

2.35.5.3 <u>Process Design Calculations</u>.

2.35.5.3.1 Calculate water loss due to evaporation, E, in/wk.

$$E = (E_f)(L_w)/(100)$$

E = water loss due to evaporation, in/wk.

E_f = percent of total applied wastewater lost through evaporation, %.

L_w = hydraulic application rate, in/wk.

2.35.5.3.2 Calculate percolation rate.

$$W_p = (W_p)_f\ (L_w)/(100).$$

W_p = percolating water rate, in/wk.

$(WP)_f$ = percent of applied wastewater lost to percolation, %.

2.35.5.3.3 Calculate runoff, R, in/wk, from water balance.

$$R = L_w + P_r - ET - W_p - E$$

R = runoff, in/wk.

P_r = design precipitation, in/wk.

ET = evapotranspiration, in/wk (crops consumptive use of water).

2.35.5.3.4 Calculate BOD_5 loading, $(L)_{BOD}$, lbs/acre-yr.

$$(L)_{BOD} = (TBOD)_i\ mg/l\ (L_w)\ in/wk\ \left(\frac{1\ gpm}{1000\ mg}\right)\left(3.785\ \frac{l}{gal}\right)$$

$$\left(\frac{1\ lb}{454\ gm}\right)\left(\frac{1\ ft}{12\ in}\right)\left(52\ \frac{wks}{yr}\right)\left(7.48\ \frac{gal}{ft^3}\right)\ (43{,}560\ ft^2/acre)$$

$$= (11.77)\ (TBOD)_i(L_w)$$

$(L)_{BOD}$ = total BOD_5 loading, lbs/acre-yr.

$(TBOD)_i$ = total BOD_5 in applied wastewater, mg/l.

2.35.5.3.4 Calculate total BOD_5 in runoff, mg/l.

$$(TBOD_5)_R = (L)_{BOD}/(R)(11.77)$$

$(TBOD_5)_R$ = BOD_5 concentration in runoff, mg/l.

2.35.5.3.5 Calculate soluble BOD_5 loading, $(L)_{SBOD_5}$, lb/ acre-yr.

$$(L)_{SBOD_5} = (11.77)\ (SBOD_5)_i\ (L_w)$$

$(L)_{SBOD_5}$ = soluble BOD_5 loading, lbs/acre-yr.

$(SBOD_5)_i$ = soluble BOD_5 in applied wastewater, mg/l.

2.35.5.3.6 Calculate soluble BOD_5 concentration in runoff, $(SBOD_5)_R$, mg/l.

$$(SBOD_5)_R = (L)_{SBOD_5}/(R)(11.77)$$

$(SBOD_5)_R$ = soluble BOD_5 concentration in runoff, mg/l.

2.35.5.3.7 Calculate total nitrogen concentration, C_n, in applied wastewater.

$$C_n = (TKN)_i + (NO_2)_i + (NO_3)_i$$

C_n = total nitrogen concentration in applied wastewater, mg/l.

$(TKN)_i$ = total kjeldahl nitrogen concentration in applied wastewater, mg/l.

$(NO_2)_i$ = nitrite-N concentration in applied wastewater, mg/l.

$(NO_3)_i$ = nitrate-N concentration in applied wastewater, mg/l.

2.35.5.3.8 Calculate wastewater nitrogen loading, lbs/acre-yr.

$$L_n = (C_n)(L_w)(13.2)$$

L_n = wastewater nitrogen loading, lbs/acre-yr.

2.35.5.3.9 Calculate total nitrogen loading, $(L_t)_N$, lbs/ acre-yr.

$$(L_t)_N = L_n + (11.77)\ (P_r)\ (0.5)$$

$(L_t)_N$ = total nitrogen loading rate, lbs/acre-yr.

0.5 = assumed total nitrogen concentration in precipitation, mg/l.

2.35.5.3.10 Calculate crop nitrogen uptake rate, $(U)_N$, lbs/acre-yr; use forage grass for ground cover.

$$(U)_N = 0.891\ [(118.73) + (0.36)\ (L_t)_N]\ \text{lbs/acre-yr}$$

2.35.5.3.11 Calculate nitrogen loss through denitrification, D, lbs/acre-yr.

$$D = (D_f)\,(L_t)_N/\,(100)$$

D = nitrogen loss through denitrification, lbs/ acres/yr.

D_f = percent of total applied nitrogen lost through denitrification, %.

2.35.5.3.12 Calculate nitrogen loss due to volatilization, AV, lbs/acre-yr.

$$AV = (AV)_f\ (L_t)_N/\,(100)$$

AV = nitrogen loss due to volatilization, lbs/acre-yr.

$(AV)_f$ = fraction of total nitrogen lost through volatilization, %.

2.35.5.3.13 Calculate sum of nitrogen losses, $(\Sigma N)_L$, lbs/acre-yr.

$$(\Sigma N)_L = (U)_N + (D) + (AV)$$

$(\Sigma N)_L$ = sum of nitrogen lost, lbs/acre-yr.

$$(\Sigma N)_L \qquad (0.8)\ (L_t)_N$$

2.35.5.3.14 From nitrogen mass balance calculate nitrogen concentration in runoff, $(C_R)_N$, mg/l.

$$(C_R)_N = [(L_t)_N - (\Sigma N)_L]/\,(R)\,(13.2)$$

2.35.5.3.15 Calculate required field area, TA, acre.

$$TA = (36.83)(Q_{av})(WWGP)/(L_w)(FAP)$$

$$TAN = (Q_{av})(36.84)(WWGP)/(L_t)_N(FAP)$$

If TA $\leq$ TAN and set TA=TAN, recalculate Lp, Lw, Ln, and $(L_t)_N$.

where

TA = required field area, acres.

Q = average wastewater flow, mgd.

WWGP = wastewater generation period, days/yr.

FAP = field application period, wks/yr.

2.35.5.3.16 Calculate phosphorus loading, L_p, lb/acre-yr.

$$L_p = (11.77)(TP)_i\ (L_w)$$

L_p = total phosphorus loading, lbs/acre-yr.

$(TP)_i$ = total phosphorus concentration in applied wastewater, mg/l.

2.35.5.3.17 Calculate soil removal of phosphorus, lbs/acre-yr.

$$(SRP) = (SRP)_f\ (L_p)/(100)$$

SRP = soil removal of phosphorus, lbs/acre-yr.

$(SRP)_f$ = percent of total phosphorus removed by the soil, %.

2.35.5.3.18 Calculate removal of phosphorus, U_p, lbs/acre-yr.

$$Up = 0.891\ [(83.386) - (0.0373)(L_p)] \quad \text{lbs/acre-yr.}$$

U_p = removal of phosphorus, lbs/acre-yr.

2.35.5.3.19 From phosphorus mass balance, calculate phosphorus concentration in runoff.

$$L_p = (U_p) + (C_p)_R\ (R)(11.77)$$

$$(C_p)_R = [(L_p) - (U_p)]/(11.77)(R)$$

$(C_p)_R$ = phosphorus content in runoff, mg/l.

$\geq (0.01)(TP)_i$

2.35.5.3.20 Calculate volume of runoff, R, in mgd.

$$R(\text{mgd}) = [(R)\ \text{in/wk}\ (\frac{1\ \text{ft}}{12\ \text{in}})(\frac{1\ \text{wk}}{7\ \text{day}})]\ (TA)\ \text{acre}\ (43{,}560\ \frac{\text{ft}^2}{\text{acre}})$$

$$(7.48\ \frac{\text{gal}}{\text{ft}^3})(1/10^6)$$

2.35.5.3.21 Calculate suspended solids concentration in runoff, mg/l, assume 93% SS removal.

$$(SS)_R = (SS)_i(0.07)$$

$(SS)_R$ and $(SS)_i$ = suspended solids concentration in runoff and applied wastewater, respectively, mg/l.

2.35.5.3.22 Calculate total and soluble COD in runoff.

$$(TCOD)_R = [(TCOD)_i - (TBOD_5)_i]\ (0.9) + (TBOD_5)_R$$

$$(SCOD)_R = [(SCOD)_i - (SBOD_5)_i]\ (0.9) + (SBOD_5)_R$$

$(TCOD)_R$ and $(SCOD)_R$ = total and soluble COD in runoff, respectively, mg/l.

$(TCOD)_i$ and $(SCOD)_i$ = total and soluble COD in applied wastewater, respectively, mg/l.

2.35.5.3.23 Nitrite-N concentration in runoff = 0.

2.35.5.3.24 Nitrate-N concentration in runoff = $(0.75)(C_R)_N$.

2.35.5.3.25 Ammonia-N concentration in runoff = $(0.25)(C_R)_N$.

2.35.5.3.26 Oil and Grease in runoff = 0.0.

2.35.5.3.27 TKN = $0.25\ (C_R)_N$.

2.35.5.3.28 SETSO = 0.0.

2.35.5.4 Process Design Output Data.

2.35.5.4.1 Hours per day operation, hours.

2.35.5.4.2 Days per week operation, days.

2.35.5.4.3 Application rate, in/week.

2.35.5.4.4 Runoff, in/week.

2.35.5.4.5 Percent denitrified, percent.

2.35.5.4.6 Percent ammonia volatilization, percent.

2.35.5.4.7 Removal of phosphorus, percent.

2.35.5.4.8 Spray evaporation, percent.

2.35.5.4.9 Wastewater generation period, days/yr.

2.35.5.4.10 Field application period, weeks/yr.

2.35.5.4.11 Buffer zone width, feet.

2.35.5.4.12 Current ground cover.

2.35.5.4.12.1 Forest, percent.

2.35.5.4.12.2 Brush, percent.

2.35.5.4.12.3 Pasture, percent.

2.35.5.4.13 Slope of site, percent.

2.35.5.4.14 Number of monitoring wells, wells,

2.35.5.4.15 Depth of Monitoring wells, feet.

2.35.5.4.16 Treatment area required, acres.

2.35.5.4.17 Volume of runoff, mgd.

2.35.5.4.18 Quality of runoff, mgd.

2.35.5.4.18.1 Suspended solids, mg/l.

2.35.5.4.18.2 Volatile solids, percent.

2.35.5.4.18.3 BOD_5, mg/l.

2.35.5.4.18.4 BOD_5 soluble, mg/l.

2.35.5.4.18.5 COD, mg/l.

2.35.5.4.18.6 COD soluble, mg/l.

2.35.5.4.18.7 PO_4, mg/l.

2.35.5.4.18.8 TKN, mg/l.

2.35.5.4.18.9 NO_2, mg/l.

2.35.5.4.18.10 NO_3, mg/l.

2.35.5.4.18.11 Oil and grease, mg/l.

2.35.5.5 Quantities Calculations.

2.35.5.5.1 Distribution Pumping.

2.35.5.5.1.1 Calculate the design flow.

$$\text{FLOW} = \frac{(Q_{avg})\ (\text{WWGP})\ (24)}{(\text{FAP})\ (\text{DPW})\ (\text{HPD})}$$

where

?sr?
Instantaneous

FLOW = ~~actual~~ daily flow to spray field, mgd.

Q_{avg} = average daily wastewater flow, mgd.

WWGP = wastewater generation period, days/yr.

FAP = field application period, wks/yr.

DPW = days per week treatment system is operated, days/wk.

HPD = hours per day treatment system is operated, hrs/day.

Using the flow calculated (FLOW), the distribution pumping will be sized and the cost estimated from the existing chapter entitled, "Intermediate Pumping".

2.35.5.5.2 Storage Requirements. Overland flow unlike rapid infiltration is dependent upon weather. Storage is required to hold the wastewater generated when application is not possible due to cold weather or heavy rains. This, of course, varies greatly for different parts of the country with different climates.

For purposes of this program the number of days of storage required will be assumed to be 50% of the number of days which wastewater cannot be applied to the field.

2.35.5.5.2.1 Calculate storage volume,

$$SV = (.5)\ [365-(\text{FAP})(7)]\ (\text{GF}\times10^6)$$

where

SV = storage volume, gal.

FAP = field application period, wks/yr.

GF = generated flow, mgd.

2.35.5.5.2.2 Calculate size and number of storage lagoons.

2.35.5.5.2.2.1 The following assumptions are made in determining size and number of lagoons:

A minimum of 2 lagoon cells will always be used.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
The largest single lagoon cell will be 40 acres which represents approximately 85 million gallons storage volume.

2.35.5.5.2.2.2 If SV $\leq$ 170,000,000 gal.

$$NLC = 2$$

2.35.5.5.2.2.3 If SV $>$ 170,000,000 gal. a trial and error solution for NLC will be used.

Assume NLC = 4; If $\frac{SV}{NLC} > 85{,}000{,}000$ gal.

Redesignate NLC = NLC+2 and repeat calculation until $\frac{SV}{NLC} \leq 85{,}000{,}000$ gal.

where

SV = storage volume, gal.

NLC = number of lagoon cells.

2.35.5.5.2.3 Calculate storage volume per cell.

$$SVC = \frac{SV}{(NLC)(7.48)}$$

where

SVC = storage volume per cell, ft^3.

SV = storage volume, gal.

NLC = number of lagoon cells.

7.48 = conversion from gal to ft^3.

2.35.5.5.2.4 Calculate lagoon cell dimensions. The following assumptions are made concerning lagoon construction:

The lagoon cells will be square.
Common levee construction will be used where possible.
Lagoons will be constructed using equal cut and fill.
Lagoon depth will be 10 ft. with 8 ft. water depth and 2 ft. freeboard.
Minimum water depth will be 1.5 ft.
Side slopes will be 3 to 1.
A 30% shrinkage factor is used for fill.

$$L = \frac{(0.615\ SVC - 1521)^{0.5} + 60}{2}$$

where

L = length of one side of lagoon cell, ft.

SVC = storage volume per cell, ft^3.

2.35.5.5.2.5 Calculate volume of earthwork required for lagoons.

2.35.5.5.2.5.1 The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = 10$$

$$VF = [\ 3(DF)^2 + 10DF\ ]\ [\ \frac{5NLC}{2} + 2\ ]\ (L)$$

$$VC = (1.3)(NLC)(DC)\ [\ L^2 - (6)(DF)(L) + 12\ DF^2 + 120\ DF - 60L + 1200]$$

2.35.5.5.2.5.2 Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC < VF then assume DC > 1 ft. and recalculate VC and VF.
If VC > VF then assume DC < 1 ft. and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the storage lagoon.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

2.35.5.5.3 Overland Flow Distribution System.

2.35.5.5.3.1 In an overland flow treatment system the wastewater is usually applied to the field with fixed sprinklers. The pipes are nomally buried and impact type irrigation sprinklers with flow from 6 to 15 gpm per sprinkler are used. There are many ways to layout a sprinkler field and where there are slopes of greater than 2% available, the natural topography of the site dictates the layout. Though every layout is different the amount and size of pipe and sprinklers used will not vary greatly.

2.35.5.5.3.2 Assume:

Field will be square.
Arrangement of headers and laterals will be as shown in Figure 2.35-4.
The treatment area will be increased by 15% to account for additional area for drainage ditches and service roads.

2.35.5.5.3.3 Calculate number of headers. The number of headers will be selected based on the following:

$$\text{Flow} \leq 1 \text{ mgd}; \ NH = 1$$
$$1 \text{ mgd} < \text{Flow} \leq 2 \text{ mgd}; \ NH = 2$$
$$2 \text{ mgd} < \text{Flow} \leq 4 \text{ mgd}; \ NH = 3$$
$$\text{Flow} > 4 \text{ mgd}; \ NH = 4$$

where

FLOW = actual daily flow to spray field.

NH = number of headers.

2.35.5.5.3.4 Calculate flow per header.

$$FPH = \frac{(FLOW \times 10^6)}{(NH)\underbrace{(HPD)(60)}_{1440}}$$

where

FPH = flow per header, gpm.

FLOW = actual daily flow to spray field, mgd.

NH = number of headers.

1440 = minutes per day

~~HPD = hours per day of operation, hrs/day.~~

~~60 = number of munites per hour, min/hr.~~

2.35.5.5.3.5 Calculate flow per sprinkler (FPN).

$$FPN = \frac{4051.7\ (AR)}{(DPW)\ (HPD)\ (FAP)}$$

where

FPN = flow per sprinkler, gpm.

AR = application rate, in/wk.

DPW = days per week treatment system is operated, days/wk.

HPD = hours per day treatment system is operated, hrs/day.

FAP = field application period, wks/yr.

4051.7 = combined conversion factors.

2.35.5.5.3.6 Calculate number of sprinklers per header.

$$SPH = \frac{FPH}{FPN}$$

If SPH < 1 set SPH = 1. Note: Flow is not sufficient, reduce operating period specification.

where

FPH = flow per header, gpm.

FPN = flow per sprinkler, gpm.

SPH = sprinklers per header.

2.35.5.5.3.7 Calculate number of laterals per header.

$$LPH = SPH/q$$

LPH must be an integer.

If LPH < 1 set LPH = 1 and recalculate the number of required sprinklers.

$$NSL = \frac{FPH}{FPN}$$

where

LPH = laterals per header.

SPH = sprinklers per header.

NSL = number of sprinklers per lateral.

FPH = flow per header, gpm.

FPN = flow per sprinkler, gpm.

2.35.5.5.3.8 Calculate flow per lateral.

$$FPL = \frac{FPH}{LPH}$$

where

FPL = flow per lateral, gpm.

FPH = flow per header, gpm.

LPH = laterals per header.

2.35.5.5.3.9 Calculate lateral diamater (DIAL).

$$DIAL = 0.286\ (FPL)^{0.5}$$

DIAL must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48. Always use the next larger diameter above the calculated diameter.

where

DIAL = diameter of lateral pipe, inches.

FPL = flow per lateral, gpm.

2.35.5.5.3.10 Calculate header pipe sizes.

The header pipe nomally decreases in size due to decreasing volume of flow as each set of lateral pipes removes part of the flow from the header pipe. There will nomally be four laterals taken off from approximately the same location. The header size will be calculated after each group of laterals.

$$DIAHN = 0.286\ [\ FPH - (N)(4)(FPL)\ ]^{0.5}$$

where

DIAHN = diameter of header pipe, inches.

FPH = flow per header, gpm.

FPL = flow per lateral, gpm.

N = number of points at which flow is removed from header.

DIAHN must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

Begin calculation with N = 0. This will give the diameter (DIAHO) of the header before the first group of laterals remove any flow. Then set N = N + 1 and repeat the calculation. This will give the diameter (DIAH 1) of the header after flow has been removed by the first group of laterals. Repeat the calculation each time redesignating N until the expression FPH-(N)(4)(FPL) is equal to zero or yields a negative number. The "N" used in this last calculation will be the number of slopes which must be constructed.

2.35.5.5.3.11 Determine length of lateral pipe.

$$LDIAL = (SPH)\ (50)\ (NH)$$

where

LDIAL = length of lateral pipe required, ft.

SPH = sprinklers per header.

50 = distance between sprinklers, ft.

2.35.5.5.3.12 Determine length of header pipes. As can be seen from Figure 2.35-4 the spray field is laid out such that the distance between lateral take-off points is 400 ft. Therefore, to determine the length of each size pipe required for a single header, sum the number of points at which the diameter is the same and multiply by 400 ft. To determine the amount of each size header pipe for the entire field, multiply by the number of headers since they are identical.

$$LDIAHN = (NH)(400) \sum N\emptyset$$

where

NH = number of headers.

LDIAHN = length of header pipe of diameter DIAHN, ft.

N∅ = number of points where same diameter pipe was chosen.

For the length of pipe from pump station to the spray field use the number of headers times the width of the spray field. The diameter will be the diameter calculated before any flow is removed from the header.

2.35.5.5.3.13 Calculate number of valves for distribution system.

2.35.5.5.3.13.1 There will be a butterfly valve in each header for flow control. These valves will be in the header upstream from the spray field and will be the same size as the initial size calculated for the header.

$$NBV = NH$$
$$DBV = DIAHN$$

where

NBV = number of butterfly valves.

DBV = diameter of butterfly valves, inches.

NH = number of headers.

2.35.5.5.3.13.2 There will be a plug valve in each lateral pipe which will be automatic but will be either fully open or fully closed. They will be the same size as the size calculated for the lateral pipes.

$$NLV = (LPH)\ (NH)$$
$$DLV = DIAL$$

where

NLV = number of lateral valves.

LPH = laterals per header.

DLV = diameter of lateral valve, inches.

DIAL = diameter of lateral pipe, inches.

NH = number of headers.

2.35.5.5.4 Construction of Overland Flow Slopes. Overland flow systems must have slopes from 2% to 8%, must be clear of trees and brush, and must be leveled to a constant slope. Not many land areas meet this criteria, therefore, in many cases the area must be cleared, slopes formed, and leveled.

2.35.5.5.4.1 Clearing and Grubbing. The areas will be classified in three categories; heavy, meadium, and light. Heavy refers to wooded areas with mature trees. Medium refers to spotted mature trees with numerous small trees and bushes. Light refers to only small trees and bushes. The user must specify the type of clearing and grubbing required as well as the percent of the treatment area requiring clearing and grubbing.

$$CAGH = \frac{(PCAGH)}{100}(TA)$$

$$CAGM = \frac{(PCAGM)}{100}(TA)$$

$$CAGL = \frac{(PCAGL)}{100}(TA)$$

where

CAGH = area which requires heavy clearing, acres.

PCAGH = percentage of treatment area requiring heavy clearing, %.

CAGM = area which requires medium clearing, acres.

PCAGM = percentage of treatment area requiring medium clearing, %.

CAGL = area which requires light clearing, acres.

PCAGL = percentage of treatment area requiring light clearing, %.

TA = treatment area, acres.

2.35.5.5.4.2 Determine Earthwork Required.

2.35.5.5.4.2.1 For areas which are flat (0-2% slope) the overland flow slopes must be formed by moving earth. For areas where slopes of from 2% to 8% exist, very little earth moving is required. The following assumptions are made to estimate the quantities of earthwork required for slopes from 0-2%.

2.35.5.5.4.2.1.1 The slopes shall be as indicated in Figure 2.35-4.

2.35.5.5.4.2.1.2 Equal cut and fill will be used.

2.35.5.5.4.2.2 Calculate volume of earthwork.

$$VSEW = (55,100)(TA)$$

where

VSEW = volume of earthwork required for slope construction, ft^3.

TA = treatment area, acres.

55,100 = volume of earthwork required per acre, ft^3.

2.35.5.5.5 Runoff collection. The overland flow system does not depend on infiltration for treatment and much of the water runs off. This water must be collected and monitored. There are basically two types of collection systems: open ditch and buried drain pipe. With both systems, the runoff from each individual slope is carried to the main collection system by small ditches or terraces.

2.35.5.5.5.1 Determine earthwork required for terraces.

$$VET = (N+2)(NH)(1000)(5)$$

where

VET = volume of earthwork for terraces, ft^3.

N = number of points which flow is removed from header.

NH = number of headers.

1000 = length of individual slope.

5 = volume of earthwork required per foot of terrace length.

2.35.5.5.5.2 Main Runoff Collection System. As stated before there are two systems which may be used, open ditches or buried drain pipe. One or the other would be used but never both.

2.35.5.5.5.2.1 Buried drain pipe.

2.35.5.5.5.2.1.1 Since an overland flow facility would not nomally be operated when the site received a rainfall in excess of 0.5 inches in 24 hours, this criteria will be used to size the drainage system. The assumption of 100% runoff will be made for design purposes.

2.35.5.5.5.2.1.2 The pipe size will vary as runoff from each slope is added. Using the layout shown in Figure 2.35-4 and the assumed rainfall, the flow from one slope would be 0.149 ft^3/sec. The following assumptions are made:

Pipe will be concrete drain pipe.
Pipe will be flowing half full.
Pipe will be laid on a .2% slope.
Friction factor is 0.013.

2.35.5.5.5.2.1.3 Calculate pipe size.

$$CDIAN = 6.38(N)^{0.375}$$

where

CDIAN = diameter of concrete drain pipe, inches.

N = number of points at which flow is removed from header.

CDIAN must be one of the following 12, 15, 18, 21, 24, 27, 30, 33, 36, 42, 48, 54, 60, 66, 72, 78, 84, 90, or 96 inches.

2.35.5.5.5.2.1.4 As in sizing the header pipe, the collection pipe will vary in size. Start with N=1 and calculate pipe size. Then set N=N+1 and repeat the calculation. Redesignate N in this manner until N is equal to total number of take off points in header pipe calculation.

2.35.5.5.5.2.1.5 Again, the length of each size pipe required will be determined by summing the number of points at which the same diameter is calculated and multiply by 400 ft. To determine the amount of pipe for the entire field multiply by NH+1 since there will be NH+1 identical collection lines.

$$LCDIAN = (400)(NH+1) \sum NCDIAN$$

where

LCDIAN = length of drain pipe of given diameter, ft.

NCDIAN = number of points where same diameter pipe was chosen.

NH = number of headers.

2.35.5.5.5.2 Open ditches.

2.35.5.5.5.2.1 Assume that the ditches will be all cut and erosion control will be required in construction.

$$LDIT = (N)(400)(NH+1)$$

where

LDIT = total length of ditches for system, ft.

N = number of points at which flow is removed from header.

400 = length of ditch between slopes, ft.

NH = number of headers.

2.35.5.5.6 Calculate total land area required.

2.35.5.5.6.1 Ditches and service roads increase the area by 15%.

$$ADR = (.15)(TA)$$

where

ADR = area for ditches and roads, acres.

2.35.5.5.6.2 Area for storage lagoons.

$$ASL = \frac{1.2(NLC)(L)^2}{43,560}$$

where

ASL = area for storage lagoons, acres.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

1.2 = additional area required for cross levee.

2.35.5.5.6.3 Area for buffer zone.

Assume:

Buffer zone will be around entire treatment area.
Assume facility will be essentially square.

2.35.5.5.6.3.1 Calculate dimensions of treatment area.

$$LTA = \left(\frac{TA + ADR + ASL}{43,560}\right)^{0.5}$$

where

LTA = length of one side of treatment area, ft.

TA = treatment area, acres.

ADR = area for ditches and service roads, acres.

ASL = area for storage lagoons, acres.

2.35.5.5.6.3.2 Calculate area for buffer zone.

$$ABZ = 4WBZ\ (LTA + WBZ)$$

where

ABZ = area required for buffer zone, acres.

LTA = length of one side of treatment area, ft.

WBZ = width of buffer zone (input by user), ft.

2.35.5.5.6.4 Total land area.

$$TLA = TA + ADR + ASL + ABZ$$

where

TLA = total land area required, acres.

TA = treatment area, acres.

ADR = area required for ditches and service roads, acres.

ASL = area required for storage lagoons, acres.

ABZ = area required for buffer zones, acres.

2.35.5.5.7 Calculate fencing required

Assume

Entire facility is to be fenced.
Facility is square.

$$LF = 834.8\ (TLA)^{0.5}$$

where

LF = length of fence required, ft.

TLA = total land area required, acres.

834.8 = combined constants.

2.35.5.5.8 Calculate operation and maintenance manpower.

2.35.5.5.8.1 Distribution System.

2.35.5.5.8.1.1 If $TA \leq 70$;

$$OMMHD = 77.91\ (TA)^{0.5373}$$

2.35.5.5.8.1.2 If $TA > 70$;

$$OMMHD = 18.12\ (TA)^{0.8814}$$

where

OMMHD = operation and maintenance manpower for distribution, MH/yr.

TA = treatment area, acres.

2.35.5.5.8.2 Runoff collection by gravity pipe.

2.35.5.5.8.2.1 If $TA \leq 100$;

$$OMMHP = 6.65\ (TA)^{0.4224}$$

2.35.5.5.8.2.2 If $TA > 100$;

$$OMMHP = 2.41\ (TA)^{0.6434}$$

where

OMMHP = operation and maintenance manpower for runoff collection by gravity pipe, MH/hr.

TA = treatment area, acres.

2.35.5.5.8.3 Runoff collection by open ditch.

2.35.5.5.8.3.1 If $TA \leq 150$;

$$OMMHO = 36.9\ (TA)^{0.3578}$$

2.35.5.5.8.3.2 If $TA > 150$;

$$OMMHO = 8.34\ (TA)^{0.6538}$$

where

OMMHO = operation and maintenance manpower for runoff collection by open ditch, MH/yr.

TA = treatment area, acres.

2.35.5.5.9 Calculate operation and maintenance material costs.

2.35.5.5.9.1 Distribution System.

2.35.5.5.9.1.1 If TA $\leq$ 500;

$$OMMPD = 0.783(TA)^{-0.0673}$$

2.35.5.5.9.1.2 If TA $>$ 500;

$$OMMPD = 9.46\ (TA)^{-0.47}$$

where

OMMPD = O&M material costs for distribution system as percent construction cost of distribution system, %.

TA = treatment area, acres.

2.35.5.5.9.2 Runoff collection by gravity pipe.

2.35.5.5.9.2.1 If TA $\leq$ 225;

$$OMMPP = 0.9566\ (TA)^{-0.2539}$$

2.35.5.5.9.2.2 If TA $>$ 225;

$$OMMPP = .242\%$$

where

OMMPP = O&M material costs for runoff collection by gravity pipe as percent construction cost for gravity pipe system, %.

TA = treatment area, acres.

2.35.5.5.9.3 Runoff collection by open ditch.

2.35.5.5.9.3.1 If TA $\leq$ 60;

$$OMMPO = 25.4\ (TA)^{-0.1383}$$

2.35.5.5.9.3.2 If TA $>$ 60;

$$OMMPO = 14.42$$

where

OMMPO = O&M material costs for runoff collection by open ditch as percent construction cost for open ditch system, %.

2.35.5.5.10 Other construction cost items. The quantities computed account for approximately 90% of the construction cost of the systems. Other miscellaneous costs such as final land leveling, connecting piping for lagoons, miscellaneous concrete structures, etc., make up the additional 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction cost.

2.35.5.6 Quantities Calculation Output Data.

2.35.5.6.1 Volume of earthwork required for lagoon construction, VLEW ft^3.

2.35.5.6.2 Diameter of lateral pipes, DIAL, inches.

2.35.5.6.3 Diameters of header pipes, DIAHN, Inches.

2.35.5.6.4 Length of lateral pipes, LDIAL, ft.

2.35.5.6.5 Length of header pipes, LDIAHN, ft.

2.35.5.6.6 Number of nozzles, NON.

2.35.5.6.7 Diameter of butterfly valves, DBV, inches.

2.35.5.6.8 Number of butterfly valves, NBV.

2.35.5.6.9 Diameter of lateral valves, DLV, inches.

2.35.5.6.10 Number of lateral valves, NLV.

2.35.5.6.11 Area which requires heavy clearing, CAGH, acres.

2.35.5.6.12 Area which requires medium clearing, CAGM, acres.

2.35.5.6.13 Area which requires light clearing, CAGL, acres.

2.35.5.6.14 Volume of earthwork required for slope construction, VSEW, ft^3.

2.35.5.6.15 Volume of earthwork for terraces, VET, ft^3.

2.35.5.6.16 Diameter of concrete drain pipe, CDIAN, inches.

2.35.5.6.17 Length of drain pipe of size CDIAN, LCDIAN, ft.

2.35.5.6.18 Total length of ditches for system, LDIT, ft.

2.35.5.6.19 Total land area required, TLA, acres.

2.35.5.6.20 Length of fence required, LF, ft.

2.35.5.6.21 Operation and maintenance manpower for distribution system, OMMHD, MH/yr.

2.35.5.6.22 Operation and maintenance manpower for runoff collection by gravity pipe, OMMHP, MH/yr.

2.35.5.6.23 Operation and maintenance manpower for runoff collection by open ditch, OMMHO, MH/yr.

2.35.5.6.24 Operation and maintenance material costs for distribution system as percent construction cost of distribution system, OMMPD, %.

2.35.5.6.25 Operation and maintenance material costs for runoff collection by gravity pipe as percent construction cost of gravity pipe system, OMMPP, %.

2.35.5.6.26 Operation and maintenance cost for runoff collection by open ditch as percent construction cost for open ditch system, OMMPO, %.

2.35.5.6.27 Correction factor for other construction cost, CF.

2.35.5.6.28 Number of headers, NH.

2.35.5.7 <u>Unit Price Input Required</u>.

2.35.5.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

2.35.5.7.2 Cost of standard size pipe (12"Ø), COSP, $/ft.

2.35.5.7.3 Cost of standard size valve (12"Ø butterfly), COSTSV, $.

2.35.5.7.4 Cost per sprinkler, COSTEN, $.

2.35.5.7.5 Cost of standard size drain pipe (24"Ø R.C. pipe), COSTCP, $/ft.

2.35.5.7.6 Unit price input for heavy clearing and grubbing, UPICG, $/acre.

2.35.5.7.7 Unit price input for fencing, UPIF, $/ft.

2.35.5.8 <u>Cost Calculations</u>.

2.35.5.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW + VSEW + VET}{27} \, UPIEX$$

where

COSTE = cost of earthwork, $.

VLEW = volume of earthwork required for lagoon construction, ft^3.

VSEW = volume of earthwork required for slope construction, ft^3.

VET = volume of earthwork for terraces, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

2.35.5.8.2 Cost of Header pipes.

2.35.5.8.2.1 Calculate total installed cost of header pipes.

$$TICHP = ICHPN$$

where

TICHP = total installed cost of header pipes, \$.

ICHPN = installed cost of various size header pipes, \$.

2.35.5.8.2.2 Calculate installed cost of each size header pipes.

$$ICHPN = (LDIAHN)\frac{(COSTPN)}{100}(COSTSP)$$

where

ICHPN = installed cost of various size header pipes, \$.

LDIAHN = length of header pipes of size DIAHN, ft.

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

2.35.5.8.2.3 Calculate COSTPN

$$COSTPN = 6.842\ (DIAHN)^{1.2255}$$

where

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

DIAHN = diameters of header pipes, inches.

2.35.5.8.2.4 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot in 4th quarter of 1977.

2.35.5.8.3 Cost of lateral pipes.

2.35.5.8.3.1 Calculate total installed cost of lateral pipes.

$$TICLP = (LDIAL) \frac{(COSTP)}{100} (COSP)$$

where

TICLP = total installed cost of lateral pipes, \$.

LDIAL = length of lateral pipes, ft.

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

2.35.5.8.3.2 Calculate COSTP.

$$COSTP = 6.842 (DIAL)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

DIAL = diameter of lateral pipes, inches.

2.35.5.8.3.3 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot in 4th quarter of 1977.

2.35.5.8.4 Calculate cost of butterfly valves.

2.35.5.8.4.1 Calculate installed cost of butterfly valves.

$$COSTBV = \frac{(COSTRV) (COSTSV) (NBV)}{100}$$

where

COSTBV = installed cost of butterfly valves, \$.

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

COSTSV = cost of standard size valve (12"Ø), \$.

NBV = number of butterfly valves.

2.35.5.8.4.2 Calculate COSTRV.

$$COSTRV = 3.99\ (DBV)^{1.395}$$

where

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

DBV = diameter of butterfly valves, inches.

2.35.5.8.4.3 Determine COSTSV. COSTSV is the cost of a 12" ∅ butterfly valve suitable for water service. This cost is $1004 for the 4th quarter of 1977.

2.35.5.8.5 Calculate cost of lateral valves.

2.35.5.8.5.1 Calculate installed cost of lateral valves.

$$COSTLV = \frac{(COSTRL)(COSTSV)(NLV)}{100}$$

where

COSTLV = installed cost of lateral valves, $.

COSTRL = cost of lateral valve of size DLV as percent of cost of standard valve, %.

COSTSV = cost of standard size valve (12"∅ butterfly), $.

NLV = number of lateral valves.

2.35.5.8.5.2 Calculate COSTRL.

$$COSTRL = 15.33\ (DLV)^{1.053}$$

where

COSTRL = cost of lateral valve of size DLV as percent of cost of standard valve, %.

DLV = diameter of lateral valve, inches.

2.35.5.8.5.3 Determine COSTSV. COSTSV is the cost of a 12" ∅ butterfly valve suitable for water service. This cost is $1004 for the 4th quarter of 1977.

2.35.5.8.6 Calculate cost of sprinklers.

2.35.5.8.6.1 Calculate installed cost of sprinklers.

$$COSTN = (1.2)(NON)(COSTEN)$$

where

COSTN = installed cost of nozzles, \$.

NON = number of nozzles.

COSTEN = cost per nozzle, \$.

1.2 = 20% cost of installation.

2.35.5.8.6.2 Determine COSTEN. The cost of an impact type rotor pop-up full circle sprinkler with a flow from 6 to 15 gpm, for the 4th quarter of 1977 is \$65.00.

$$COSTEN = \$65.00.$$

For better cost estimation, COSTEN should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTEN = \$65.00 \frac{MSECI}{518.4}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

518.4 = Marshall and Swift Cost Index 4th quarter 1977.

2.35.5.8.7 Calculate total cost of distribution system.

$$TCDS = TICHP + TICLP + COSTBV + COSTLV + COSTN$$

where

TCDS = total cost of distribution system, \$.

TICHP = total installed cost of header pipe, \$.

TICLP = total installed cost of lateral pipe, \$.

COSTBV = installed cost of butterfly valves, \$.

COSTLV = installed cost of lateral valves, \$.

COSTN = installed cost of nozzles, \$.

2.35.5.8.8 Determine cost of runoff collection by open ditch.

$$COSTOD = (.57)(LDIT)UPIEX$$

where

COSTOD = cost of runoff collection by open ditch, \$.

LDIT = total length of ditches for system, ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.35.5.8.9 Detemine cost of runoff collection by gravity pipe.

2.35.5.8.9.1 Calculate total installed cost of collection system.

$$TICRC = \sum ICRCN$$

where

TICRC = total installed cost of runoff collection by gravity pipe, \$.

ICRCN = installed cost of each size drain pipe, \$.

2.35.5.8.9.2 Calculate installed cost of each size drain pipe.

$$ICRCN = \frac{(LCDIAN)(COSTRN)(COSTSC)}{100}$$

where

ICRCN = installed cost of each size drain pipe, \$.

LCDIAN = length of drain pipe of size CDIAN, ft.

COSTRN = cost of drain pipe of size CDIAN as percent of cost of standard size (24"Ø) drainage pipe, %.

COSTCP = cost of standard size (24"Ø R.C. pipe), \$/ft.

2.35.5.8.9.3 Calculate COSTRN.

$$COSTRN = 0.7044\ (CDIAN)^{1.6587}$$

where

COSTRN = cost of gravity pipe of size CDIAN as percent of cost of standard size (24"Ø) gravity pipe, %.

CDIAN = diameter of concrete drain pipe, inches.

2.35.5.8.9.4 Determine COSTCP. COSTCP is the cost per foot of 24"Ø reinforced concrete drain pipe with gasket joints. This cost for the 4th quarter of 1977 is $10.76/ft.

$$COSTCP = \$10.76/ft.$$

2.35.5.8.10 Calculate cost for clearing and grubbing.

$$COSTCG = (CAGH + 0.306\ CAGM + 0.092\ CAGL)\ UPICG$$

where

COSTCG = cost for clearing and grubbing site, $.

CAGH = area which requires heavy clearing, acres.

CAGM = area which requires medium clearing, acres.

CAGL = area which requires light clearing, acres.

UPICG = unit price input for heavy clearing and grubbing, $/acre.

2.35.5.8.11 Calculate cost of fencing.

$$COSTF = (LF)(UPIF)$$

where

COSTF = installed cost of fencing, $.

LF = length of fencing required, ft.

UPIF = unit price input for fencing, $/ft.

2.35.5.8.12 Calculate O & M material costs.

$$OMMC = \frac{(OMMPD)(TCDS) + (OMMPP)(TICRC) + (OMMPO)(COSTOD)}{100}$$

where

OMMC = total O&M material cost, $.

OMMPD = O&M material costs for distribution system as percent construction cost for distribution system, %.

TCDS = total cost of distribution system, $.

OMMPP = O&M material costs for runoff collection by gravity pipe as percent of construction cost of gravity pipe system, %.

TICRC = total installed cost of runoff collection by gravity pipe, $.

OMMPO = O&M material cost for runoff collection by open ditch as percent construction cost for open ditch system, %.

COSTOD = cost of runoff collection by open ditch, $.

2.35.5.8.13 Total bare construction cost.

$$TBCCOF = (1.11)(TCDS + TICRC + COSTOD + COSTE + COSTCG + COSTF + COSTL)$$

where

TBCCOF = total bare construction cost for overland flow, $.

TCDS = total cost of distribution system, $.

TICRC = total installed cost of runoff collection by gravity pipe, $.

COSTOD = cost of runoff collection by open ditch, $.

COSTE = cost of earthwork, $.

COSTCG = cost of clearing and grubbing site, $.

COSTF = installed cost of fencing, $.

COSTL = cost of land for facility, $.

2.35.5.9 Cost Calculations Output Data.

2.35.5.9.1 Total bare construction cost for overland flow, TBCCOF, $.

2.35.5.9.2 O&M material cost, OMMC, $/yr.

2.35.6 Rapid Infiltration.

2.35.6.1 Input Data.

2.35.6.1.1 Wastewater flow, Q, mgd.

2.35.6.1.1.1 Minimum flow, mgd.

2.35.6.1.1.2 Average flow, mgd.

2.35.6.1.1.3 Maximum flow, mgd.

2.35.6.1.2 Wastewater characteristics.

2.35.6.1.2.1 Suspended solids, mg/l.

2.35.6.1.2.2 Volatile suspended solids, % of suspended solids.

2.35.6.1.2.3 Settleable solids, mg/l.

2.35.6.1.2.4 BOD_5 (soluble and total), mg/l.

2.35.6.1.2.5 COD (soluble and total), mg/l.

2.35.6.1.2.6 Phosphorus (as PO_4), mg/l.

2.35.6.1.2.7 Total kjeldahl nitrogen (TKN), mg/l.

2.35.6.1.2.8 Ammonia-nitrogen, NH_3, mg/l.

2.35.6.1.2.9 Nitrite-nitrogen, NO_2, mg/l.

2.35.6.1.2.10 Nitrate-nitrogen, NO_3, mg/l.

2.35.6.1.2.11 Temperature, °C.

2.35.6.1.2.12 pH, units.

2.35.6.1.2.13 Oil and grease, mg/l.

2.35.6.1.2.14 Cations, mg/l.

2.35.6.1.2.15 Anions, mg/l.

2.35.6.2 Design Parameters.

2.35.6.2.1 Application rate, L_w, in/wk = 4.0 - 150 in/wk.

2.35.6.2.2 Precipitation rate, P_r, in/wk.

2.35.6.2.3 Evapotranspiration, ET, in/wk.

2.35.6.2.4 Runoff, R, in/wk = 0.0.

2.35.6.2.5 Wastewater generation period, WWGP, days/yr.

2.35.6.2.6 Field application period, FAP, wks/yr.

2.35.6.2.7 Recovery systems (specify one).

2.35.6.2.7.1 Recovery wells (number; diameter, in., and depth, ft).

2.35.6.2.7.2 Underdrains.

2.35.6.2.7.3 No recovery.

2.35.6.2.8 Buffer width, ft (site dependent) = 0.0 - 500 ft.

2.35.6.2.9 Monitoring wells.

2.35.6.2.9.1 Number.

2.35.6.2.9.2 Depth per well, ft.

2.35.6.2.10 Fraction denitrified, D, % = 30-60%.

2.35.6.2.11 Ammonia volatilization, AV, % = 0.0%.

2.35.6.2.12 Removal of phosphorus, % = 90%.

2.35.6.3 Process Design Calculations.

2.35.6.3.1 Calculate total nitrogen concentration, C_n, in the applied wastewater.

$$C_n = (TKN)_i + (NO_2)_i + (NO_3)_i$$

where

C_n = total nitrogen concentration in applied wastewater, mg/l.

$(TKN)_i$ = total Kjeldahl nitrogen concentration in applied wastewater, mg/l.

$(NO_2)_i$ = nitrite-N concentration in applied wastewater, mg/l.

$(NO_3)_i$ = nitrate-N concentration in applied wastewater, mg/l.

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL. 11/18

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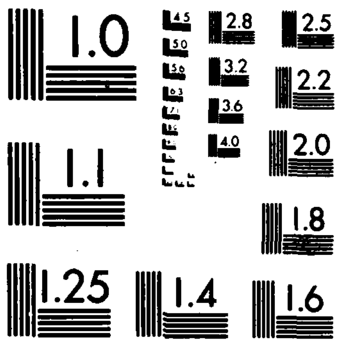

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

2.35.6.3.2 Calculate wastewater nitrogen loading, L_n, lbs/acre-d.

$$L_n = 11.77\ C_n L_w$$

where

L_n = wastewater nitrogen loading, lbs/acre-yr.

L_w = wastewater hydraulic loading rate, in/wk.

2.35.6.3.3 From water balance, calculate percolating water rate, W_p, in/wk.

$$W_p = L_w + (P_r - ET) - R$$

where

W_p = percolating water rate, in/wk.

P_r = design precipitation, in/wk.

ET = evapotranspiration (or crops consumptive use of water), in/wk.

R = net runoff, in/wk.

2.35.6.3.4 Calculate total nitrogen loading, $(L_t)_N$, lb/acre-yr.

$$(L_t)_N = L_n + 11.77\ (P_r)(0.5)$$

where

$(L_t)_N$ = total nitrogen loading rate, lb/acre-yr.

0.5 = assumed nitrogen concentration in precipitation water, mg/l.

2.35.6.3.5 Assume crop nitrogen uptake, $(U)_N$, lb/acre-yr.

2.35.6.3.6 Calculate nitrogen loss through denitrification, D, lb/acre-yr.

$$D = (D_f)(L_t)_N/(100)$$

where

D = nitrogen loss through denitrification, lb/acre-yr.

D_f = nitrogen loss as a percent of total applied nitrogen, %.

2.35.6.3.7 Calculate nitrogen loss due to volatilization, AV, lb/acre-yr.

$$AV = (AV)_f\ (L_t)_N/(100)$$

where

AV = nitrogen loss due to volatilization, lb/acre-yr.

$(AV)_f$ = percent of total nitrogen applied lost to volatilization, %.

2.35.6.3.8 Calculate sum of nitrogen losses, $(\Sigma N)_L$, lb/acre-yr.

$$(\Sigma N)_L = (U)_N + D + AV$$

where

$(\Sigma N)_L$ = sum of total nitrogen lost, lb/acre-yr.

2.35.6.3.9 Check total nitrogen against 0.8 $(L_t)_N$.

$$(\Sigma N)_L \leq 0.8\ (L_t)_N$$

$$\text{if } (\Sigma N)_L > 0.8\ (L_t)_N$$

$$\text{set } (\Sigma N)_L = 0.8\ (L_t)_N$$

2.35.6.3.10 From nitrogen balance, calculate nitrogen concentration in percolate, $(C_p)_N$, mg/l.

$$(L_t)_N = (11.77)(W_p)(C_p)_N + (\Sigma N)_L$$

$$(C_p)_N = [(L_t)_N - (\Sigma N)_L]/(11.77)(W_p)$$

where

$(C_p)_N$ = nitrogen concentration in percolate, mg/l.

2.35.6.3.11 Calculate required treatment acre, TA, acres.

$$TA = \frac{(WWGP)(36.84)(Q)}{(FAP)(L_w)}$$

where

TA = required field area, acres.

Q = average wastewater flow, mgd.
L_w = wastewater hydraulic loading rate, in/wk.

2.35.6.3.12 Calculate phosphorus loading, L_p, lb/acre-yr.

$$L_p = 11.77\ (TP)_i(L_w)$$

where

L_p = total phosphorus loading, lbs/acre-yr.

$(TP)_i$ = total phosphorus concentration in applied wastewater, mg/l.

2.35.6.3.13 Calculate soil removal of phosphorus, SRP, lb/ acre-yr.

$$(SRP) = 0.891\ [(94.544 - 0.0041)(L_p)]$$

where

SRP = soil removal of phosphorus, lbs/acre-yr.

2.35.6.3.14 From phosphorus mass balance, calculate phosphorus concentration of percolate water.

$$(C_p)_p = [(L_p) - (SRP)]/(11.77)(W_p)$$

where

$(C_p)_p$ = phosphorus content of percolate water, mg/l.
(0.10) $(TP)_i$

2.35.6.3.15 Calculate percolate rate, W_p, mgd.

$$W_p\ (mgd) = [(W_p)\ in/wk\ (\frac{1\ ft}{12\ in})(\frac{1\ wk}{7\ day})]\ (TA)\ acre\ (43{,}560\ \frac{ft^2}{acre})$$
$$(7.48\ \frac{gal}{ft^3})(1/10^6)$$

2.35.6.3.16 Calculate suspended solids concentration in percolate, mg/l, assume 99% removal.

$$(SS)_p = (0.01)(SS)_i$$

where

$(SS)_p$ = suspended solids concentration in percolate, mg/l.

$(SS)_i$ = suspended solids concentration in applied wastewater, mg/l.

2.35.6.3.17 Calculate total and soluble BOD_5 concentration in percolate, mg/l, assume 99% removal of total BOD_5.

$$(TBOD_5)_p = (TBOD_5)_i \ (0.01)$$

$$(SBOD_5)_p = (SBOD_5)_i \ (0.01)$$

where

$(TBOD_5)_p$ = total BOD_5 concentration in percolate, mg/l.

$(TBOD_5)_i$ = total BOD_5 concentration in applied wastewater, mg/l.

$(SBOD_5)_p$ = soluble BOD_5 in percolate, mg/l.

$(SBOD_5)_i$ = soluble BOD_5 in applied wastewater, mg/l.

2.35.6.3.18 Calculate total and soluble COD concentration in percolate, mg/l.

$$(TCOD)_p = [(TCOD)_i - (TBOD_5)_i] \ (0.5) + 0.01 \ (TBOD_5)_i$$

$$(SCOD)_p = [(SCOD)_i - (SBOD_5)_i] \ (0.5) + 0.01 \ (SBOD_5)_i$$

where

$(TCOD)_p$ and $(TCOD)_i$ = total COD concentration in percolate and applied wastewater, respectively, mg/l.

$(SCOD)_p$ and $(SCOD)_i$ = soluble COD concentration in percolate and applied wastewater, respectively, mg/l.

2.35.6.3.19 Nitrite-N concentration in percolate = 0.0.

2.35.6.3.20 Nitrate-N concentration in percolate = 0.4 $(C_p)_N$.

2.35.6.3.21 Ammonia-N concentration in percolate = 0.35 $(C_p)_N$. Total Kjeldahl-nitrogen concentration in percolate = .60 $(C_p)_N$.

2.35.6.3.22 Oil and grease concentration in percolate = 0.0. SETSO = 0.0.

2.35.6.4 Process Design Output Data.

2.35.6.4.1 Application rate, in/week.

2.35.6.4.2 Evapotranspiration rate, in/week.

2.35.6.4.3 Precipitation rate, in/week.

2.35.6.4.4 Runoff, in/week.

2.35.6.4.5 Percent denitrified, percent.

2.35.6.4.6 Percent ammonia volatilization, percent.

2.35.6.4.7 Removal of phosphorus, percent.

2.35.6.4.8 Wastewater generation period, days/yr.

2.35.6.4.9 Field application period, weeks/yr.

2.35.6.4.10 Surface flooding.

2.35.6.4.11 Buffer zone width, feet.

2.35.6.4.12 Number of monitoring wells, wells,

2.35.6.4.13 Depth of monitoring wells, feet.

2.35.6.4.14 Treatment area required, acres.

2.35.6.4.15 Volume of percolate, mgd.

2.35.6.4.16 Quality of percolate.

2.35.6.4.16.1 Suspended solids, mg/l.

2.35.6.4.16.2 Volatile solids, percent.

2.35.6.4.16.3 BOD_5, mg/l.

2.35.6.4.16.4 BOD_5 soluble, mg/l.

2.35.6.4.16.5 COD, mg/l.

2.35.6.4.16.6 COD soluble, mg/l.

2.35.6.4.16.7 PO_4, mg/l.

2.35.6.4.16.8 TKN, mg/l.

2.35.6.4.16.9 NO_2, mg/l.

2.35.6.4.16.10 NO_3, mg/l.

2.35.6.4.16.11 Oil and grease, mg/l.

2.35.6.5 Quantities Calculations.

2.35.6.5.1 Distribution Pumping. Distribution pumping will be taken from the section entitled "Intermediate Pumping".

2.35.6.5.2 Determine number and size of basins required.

2.35.6.5.2.1 Assume:

Use minimum depth of 4 feet.
Use a minimum of 4 infiltration basins.
Infiltration basins will be a maximum of 10 acres in area and will be square.

2.35.6.5.2.2 If TA $\leq$ 4 acres

$$NIB = 2$$

$$IBA = \frac{TA}{2}$$

If IBA < .1 set IBA = .1

where

TA = treatment area, acres.

NIB = number of infiltration basins.

IBA = area of individual infiltration basins, acres.

2.35.6.5.2.3 If TA is less than or equal to 40 acres use 4 equal sized basins.

$$4 < TA \leq 40; \; NIB = 4$$

$$IBA = \frac{TA}{4}$$

where

TA = treatment area

NIB = number of infiltration basins

IBA = area of individual infiltration basins

2.35.6.5.2.4 If TA is greater than 40 acres.

$$TA > 40; \; NIB = \frac{TA}{10}$$

NIB must be an integer.

$$IBA = \frac{TA}{NIB}$$

2.35.6.5.3 Calculate volume of earthwork for basins.

2.35.6.5.3.1 Assume:

Levees will be built on top of natural ground with fill hauled in from off the site.
Levee side slopes will be 3 to 1.
Top of the levee will be 10 feet wide.
Basins will be 4 feet deep.
Basins will be square.

2.35.6.5.3.2 Calculate dimensions of basins.

$$L = 208.7\ (IBA)^{0.5}$$

where

L = Length of one side of the basin.

2.35.6.5.3.3 Volume of earthwork.

$$V_{ew} = NIB\ (352L + 11{,}968)$$

where

V_{ew} = volume of fill required to construct levees.

L = length of one side of the basin.

NIB = number of infiltration basins.

2.35.6.5.4 Calculate header size to feed infiltration basins.

If GF $\leq$ 40 mgd calculate PIPE using GF if GF > 40 mgd calculate PIPE using GF/2.
Assume velocity (V) = 4 fps.

$$PIPE = 8.42\ (GF)^{0.5}$$

Pipe must be one of the following: 2,3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

Check velocity (V) using selected pipe size.

$$V = \frac{(283.6)\ (GF)}{(PIPE)^2} \text{ or } \frac{(283.6)\ (GF/2)}{(PIPE)^2}$$

If $V \leq 1$ fps use next smallest diameter.

If $V > 5$ fps use next largest diameter.

where

PIPE = diameter of pipe (inches).

GF = generated flow (mgd).

V = velocity of water in pipe (fps).

2.35.6.5.5 Calculate quantity of header pipe required.

If $GF \leq 40$ mgd LPIPE = (NIB)(L)

If $GF > 40$ mgd LPIPE = 2(NIB)(L)

where

LPIPE = length of header pipe required, ft.

2.35.6.5.6 Calculate pipe size for lateral to each infiltration basin.

2.35.6.5.6.1 Calculate flow.

$$FLOW = (.012)(AR)(IBA)$$

If $FLOW \leq 62$ ft^3/sec calculate DIA using FLOW.

If $FLOW > 62$ ft^3/sec calculate DIA using FLOW/2.

2.35.6.5.6.2 Calculate diameter.

Assume velocity (V) = 4 fps.

$$DIA = 6.77 \ (FLOW)^{0.5}$$

DIA must be one of the following 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 30, 36, 42, 48.

2.35.6.5.6.3 Select diameter closest to the calculated diameter.

2.35.6.5.6.4 Check the velocity (V) using the selected diameter.

$$V = \frac{(183.3)\ FLOW}{(DIA)^2}$$

If $V \leq 1$ fps use next smallest diameter.

If $V > 5$ fps use next largest diameter.

where

FLOW = the wastewater flow to each basin (ft^3/sec).

DIA = diameter of lateral pipe (inches).

V = velocity of water in pipe (ft/sec).

AR = application rate, (in/wk).

2.35.6.5.7 Determine size and number of valves for distribution system.

2.35.6.5.7.1 Assume:

Each lateral will have a valve to cut off flow to that infiltration basin.
Valves will be butterfly valves suitable for use in water service.

Valves will be the same size as lateral pipe (DIA).

2.35.6.5.7.2 If FLOW $\leq 62\ ft^3/sec$.

NBV = NIB

2.35.6.5.7.3 If FLOW $> 62\ ft^3/sec$

NBV = 2 NIB

where

NBV = Number of butterfly valves.

NIB = Number of infiltration basins.

FLOW = the wastewater flow to each basin, ft^3/sec

DIA = diameter of lateral pipe, inches.

2.35.6.5.8 Calculate quantity of lateral pipe required.

If FLOW $\leq$ 62 ft^3/sec; LLAT = (100) (NIB)

If FLOW $>$ 62 ft^3/sec; LLAT = (2) (100) (NIB)

where

FLOW = the wastewater flow to each basin (ft^3/sec).

LLAT = length of lateral pipe of diameter DIA, (ft).

NIB = number of infiltration basins.

2.35.6.5.9 Recovery of renovated water.

Two recovery systems are commonly used, underdrains and recovery wells. The user must designate which system is to be used if the water is to be recovered.

2.35.6.5.9.1 Underdrain system.

2.35.6.5.9.1.1 The following assumptions are made:

Perforated PVC pipe 6 inches in diameter will be used for underdrain laterals in basins.
100% of the applied wastewater will be recovered.
The 6 inch pipe will be laid on 1% slope and assumed to flow 1/2 full.
Concrete sewer pipe will be used as collection headers.

2.35.6.5.9.1.2 Calculate quantity of underdrain pipe required.

$$DPIPE = (.0105)(L)(IBA)(AR)(NIB)$$

where

DPIPE = length of 6" drain pipe required (ft).

L = length of one side of infiltration basin, (ft).

IBA = area of individual infiltration basins, (acres).

AR = application rate, (inches/wk).

NIB = number of infiltration basins.

0.0105 = accumulated constants.

2.35.6.5.9.1.3 Calculate size and quantity of collection header pipe.

Assume:

Class III concrete sewer pipe will be used.
Pipe will be laid on 1% slope.
Pipe will be sized by Manning formula assumed flowing half full with "N" factor 0.013.

$$CDIA = 9.56\ (FLOW)^{0.375}$$

where

CDIA = diameter of collection header pipe, inches.

FLOW = the wastewater flow to each basin, ft^3/sec.

9.56 = accumulated constants.

$$LDCH = (L)(NIB)$$

where

LDCH = length of drain collection header pipe of diameter CDIA, ft.

L = length of one side of infiltration basins, ft.

NIB = number of infiltration basins.

2.35.6.5.9.2 Recovery wells.

User must specify number of wells (NW), size of wells (WDIA), and depth of wells (DW).

where

NW = number of recovery wells required.

WDIA = diameter of recovery wells, inches.

DW = depth of recovery wells, ft.

2.35.6.5.10 Monitoring System.

Monitoring wells shall be 4" in diameter. User must specify the number of monitoring wells, (NMW) and depth of monitoring wells (DMW).

where

NMW = number of monitoring wells.

DMW = depth of monitoring wells, (ft).

2.35.6.5.11 Operation and maintenance manpower requirements.

2.35.6.5.11.1 Distribution System.

2.35.6.5.11.1.1 If $TA \leq 15$

$$OMMHD = 128.5\ (TA)^{0.6285}$$

2.35.6.5.11.1.2 If TA > 15

$$OMMHD = 78.8\ (TA)^{0.8092}$$

where

TA = treatment area, acres.

OMMHD = operation and maintenance manpower for distribution, MH/yr.

2.35.6.5.11.2 Water recovery by wells.

$$OMMHW = 384.64\ (GF)^{0.5981}$$

where

OMMHW = operation and maintenance manpower for water recovery by wells, MH/yr.

GF = generated flow, mgd.

2.35.6.5.11.3 Water recovery by underdrains.

2.35.6.5.11.3.1 If TA ≤ 80

$$OMMHU = 54.71\ (TA)^{-0.2414}$$

2.35.6.5.11.3.2 If TA > 80

$$OMMHU = 10.12\ (TA)^{0.6255}$$

where

OMMHU = operation and maintenance manpower for water recovery by underdrains, MH/yr.

TA = treatment area, acres.

2.35.6.5.11.4 Monitoring wells.

$$OMMHM = 6.39(NMW)(DMW)^{0.2760}$$

where

OMMHM = operation and maintenance manpower for monitoring wells, MH/yr.

NMW = number of monitoring wells.

DMW = depth of monitoring wells, ft.

2.35.6.5.12 Operation and Maintenance Materials Cost. This item includes repair and replacement material costs. It is expressed as a percentage of the capital costs for the various areas of construction of the rapid infiltration system.

2.35.6.5.12.1 Distribution System.

2.35.6.5.12.1.1 If $TA \leq 19$

$$OMMPD = 2.64\ (TA)^{-0.2101}$$

2.35.6.5.12.1.2 IF $TA > 19$

$$OMMPD = 1.59\ (TA)^{-0.0399}$$

where

OMMPD = O&M material cost for distribution system as percentage of construction cost of distribution system.

TA = treatment area, acres.

2.35.6.5.12.2 Water recovery by wells.

2.35.6.5.12.2.1 If $GF \leq 5$

$$OMMPW = 1.53\ (GF)^{0.6570}$$

2.35.6.5.12.2.2 If $5 < GF \leq 10$

$$OMMPW = 2.76\ (GF)^{0.2894}$$

2.35.6.5.12.2.3 If $GF > 10$

$$OMMPW = 4.55\ (GF)^{0.0715}$$

where

OMMPW = O&M material cost for water recovery wells as percentage of construction cost of recovery wells.

GF = generated flow, mgd.

2.35.6.5.12.3 Water recovery by underdrains.

2.35.6.5.12.3.1 If $T \leq 200$

$$OMMPU = 14.13\ (TA)^{-0.1392}$$

2.35.6.5.12.3.2 If $TA > 200$

$$OMMPU = 30.95 \ (TA)^{-0.2860}$$

where

OMMPU = O&M material costs for water recovery by underdrains as percentage of construction cost of underdrains.

TA = Treatment area, acres.

2.35.6.5.12.4 Monitoring wells.

$$OMMPM = 2.28 \ (DMW)^{0.0497}$$

where

OMMPM = O&M material cost for monitoring wells as percentage of construction cost of monitoring wells.

DMW = depth of monitoring wells.

2.35.6.5.13 Electrical energy requirements.

2.35.6.5.13.1 Recovery wells.

Assume:

Pump efficiency of 60%.
Motor efficiency of 90%.
Total head is equal to the well depth plus 40 ft.

$$KWH = (12.6) \ (GF) \ (DW+40) \ (DPW) \ (HPD)$$

where

KWH = energy required, kwhr/yr.

GF = generated flow, mgd.

DW = depth of well, ft.

DPW = days per week of operations, day/wk.

HPD = hours per day of operation, hr/day.

2.35.6.5.14 Other construction cost items.

The quantities and items computed account for approximately 85% of the cost of the systems. Other miscellaneous items such as concrete head walls, pneumatic piping, etc., make up the other 15%.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other construction costs.

2.35.6.6 Quantities Calculations Output Data.

2.35.6.6.1 Area of individual infiltration basins, IBA, acres.

2.35.6.6.2 Number of infiltration basins, NIB.

2.35.6.6.3 Length of side of basin, L, ft.

2.35.6.6.4 Volume of earthwork for infiltration basins, V_{ew}, cu.ft.

2.35.6.6.5 Diameter of distribution header, PIPE, inches.

2.35.6.6.6 Length of distribution header pipe required, LPIPE, ft.

2.35.6.6.7 Diameter of lateral pipe, DIA, inches.

2.35.6.6.8 Length of lateral pipe of diameter DIA, LLAT, ft.

2.35.6.6.9 Length of 6 inch diameter drain pipe required, DPIPE, ft.

2.35.6.6.10 Number of recovery wells, NW.

2.35.6.6.11 Diameter of recovery wells, WDIA, inches.

2.35.6.6.12 Depth of recovery wells, DW, ft.

2.35.6.6.13 Number of monitoring wells, NMW.

2.35.6.6.14 Depth of monitoring wells, DMW, ft.

2.35.6.6.15 Operation and Maintenance manpower for distribution, OMMHD, MH/yr.

2.35.6.6.16 Operation and maintenance manpower for recovery wells, OMMHU, MH/yr.

2.35.6.6.17 Operation and maintenance manpower for underdrain system, OMMHU, MH/yr.

2.35.6.6.18 Operation and maintenance manpower for monitoring wells, OMMHM, MH/yr.

2.35.6.6.19 O&M material costs for distribution system, OMMPD, %.

2.35.6.6.20 O&M material costs for recovery wells, OMMPW, %.

2.35.6.6.21 O&M material cost for underdrain system, OMMPU, %.

2.35.6.6.22 O&M material costs for monitoring wells, OMMPM, %.

2.35.6.6.23 Diameter of drain collection header pipe, CDIA, inches.

2.35.6.6.24 Length of drain collection header pipe, LDCH, ft.

2.35.6.6.25 Number of butterfly vlaves, NBV.

2.35.6.6.26 Energy required for recovery wells, KWH, Kw hr/yr.

2.35.6.6.27 Correction factor for other construction costs, CF.

2.35.6.7 Unit Price Input Required.

2.35.6.7.1 Unit price input for earthwork assuming hauled from offsite and compacted, UPIEW, $/cu yd.

2.35.6.7.2 Cost of standard size steel pipe (12"Ø), COSP, $/ft.

2.35.6.7.3 Cost of standard size butterfly valve (12"Ø) COSTSV, $.

2.35.6.7.4 Unit price input for 6" PVC perforated drain pipe, UPIPP, $/ft.

2.35.6.7.5 Cost of standard size (24"Ø) reinforced concrete drain pipe (Class III) COSTCP, $/ft.

2.35.6.8 Cost Calculations.

2.35.6.8.1 Cost of earthwork.

$$COSTE = \frac{(V_{ew})\ (UPIEW)}{27}$$

where

COSTE = Cost of earthwork for levees, $.

V_{ew} = Volume of earthwork for infiltration basins, cu ft

UPIEW = unit price input for earthwork assuming hauled from offsite and compacted, $/cu.yd.

2.35.6.8.2 Cost of header pipe.

2.35.6.8.2.1 Calculate installed cost header pipe (excluding trenching and backfilling).

$$COSTP = \frac{(COSTP)\ (COSTRP)}{100}$$

where

COSTP = cost of pipe of diameter PIPE, \$/ft.

COSP = cost of standard size pipe (12" diameter), \$/ft.

COSTRP = cost of pipe of diameter PIPE as percent of cost of standard size pipe, percent.

2.35.6.8.2.2 Calculate COSTRP.

$$COSTRP = 5.48\ (PIPE)^{1.1655}$$

where

PIPE = diameter of header pipe, inches.

COSTRP = cost of pipe of diameter PIPE as percent of cost of standard size pipe, percent.

2.35.6.8.2.3 Determine COSP.

COSP is the cost per foot of 12" diameter welded steel pipe in place (excluding cost for trenching and backfilling).

2.35.6.8.2.4 Calculate cost for trenching and backfilling. This cost is computed as a fraction of the cost of the pipe.

$$\text{If } PIPE \leq 12" \quad EBF = 0.334\ (PIPE)^{-0.6840}$$

$$\text{If } PIPE > 12" \quad EBF = 0.061$$

where

EBF = fraction of pipe cost for trenching and backfilling.

PIPE = diameter of header pipe, inches.

2.35.6.8.2.5 Total installed cost of header pipe.

$$TICHP = (1 + EBF)\ (COSTP)\ (LPIPE)$$

where

TICHP = total installed cost of header pipe, \$.

EBF = fraction of pipe cost for trenching and backfilling.

COSTP = cost of pipe of diameter PIPE, \$/ft.

LPIPE = length of header pipe required, ft.

2.35.6.8.3 Cost of lateral piping to infiltration basins.

2.35.6.8.3.1 Calculate installed cost of lateral piping (excluding trenching and backfilling).

$$COSTLP = \frac{(COSP)\ (COSTRL)}{100}$$

where

COSTLP = cost of lateral pipe of diameter DIA, \$/ft.

COSP = cost of standard size pipe (12" diameter), \$/ft.

COSTRL = cost of lateral pipe of diameter DIA as percent of cost of standard size pipe, percent.

2.35.6.8.3.2 Calculate COSTRL

$$COSTRL = 5.48\ (DIA)^{1.1655}$$

where

DIA = diameter of lateral pipe, inches.

COSTRL = cost of lateral pipe of diameter DIA as percent of cost of standard size pipe, percent.

2.35.6.8.3.3 Calculate cost of trenching and backfilling. This cost is computed as a fraction of the cost of the pipe.

$$\text{If } DIA \leq 12'' \quad EBFL = 0.334\ (DIA)^{-0.6840}$$

$$\text{If } DIA > 12'' \quad EBFL = 0.061$$

where

EBFL = fraction of pipe cost for trenching and backfilling.

DIA = diameter of lateral pipe, inches.

2.35.6.8.3.4 Total installed cost of lateral pipe.

$$TICLP = (1 + EBFL)\ (COSTLP)\ (LLAT)$$

where

TICLP = total installed cost of lateral pipe, \$.

EBFL = fraction of pipe cost for trenching and backfilling.

COSTLP = cost of lateral pipe of diameter DIA, \$/ft.

LLAT = length of lateral pipe required, ft.

2.35.6.8.4 Cost of butterfly valves.

2.35.6.8.4.1 Calculate installed cost of butterfly valves.

$$COSTBV = \frac{(COSTSV)\ (COSTRV)\ (NBV)}{100}$$

where

COSTBV = installed cost of butterfly valves, \$.

COSTSV = cost of standard size valve (12" ∅), \$.

COSTRV = cost of valve of size DIA as a percent of the cost of the standard size valve, %.

NBV = number of butterfly valves.

2.35.6.8.4.2 Calculate COSTRV

$$COSTRV = 3.99\ (DIA)^{1.395}$$

2.35.6.8.4.3 Determine COSTSV

COSTSV is the installed cost of a 12"Ø butterfly valve suitable for water service.

2.35.6.8.5 Total cost of distribution system.

$$TCDS = COSTE + TICHP + TICLP + COSTBV$$

where

TCDS = total cost of distribution system, $.

2.35.6.8.6 Cost of underdrain system.

2.35.6.8.6.1 Cost of underdrain laterals.

$$COSTUL = (DPIPE)\ (UPIPP)\ (1.1)$$

where

COSTUL = installed cost of underdrain laterals, $.

DPIPE = length of 6" drain pipe required, ft.

UPIPP = unit price input for 6" PVC perforated drain pipe, $/ft.

1.1 = 10% adjustment for trenching and backfilling.

2.35.6.8.6.2 Cost of underdrain collection header pipe.

$$ICUCH = \frac{(COSTRC)\ (COSTCP)\ (LDCH)\ (1 + EBFD)}{100}$$

where

ICUCH = installed cost of underdrain collection header.

COSTRC = cost of underdrain collection header pipe of diameter CDIA as percent of standard size pipe, %.

COSTCP = cost of standard size pipe (24"Ø reinforced concrete pipe Class III), $/ft.

LDCH = length of underdrain collection header required, ft.

EBFD = cost for trenching and backfilling as fraction of pipe cost.

2.35.6.8.6.2.1 Calculate COSTRC.

$$COSTRC = 0.489\ (CDIA)^{1.686}$$

where

CDIA = diameter of underdrain collection header pipe, inches.

2.35.6.8.6.2.2 Determine COSTCP.

COSTCP is the cost per foot of 24"Ø Class III reinforced concrete sewer pipe with gasket joints.

2.35.6.8.6.2.3 Calculate EBFD

$$EBFD = 0.392\ (CDIA)^{-0.2871}$$

where

EBFD = cost for trenching and backfilling as a fraction of pipe cost.

2.35.6.8.6.3 Calculate total cost of underdrain system.

$$TCUS = COSTUL + ICUCH$$

where

TCUS = total cost of underdrain system, \$.

2.35.6.8.7 Calculate cost of recovery wells and pump.

2.35.6.8.7.1 Calculate cost of well.

2.35.6.8.7.1.1 Calculate COSTW.

$$COSTW = RWC\ (DW)\ (NW)\ (COSP)$$

where

COSTW = cost of recovery wells, \$.

RWC = recovery well cost as fraction of cost of standard pipe.

COSP = cost of standard size pipe (12"Ø welded steel), \$/ft.

2.35.6.8.7.1.2 Calculate RWC.

2.35.6.8.7.1.2.1 If $4" \leq WDIA \leq 10"$

$$RWC = 160.4\ (DW)^{-0.7033}$$

2.35.6.8.7.1.2.2 If $12" \leq WDIA \leq 20"$

$$RWC = 159.8\ (DW)^{-0.6209}$$

2.35.6.8.7.1.2.3 If $24" \leq WDIA \leq 34"$

$$RWC = 142.7\ (DW)^{-0.5286}$$

2.35.6.8.7.1.2.4 If $36" \leq WDIA \leq 42"$

$$RWC = 206.5\ (DW)^{-0.445}$$

where

WDIA = diameter of the well, inches.

RWC = recovery well cost as fraction of cost of standard size pipe.

DW = depth of recovery wells, ft.

2.35.6.8.7.1.3 Determine COSP.

COSP is the cost per foot of 12"Ø welded steel pipe. This cost in the 1st quarter of 1977 is \$13.50/ft. For the best cost estimation COSP should be a current price input from a vendor, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSP = \$13.50\ \frac{MSECI}{491.6}$$

where

COSP = cost of standard size pipe (12"Ø welded steel), \$/ft.

MSECI = current value for Marshall and Swift Equipment Cost Index.

2.35.6.8.7.2 Calculate cost of pump for recovery wells.

2.35.6.8.7.2.1 Calculate COSTWP.

$$COSTWP = (COSTPS)\ (WPR)\ (NW)$$

where

COSTWP = cost of pumps for recovery wells, \$.

COSTPS = cost of standard size pump (3000 gpm), \$.

WPR = cost of well pump as fraction of cost of standard pump.

NW = number of recovery wells.

2.35.6.8.7.2.2 Calculate WPR.

$$WPR = 0.00048\ (WDIA)^{1.791}\ (DW)^{0.658}$$

where

WPR = cost of well pump as fraction of cost of standard pump.

WDIA = diameter of recovery wells, inches.

DW = depth of recovery wells, ft.

2.35.6.8.7.2.3 Determine COSTPS.

COSTPS is the cost of a 3000 gpm pump. This cost is \$17,250 for the first quarter of 1977. For the best cost estimate the user should input a current value for COSTPS, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250\ \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump (3000 gpm), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

2.35.6.8.7.3 Calculate total cost of recovery wells.

$$COSTRW = COSTW + COSTWP$$

where

COSTRW = total cost of recovery wells, \$.

COSTW = cost of recovery well, \$.

COSTWP = cost of pumps for recovery wells, \$.

2.35.6.8.8 Cost of monitoring wells and pumps.

2.35.6.8.8.1 Calculate cost of wells.

2.35.6.8.8.1.1 Calculate COSTM.

$$COSTM = (RMWC)(DMW)(NMW)(COSP)$$

where

COSTM = cost of monitoring wells, \$.

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

NMW = number of monitoring wells.

COSP = cost of standard size pipe (12" welded steel), \$/ft.

2.35.6.8.8.1.2 Calculate RMWC.

$$RMWC = 160.4\ (DMW)^{-0.7033}$$

where

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

2.35.6.8.8.1.3 Determine COSP.

COSP is the cost per foot of 12"ϕ welded steel pipe. This cost in first quarter of 1977 is \$13.50/ft. For the best cost estimate COSTSP should be a current price input from a vendor, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSP = \$13.50\ \frac{MSECI}{491.6}$$

where

COSP = cost of standard size pipe (12"ϕ welded steel), \$/ft.

MSECI = current Marshall and Swift Equipment Cost Index.

2.35.6.8.8.2 Calculate cost of pumps for monitoring wells.

2.35.6.8.8.2.1 Calculate COSTMP.

$$COSTMP = (COSTPS)(MWPR)(NMW)$$

where

COSTMP = cost of pumps for monitoring wells, \$.

COSTPS = cost of standard size pump (3000 gpm), \$.

MWPR = cost of well pump as fraction of cost of standard pump.

NMW = number of monitoring wells.

2.35.6.8.8.2.2 Calculate MWPR.

$$MWPR = 0.0551\ (DMW)^{0.658}$$

where

MWPR = cost of well pump as fraction of cost of standard size pump.

DMW = depth of monitoring wells, ft.

2.35.6.8.8.2.3 Detemine COSTPS.

COSTPS is the cost of a 3000 gpm pump. The cost is \$17,250 for the first quarter of 1977. For the best cost estimate the user should input a current value of COSTPS, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250\ \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump (3000 gpm), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

2.35.6.8.8.3 Calculate total cost of monitoring wells.

$$COSTMW = COSTM + COSTMP$$

where

COSTMW = total cost of monitoring wells, \$.

COSTM = cost of monitoring wells, \$.

COSTMP = cost of pumps for monitoring wells, \$.

2.35.6.8.9 Operation and maintenance material cost.

$$OMMC = \frac{(TCDS)(OMMPD) + (TCUS)(OMMPW) + (COSTRW)(OMMPW) + (COSTMW)(OMMPM)}{100}$$

where

OMMC = O&M material costs, \$/yr.

2.35.6.8.10 Total bare construction cost.

$$TBCCRI = (TCDS + TCUS + COSTRW + COSTMW)(1.18)$$

where

TBCCRI = total bare construction cost for rapid infiltration, \$.

TCDS = total cost of distribution system, \$.

TCUS = total cost of underdrain system, \$.

COSTRW = installed cost of recovery wells, \$.

COSTMW = installed cost of monitoring wells, \$.

2.35.6.9 Cost Calculations Output Data.

2.35.6.9.1 Total bare construction cost for rapid infiltration, TBCCRI, \$.

2.35.6.9.2 Operation and maintenance material cost, OMMC, \$/yr.

2.35.7 Slow Infiltration.

2.35.7.1 Input Data.

2.35.7.1.1 Wastewater flow, Q, mgd.

2.35.7.1.1.1 Minimum flow, mgd.

2.35.7.1.1.2 Average flow, mgd.

2.35.7.1.1.3 Maximum flow, mgd.

2.35.7.1.2 Wastewater characteristics.

2.35.7.1.2.1 Suspended solids, mg/l.

2.35.7.1.2.2 Volatile suspended solids, % of suspended solids.

2.35.7.1.2.3 Settleable solids, mg/l.

2.35.7.1.2.4 BOD_5 (soluble and total), mg/l.

2.35.7.1.2.5 COD (soluble and total), mg/l.

2.35.7.1.2.6 Phosphorus (as PO_4), mg/l.

2.35.7.1.2.7 Total Kjeldahl Nitrogen (TKN), mg/l.

2.35.7.1.2.8 Ammonia-Nitrogen, NH_3, mg/l.

2.35.7.1.2.9 Nitrite-Nitrogen, NO_2, mg/l.

2.35.7.1.2.10 Nitrate-Nitrogen, N_2O_3, mg/l.

2.35.7.1.2.11 Temperature, °C.

2.35.7.1.2.12 pH, units.

2.35.7.1.2.13 Oil and Grease, mg/l.

2.35.7.1.2.14 Cations, mg/l.

2.35.7.1.2.15 Anions, mg/l.

2.35.7.2 Design Parameters.

2.35.7.2.1 Crops classification (specify).

2.35.7.2.1.1 Forage grass.

2.35.7.2.1.2 Corn.

2.35.7.2.2 Application rate, L_w.

2.35.7.2.2.1 Average application rate, in/wk = (0.5 - 4 in/wk).

2.35.7.2.2.2 Maximum application rate, in/hr = (0.1-0.5 in/hr).

2.35.7.2.3 Precipitation rate, P_r, in/wk.

2.35.7.2.4 Evapotranspiration rate, ET, in/wk.

2.35.7.2.5 Runoff, R, in/wk.

2.35.7.2.6 Wastewater generation period, WWGP, days/yr.

2.35.7.2.7 Field application period, FAP, wks/yr.

2.35.7.2.8 Piping classification (specify one).

2.35.7.2.8.1 Solid set piping.

2.35.7.2.8.2 Center pivot piping.

2.35.7.2.9 Storage requirements, days/yr (specify one).

2.35.7.2.9.1 Minimum storage, days/yr.

2.35.7.2.9.2 No storage.

2.35.7.2.10 Liner required (liner should only be used with storage).

2.35.7.2.11 Embankment protection (should only be used with storage).

2.35.7.2.12 Recovery system (specify one).

2.35.7.2.12.1 Underdrains recovery system.

2.35.7.2.12.2 No recovery system.

2.35.7.2.13 Buffer zone width, ft (site dependent) = 0.0-500 ft.

2.35.7.2.14 Current ground cover, %.

2.35.7.2.14.1 Forest, % (require heavy clearing).

2.35.7.2.14.2 Brush, % (require medium clearing).

2.35.7.2.14.3 Pasture, % (require light clearing).

2.35.7.2.15 Slope, %.

2.35.7.2.15.1 Cultivated land 20%.

2.35.7.2.15.2 None cultivated land 40%.

2.35.7.2.16 Monitoring wells.

2.35.7.2.16.1 Number.

2.35.7.2.16.2 Depth per well, ft.

2.35.7.2.17 Fraction denitrified, D, % = 15 - 25%.

2.35.7.2.18 Ammonia volatilization, AV, % = 0.0%.

2.35.7.2.19 Soil removal of phosphorus, % = 80%.

2.35.7.2.20 Hours per day operation, HPD, hrs.

2.35.7.2.21 Days per week operation, DPW, days.

2.35.7.3 Process Design Calculations.

2.35.7.3.1 Calculate total nitrogen concentration, C_n, in the applied wastewater.

$$C_n = (TKN)_i + (NO_2)_i + (NO_3)_i$$

where

C_n = total nitrogen concentration in applied wastewater, mg/l.

$(TKN)_i$ = total Kjeldahl nitrogen concentration in applied wastewater, mg/l.

$(NO_2)_i$ = nitrite-N concentration in applied wastewater, mg/l.

$(NO_3)_i$ = nitrate-N concentration in applied wastewater, mg/l.

2.35.7.3.2 Calculate wastewater nitrogen loading, L_n, lbs/acre-d.

$$= 11.77\ C_n L_w$$

where

L_n = wastewater nitrogen loading, lbs/acre-yr.

L_w = wastewater hydraulic loading rate, in/wk.

2.35.7.3.3 From water balance, calculate percolating water rate, W_p, in/wk.

$$W_p = L_w + (P_r - ET) - R$$

where

W_p = percolating water rate, in/wk.

P_r = design precipitation, in/wk.

ET = evapotranspiration (or crops consumptive use of water), in/wk.

R = net runoff, in/wk.

2.35.7.3.4 Calculate total nitrogen loading, $(L_t)_N$, lb/acre-yr.

$$(L_t)_N = L_n + 11.77\ (P_r)(0.5)$$

where

$(L_t)_N$ = total nitrogen loading rate, lb/acre-yr. water, mg/l.

2.35.6.3.5 Assume crop nitrogen uptake, $(U)_N$, lb/acre-yr = 0.9.

2.35.7.3.5.1 For forage grass.

$$(U)_N = 0.891\ [(118.73) + 0.36\ (L_t)_N]\ \text{lb/acre-yr}$$

2.35.7.3.5.2 For corn.

$$(U)_N = 0.891[(176.53) - 0.0476(L_t)_N]\ \text{lb/acre-yr}$$

where

$(U)_N$ = crop nitrogen uptake, lb/acre-yr.

2.35.7.3.6 Calculate nitrogen loss through denitrification, D, lb/acre-yr.

$$D = (D_f)(L_t)_N/(100)$$

where

D = nitrogen loss through denitrification, lb/acre-yr.

D_f = nitrogen loss as a percent of total applied nitrogen, %.

2.35.7.3.7 Calculate nitrogen loss due to volatilization, AV, lb/acre-yr.

$$AV = (AV)_f \ (L_t)_N/(100)$$

where

AV = nitrogen loss due to volatilization, lb/acre-yr.

$(AV)_f$ = percent of total nitrogen applied lost to volatilization, %.

2.35.7.3.8 Calculate sum of nitrogen losses, $(\Sigma N)_L$, lb/acre-yr.

$$(\Sigma N)_L = (U)_N + D + AV$$

where

$(\Sigma N)_L$ = sum of total nitrogen lost, lb/acre-yr.

2.35.7.3.9 Check total nitrogen against (0.8 $(L_t)_N$.

$$(\Sigma N)_L \quad 0.8\ (L_t)_N$$

$$\text{if } (\Sigma N)_L \quad 0.8\ (L_t)_N$$

$$\text{set } (\Sigma N)_L = 0.8\ (L_t)_N$$

2.35.7.3.10 From nitrogen balance, calculate nitrogen concentration in percolate, $(C_p)_N$, mg/l.

$$(C_p)_N = [(L_t)_N - (\Sigma N)_L]/(11.77)(W_p)$$

where

$(C_p)_N$ = nitrogen concentration in percolate, mg/l.

2.35.7.3.11 Calculate required treatment acre, TA, acres.

$$TA = (258.5)(Q)/(L_w)$$

where

TA = required field area, acres.

Q = average wastewater flow, mgd.

2.35.7.3.12 Calculate volume of required storage, acre-ft.

$$(SV) = (SR)\ (Q_{ave})\ (10^6)/(7.48)\ (43{,}560)$$

where

SV = volume of required storage, acre-ft.

2.35.7.3.13 Calculate phosphorus loading, L_p, lb/acre-yr.

$$L_p = 11.77\ (TP)_i(L_w)$$

where

L_p = total phosphorus loading, lbs/acre-yr.

$(TP)_i$ = total phosphorus concentration in applied wastewater, mg/l.

3.35.7.3.14 Calculate soil removal of phosphorus, lb/acre-yr.

$$(SRP) = (SRP)_f\ (L_p)/(10C)$$

where

SRP = soil removal of phosphorus, lb/acre-yr.

$(SRP)_f$ = percent of total applied phosphorus removed by the soil, %.

2.35.7.3.15 Calculate plant uptake of phosphorus, U_p, lb/ acre-yr.

$$U_p = 0.891\ [215.54 - 37.11\ \log_e\ (L_p)]$$

where

U_p = plant uptake of phosphorus, lbs/acre-yr.

2.35.7.3.16 From phosphorus mass balance, calculate phosphorus concentration of percolate water.

$$(C_p)_p = [(L_p) - (SRP) - (U_p)]/(11.77)(W_p)$$

where

$(C_P)_P$ = phosphorus content of percolate water, mg/l. $\geq (0.01)\ (TP)_i$

2.35.7.3.17 Calculate percolate rate, W_p, mgd.

$$W_p(\text{mgd}) = [(W_p)\text{in/wk}(\frac{1\text{ ft}}{12\text{ in}})(\frac{1\text{ wk}}{7\text{ day}})]\ (TA)\text{ acre }(43{,}560\ \frac{\text{ft}^2}{\text{acre}})\ (7.48\ \frac{\text{gal}}{\text{ft}^3})(1/10^6)$$

2.35.7.3.18 Calculate suspended solids concentration in percolate, mg/l, assume 97% removal.

$$(SS)_p = (0.03)(SS)_i$$

where

$(SS)_p$ = suspended solids concentration in percolate, mg/l.

$(SS)_i$ = suspended solids concentration in applied wastewater, mg/l.

2.35.7.3.19 Calculate total and soluble BOD_5 concentration in percolate, mg/l, assume 95% removal of total BOD_5.

$$(TBOD_5)_p = (TBOD_5)_i\ (0.05)$$

$$(SBOD_5)_p = (SBOD_5)_i\ (0.05)$$

where

$(TBOD_5)_p$ = total BOD_5 concentration in percolate, mg/l.

$(TBOD_5)_i$ = total BOD_5 concentration in applied wastewater, mg/l.

$SBOD_5$ = soluble BOD_5, mg/l.

2.35.7.3.20 Calculate total and soluble COD concentration in percolate, mg/l, assume COD removal of 95%.

$$(TCOD)_p = (TCOD)_i\ (0.05)$$

$$(SCOD)_p = (SCOD)_i\ (0.05)$$

where

$(TCOD)_p$ and $(TCOD)_i$ = total COD concentration in percolate and applied wastewater, respectively, mg/l.

$(SCOD)_p$ and $(SCOD)_i$ = soluble COD concentration in percolate and applied wastewater, respectively, mg/l.

2.35.7.3.21 Nitrite-N concentration in percolate = 0.0.

2.35.7.3.22 Nitrate-N concentration in percolate = $(C_P)_N$.

2.35.7.3.23 Ammonia-N concentration in percolate = 0.0. Total Kjeldahl-nitrogen concentration in percolate = 0.0.

2.35.7.3.24 Oil and grease concentration in percolate = 0.0. SETSO = 0.0.

2.35.7.4 Process Design Output Data.

2.35.7.4.1 Hours per day operation, hours.

2.35.7.4.2 Days per week operation, days.

2.35.7.4.3 Forage grasses.

2.35.7.4.4 Application rate, in/week.

2.35.7.4.5 Maximum application rate, in/hour.

2.35.7.4.6 Evapotransportation rate, in/week.

2.35.7.4.7 Precipitation rate, in/week.

2.35.7.4.8 Runoff, in/week.

2.35.7.4.9 Percent denitrified, percent.

2.35.7.4.10 Percent ammonia volatilization, percent.

2.35.7.4.11 Removal of phosphorus, percent.

2.35.7.4.12 Wastewater generation period, days/yr.

2.35.7.4.13 Field application period, weeks/yr.

2.35.7.4.14 Solid set piping and pumping.

2.35.7.4.15 Calculated storage required, acre-ft.

2.35.7.4.16 Buffer zone width, feet.

2.35.7.4.17 Current ground cover.

2.35.7.4.17.1 Forest, percent.

2.35.7.4.17.2 Brush, percent.

2.35.7.4.17.3 Pasture, percent.

2.35.7.4.18 Slope of site, percent.

2.35.7.4.19 Number of monitoring wells, wells.

2.35.7.4.20 Depth of monitoring wells, feet.

2.35.7.4.21 Treatment area required, acres.

2.35.7.4.22 Volume of percolate, mgd.

2.35.7.4.23 Quality of percolate.

2.35.7.4.23.1 Suspended solids, mg/l.

2.35.7.4.23.2 Volatile solids, percent.

2.35.7.4.23.3 BOD_5, mg/l.

2.35.7.4.23.4 BOD_5 soluble, mg/l.

2.35.7.4.23.5 COD, mg/l.

2.35.7.4.23.6 COD soluble, mg/l.

2.35.7.4.23.7 PO_4, mg/l.

2.35.7.4.23.8 TKN, mg/l.

2.35.7.4.23.9 NO_2, mg/l.

2.35.7.4.23.10 NO_3, mg/l.

2.35.7.4.23.11 Oil and grease, mg/l.

2.35.7.5 Quantities Calculations.

2.35.7.5.1 Distribution pumping. User must input the operating schedule, days per week (DPW) and hours per day (HPD) of operation.

2.35.7.5.1.1 Calculate the design flow.

$$\text{FLOW} = \frac{(Q_{avg})\ (\text{WWGP})\ (24)}{(\text{FAP})\ (\text{DPW})\ (\text{HPD})}$$

where

FLOW = actual daily flow to spray field, mgd.

Q = average daily flow, mgd.

WWGP = wastewater generation period, days/yr.

FAP = field application period, wks/yr.

DPW = days per week treatment system is operated, days/wk.

HPD = hours per day treatment system is operated, hrs/day.

24 = conversion from days to hours, hrs/day.

Using the flow calculated (FLOW), the distribution pumping will be sized and the cost estimated from the existing section entitled "Intemediate Pumping".

2.35.7.5.2 Storage requirements. The slow rate system, like overland flow, is dependent upon weather. Also if crops are grown it is dependent upon growing seasons. The user must input the number of days of storage required based on anticipated crops, and climatic data for the particular area.

2.35.7.5.2.1 Calculate storage volume.

$$SV = (SR)\ (Q \times 10^6)$$

where

SV = storage volume, gal.

SR = storage required, days/yr.

Q = average daily flow, mgd.

2.35.7.5.2.2 Calculate size and number of storage lagoons.

2.35.7.5.2.2.1 The following assumptions are made in determining size and number of lagoons:

A minimum of 2 lagoon cells will always be used.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
The largest single lagoon cell will be 40 acres which represents approximately 85 million gallons storage volume.

2.35.7.5.2.2.2 If SV $\leq$ 170,000,000 gal.

$$NLC = 2$$

2.35.7.5.2.2.3 If $SV > 170,000,000$ gal. a trial and error solution for NLC will be used.

Assume NLC = 4; if $\frac{SV}{NLC} > 85,000,000$ gal.

Redesignate NLC = NLC + 2 and repeat calculation until $\frac{SV}{NLC} \leq 85,000,000$ gal.

where

SV = storage volume, gal.

NLC = number of lagoon cells.

2.35.7.5.2.3 Calculate storage volume per cell.

$$SVC = \frac{SV}{(NLC)\ (7.48)}$$

where

SVC = storage volume per cell, ft^3.

SV = storage volume, gal.

NLC = number of lagoon cells.

7.48 = conversion from gal. to ft^3, gal/ft^3.

2.35.7.5.2.4 Calculate lagoon cell dimensions. The following assumptions are made concerning lagoon construction:

The lagoon cells will be square.
Common levee construction will be used where possible.
Lagoons will be constructed using equal cut and fill.
Lagoon depth will be 10 ft with 8 ft water depth and 2 ft freeboard.
Minimum water depth will be 1.5 ft.
Side slopes will be 3 to 1.
A 30% shrinkage factor will be used for fill.

$$L = \frac{(0.615\ SVC - 1521)^{0.5} + 60}{2}$$

where

L = length of one side of lagoon cell, ft.

SVC = storage volume per cell, ft^3.

2.35.7.5.2.5 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = 10$$

$$VF = [\ 3\ (DF)^2 + 10DF\ ]\ [\ \frac{5NLC}{2} + 2]\ \ (L)$$

$$VC = (1.3)(NLC)(DC)\ [\ L^2 - (6)(DF)(L) + 12\ (DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If $VC < VF$ then assume $DC > 1$ ft and recalculate VC and VF.

If $VC > VF$ then assume $DC < 1$ ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the storage lagoon.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

2.35.7.5.3 Slow rate distribution system. In a slow rate land treatment system the wastewater is usually applied to the field in one of two ways, buried solid set sprinklers or center pivot sprinkler systems. Both of these distribution methods will be addressed.

2.35.7.5.3.1 Buried solid set sprinklers. The selection of the optimum sprinkler type, size, spacing is very dependent on the site conditions. Certain assumptions will be made on these parameters to simplify the calculations. While these assumptions, if used to design some systems, would drastically affect perfomance, they will have little affect on the overall costs.

Assume:

The treatment area will be square.
The spacing between laterals will be 50 ft.
The spacing between sprinklers will be 50 ft.
Sprinklers will be arranged in square patterns.

2.35.7.5.3.1.2 Calculate dimensions of treatment area.

$$LTA = [(TA)\ (43,560)]^{0.5}$$

where

LTA = length of one side of treatment area, ft.

TA = treatment area, acres.

43,560 = conversion from acres to ft^2, ft^2/acre.

2.35.7.5.3.1.3 Calculate flow per sprinkler.

$$FPS = \frac{2,500\ (MAR)}{96.3}$$

where

FPS = flow per sprinkler, gpm.

MAR = Maximum application rate, inches/hr. (Must be input by user based on crop and infiltration rate)

2500 = application area for each sprinkler, ft^2.

96.3 = combined conversion factors.

2.35.7.5.3.1.4 Calculate number of headers. This will be a trial and error process. The governing assumption will be that the header pipes will be less than 48"Ø. Assume NH = 1.

2.35.7.5.3.1.4.1 Calculate the length of laterals.

$$LL = \frac{LTA}{2NH}$$

where

LL = length of laterals, ft.

LTA = length of one side of treatment area, ft.

NH = number of headers.

2.35.7.5.3.1.4.2 Calculate number of sprinklers per lateral.

$$NSL = \frac{LL}{50}$$

where

NL = number of laterals per header.

LTA = length of one side of treatment area.

2.35.7.5.3.1.4.3 Calculate number of laterals per header.

$$NL = \frac{LTA}{50}$$

where

NL = number of laterals per header.

LTA = length of one side of treatment area, ft.

50 = spacing of laterals on header, ft.

2.35.7.5.3.1.4.4 Calculate number of sprinklers per header.

$$NSH = (NL)\ (NSL)$$

where

NSH = number of sprinklers per header.

NL = number of laterals per header.

NSL = number of sprinklers per lateral.

2.35.7.5.3.1.4.5 Calculate flow per header.

$$FPH = (FPS)\ (NSH)$$

If FPH $> 16{,}000$ gpm; assume NH = NH + 1 and recalculate FPH until FPH $\leq 16{,}000$ gpm.

where

FPH = flow per header, gpm.

FPS = flow per sprinkler, gpm.

NSH = number of sprinkler per header.

Adjust

$$FPH = \frac{FLOW \times 10^6}{(NH)(1440}$$

2.35.7.5.3.1.5 Calculate pipe size for laterals.

$$DIAL = 0.286\ [(NSL)\ (FPS)]^{0.5}$$

DIAL must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48. Always use the next larger diameter above the calculated diameter.

where

DIAL = diameter of lateral, inches.

NSL = number of sprinklers per lateral.

FPS = flow per sprinkler, gpm.

0.286 = combined conversion factors.

2.35.7.5.3.1.6 Calculate quantity of lateral pipe required.

$$TLL = (LL)\ (NLH)\ (NH)$$

where

TLL = total length of lateral pipe, ft.

LL = length of laterals, ft.

NLH = number of laterals per header.

NH = number of headers.

2.35.7.5.3.1.7 Calculate pipe sizes for headers. The header pipe nomally decreases in size due to decreasing volume of flow as each lateral pipe removes part of the flow from the header pipe. The header size will be calculated after each lateral on the header.

$$DIAHN = 0.286\ [FPH - (N)\ (FPL)]^{0.5}$$

Begin calculation with N = 0. This will give the diameter of the header (DIAHO) before any flow is removed. Then set N = N + 1 and repeat the calculation. This will give the diameter (DIAH 1) of the header after the first lateral has removed a part of the flow. Repeat the calculation each time redesignating N until N = NLH.

DIAHN must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

where

DIAHN = diameter of header pipe, inches.

FPH = flow per header, gpm.

N = number of laterals.

FPL = flow per lateral, gpm.

0.286 = combined conversion factors.

2.35.7.5.3.1.8 Calculate length of header pipe.

$$LDIAHN = (50)\ (SUM)\ (NH)$$

where

LDIAHN = length of header pipe of diameter DIAHN, ft.

SUM = the number of points with the same diameter.

50 = spacing between laterals, ft.

2.35.7.5.3.1.9 Calculate number of butterfly valves for distribution system. There will be a butterfly valve in each header for flow control. These valves will be in the header upstream from the spray field and will be the same as the initial size calculated for the header.

$$NBV = NH$$

$$DBV = DIAHN$$

where

NBV = number of butterfly valves.

NH = number of headers.

DBV = diameter of butterfly valves, inches.

DIAHN = diameter of header calculated when N = 0, inches.

2.35.7.5.3.1.10 Calculate number of valves for lateral lines. There will be a plug valve in each lateral line which will be automatic but will be either fully open or fully closed. They will be the same size as the size calculated for the lateral pipes.

$$NLV = (NLH)\ (NH)$$

$$DLV = DIAL$$

where

NLV = number of lateral valves.

NLH = number of laterals per header.

NH = number of headers.

DLV = diameter of lateral valves, inches.

DIAL = diameter of lateral pipes, inches.

2.35.7.5.3.1.11 Calculate number of sprinklers.

$$NS = (NSL)\ (NLH)\ (NH)$$

where

NS = number of sprinklers

NSL = number of sprinklers per lateral.

NLH = number of laterals per header.

NH = number of headers.

2.35.7.5.3.2 Center pivot system.

2.35.7.5.3.2.1 Determine size and number of center pivot systems. Center pivot systems are available in sizes which cover from 2 to 450 acres. Because of weight and structural consideration, the largest pipe available in the system is 8 inches. For this reason, hydraulics sometimes control the sizing rather than area of coverage.

2.35.7.5.3.2.1.1 The following assumptions will be made:

The systems will operate 24 hours per day, 7 days per week.
A minimum of 2 units will be used.
Ten percent of the treatment area will not be irrigated because of the circular configuration.

$$SCP = \frac{(TA)\ (1.1)}{N}$$

Begin with N = 2 and if SCP > 450 redesignate N = N + 1 and recalculate.

2.35.7.5.3.2.1.2 Because of hydraulic considerations check system velocity.

$$V = (.017)\ (AR)\ (SCP)$$

If V > 10 fps redesignate N = N + 1 and recalculate SCP and V. When V ≤ 10 fps use the calculated SCP and N.

where

SCP = size of center pivot system, acres.

TA = treatment area, acres.

N = number of center pivot systems.

V = velocity in system, ft/sec.

AR = application rate, in/hr.

2.89 = combined constants and conversion factors.

2.35.7.5.3.2.2 Determine size of header pipe. Assume each center pivot system takes flow from header consecutively.

$$CDIAHN = 0.832 [(N) (AR) (SCP)]^{0.5}$$

This gives the header size to the first unit. DIAHN must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48 inches. Always use next higher pipe size.

Set N = N - 1 and repeat the calculation. This gives the diameter of the header pipe between the first and second unit.

Redesignate N after each calculation until N = 0.

where

CDIAHN = diameter of segments of header, inches.

N = number of center pivot systems.

AR = application rate, in/hr.

SCP = size of center pivot system, acres.

2.35.7.5.3.2.3 Determine length of segments of header pipe.

$$LCDIAN = 235.5 (SCP)^{0.5}$$

Each segment of header pipe is approximately the same length and is essentially equal to the diameter of the center pivot system.

where

LCDIAN = length of segment of header pipe of diameter CDIAHN, ft.

SCP = size of center pivot system, acres.

235.5 = combined constants and conversion factors.

2.35.7.5.4 Underdrain system for groundwater control. Practical drainage systems for wastewater applications will be at depths of 4 to 8 ft and spaced 200 ft apart. The following assumptions will be made concerning the drainage system.

6" diameter perforated PVC pipe will be used.
Spacing will be 200 ft.
Depth of burial will be 4 to 8 ft.

$$LDP = [\frac{LTA}{200} + 2] \quad LTA$$

where

LDP = length of drain pipe, ft.

LTA = length of one side of treatment area, ft.

2.35.7.5.5 Land preparation. Unlike overland flow there is very little land forming required for slow rate. The land will, however, require clearing and grubbing.

For clearing and grubbing the areas will be classified in three categories: heavy, medium and light. Heavy refers to wooded areas with mature trees. Medium refers to spotted mature trees with numerous small trees and bushes. Light refers to only small trees and bushes. The user must specify the type of clearing and grubbing required as well as the percent of the treatment area requiring clearing and grubbing.

$$CAGH = \frac{PCAGH}{100} (TA)$$

$$CAGM = \frac{PCAGM}{100} (TA)$$

$$CAGL = \frac{PCAGL}{100} (TA)$$

where

CAGH = area which requires heavy clearing, acres.

PCAGH = percentage of treatment area requiring heavy clearing, %.

CAGM = area which requires medium clearing, acres.

PCAGM = percentage of treatment area requiring medium clearing, %.

CAGL = area which requires light clearing, acres.

PCAGL = percentage of treatment area requiring light clearing, %.

TA = treatment area, acres.

2.35.7.5.6 Determine total land requirement. The land requirement will be different depending on which application method is used center pivot, or fixed sprinklers.

2.35.7.5.6.1 Total treatment area.

2.35.7.5.6.1.1 Center pivot. The actual treatment area for the center pivot system will be increased by 10% because there will be unwetted areas due to the circular configuration.

$$TTA = (TA)\ (1.1)$$

where

TTA = total treatment area, acres.

TA = treatment area, acres.

1.1 = factor for unwetted area.

2.35.7.5.6.1.2 Buried solid set sprinklers. The treatment area will be increased by approximately 5% for service roads.

$$TTA = (1.05)\ (TA)$$

where

TTA = total treatment area, acres.

TA = treatment area, acres.

1.05 = factor for service roads.

2.35.7.5.6.2 Area for storage lagoons.

$$ASL = \frac{(1.2)\ (NLC)\ (L)^2}{43{,}560}$$

where

ASL = area for storage lagoons, acres.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

1.2 = additional area required for cross levee.

2.35.7.5.6.3 Area for buffer zone.

Assume:

Buffer zone will be around entire treatment area.
Facility will be square.

$$ABZ = \frac{4\ WBZ\ [\ (43{,}560\ TTA)^{0.5} + WBZ\ ]}{43{,}560}$$

where

ABZ = area required for buffer zone, acres.

WBZ = width of buffer zone (must be input by user), ft.

TTA = total treatment area, acres.

2.35.7.5.6.4 Total land area

$$TLA = TTA + ASL + ABZ$$

where

TLA = total land area required, acres.

TTA = total treatment area, acres.

ASL = area required for storage lagoons, acres.

ABZ = area required for buffer zone, acres.

2.35.7.5.7 Calculate fencing required.

$$LF = 834.8\ (TLA)^{0.5}$$

where

LF = length of fence required, ft.

TLA = total land area required, acres.

834.8 = combined conversion factors and constants.

2.35.7.5.8 Calculate operation and maintenance manpower.

2.35.7.5.8.1 Distribution system.

2.35.7.5.8.1.1 Solid set sprinklers.

2.35.7.5.8.1.1.1 If $TA \leq 60$;

$$OMMHD = 158.32\ (TA)^{0.4217}$$

2.35.7.5.8.1.1.2 If $TA > 60$;

$$OMMHD = 26.73\ (TA)^{0.8561}$$

2.35.7.5.8.1.2 Center pivot.

2.35.7.5.8.1.2.1 If $TA \leq 100$;

$$OMMHD = 209.86\ (TA)^{0.4467}$$

2.35.7.5.8.1.2.2 If $TA > 100$;

$$OMMHD = 32.77\ (TA)^{0.8481}$$

where

TA = treatment area, acres.

OMMHD = O&M manpower for distribution system, MH/yr.

2.35.7.5.8.2 Underdrain system.

2.35.7.5.8.2.1 If $TA \leq 80$;

$$OMMHU = 54.71\ (TA)^{0.2414}$$

2.35.7.5.8.2.2 If $TA > 80$;

$$OMMHU = 10.12\ (TA)^{0.6255}$$

where

TA = treatment area, acres.

OMMHU = O&M manpower for underdrain system, MH/yr.

2.35.7.5.8.3 Monitoring wells.

$$OMMHM = 6.39 (NMW) (DMW)^{0.2760}$$

where

OMMHM = operation and maintenance manpower for monitoring wells, MH/yr.

NMW = number of monitoring wells.

DMW = depth of monitoring wells, ft.

2.35.7.5.9 Calculate O&M material costs.

2.35.7.5.9.1 Distribution system.

2.35.7.5.9.1.1 Solid set sprinklers.

$$OMMPD = 0.906\ (TA)^{-0.0860}$$

2.35.7.5.9.1.2 Center pivot.

2.35.7.5.9.1.2.1 If $TA \leq 175$;

$$OMMPD = 1.06\ (TA)^{0.0696}$$

2.35.7.5.9.1.2.2 If $TA > 175$;

$$OMMPD = 2.92\ (TA)^{-0.1261}$$

where

TA = treatment area, acres.

OMMPD = O&M material cost as percent of construction cost of distribution system, %.

2.35.7.5.9.2 Monitoring wells.

$$OMMPM = 2.28\ (DMW)^{0.0497}$$

where

OMMPM = O&M material cost as percent of construction cost of monitoring wells.

DMW = depth of monitoring wells, ft.

2.35.7.5.9.3 Underdrain system.

2.35.7.5.9.3.1 If $TA \leq 200$;

$$OMMPU = 14.13\ (TA)^{-0.1392}$$

2.35.7.5.9.3.2 If $TA > 200$;

$$OMMPU = 30.95\ (TA)^{-0.2860}$$

where

OMMPU = O&M material cost as percent of construction cost of underdrain system, %.

TA = treatment area, acres.

2.35.7.5.10 Other construction cost items. The quantities computed account for approximately 85% of the construction cost of the systems. Other miscellaneous costs such as connecting piping for lagoons, lagoon influent and effluent structures, miscellaneous concrete structures, etc., make up the additional 15%.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other construction costs.

2.35.7.6 Quantities Calculations Output Data.

2.35.7.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

2.35.7.6.2 Diameter of lateral pipes, DIAL, inches.

2.35.7.6.3 Total length of lateral pipe, TLL, ft.

2.35.7.6.4 Diameters of header pipes, DIAHN, inches.

2.35.7.6.5 Length of header pipes of diameters DIAHN, LDIAHN, ft.

2.35.7.6.6 Number of butterfly valves, NBV.

2.35.7.6.7 Diameter of butterfly valves, DBV, inches.

2.35.7.6.8 Number of lateral valves, NLV.

2.35.7.6.9 Diameter of lateral valves, DLV, inches.

2.35.7.6.10 Number of sprinklers, NS.

2.35.7.6.11 Number of center pivot systems, N.

2.35.7.6.12 Size of center pivot system, SCP, acres.

2.35.7.6.13 Diameter of segments of header pipe for center pivot, CDIAHN, inches.

2.35.7.6.14 Length of segment of header pipe of diameter CDIAHN for center pivot, LCDIAN, ft.

2.35.7.6.15 Length of drain pipe, LDP, ft.

2.35.7.6.16 Area which requires heavy clearing, CAGH, acres.

2.35.7.6.17 Area which requires medium clearing, CAGM, acres.

2.35.7.6.18 Area which requires light clearing, CAGL, acres.

2.35.7.6.19 Total land area required, TLA, acres.

2.35.7.6.20 Length of fencing required, LF, ft.

2.35.7.6.21 O&M manpower for distribution system, OMMHD, MH/yr.

2.35.7.6.22 O&M manpower for underdrain system, OMMHU, MH/yr.

2.35.7.6.23 O&M material costs as percent of construction cost of distribution system, OMMPD, %.

2.35.7.6.24 O&M material costs as percent of construction cost of underdrain system, OMMPU, %.

2.35.7.6.25 Correction factor for other construction costs, CF.

2.35.7.7 Unit Price Input Required.

2.35.7.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

2.35.7.7.2 Cost of standard size pipe (12"Ø), COSP, $/ft.

2.35.7.7.3 Cost of standard size valve (12"Ø butterfly), COSTSV, $.

2.35.7.7.4 Cost per sprinkler, COSTEN, $.

2.35.7.7.5 Unit price input for 6" PVC perforated drain pipe, UPIPP, $/ft.

2.35.7.7.6 Unit price input for heavy clearing and grubbing, UPICG, $/acre.

2.35.7.7.7 Unit price input for fence, UPIF, $/ft.

2.35.7.8 Cost Calculations.

2.35.7.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW}{27} \, UPIEX$$

where

COSTE = cost of earthwork, $.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, $/cu yd.

2.35.7.8.2 Cost of distribution system for solid set sprinklers.

2.35.7.8.2.1 Cost of header pipes.

2.35.7.8.2.1.1 Calculate total installed cost of header pipe.

$$TICHP = \sum ICHPN$$

where

TICHP = total installed cost of header pipes, $.

ICHPN = installed cost of various size header pipes, $.

2.35.7.8.2.1.2 Calculate installed cost of each size header pipe.

$$ICHPN = (LDIAHN) \frac{COSTPN}{100} (COSP)$$

where

ICHPN = installed cost of various size header pipes, \$.

LDIAHN = length of header pipes of size DIAHN, ft.

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

2.35.7.8.2.1.3 Calculate COSTPN.

$$COSTPN = 6.842\ (DIAHN)^{1.2255}$$

where

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

DIAHN = diameters of header pipes, inches.

2.35.7.8.2.1.4 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot 4th quarter, 1977.

2.35.7.8.2.2 Cost of lateral pipes.

2.35.7.8.2.2.1 Calculate total installed cost of lateral pipe.

$$TICLP = (TLL) \frac{(COSTP)}{100} (COSP)$$

where

TICLP = total installed cost of lateral pipe, \$.

TLL = total length of lateral pipe.

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

2.35.7.8.2.2.2 Calculate COSTP.

$$COSTP = 6.842\ (DIAL)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

DIAL = diameter of lateral pipes, inches.

2.35.7.8.2.2.3 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is $13.50 per foot in 4th quarter, 1977.

2.35.7.8.2.3 Calculate cost of butterfly valves.

2.35.7.8.2.3.1 Calculate installed cost of butterfly valves.

$$COSTBV = \frac{(COSTRV)\ (COSTSV)\ (NBV)}{100}$$

where

COSTBV = installed cost of butterfly valves, $.

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

COSTSV = cost ot standard size valve, $.

NBV = number of butterfly valves.

2.35.7.8.2.3.2 Calculate COSTRV.

$$COSTRV = 3.99(DBV)^{1.395}$$

where

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

DBV = diameter of butterfly valves, inches.

2.35.7.8.2.3.3 Determine COSTSV. COSTSV is the cost of a 12" ϕ butterfly valve suitable for water service. This cost is $1,004 for 4th quarter, 1977.

2.35.7.8.2.4 Calculate cost of lateral valves.

2.35.7.8.2.4.1 Calculate installed cost of lateral valves.

$$COSTLV = \frac{(COSTRL)\ (COSTSV)\ (NLV)}{100}$$

where

COSTLV = installed cost of lateral valves, $.

COSTRL = cost of lateral valve of size DLV as percent of cost of standard size valve, %.

COSTSV = cost of standard size valve (12" ϕ butterfly), $.

NLV = number of lateral valves.

2.35.7.8.2.4.2 Calculate COSTRL.

$$COSTRL = 15.33\ (DLV)^{1.053}$$

where

COSTRL = cost of lateral valve of size DLV as percent of cost of standard size valve, %.

DLV = diameter of lateral valves, inches.

2.35.7.8.2.4.3 Determine COSTSV. COSTSV is the cost of a 12" ∅ butterfly valve suitable for water service. This cost is $1004 for the 4th quarter of 1977.

2.35.7.8.2.5 Calculate cost of sprinklers.

2.35.7.8.2.5.1 Calculate installed cost of sprinklers.

$$COSTS = 1.2\ (NS)\ COSTEN$$

COSTS = installed cost of sprinklers, $.

NS = number of sprinklers.

COSTEN = cost per sprinkler, $.

1.2 = 20% for cost of installation.

2.35.7.8.2.5.2 Determine COSTEN. COSTEN is the cost of an *impact type rotary pop-up*, full circle sprinkler with a flow from 6 to 15 gpm. This cost is $65.00 for the 4th quarter of 1977.

$$COSTEN = \$65.00$$

For better cost estimation, COSTEN should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTEN = \$65.00\ \frac{MSECI}{518.4}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

518.4 = Marshall and Swift Equipment Cost Index 4th quarter of 1977.

2.35.7.8.2.6 Calculate total cost of distribution system for solid set sprinklers.

$$TCDSS = TICHP + TICLP + COSTBV + COSTLV + COSTS$$

where

TCDSS = total cost of distribution system, \$.

TICHP = total installed cost of header pipes, \$.

TICLP = total installed cost of lateral pipes, \$.

COSTBV = installed cost of butterfly valves, \$.

COSTLV = installed cost of lateral valves, \$.

COSTS = installed cost of sprinklers, \$.

2.35.7.8.3 Cost of distribution system for center pivot system.

2.35.7.8.3.1 Cost of center pivot systems.

2.35.7.8.3.1.1 Calculate cost of center pivot systems.

$$COSTCP = \frac{(N)\ (COSTRC)\ (COCP)}{100}$$

where

COSTCP = total cost of center pivot systems, \$.

N = number of center pivot systems required.

COSTRC = cost of center pivot system of size SCP as percent of standard size system, %.

COCP = cost of standard size system (200 acres), \$.

2.35.7.8.3.1.2 Calculate COSTRC.

$$COSTRC = 12.25\ (SCP)^{0.4559}$$

where

COSTRC = cost of center pivot system of size SCP as percent of standard size system, %.

SCP = size of center pivot system, acres.

2.35.7.8.3.1.3 Determine COCP. COCP is the cost of a center pivot sprinkler system capable of irrigating 200 acres. The cost is \$29,200 for the 4th quarter of 1977.

$$COCP = \$29{,}200$$

For better cost estimation, COCP should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTCP = \$29,200 \frac{MSECI}{518.4}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

518.4 = Marshall and Swift Cost Index 4th quarter of 1977.

2.35.7.8.3.2 Cost of header pipe for center pivot.

2.35.7.8.3.2.1 Total cost header pipe.

$$TCHPC = \sum CHPCN$$

where

TCHPC = total cost header pipe for center pivot, \$.

CHPCN = cost of various size header pipe for center pivot, \$.

2.35.7.8.3.2.2 Calculate cost of each size header pipe.

$$CHPCN = \frac{(LCDIAN)\ (COSTPN)\ (COSP)}{100}$$

where

CHPCN = installed cost of various size header pipe, \$.

LCDIAN = length of header pipes of diameter CDIAHN, ft.

COSTPN = cost of pipe of diameter CDIAHN as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

2.35.7.8.3.2.3 Calculate COSTPN.

$$COSTPN = 6.842\ (CDIAHN)^{1.2255}$$

where

COSTPN = cost of pipe of diameter CDIAHN as percent of cost of standard size pipe, %.

CDIAHN = diameter of segments of header pipe, inches.

2.35.7.8.3.2.4 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot in 4th quarter of 1977.

2.35.7.8.3.3 Calculate total cost of distribution system for center system.

$$TCDCP = COSTCP + TCHPC$$

where

TCDCP = total cost of distribution system for center pivot system, $.

COSTCP = cost of center pivot systems, $.

TCHPC = total cost of header pipe for center pivot, $.

2.35.7.8.4 Cost of underdrain system.

$$COSTU = (LDP)(UPIPP)(1.1)$$

where

COSTU = cost of underdrain system, $.

LDP = length of drain pipe, ft.

UPIPP = unit price input for 6" PVC perforated drain pipe, $/ft.

1.1 = 10% adjustment for trenching and backfilling.

2.35.7.8.5 Calculate cost of clearing and grubbing.

$$COSTCG = (CAGH + 0.306\ CAGM + 0.092\ CAGL)\ UPICG$$

where

COSTCG = cost for clearing and grubbing site, $.

CAGH = area which requires heavy clearing, acres.

CAGM = area which requires medium clearing, acres.

CAGH = area which requires light clearing, acres.

UPICG = unit price input for heavy clearing and grubbing $/acres.

2.35.7.8.6 Calculate cost of fencing.

$$COSTF = (LF)\ (UPIF)$$

where

COSTF = installed cost of fencing, $.

LF = length of fencing required, ft.

UPIF = unit price input for fencing, \$/ft.

2.35.7.8.7 Cost of monitoring wells and pumps.

2.35.7.8.7.1 Calculate cost of wells.

2.35.7.8.7.1.1 Calculate COSTM.

$$COSTM = (RMWC)(DMW)(NMW)(COSP)$$

where

COSTM = cost of monitoring wells, \$.

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

NMW = number of monitoring wells.

COSP = cost of standard size pipe (12" welded steel), \$/ft.

2.35.7.8.7.1.2 Calculate RMWC.

$$RMWC = 160.4\ (DMW)^{-0.7033}$$

where

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

2.35.7.8.7.1.3 Determine COSP.

COSP is the cost per foot of 12"Ø welded steel pipe. This cost in first quarter of 1977 is \$13.50/ft. For the best cost estimate COSTSP should be a current price input from a vendor, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSP = \$13.50\ \frac{MSECI}{491.6}$$

where

COSP = cost of standard size pipe (12"Ø welded steel), \$/ft.

MSECI = current Marshall and Swift Equipment Cost Index.

2.35.7.8.7.2 Calculate cost of pumps for monitoring wells.

2.35.7.8.7.2.1 Calculate COSTMP.

$$COSTMP = (COSTPS)(MWPR)(NMW)$$

where

COSTMP = cost of pumps for monitoring wells, $.

COSTPS = cost of standard size pump (3000 gpm), $.

MWPR = cost of well pump as fraction of cost of standard pump.

NMW = number of monitoring wells.

2.35.7.8.7.2.2 Calculate MWPR.

$$MWPR = 0.0551\ (DMW)^{0.658}$$

where

MWPR = cost of well pump as fraction of cost of standard size pump.

DMW = depth of monitoring wells, ft.

2.35.7.8.7.2.3 Determine COSTPS.

COSTPS is the cost of a 3000 gpm pump. The cost is $17,250 for the first quarter of 1977. For the best cost estimate the user should input a current value of COSTPS, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250\ \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump (3000 gpm), $.

MSECI = current Marshall and Swift Equipment Cost Index.

2.35.7.8.7.3 Calculate total cost of monitoring wells.

$$COSTMW = COSTM + COSTMP$$

where

COSTMW = total cost of monitoring wells, $.

COSTM = cost of monitoring wells, \$.

COSTMP = cost of pumps for monitoring wells, \$.

2.35.7.8.8 Calculate O&M material costs.

$$OMMC = \frac{(OMMPD)(TCDSS) + (OMMHD)(TCDCP) + OMMPU(COSTU) + OMMPM(COSTMW)}{100}$$

where

OMMC = total O&M material cost, \$.

OMMPD = O&M material cost for distribution system as percent of construction cost for distribution system, \$.

TCDSS = total cost of distribution system for solid set sprinklers, \$.

OMMPM = O&M material cost for monitoring well as percent of construction cost of monitoring well, %.

COSTMW = total cost of monitoring wells, \$.

TCDCP = total cost of distribution system for center pivot, \$.

OMMPU = O&M material cost for underdrain system as percent of construction cost of underdrain system, \$.

COSTU = cost of underdrain system, \$.

2.35.7.8.9 Total bare construction cost.

$$TBCCSR = (1.18)(TCDSS + TCDCP + COSTU + COSTE + COSTCG + COSTF + COSTMW)$$

where

TBCCSR = total bare construction cost for slow rate land treatment, \$.

TCDSS = total cost of distribution system for solid set sprinklers, \$.

TCDCP = total cost of distribution system for center pivot, \$.

COSTU = cost of underdrain system, \$.

COSTE = cost of earthwork, \$.

COSTCG = cost of clearing and grubbing, \$.

2.35.7.9 Cost Calculations Output Data.

2.35.7.9.1 Total bare construction cost for slow rate land treatment, TBCCSR, $.

2.35.7.9.2 O&M material cost, OMMC, $.

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2.36 TWO-STAGE LIME TREATMENT

2.36.1 Background.

2.36.1.1 Two-Stage treatment systems are used for phosphorus removal from wastewaters. In principle, for a two-stage system, sufficient lime is added to the wastewater in the first stage to raise the pH to above 11. At this pH precipitation of hydroxyapatite, calcium carbonate, and magnesium hydroxide takes place. Carbon dioxide is then added following the first-stage clarifier to reduce the pH to a value of 9.5-10, where calcium carbonate precipitation results. The sludge, which is mainly $CaCO_3$, is then separated in a clarifier and the pH of the wastewater is adjusted to around 7 for further treatment or final disposal. A flow diagram of a two-stage lime treatment system is presented in Figure 2.36-1.

2.36.1.2 The design of a two-stage lime treatment system will be a combination of (1) chemical coagulation-precipitation basin using lime as a coagulate; (2) a primary recarbonation stage; (3) a clarifier designed as a primary clarifier; and (4) a secondary recarbonation stage.

2.36.2 Input Data.

2.36.2.1 Chemical Coagulation.

2.36.2.1.1 Wastewater flow, mgd.

2.36.2.1.1.1 Average daily flow, mgd.

2.36.2.1.1.2 Variation in flow, mgd.

2.36.2.1.2 Wastewater characteristics.

2.36.2.1.2.1 BOD, total and soluble, mg/l.

2.36.2.1.2.2 COD, total and soluble, mg/l.

2.36.2.1.2.3 Phosphorus, mg/l.

2.36.2.1.2.4 Suspended solids, mg/l.

2.36.2.1.2.5 pH.

2.36.2.1.2.6 Alkalinity, mg/l.

2.36.2.2 Primary Clarifier.

2.36.2.2.1 Wastewater flow.

2.36.2.2.1.1 Average flow, mgd.

2.36.2.2.1.2 Peak flow, mgd.

2.36.2.2.2 Wastewater characteristics.

2.36.2.2.2.1 Suspended solids, mg/l.

2.36.2.2.2.2 Volatile suspended solids, percent.

2.36.2.3 Recarbonation.

2.36.2.3.1 Wastewater flow.

2.36.2.3.1.1 Average flow.

2.36.2.3.1.2 Peak flow.

2.36.2.3.2 Wastewater characteristics.

2.36.2.3.2.1 Alkalinity.

2.36.2.3.2.2 Hydroxide alkalinity, mg/l as $CaCO_3$.

2.36.2.3.2.3 Carbonate alkalinity, mg/l as $CaCO_3$.

2.36.2.3.3 pH.

2.36.2.4 Primary Clarifier.

2.36.2.4.1 Wastewater flow.

2.36.2.4.1.1 Average flow, mgd.

2.36.2.4.1.2 Peak hourly flow, mgd.

2.36.2.4.2 Suspended solids, mg/l.

2.36.2.4.3 Volatile suspended solids, percent.

2.36.3 Design Parameters

2.36.3.1 Chemical Coagulation.

2.36.3.1.1 Desired quality of treated effluent, mg/l.

2.36.3.1.2 Coagulant dosage, mg/l (jar test).

2.36.3.1.3 Detention time of rapid mix basin (1-3 min).

2.36.3.1.4 Detention time of flocculator basin (15-60 min).

2.36.3.2 Primary Clarifier.

2.36.3.2.1 Overflow rate, gpd/ft^2. (Tables 2.36-1 and 2.36-2).

2.36.3.2.2 Detention time, hr. (Table 2.36-2).

2.36.3.2.3 Specific gravity of sludge (Table 2.36-3).

2.36.3.2.4 Solids content of underflow, percent (4 to 6 percent).

2.36.3.2.5 Removal efficiency of suspended solids, percent. (Figure 2.36-1).

2.36.3.2.6 Weir loading, gpd/ft (10,000 to 15,000 gpd/ft.)

2.36.3.3 Recarbonation.

2.36.3.3.1 Contact time, min (15-30).

2.36.3.3.2 Carbon dioxide dose, lb/million gal.

2.36.3.3.3 Desired effluent pH.

2.36.3.4 Primary Clarifier.

2.36.3.4.1 Overflow rate, gpd/ft^2. (Tables 2.36-1 and 2.36-2).

2.36.3.4.2 Detention time, hr. (Table 2.36-2).

2.36.3.4.3 Specific gravity of sludge. (Table 2.36-3).

2.36.3.4.4 Solids content of underflow, percent (4 to 6 percent).

2.36.3.4.5 Removal efficiency of suspended solids, percent (Figure 2.36-1).

2.36.3.4.6 Weir loading, gpd/ft (10,000 to 15,000 gpd/ft).

2.36.4 <u>Process Design Calculations</u>.

2.36.4.1 Chemical Coagulation.

2.36.4.1.1 Calculate coagulant requirements.

$$CR = (CD)(Q_{avg})(8.34)$$

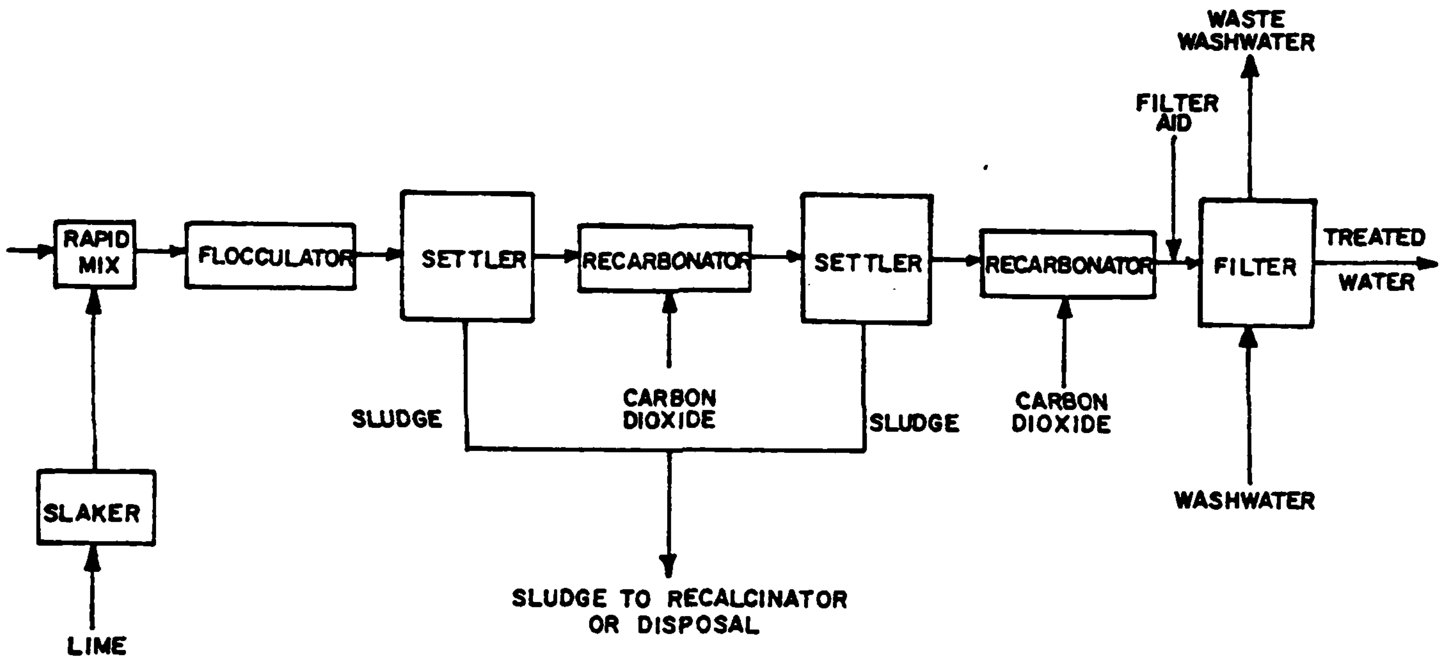

Figure 2.36-1. Two-stage lime treatment system.

where

CR = coagulant requirements, lb/day.

CD = coagulant dosage, mg/l.

Q_{avg} = average daily flow, mgd.

2.36.4.1.2 Calculate volume of flash mixing basin.

$$VFM = \frac{(Q_p)(DTFM)(10^6)}{1440}$$

where

VFM = volume of flash mix basin, gal.

Q_p = peak flow, mgd.

$DTFM$ = detention time of flash mixer, min (1-3 min).

2.36.4.1.3 Calculate voluem of flocculator basin.

$$VFL = \frac{(Q_p)(DTFL)(10^6)}{1440}$$

where

VFL = volume of flocculator basin, gal.

Q_p = peak flow, mgd.

$DTFL$ = detention time of flocculator basin, min (15-60 min).

2.36.4.2 Primary Clarifier.

2.36.4.2.1 Select an overflow rate by using Table 2.36-1 or by laboratory methods and calculate surface area:

$$SA = \frac{Q_p \times 10^6}{OFR}$$

where

SA = surface area, ft^2.

Q_p = peak flow, mgd.

OFR = overflow rate, $gal/ft^2/day$.

2.36.4.2.2 Select detention time and calculate volume (Table 2.36-2).

$$V = (Q_{avg})(t) \times \frac{1}{7.48} \times \frac{1}{24} \times 10^6$$

where

V = volume of tank, ft^3.

Q_{avg} = average flow, mgd.

t = time, hr.

2.36.4.2.3 Calculate side water depths.

$$SWD = \frac{V}{SA}$$

where

SWD = side water depth, ft.

V = volume, ft^3.

SA = surface area, ft^2.

2.36.4.2.4 Check solids loading rate:

$$SLR = \frac{(Q_{avg})(SSI)(8.34)}{(SA)}$$

where

SLR = solids loading rate, $lb/ft^2/day$.

Q_{avg} = average flow, mgd.

SSI = influent solids concentration, mg/l.

SA = surface area, ft^2.

2.36.4.2.5 Select weir loading rate and calculate weir length.

$$WL = \frac{Q_p}{WLR} \times 10^6$$

where

WL = weir length, ft.

Q_p = peak flow, mgd.

WLR = weir loading rate, gal/ft/day.

2.36.4.2.6 Detemine percentage of suspended solids removed from Figure 2.36-1.

2.36.4.2.7 Calculate amount of primary sludge produced.

$$PSP = (Q_{avg})(SSI)(SSR)(10^{-2})(8.34)$$

where

PSP = primary sludge produced, lb/day.

Q_{avg} = average flow, mgd.

SSI = influent solids concentration, mg/l (50 to 60 percent for municipal systems).

SSR = suspended solids removed, percent.

2.36.4.2.8 Select underflow concentration (3 to 6 percent) and sludge specific gravity and calculate the volume flow of primary sludge produced.

$$VPSP = \frac{PSP(100)}{(\text{specific gravity})(UC)(8.34)}$$

where

VPSP = volume flow of primary sludge produced, gal/day.

PSP = primary sludge produced, gal/day.

UC = underflow concentration (3 to 6 percent).

2.36.4.3 Recarbonation (1st stage).

2.36.4.3.1 Calculate tank volume.

$$V = \frac{Q(t)10^6}{24(60)}$$

where

V = tank volume, gal.

Q = wastewater flow, mgd.

t = contact time, min (15-30).

2.36.4.3.2 Calculate CO_2 requirement. Assume pH to be adjusted to 9.3.

$$CO_2 = (3.7)(OH^-)(Q)/0.116/1440$$

Table 2.36-1. Recommended Surface-Loading Rates for Various Suspensions

Suspension	Loading Rate, gpd/ft^2 Range	Loading Rate, gpd/ft^2 Peak Flow
Untreated Wastewater	600 to 1200	1200
Alum floc[a]	360 to 600	600
Iron floc[a]	540 to 800	800
Lime floc[a]	540 to 1200	1200

[a]Mixed with the settleable suspended solids in the untreated wastewater and colloidal or other suspended solids swept out by the floc.

Table 2.36-2. Detention Times for Various Surface-Loading Rates and Tank Depths

Surface-Loading Rate, gpd/ft^2	Detention Time, hr. 7-ft Depth	8-ft Depth	10-ft Depth	12-ft Depth
400	3.2	3.6	4.5	5.4
600	2.1	2.4	3.0	3.6
800	1.6	1.8	2.25	2.7
1000	1.25	1.4	1.8	2.2

Table 2.36-3. Specific Gravity of Raw Sludge Produced from Various Types of Sewage

Type of Sewerage System	Strength of Sewage	Specific Gravity
Sanitary	Weak	1.02
Sanitary	Medium	1.03
Combined	Medium	1.05
Combined	Strong	1.07

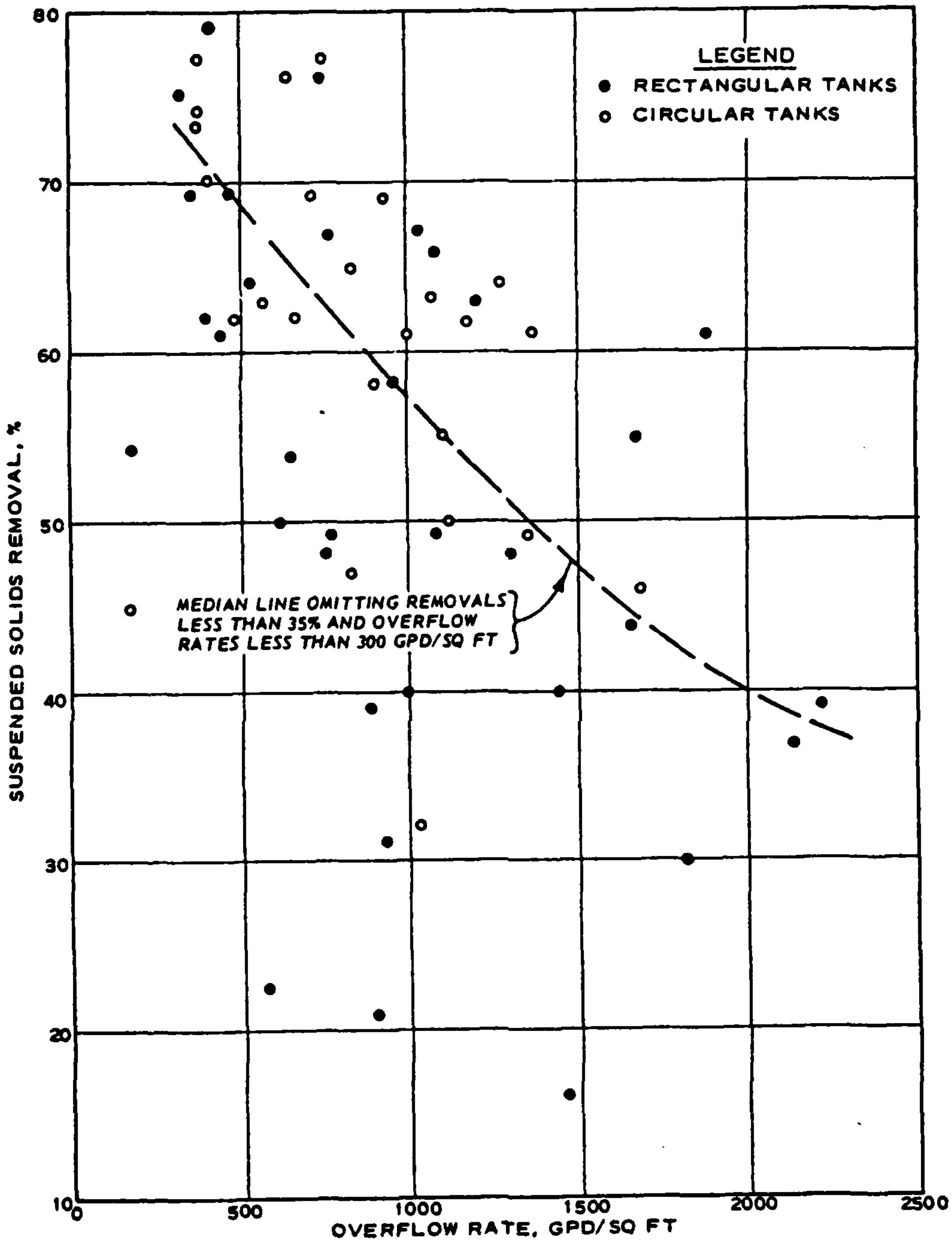

Figure 2.36-2. Suspended solids removal versus overflow rate for secondary clarifiers.

where

3.7 = stochiometric value to convert hydroxide to carbonate.

OH^- = hydroxide alkalinity, mg/l as $CaCO_3$.

Q = wastewater flow, mgd.

0.116 = density of CO_2, lb/ft^3.

1440 = min/day.

2.36.4.4 Primary Clarifier

2.36.4.4.1 Select an overflow rate by using Table 2.36-1 or by laboratory methods and calculate surface area.

$$SA = \frac{Q_p \times 10^6}{OFR}$$

where

SA = surface area, ft^2.

Q_p = peak flow, mgd.

OFR = overflow rate, $gal/ft^2/day$.

2.36.4.4.2 Select detention time and calculate volume (Table 2.36-2).

$$V = (Q_{avg})(t) \times \frac{1}{7.48} \times \frac{1}{24} \times 10^6$$

where

V = volume of tank, ft^3.

Q_{avg} = average flow, mgd.

t = time, hr.

2.36.4.4.3 Calculate side water depth.

$$SWD = \frac{V}{SA}$$

where

SWD = side water depth, ft.

V = volume, ft^3.

SA = surface area, ft^2.

2.36.4.4.4 Check solid loading rate.

$$SLR = \frac{(Q_{avg})(SSI)(8.34)}{(SA)}$$

where

SLR = solid loading rate, lb/ft^2/day.

Q_{avg} = average flow, mgd.

SSI = influent solids concentration, mg/l.

SA = surface area, ft^2.

2.36.4.4.5 Select weir loading rate and calculate weir length.

$$WL = \frac{Q_p}{WLR} \times 10^6$$

where

WL = weir length, ft.

Q_p = peak flow, mgd.

WLR = weir loading rate, gal/ft/day.

2.36.4.4.6 Determine percentage of suspended solids removed from Figure 2.36-1.

2.36.4.4.7 Calculate amount of primary sludge produced.

$$PSP = (Q_{avg})(SSI)SSR)(10^{-2})(8.34)$$

where

PSP = primary sludge produced, lb/day.

Q_{avg} = average flow, mgd.

SSI = influent solids concentration, mg/l (50 to 60 percent for municipal systems).

SSR = suspended solids removed, percent.

2.36.4.4.8 Select underflow concentration (3 to 6 percent) and sludge specific gravity (Table 2.36-3), and calculate the volume flow of primary sludge produced.

$$VPSP = \frac{PSP(100)}{(\text{specific gravity})(UC)(8.34)}$$

where

VPSP = volume flow or primary sludge produced, gal/day.

PSP = primary sludge produced, gal/day.

UC = underflow concentration (3 to 6 percent).

2.36.4.5 Recarbonation (2nd stage).

2.36.4.5.1 Calculate tank volume.

$$V = \frac{Q(t)10^6}{24(60)}$$

where

V = volume, gal.

Q = wastewater flow, mgd.

t = contact time, min (15-30 min).

2.36.4.5.2 Calculate CO_2 requirement. Assume pH to be adjusted to 7.0.

$$CO_2 = 3.7(CO_3^= + OH^-)(Q)/0.116/1440$$

where

3.7 = stochiometric value to convert hydroxide to carbonate.

OH^- = hydroxide alkalinity, mg/l as $CaCO_3$.

Q = wastewater flow, mgd.

0.116 = density of CO_2, lb/ft^3.

1440 = min/day.

2.36.5 <u>Process Design Output Data</u>.

2.36.5.1 Chemical Coagulation.

2.36.5.1.1 Coagulant dosage, mg/l.

2.36.5.1.2 Optimum pH.

2.36.5.1.3 Rapid mix detention time, min.

2.36.5.1.4 Flocculator detention time, min.

2.36.5.1.5 Coagulant requirement, lb/day.

2.36.5.1.6	Volume of flash mix basin, gal.
2.36.5.1.7	Volume of flocculator basin, gal.
2.36.5.2	Clarifier.
2.36.5.2.1	Overflow rate, gal/day ft^2.
2.36.5.2.2	Surface area, ft^2.
2.36.5.2.3	Side water depth, ft.
2.36.5.2.4	Detention time, hr.
2.36.5.2.5	Solid loading, lb/ft^2/day.
2.36.5.2.6	Weir loading, gal/ft/day.
2.36.5.2.7	Weir length, ft.
2.36.5.2.8	Volume of sludge produced, gal/day.
2.36.5.2.9	Suspended solids removal, percent.
2.36.5.3	Recarbonation unit, (1st stage).
2.36.5.3.1	Volume of tank, million gal.
2.36.5.3.2	Carbon dioxide requirement, cfm/mgd.
2.36.5.3.3	Final pH.
2.36.5.3.4	Contact time, min.
2.36.5.4	Clarifier.
2.36.5.4.1	Overflow rate, gal/day/ft^2.
2.36.5.4.2	Surface area, ft^2.
2.36.5.4.3	Side water depth, ft.
2.36.5.4.4	Detention time, hr.
2.36.5.4.5	Solid loading, lb/ft^2/day.
2.36.5.4.6	Weir loading, gal/ft/day.
2.36.5.4.7	Weir length, ft.
2.36.5.4.8	Volume of sludge produced, gal/day.
2.36.5.4.9	Suspended solids removal, percent.

2.36.5.5 Recarbonation unit (2nd stage).

2.36.5.5.1 Volume of tank, million gal.

2.36.5.5.2 Carbon dioxide requirement, cfm/mgd.

2.36.5.5.3 Final pH.

2.36.5.5.4 Contact time, min.

2.36.6 Quantities Calculations.

2.36.6.1 Operation and Maintenance Labor.

$$OMMH = 2,683\ (Q)^{0.46}$$

where

OMMH = annual operation and maintenance manpower, MH.

Q = average daily wastewater flow, mgd.

2.36.7 Quantities calculations output data.

2.36.7.1 Annual operation and maintenance manpower, MH.

2.36.8 Cost Calculations

2.36.8.1 Total bare construction costs.

$$TBCC = 294,457 + 30,075\ (Q)$$

where

TBCC = total bare construction costs, $.

Q = average daily wastewater flow, mgd.

2.36.8.2 O&M material and supply costs.

$$OMMC = 1,824\ (Q)^{0.65}$$

where

OMMC = O&M material and supply costs, $/yr.

Q = average daily wastewater flow, mgd.

2.36.9 Cost Calculations Output Data.

2.36.9.1 Total bare construction cost, TBCC, $.

2.36.9.2 O&M material and supply costs, $/yr.

2.36.10 Bibliography.

2.36.10.1 American Water Works Association, Water Quality and Treatment, McGraw-Hill, New York, 1971.

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2.37 MICROSCREENING

2.37.1 Background. Microscreening is a method of removing suspended solids from water utilizing a rotating drum covered with a screen having 20 to 40 micron openings. The water flows into the drum and out through the screen. As the drum rotates, backwash jets remove the collected solids from the screen. The backwash water is usually product water. The dirty backwash which is about 3 to 5 percent of the applied flow is collected and returned to the front of the treatment plant. The wastewater application rate is said to be 5 to 10 gpm/ft^2 of the submerged screen area. The rotational speed can be varied with the flow utilizing level detection equipment which will send a signal to a speed controller. The higher the flow, the faster the speed would be. Experience with wastewater indicates slime growth could be a problem. To minimize this, one manufacturer utilizes ultraviolet lights. Another method is to periodically wash the screen with a hypochlorite solution, drain the tanks and thoroughly clean the screen with a backwash. Grease accumulation has also been a problem and a hosing with hot water has been used to minimize this. Wastewater flow through the microscreen device should be by gravity since pumping will break the suspended solids into smaller sizes and hinder removal. If pumping is required, it should be on the downstream side of the unit. The backwash should be pumped with about 50 psig at the unit. The dirty backwash would be collected, transported to a wet well and pumped to the front of the plant. The largest unit available in a standard size is 10 ft. in diameter and 16 ft. long; however, vendors state that larger sizes are available which would be custom built on special order. The suspended solids removal efficiency is 50 to 90%.

2.37.2 Input Data.

2.37.2.1 Wastewater flow, average daily, mgd

2.37.2.2 Peak wastewater flow, mgd

2.37.2.3 Influent suspended solids, mg/l

2.37.2.4 Influent BOD_5, mg/l

2.37.2.5 Microscreen hydraulic loading, gpm/ft^2 (optional)

2.37.3 Design Parameters.

2.37.3.1 Microscreen hydraulic loading rate, 2.5 - 10 gpm/ft^2

2.37.3.2 Suspended solids removal efficiency, 50-80%

2.37.3.3 BOD removal efficiency, 40-70%

2.37.3.4 Wash water quantity as percent of Q_{AVG}, 3-5%

2.37.3.5 Fabric aperture, 23-35 micron

2.37.4 Process Design Calculations.

2.37.4.1 Calculate area of microscreen required

$$A_s = \frac{1000}{1.44} \frac{Qp}{Vs}$$

where

A_s = area of microscreens, ft^2

Q_p = Peak flow rate, mgd

V_s = Microscreen loading rate, gpm/ft^2, 4 gpm/ft^2 is the default value suggested

2.37.4.2 Effluent Characteristics.

2.37.4.2.1 BOD.

$$BODE = (BODI)(1 - \frac{BODR}{100})$$

$$S_e = BODS$$

If BODE < S_e, Set BODE = S_e

where

BODE = effluent BOD_5 concentration, mg/l.

BODI = influent BOD_5 concentration, mg/l.

S_e = effluent BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, %.

BODS = influent BOD_5 concentration, mg/l.

2.37.4.2.2 COD.

$$CODE = CODI\ (1 - \frac{CODR}{100})$$

where

CODE = effluent COD concentration, mg/l.

CODI = influent COD concentration, mg/l.

CODR = COD removal rate, %.

2.37.4.2.8 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNE}{1})$$

$$NH3E = NH3$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNR = Kjedahl nitrogen removal rate, %.

NH3E = effluent ammonia nitrogen concentration, mg/l.

NH3 = influent ammonia nitrogen concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

2.37.4.2.4 Suspended Solids.

$$SSE = SSI\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, mg/l.

2.37.4.2.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.37.4.2.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.37.4.3 Wash Water Pumping Rate. It is shown that approximately 3-5% of the flow processed through miscroscreen will have to be used for backwashing the screen. However, the fim pump capacity of the wash water system can be as high as 15% of the processed flow. Thus, the pump capacity is estimated to be:

$$Q_{pump} = 0.15 \times Q_{avg}.$$

where

Q_{pump} = Wash water pump capacity, mgd

2.37.5 <u>Process Design Output Data.</u>

2.37.5.1 Microscreen area requirement, A_s, ft^2

2.37.5.2 Effluent BOD concentration, BODE, mg/l

2.37.5.3 Effluent suspended solids concentration, SSE, mg/l

2.37.5.4 Wash water pump capacity, mgd

2.37.5.5 Effluent COD concentration, CODE, mg/l.

2.37.5.6 Effluent oil and grease concentration, OAGE, mg/l.

2.37.6 <u>Quantities Calculations.</u>

2.37.6.1 Selection of number of microscreen to be installed. The following table will be utilized in the section of number of units to be installed:

Q_{avg} (mgd)	Number of Unit, NU
0.5- 1.0	2
1.0- 4.0	3
4.0- 8.0	4
8.0- 15.0	8
15.0- 30.0	15
30.0- 50.0	25
50.0-100.0	55

When Q_{avg} is larger than 100 mgd, several batteries of microscreens will be used. See next section for details.

2.37.6.2 Selection of number of screens and number of batteries of screens when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewer treatment plants that several batteries of screens, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.37.6.2.1 When $Q_{avg} \leq$ 100 mgd, only one battery of microscreens will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units

2.37.6.2.2 When $100 < Q_{avg} \leq$ 200 mgd, the system will be designed as two identical batteries of microscreens. Each battery would handle half of the wastewater. The number of screens in each battery would be selected according to the rules established in subsection (a) by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.37.6.2.3 When $Q_{avg} >$ 200 mgd, the design will be perfomed to use three batteries of screens, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.37.6.3 Sizing individual microscreens: The screen area of each individual microscreen can be calculated by using

$$A_{SI} = \frac{A_s}{NB \times NU}$$

where

ASI = Surface area of each microscreen, sq.ft.

AS = Area of microscreen required, sq.ft.

NB = Number of battery

NU = Number of units in each battery

The following table lists the commercially available sizes of microscreens.

Drum Sizes ft		
Diam.	Width	Screen Area ft^2
4	2	24
4	4	48
6	4	72
6	6	108
6	8	144
10	10	315

A_{SI} should confom with one of the sizes commercially available.

2.37.6.4 Calculate installed concrete requirement

when $A_s \leq 2820\ ft^2$

$$V_{cw} = 2.5\ A_s^{0.69}$$

when $A_s > 2820$

$$V_{cw} = 0.213 \times A_s$$

where

V_{cw} = Reinforced concrete wall inplace, cu.yd.

A_s = Area of microscreen required, sq.ft.

2.37.6.5 Calculate of earth work requirement

$$V_e = 1.589\ A_s^{0.929}$$

where

V_2 = Quantity of earth work required, cu.yd.

2.37.6.6 Calculate electrical power requirement for operation

when $A_s \leq 4500$ sq.ft.

$$KWH = 1000\ A_s^{0.866}$$

when $A_s > 4500$ sq.ft.

$$KWH = 327\ A_s$$

where

KWH = Power consumption of microscreens, Kwhr/yr.

A_s = Media area of microscreen, sq.ft.

2.37.6.7 Building area requirement. Ordinarily the microscreen system is housed in a building to protect it from weather. The building surface area can be calculated as:

$$A_B = 1.322\ A_s^{0.90}$$

where

A_B = Building area requirement, sq.ft.

2.37.6.8 Calculate operational labor

$$OMH = 160 \times Q_{avg}$$

where

OMH = Operating labor, man-hrs/yr

Q_{avg} = Average design flow, mgd

2.37.6.9 Calculate maintenance labor

$$MMH = 103 \times Q_{avg}$$

where

MMH = Maintenance labor, man-hr/yr.

2.37.6.10 Calculate O & M material and supply cost. Material and supply costs include such items as lubrication, screen and UV lamp replacement, etc. These costs are estimated as 15% of the installed equipment cost. This high material supply cost is due to the frequent replacement of the screen media panels.

2.37.7 Quantities Calculations Output Data.

2.37.7.1 Number of batteries, NB

2.37.7.2 Number of unit per battery, NU

2.37.7.3 Individual screen area of each unit, sq.ft. A_{SI}.

2.37.7.4 Quantity of earthwork, cu.yd., V_e

2.37.7.5 Quantity of concrete wall, cu.yd., V_{cw}

2.37.7.6 Building area requirement, sq.ft., A_B

2.37.7.7 Operation electricity, Kwhr/yr., KWH

2.37.7.8 Operating labor requirement, OMH, man-hrs/yr

2.37.7.9 Maintenance labor requirement, MMH, man-hrs/yr

2.37.8 Unit Price Input Required.

2.37.8.1 Standard size microscreen equipment cost, COSTSM, \$, (optional).

2.37.8.2 Unit price input of earthwork, UPIEX, \$/cu yd.

2.37.8.3 Unit price input of concrete wall inplace, UPICW, \$/cu yd.

2.37.8.4 Building construction costs, UPIBC, \$/sq ft.

2.37.8.5 Current Marshall and Swift Equipment Cost Index, MSECI.

2.37.9 Cost Calculations.

2.37.9.1 Cost of earthwork, COSTE.

$$COSTE = V_e \times UPIEX$$

where

COSTE = Cost of earthwork, \$.

V_e = Quantity of earthwork, cu.yd.

UPIEX = Unit price input of earthwork, \$/cu.yd.

2.37.9.2 Cost of concrete inplace, COSTC.

$$COSTC = V_{cw} \times UPICW$$

where

COSTC = Cost of concrete inplace, \$.

V_{cw} = Quantity of concrete, cu.yd.

UPICW = Unit price input of concrete wall \$/cu.yd.

2.37.9.3 Cost of installed equipment.

2.37.9.3.1 Purchase cost of microscreens includes the rotating drum, screen motor, backwashing assembly, UV lamps and other miscellaneous items. The cost can be estimated by using the following fomula:

$$COSTMS = COSTSM \times COSTR \times N_B \times N_u$$

where

COSTMS = Total purchase cost of microscreens and accessories, \$

COSTSM = Purchase cost of standard size screen (108 sq.ft.), \$

COSTR = Cost of microscreen of area A_{SI} as ratio of the cost of the standard size screen

N_B = Number of batteries

N_U = Number of units per battery

2.37.9.3.2 Calculate COSTR

when $A_{SI} \leq 108$ sq.ft.

$$COSTR = 0.231\ A_{SI}^{0.318}$$

when $A_{SI} > 108$ sq.ft.

$$COSTR = 0.032\ A_{SI}^{0.748}$$

where

A_{SI} = Screen area of the microscreen selected, sq.ft.

2.37.9.3.3 Purchase cost of standard size screen. The 108 sq.ft. microscreen was selected as the standard size unit as it is midrange of the sizes available. The costs of a 108 sq.ft. screen and accessories for the first quarter of 1977 are:

$$COSTSM = 52{,}000$$

For better estimation, COSTSM should be obtained from the equipment vendor and treated as a unit price input. If COSTSM is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift equipment cost index.

$$COSTSM = \$52{,}000 \times \frac{MSECI}{491.6}$$

where

MSECI = Current Marshall and Swift equipment cost index

491.6 = Marshall and Swift equipment cost index 1st quarter 1977

2.37.9.3.4 Calculate the installed cost of equipment. It is estimated that a 25% markup should be added for installation of microscreen equipment. Thus, the installed equipment cost, IEC

$$IEC = 1.25 \times COSTMS$$

where

IEC = Installed equipment cost, \$.

2.37.9.4 Cost of building.

$$COSTB = A_B \times UPIBC$$

where

COSTB = Cost of filter building, \$.

A_B = Area of building, sq.ft.

UPIBC = Unit price input for building construction costs, \$/sq.ft.

2.37.9.5 Other costs. The cost of piping, control system, electrical work and other minor items can be added as a percent of the total bare construction cost. It is estimated that a factor of 15% should be used.

2.37.9.6 Total bare construction cost

$$TBCC = 1.15\ (COSTE + COSTC + IEC + COSTB)$$

where

TBCC = Total bare construction cost, \$.

2.37.9.7 Calculate operation and maintenance material and supply costs.

$$OMMC = (IEC) \times 0.15$$

where

OMMC = Operation and maintenance material and supply costs, \$/yr.

IEC = Total installed equipment costs, \$.

2.37.10 Cost Calculations Output Data.

2.37.10.1 Total bare construction cost of the microscreen system, TBCC, \$.

2.37.10.2 Operational and maintenance material cost, OMMC, \$/yr.

2.37.11 Bibliography.

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2.37.11.2 EPA, Process Design Manual for Suspended Solids Removal, USEPA, 1975

2.37.11.3 Metcalf and Eddy, Inc., "Water Pollution Abatement Technology: Capabilities and Cost, Public Owned Treatment Works", P.B.-250/690-01. March, 1976, NTIS

2.39 NEUTRALIZATION

2.39.1 Background.

2.39.1.1 Neutralization is a unit operation in which pH adjustment of highly acidic or highly alkaline wastewaters takes place. Neutralization of such wastes is necessary prior to:

2.39.1.1.1 Biological waste treatment where the optimum pH for bacterial activity (ph 6.5-8.5) must be maintained.

2.39.1.1.2 Chemical treatment to provide an optimum pH for the reaction.

2.39.1.1.3 Disposal to the receiving streams.

Neutralization is applied mainly to the treatment of industrial wastes.

2.39.1.2 There are many acceptable methods for the neutralization of acidic and alkaline wastes. Acidic wastes may be neutralized by reaction with caustic soda, lime, or by passing the wastewater through a limestone bed. Neutralization through limestone beds may be accomplished through upflow or downflow systems to ensure sufficient retention time. Maximum acid concentration of 0.6 percent H_2SO_4 is suggested to avoid coating the limestone with nonreactive calcium sulfate and to prevent excessive evolution of carbon dioxide, both of which limit complete neutralization. Higher hydraulic rates may be used for upflow beds since the products are swept out before precipitation. However, the disposal of exhausted limestone beds may be a serious drawback to this method of neutralization.

2.39.1.3 Mixing acid wastes with lime slurries is an effective means of neutralization. The reaction is similar to that obtained with limestone beds. Lime is relatively inexpensive and posseses a high neutralizing power. Hydrated lime may produce a problem in handling since it has a tendency to arch or bridge over the outlet in storage bins and has poor flow characteristics. Both caustic soda and sodium carbonate are more powerful than lime, and the reaction products are soluble and do not increase the hardness of the receiving waters.

2.39.1.4 Alkaline wastes can be neutralized with acid (mostly sulfuric or hydrochloric), with flue gas containing 14 percent carbon dioxide, or with bottled carbon dioxide. Carbon dioxide will form carbonic acid when dissolved in water which will neutralize alkaline wastes. The reaction is slow, but sufficient, if the desired pH is near 7 or 8. The use of acid to neutralize alkaline wastes is fairly common. The reaction rate is almost instantaneous. A titration curve of the alkaline waste neutralized with various amounts of acid is helpful to ascertain the quantities of acid required for neutralization.

2.39.1.5 The selection of a pH control system may prove to be one of the most troublesome tasks facing the design engineer because of the following factors.

2.39.1.5.1 The relation between the amount of reagent and pH is nonlinear.

2.39.1.5.2 The input pH can vary rapidly over a range of several decades in a short time.

2.39.1.5.3 The flow can change while the pH is changing, and the two are not related.

2.39.1.5.4 The pH at neutrality is so sensitive to the addition of reagent that a slight excess can cause large deviations from the setpoint.

2.39.1.5.5 Measurement of the pH can be affected by materials which coat the electrodes.

2.39.1.5.6 The buffer capacity of the waste has a profound effect on the relation between reagent feed and pH and may not remain constant.

2.39.1.6 Several types of pH control schemes have been applied in waste neutralization systems, including manual control, open loop control, closed loop systems, combined open and closed loop systems, feedback control, and feed forward control.

2.39.1.7 Figure 2.39-1 shows an acid-waste neutralization system.

2.39.2 Input Data.

2.39.2.1 Wastewater flow.

2.39.2.1.1 Variations in waste flow (maximum and minimum), mgd.

2.39.2.1.2 Average daily flow, mgd.

2.39.2.2 pH ranges at various waste flows.

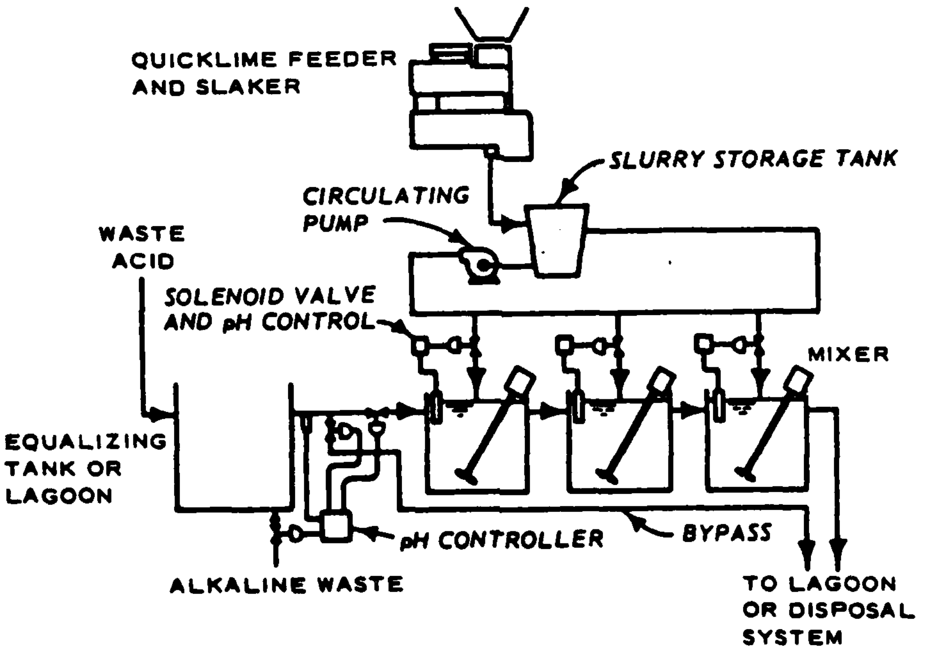

Figure 2.39-1. Schematic of acid-waste neutralization system

2.39.2.3 Acidity or alkalinity, mg/l.

2.39.3 <u>Design Parameters</u>.

2.39.3.1 Desired pH level and range.

2.39.3.2 Buffer capacity.

2.39.3.2.1 Pound reagent/gallon waste to neutralize to desired pH (from titration curves).

2.39.3.2.2 Flow rate, depth, and concentration of feed for limestone neutralization (from laboratory investigation).

2.39.3.3 Residence time (depends on the degree of mixing).

2.39.3.4 Degree of mixing (hp/1000 gal) (manufacturer, 0.2-0.4 hp/1000 gal).

2.39.4 <u>Process Design Calculations</u>.

2.39.4.1 Select reagent.

2.39.4.1.1 Acid waste (limestone bed or lime slurries or caustic soda).

2.39.4.1.2 Alkaline wastes (sulfuric or hydrochloric acids, carbon dioxide gas).

2.39.4.2 For limestone beds, select depth of bed and rate of flow (from laboratory investigation) and calculate surface area.

$$SA = \frac{Q_{avg} \times 10^6}{(24)(60)FR}$$

where

SA = surface area, ft^2.

Q_{avg} = average flow, mgd.

FR = flow rate, gpm/ft^2.

2.39.4.3 Calculate reagent feed using flow data and titration curves.

$$RF = (Q)(R)(10^6)$$

where

RF = reagent feed rate, lb/day.

Q = waste flow, mgd.

R = buffer capacity, lb reagent/gal waste to neutralize to desired pH.

2.39.4.4 Calculate volume of mixing tank.

$$Vol = (Q)(\sigma)\frac{(10^6)}{(24)(60)(7.48)}$$

where

Vol = volume of mixing tank, ft^3.

Q = waste flow, mgd.

σ = mixing time, min (from manufacturer, 5-10 min).

2.39.4.5 Calculate horsepower for mixing.

$$hp = \frac{(HPR)(Vol)(7.48\ gal/ft^3)}{(1000)}$$

where

hp = horsepower.

HPR = horsepower for mixing per 1000 gal (0.2-0.4 hp/1000 gal).

Vol = volume of mixing tank, ft^3.

2.39.4.6 For feedback control, may be calculated by equation proposed by Greer.

$$\Delta A = \frac{\partial A}{\partial t}\left(t - \sigma + \sigma e^{-\frac{t}{\sigma}}\right)$$

where

ΔA = allowable deviation in acidity or alkalinity from a given set point in the effluent.

$\frac{\partial A}{\partial t}$ = maximum rate of acidity or alkalinity change in feed to the unit.

t = overall process lag time = reaction time + measurement and transport lag ($\approx$ 0.5 min).

σ = minimum nominal residence time (time required for perfectly mixed basin to give process controllability).

2.39.4.7 Calculate and apply a factor of safety of 2-3.

2.39.4.8 PH.

$$PH = 7.0$$

where

PH = effluent pH.

2.39.5 <u>Process Design Output Data</u>.

2.39.5.1 Buffer capacity, lb/day.

2.39.5.2 Volume of mixing tank, ft^3.

2.39.5.3 Detention time, min.

2.39.5.4 Horsepower.

2.39.6 <u>Quantities Calculations</u>. Not used.

2.39.7 <u>Quantities Calculations Output Data</u>. Not used.

2.39.8 <u>Unit Price Input Required</u>. Not used.

2.39.9 <u>Cost Calculations</u>.

2.39.9.1 Unit costing is not available for this treatment process. Parametric costing will be used.

2.39.9.2 Calculate total bare construction cost.

$$TBCC = 6.0 \times 10^4 \, (Q_{avg})^{0.7}$$

where

TBCC = total bare construction cost, \$.

Q_{avg} = average daily flow, mgd.

2.39.10 Cost Calculations Output Data.

2.39.10.1 Total bare construction cost, TBCC, \$.

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2.41 NITRIFICATION

2.41.1 Background.

2.41.1.1 Nitrification is the process which converts organic and ammonia nitrogen to nitrate nitrogen. Nitrification may be coupled with denitrification, which reduces nitrate to nitrogen gas and removes the nitrogen from the water.

2.41.1.2 Conversion of ammonia nitrogen in wastewater to nitrates or atmospheric nitrogen is desirable for several reasons: 1) Nitrogen is one of the growth limiting nutrients. Control of nitrogen in wastewater effluents will help to limit the growth of algae and aquatic organisms and will slow the eutrophication of the receiving water body. 2) Ammonia nitrogen in the molecular form (NH_3) is toxic to fish. 3) Biological oxidation of ammonia to nitrite and then nitrate in the receiving water adds to the oxygen demand in the water. 4) Chlorination of wastewater containing ammonia requires high dosages because the chlorine combines with the ammonia to form chloramines which are less effective disinfectants.

2.41.1.3 Nitrification Reactions. The two principal genera of bacteria involved in the nitrification processes are Nitrosomonas and Nitrobacter. These bacteria are classified as autotrophic organisms. They derive energy from the oxidation of inorganic nitrogen compounds rather than from organic compounds, and they use inorganic carbon (carbon dioxide) rather than organic carbon for synthesis.

2.41.1.4 Nitrosomonas can oxidize ammonia to nitrite, and Nitrobacter can oxidize nitrite to nitrate. Both organisms must be present in a biological nitrification system.

2.41.1.5 The overall nitrification reactions, assuming an emperical formula for bacterial cells of $C_5H_7NO_2$, is given in the equations below:

$$15\ CO_2 + 13\ NH_4^+ \longrightarrow 10\ NO_2^- + 3\ C_5H_7NO_2 + 23\ H^+ + 4\ H_2O \quad \text{(Nitrosomonas)}$$

$$5\ CO_2 + NH_4^+ + 10\ NO_2^- + 2\ H_2O \longrightarrow 10\ NO_3^- + C_5H_7NO_2 + H^+ \quad \text{(Nitrobacter)}$$

2.41.1.6 These equations assume that the bacterial cells utilize gaseous CO_2 and produce free H^+. In actuality, these reactions take place in the aqueous carbonic acid system. The organisms utilize the dissolved form of carbon dioxide, carbonic acid (H_2CO_3), and produce free hydrogen ions (H^+) which immediately combine with the bicarbonate ion (HCO_3^-) to form carbonic acid. Nitrification tends to increase the H_2CO_3 in solution which can lower the pH of the water and interfere with the nitrification reactions.

2.41.1.7 Experiments have determined that approximately 7.14 mg/l alkalinity is destroyed per mg of ammonia nitrogen oxidized. Sufficient alkalinity in the form of HCO_3^- must be provided to maintain a pH favorable to the growth of nitrifying bacteria.

2.41.1.8 Experiments have also shown that the aeration requirement for nitrification systems is 4.6 mg/l O_2 per one mg NH_4^+-N. Excess air should be provided for oxidation of carbonaceous or other oxygen demanding materials other than NH_4^+-N that are present in the wastewater.

2.41.1.9 The yield of nitrifiers is very low. Thus, ordinarily, the nitrifier sludge production is eliminated in engineering calculations.

2.41.1.10 Nitrification Systems. Biological nitrification and carbon oxidation may be combined in conventional secondary treatment processes or they may be divided into separate stages. Separate stage nitirification is preferred in most cases.

2.41.1.11 Also, in a separate stage nitrification system the ratio of BOD_5 to Total Kjeldahl Nitrogen (TKN) is less than 3.0 as compared to 5.0 for a combined system. The lower BOD_5 load relative to the ammonia load allows a higher proportion of nitrifiers to heterotrophic bacteria which results in higher rates of nitrification.

2.41.1.12 Separate stage nitrification may be carried out in three types of systems: 1) suspended growth activated sludge; 2) attached growth trickling filters; and 3) attached growth rotating biological contactors (RBC).

2.41.2 <u>General Description Rotating Biological Contactor Nitrification</u>.

2.41.2.1 The rotating biological contactor (RBC) is an attached growth nitrification system. The RBC process consists of a series of large diameter plastic disks which are mounted on a horizontal shaft and placed in a concrete tank. The disks are slowly rotated while approximately 40 percent of the surface area is immersed in the water to be treated. A layer of biomass grows on the surface of the disks. A thin film of water is picked up on the biomass and flows down as the RBC rotates, absorbing oxygen from the air. The disk units are normally housed to avoid temperature drops, prevent algae growth and protect the slime layer from rain or hail which can wash the slime layer off the disks.

2.41.2.2 Special RBC media have been developed for nitrification. The minimal biomass that forms in a separate stage RBC allows 50 percent more subsurface area per standard shaft. As in trickling filter nitrification systems, the low growth rate of solids produces an effluent with a suspended solids concentration equal to the influent and eliminates the need for final clarification.

2.41.3 General Description Suspended Growth Nitrification.

2.41.3.1 Suspended growth nitrification systems are very similar in design to carbon oxidation activated sludge systems. The biological growth is suspended in an aeration basin. Mechanical or diffused aerators provide oxygen for nitrification and provide a mixing action which keeps the growth in suspension. The aeration mixed liquor is clarified to remove suspended solids and concentrate the sludge for recycle. The sludge retention time in the nitrification system is longer than that in a carbon oxidation system due to the slower growth rate of the nitrifiers as compared to heterotrophic bacteria.

2.41.3.2 The suspended growth activated sludge nitrification system can be operated as a complete mix or plug flow system and may utilize either mechanical or diffused aeration. In this design only the plug flow system is addressed, however both diffused and mechanical aeration is considered. This system is preferred due to its superior shock load dampening capacity. The design is based on the sludge retention time required for the desired level of nitrification. In order to promote the growth of nitrifiers and still maintain adequate solids in the aeration tank with good settleability, the influent BOD_5 concentration must be controlled. Separate stage nitrification requires a BOD_5 to TKN ratio of less than 3.0 in order to promote nitrification.[5] However, the influent BOD_5 concentration must be greater than approximately 50 mg/l. This can be accomplished by adding a small amount of primary effluent directly to the nitrification basin. Another solution is to add waste sludge from the system secondary aeration tank to maintain solids in the nitrification tank.

2.41.4 General Description Trickling Filter Nitrification.

2.41.4.1 In separate stage nitrification application, trickling filters can follow high rate trickling filters with intermediate clarification or an activated sludge process. The rate of nitrification is proportional to the surface area of the media exposed to the liquid being nitrified. Very little biological film develops in separate stage nitrification trickling filter. Hence, a media of higher specific surface area can be used without plugging or ponding which can be a problem in combined oxidation-nitrification trickling filters. Also, because of the small growth of solids on the media, clarification is not necessary. In this section, only the separate stage fixed film system is modeled; the influent BOD and suspended solids concentration are in the range of secondary effluent levels.

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL. 12/18

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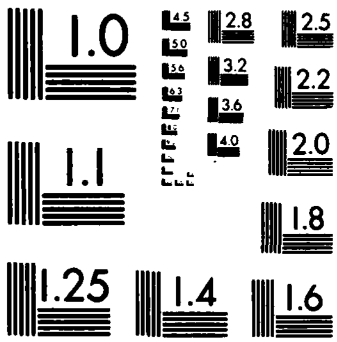

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2.41.5 RBC Nitrification.

2.41.5.1 Input Data.

2.41.5.1.1 TS, Summer Wastewater Temperature, ^{o}C

2.41.5.1.2 TW, Winter Wastewater Temperature, ^{o}C

2.41.5.1.3 N1S, Summer Effluent Standard, mg/l as N

2.41.5.1.4 N1W, Winter Effluent Standard, mg/l as N

2.41.5.1.5 NO, Influent TKN Concentration, mg/l as N

2.41.5.1.6 Q_{avg}, Average Design Flow, mgd

2.41.5.1.7 Q_{pk}, Peak Design Flow, mgd

2.41.5.2 Process Design Calculations.

2.41.5.2.1 Calculate Media Surface Required

2.41.5.2.1.1 The relationships between surface nitrification rate and the exposed ammonia concentration for secondary effluent is related to the temperature and the effluent ammonia concentration desired. This relationship can be expressed by the following equation.

$$A_N = \frac{14135.6}{TCF} [8.6 \ln (\frac{NO}{N1}) + (NO-N1)]$$

where

A_N = Surface area required per million gallons of wastewater flow, sq ft/MG

NO = Influent NH3 concentration, mg/l as N

TCF = Temperature correction factor

N1 = effluent NH3 concentration, mg/l as N.

and TCF, Temperature Correction Factor can be defined as:

When T is larger than 18^{o} C

TCF = 1.41

When T is smaller than 18^{o} C

TCF = 0.0787 x T

Where T = wastewater temperature, ^{o}C

2.41.5.2.1.2 If different effluent concentration limitations are given for both summer and winter conditions, determine the surface area requirement for each condition and base the final design on the condition that requires a larger media surface.

2.41.5.2.1.3 Safety Factor for Variation of Influent Flow. The design formula used in the above procedure was empirically obtained under the condition that no great variation of flow was received at the plant.

2.41.5.2.1.4 One of the RBC manufacturers suggested that when the peak flow to average flow ratio is more than 2.5, a safety factor should be used to give a conservative design. The safety factor is calculated by using the following equation:

$$SF = 1 + 0.143\left(\frac{Q_{pk}}{Q_{avg}} - 2.5\right)$$

where

SF = safety factor

Q_{pk} = peak flow in mgd

Q_{avg} = average design flow, mgd

2.41.5.2.1.5 Calculate the required media surface area.

$$A = Q_{avg} \times A_N \times SF$$

where

A = Required media surface area, sq ft

Q_{avg} = average design flow, mgd

SF = safety factor

2.41.5.2.2 Calculate design hydraulic loading

$$H_d = \frac{Q_{avg} \times 10^6}{A}$$

where

Hd = Design hydraulic loading, gpd/sq ft

2.41.5.2.3 Secondary Clarifier. If the influent quality is that of a secondary effluent, no clarification is needed and the BOD and suspended solid concentration would be the same as those in the influent.

2.41.5.2.4 Effluent Quality.

2.41.5.2.4.1 BOD_5.

$$BODE = BOD$$

where

BODE = effluent BOD_5 concentration, mg/l.

2.41.5.2.4.2 COD.

$$CODE = 1.5\ BODE$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

2.41.5.2.4.3 Suspended Solids.

$$SSE = SS$$

where

SSE = effluent suspended solids concentration, mg/l.

SS = influent suspended solids concentration, mg/l.

2.41.5.2.4.4 Nitrogen.

$$TKNE = (0.1)\ (SSE) + NH3E$$

$$NO3E = NO3 + (TKN - TKNE)$$

$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

SSE = effluent suspended solids concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

NO2 = influent NO_2 concentration, mg/l.

2.41.5.2.4.5 pH.

$$PH = 7.2$$

where

PH = effluent pH.

2.41.5.2.4.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.41.5.2.4.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.41.5.2.4.8 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.41.5.3 Process Design Output Data.

2.41.5.3.1 Required media surface area, A, sq ft

2.41.5.3.2 Design hydraulic loading, H_d, gpd/ sq ft

2.41.5.4 Quantities Calculations.

2.41.5.4.1 Calculate number of shafts necessary. In this design, the high density media with 150,000 sq ft of surface area per shaft is used.

$$N_{sh} = \frac{A}{A_{sh}}$$

If N_{sh} is not an integer, use the next larger integer.

where

N_{sh} = number of shafts necessary

A = total media surface area, ft^2

A_{sh} = media surface area per shaft, ft^2 (default value is 150,000)

Since there is not a set rule for staging the nitrification RBC system, the minimal practical number of shafts is the same as N_{sh}.

$$N_{mp} = N_{sh}$$

where

N_{mp} = minimum practical number of shafts.

2.41.5.4.2 For simplicity of design, it is assumed that shafts will be arranged in groups of eight. Each group will be called a bank and will consist of two end shafts and six intermediate shafts. Any shafts in excess of a multiple number of eight will form a partial bank of from one to seven shafts as needed. Many other configurations are possible; however, varying the configuration should not affect earth work and concrete requirements significantly.

2.41.5.4.2.1 Calculate number of banks needed.

$$N_b = N_{mp}/8$$

If N_b is not an integer, use next larger integer.

where

N_b = number of full and partial banks.

N_{mp} = minimum practical number of shafts needed.

2.41.5.4.2.2 Calculate number of end and intermediate shafts needed.

$$N_{es} = 2\ N_b$$

$$N_{is} = N_{mp} - N_{es}$$

where

N_{es} = number of end shafts needed.

N_b = number of banks needed.

N_{is} = number of intemediate shafts needed.

N_{mp} = minimum practical number of shafts needed.

2.41.5.4.3 Calculate earthwork requirements.

2.41.5.4.3.1 Dimensions used in calculating earth work requirements are shown in Figure 3.

2.41.5.4.3.2 $V_{ew} = 130\ N_{es} + 142\ N_{is}$

where

V_{ew} = volume of earth work required, yd^3

N_{es} = number of end shafts needed.

N_{is} = number of intemediate shafts needed.

2.41.5.4.4 Calculate reinforced concrete requirements.

2.41.5.4.4.1 Dimensions used in calculating concrete requirements are shown in Figure 3.

2.41.5.4.4.2 $V_{sc} = 23\ N_{es} + 20.7\ N_{is}$

$$V_{wc} = 11.5\ N_{es} + 8.6\ N_{is}$$

where

V_{sc} = volume of slab concrete needed, yd^3

V_{wc} = volume of wall concrete needed, yd^3

N_{es} = number of end shafts needed.

N_{is} = number of intemediate shafts needed.

2.41.5.4.5 Calculate annnual power consumption.

2.41.5.4.5.1 For a hydraulic loading between 0.5 and 2.5 mgd.

$$KWH = [3^{(3.9 - 1.21\ H_d)} + 7.12]\ 6{,}534\ Q_{avg}$$

2.41.5.4.5.2 For a hydraulic loading between 2.5 and 4.5 gpd/ft^2

$$KWH = (17.05 - 2.5H_d)\ 6{,}534\ Q_{avg}$$

where

KWH = annual power consumption, kwh/yr.

H_d = hydraulic loading, gpd/ft^2

Q_{avg} = design flow rate, mgd.

2.41.5.4.6 Calculate annual operation and maintenance labor requirements.

2.41.5.4.6.1 For a plant with less than 30 shafts

$$L_{om} = (1.25 - 0.025\ N_{mp})\ 52\ N_{mp}$$

2.41.5.4.6.2 For a plant with 30 shafts or more

$$L_{om} = 26\ N_{mp}$$

where

L_{om} = annual operation and maintenance labor requirements, person-hrs./yr.

N_{mp} = minimum practical number of shafts needed.

2.41.5.4.6.3 It is assumed that 70% of the man hour requirement is allocated for maintenance and the rest to operation. Thus,

Operation man hour requirement, OMH, man hr/yr

$$OMH = 0.3 \times L_{om}$$

Maintenance man-hour requirement, MMH, man-hr/yr

$$MMH = 0.7 \times L_{om}$$

2.41.5.4.7 Other Operation and Maintenance Material Costs. This item includes repair and replacement material cost and other minor costs, such as lubricating oil. It is expressed as a percent of installed costs of the RBC equipment.

OMMP = 2%

OMMP = percent of the installed RBC equipment costs

2.41.5.4.8 Other Construction Cost Items. From the above estimation, approximately 85% of the total construction costs have been accounted for. Other minor cost items such as piping, site work, control system, etc., would be the other 15 percent.

CF, correction factor would be $\frac{1}{.85} = 1.18$

2.41.5.5 <u>Quantities Calculations Output Data.</u>

2.41.5.5.1 Minimum practical number of shafts need, N_{mp}.

2.41.5.5.2 Earthwork required, V_{ew}, cu yd.

2.41.5.5.3 R.C. wall quantity, V_{ws}, cu yd.

2.41.5.5.4 R.C. slab quantity, V_{wc}, cu yd.

2.41.5.5.5 Annual power consumption, KWH, kwhr/yr.

2.41.5.5.6 Operational manhour requirement, man-hr/yr.

2.41.5.5.7 Maintenance manhour requirement, man-hr/yr.

2.41.5.5.8 Other operation and maintenance material cost, OMMP, percent.

2.41.5.5.9 Correction factor, CF.

2.41.5.6 Unit Price Inputs Required.

2.41.5.6.1 Cost of earthwork, UPIEX, $/cu yd.

2.41.5.6.2 Cost of R.C. wall in-place, UPICW, dollars/cu yd.

2.41.5.6.3 Cost of R.C. slab in-place, UPICS, dollars/cu yd.

2.41.5.6.4 Standard size RBC equipment cost, COSRBC, $ (optional).

2.41.5.6.5 Marshall and Swift equipment cost index, MSECI.

2.41.5.7 Cost Calculations.

2.41.5.7.1 Cost of earthwork, COSTE.

$$COSTE = V_{ew} \times UPIEX$$

where

COSTE = cost of earthwork, $.

V_{ew} = quantity of earthwork, cu yd.

UPIEX = unit price input of earthwork, $/cu yd.

2.41.5.7.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = V_{cw} \times UPICW$$

where

COSTCW = cost of concrete wall in-place, $.

V_{cw} = quantity of concrete wall, cu yd.

UPICW = unit price input cost of concrete wall in-place, \$/cu yd

2.41.5.7.3 Cost of concrete slab in-place COSTCS.

$$COSTCS = V_{cs} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of concrete slab, cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.41.5.7.4 Cost of Installed Equipment.

2.41.5.7.4.1 Purchase cost of RBC equipment. This includes the cost for the media, shaft and fiberglass cover. The first quarter, 1977 cost of a shaft with media surface area of 150,000 sq ft is approximately \$34,500. In this procedure, it is preferred that the cost of equipment is input from user. If no user input is given, the default value should be used.

$$COSRBC = 34{,}500 \times \frac{MSECI}{491.6}$$

where

COSRBC = purchase cost of standard size media with surface area of 100,000 sq ft, \$.

MSECI = current Marshall and Swift equipment cost index from input.

481.6 = Marshall and Swift cost index first quarter 1977

2.41.5.7.4.2 Installation cost. It is estimated that a 15% markup should be added for installation of the RBC equipment. Thus, the installed equipment cost, IEC.

$$IEC = 1.15 \times COSRBC \times N_{mp}$$

where

IEC = installed equipment cost, \$.

2.41.5.7.5 Other Cost Items. This category includes cost of piping, walkways, electrical control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.41.5.7.6 Total bare construction costs, TBCC.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC) \times CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

2.41.5.7.7 Operation and Maintenance Material Costs. Since this item of the operation and maintenance costs is expressed as a percentage of the installed equipment cost, it can be calculated by:

$$OMMC = \frac{OMMP}{100} \times IEC$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = percentage of the total bare construction costs as operation and maintenance material cost/percent.

2.41.5.8 Cost Calculations Output Data.

2.41.5.8.1 Total bare construction costs of the RBC system, TBCC, \$.

2.41.5.8.2 Operation and maintenance material costs, OMMC, \$/yr.

2.41.6 <u>Suspended Growth Nitrification (Diffused Aeration)</u>.

2.41.6.1 <u>Input Data</u>.

2.41.6.1.1 TS, summer operating temperature, ^{o}C

2.41.6.1.2 TW, winter operating temperature, ^{o}C

2.41.6.1.3 NO, influent total kjeldahl nitrogen concentration, mg/l as N

2.41.6.1.4 N1S, effluent ammonia nitrogen concentration, summer effluent standard, mg/l as N

2.41.6.1.5 N1W, effluent ammonia nitrogen concentration, winter effluent standard, mg/l as N

2.41.6.1.6 DO, dissolved oxygen concentration, 20 mg/l

2.41.6.1.7 Qp, peak flow rate, mgd

2.41.6.1.8 Q_{avg}, averge flow rate, mgd

2.41.6.1.9 Kd, decay coefficient, 0.05 day^{-1}

2.41.6.1.10 Yb, heterotrophic yield coefficient, 0.65 lb VSS grown/lb BOD_5 removed.

2.41.6.1.11 So, influent BOD_5 (must be $\geq$ 50 mg/10.

2.41.6.1.12 Km, metabolic factor, 15 hr^{-1} at 20°C.

2.41.6.1.13 SSI, influent inert suspended solids, mg/l.

2.41.6.1.14 XW, sludge solids concentration, mg/l.

2.41.6.1.15 STE, standard transfer efficiency of diffusers, 5 percent.

2.41.6.1.16 ALPHA, oxygen transfer coefficient, 0.9.

2.41.6.1.17 BETA, oxygen transfer coefficient, 0.9.

2.41.6.1.18 P, pressure correction factor, 1.0.

2.41.6.1.19 CS, O_2 saturation concentration at summer operating temperature.

2.41.6.1.20 CL, minimum O_2 concentration, 2.0 mg/l.

2.41.6.1.21 IALK, influent alkalinity, mg/l as $CaCO_3^{-}$.

2.41.6.1.22 SLR, solids loading rate to clarifier, 25 lb/ft^2/day.

2.41.6.1.23 OFR, maximum surface overflow rate to clarifier, 450 gpd/ft^2.

2.41.6.1.24 SAC, standard rated aerator capacity, 4.0 lb O_2 transfer/HP-hr mechanical systems.

2.41.6.2 Process Design Calculations.

2.41.6.2.1 Determine Sludge Retention Time (SRT). The sludge retention time is calculated from the temperature of the waste, the pH of the waste, the effluent concentration of ammonia nitrogen required and the dissolved oxygen concentration in the waste. In many states the effluent concentration standards differ for summer and winter operation. The design sludge retention time must be calculated for both summer temperature and effluent concentration and winter temperature and effluent concentration. The longer retention time should be used in the design of the nitrification system.

2.41.6.2.1.1 The first step required to calculate the desired sludge retention time in a plug flow nitrification system is to determine the growth rate of the nitrifying bacteria. Since the maximum growth rate of nitrobactor is considerably larger than the maximum growth rate of nitrosomonas and the value of substrate concentration at half maximum growth rate is less than 1 mg/l-N for both organisms, nitrite does not accumulate in large amounts in biological treatment systems. For this reason nitrifier growth can be modeled using the rate limiting step, ammonia conversion to nitrite, and the growth rate is of the nitrosomonas bacteria.

2.41.6.2.1.2 The growth rate of the nitrosomonas bacteria in plug flow is given by:

for pH < 7.2

$$MUN = 0.47\ [\ e^{\,0.098(T-15)}]\ x\ \ [1-0.833\ (7.2-ph)]$$

$$x\ [\ \frac{NO-N1}{(NO-N1)\ +\ 10^{0.051T-1.158}\ \ln \frac{NO}{N1}}\ ]\ x\ [\ \frac{DO}{DO\ +\ 1.3}]\ day^{-1}$$

For pH 7.2

$$MUN = 0.47\ [\ \ e^{\ 0.098(T-15)}\ ]\ x\ [\frac{NO-N1}{(NO-N1)\ +\ 10^{0.051T-1.158}\ln \frac{NO}{N1}}]$$

$$x\ [\frac{DO}{DO\ +\ 1.3}]\ \ day^{-1}$$

where

MUN = Growth rate, day^{-1}

T = Temperature, ^{o}C ($8^{o}C$ to $30^{o}C$)

NO = Influent TKN concentration (NH_4^+-N), mg/l

N1 = Effluent ammonia concentration (NH_4^+-N), mg/l

DO = Dissolved oxygen concentration, mg/l
default value of 2.0 mg/l

2.41.6.2.1.3 The minimum solids retention time is given by:

$$SRTMIN = \frac{1}{MUN}$$

where

SRTMIN = Minimum SRT required, days

2.41.6.2.1.4 The minimum solids retention time is the time that is theoretically required to obtain the desired effluent concentration of NH_3-N. Variation in flow may significantly affect the operation of a biological nitrification system. The degree of ammonia removal required may necessitate a longer SRT to accommodate the variations in flow. A safety factor must be multiplied by the minimum SRT to obtain the design SRT. The value of this safety factor is dependent on the allowable effluent ammonia concentration, the average flow rate, and the ratio of the peak flow rate to the average flow rate.

Estimates for the safety factor can be obtained for the following equations:

$$SF1 = 0.375 + 0.125 \frac{Q_p}{Q_{avg}}$$

$$SF2 = 0.935 - 0.271 \log (Q_{avg})$$

$$SF3 = 1.03 - 0.053 \, N1$$

where

Q_p = Peak flow, mgd

Q_{avg} = Average flow, mgd

N1 = Effluent ammonia limitation, NH_3-N mg/l

The safety factor to be used is the sum of SF1, SF2, and SF3:

$$SF = SF1 + SF2 + SF3$$

2.41.6.2.1.5 The design sludge retention time is calculated by:

$$SRT = (SF)(SRTMIN)$$

where

SRT = Design sludge retention time, day

SF = Retention time safety factor

SRTMIN = Minimum sludge retention time

2.41.6.2.1.6 If the effluent concentration standards differ for summer and winter, repeat the design sludge retention time calculation for each season's parameters. The largest SRT is used in the design procedure below.

2.41.6.2.2 Calculate Organic Removal Rate

$$q_b = \frac{\frac{1}{SRT} + Kd}{Y_b}, \text{ (lb BOD removal/lb MLVSS/day)}$$

Y_b = 0.65 lb VSS/lb BOD removal (heterotrophic yield coef.)

K_d = 0.05 day $^{-1}$ (decay coefficient)

SRT = Sludge retention time

q_b = Organic removal rate, (lb BOD removal/lb MLVSS/day)

2.41.6.2.3 Hydraulic Detention Time

$$HT = \frac{(S_o - S)}{(q_b)(MLVSS)}$$

where

HT = Hydraulic detention time (days)

S_o = Influent BOD_5 (mg/l)

S = Effluent soluble BOD_5 (assume S = 0 mg/l)

q_b = Organic removal rate (lb BOD_5 removal/lb MLVSS/day)

MLVSS = Mixed liquor volatile suspended solids (≈2000 mg/l)

2.41.6.2.4 Volume of Aeration Tank

$$V = Q_{avg}\,(HT)$$

where

V = Volume, million gallons

HT = Hydraulic detention time, days

Q_{avg} = Average flow rate, mgd

2.41.6.2.5 Calculate Effluent Soluble BOD_5

$$S = \frac{S_o}{K_m\,HT(24) + 1}$$

where

S = Effluent soluble BOD_5, mg/l

S_o = Influent BOD_5, mg/l

K_m = Metabolic factor = 15 hr^{-1} at 20°C

HT = Hydraulic detention time, days

2.41.6.2.6 Recalculate MLVSS

$$MLVSS = \frac{S_o - S}{(Q_b)\,(HT)}$$

where

S_o = Influent BOD_5, mg/l

S = Effluent soluble BOD_5, mg/l

q_b = Organic removal rate, lb BOD_5 removal/lb MLSS/day

MLVSS = Mixed liquor volatile suspended solids, mg/l

HT = Hydraulic detention time, days

2.41.6.2.7 Calculate MLSS in Aeration Tank

$$MLSS = MLVSS + \text{inert solids}$$

$$\text{inert solids} = SSI \frac{SRT}{HT} + 0.1\ MLVSS$$

$$MLSS = 1.1\ MLVSS + SSI \frac{SRT}{HT}$$

where

MLSS = Mixed liquor suspended solids, mg/l

MLVSS = Mixed liquor volatile suspended solids, mg/l

SSI = Influent inert suspended solids, mg/l (a default value of 20 mg/l)

SRT = Sludge retention time, days

HT = Hydraulic detention time, days

2.41.6.2.8 Sludge Waste Schedule

2.41.6.2.8.1 Sludge inventory

$$I = 8.34\ (MLSS)(V)$$

where

I = Sludge under aeration, lbs

MLSS = Mixed liquor suspended solids, mg/l

V = Aeration tank volume, million gallons

2.41.6.2.8.2 Total sludge wasted per day

$$SW = \frac{I}{SRT}$$

where

SW = Sludge wasted, lb/day

SRT = Sludge retention time, day

I = Sludge under aeration, lbs

2.41.6.2.8.3 Waste sludge flow rate. For a conservative estimate, assume that the effluent suspended solids is negligible compared to the sludge MLSS.

$$W = \frac{10^6\ SW}{8.34(XW)}$$

where

W = Sludge flow rate, gal/day

SW = Sludge wasted, lb/day

XW = Sludge suspended solids concentration, mg/l (12,000 mg/l)

2.41.6.2.8.4 Calculate return sludge rate

$$RSR = \frac{W}{Q_{avg}} \times 100$$

RSR = Return sludge rate, % of Q_{avg} (usually 100%)

W = Sludge flow rate, mgd

Q_{avg} = Average flow rate, mgd

2.41.6.2.9 Oxygen Requirement

2.41.6.2.9.1 Carbonaceous oxygen requirement

$$COR = \frac{1.5\ (S_o - S)}{(HT)24} - \frac{1.42\ (MLVSS)}{(SRT)(24)}\ ,\ (mg/l/hr)$$

where

S_o = Influent BOD mg/l

S = Effluent BOd mg/l

HT = Hydraulic detention time, days.

MLVSS = Mixed liquor volatile suspended solids mg/l

SRT = Design SRT, days.

2.41.6.2.9.2 Nitrogenous oxygen requirement

$$NOR = \frac{4.6\ (NO-N1)}{HT}\ ,\ (mg/l/hr)$$

NOR = nitrogenous oxygen requirement, mg/l

NO = Influent nitrogen concentration, mg/l

N1 = Effluent nitrogen concentration, mg/l

HT = Hydraulic detention time, hours

2.41.6.2.9.3 Oxygen demand safety factor

$$\text{if } \frac{Q_p}{Q_{avg}} \geq 2.2$$

$$SFOX = (0.55)\frac{Q_p}{Q_{avg}}$$

$$\text{if } \frac{Q_p}{Q_{avg}} < 2.2$$

$$SFOX = 1.2$$

where

SFOX = safety factor for oxygen demand

2.41.6.2.9.4 Oxygen requirement (OR)

$$OR = SFOX\ (COR + NOR)\ ,\ (mg/l/hour)$$

2.41.6.2.9.5 Calculate oxygen requirement in lb/day

$$O2 = (OR)(V)(200)\ lb/day$$

where

O2 = Oxygen requirement, lbs/day

V = Aeration tank volume, million gallons

2.41.6.2.10 Design the Aeration System

2.41.6.2.10.1 Assume following parameters

STE = 5.0 percent

ALPHA = 0.9

BETA = 0.9

P = Pressure correction factor, 1.0

2.41.6.2.10.2 Select summer operating temperature and detemine O_2 saturation

$$TS = 25^{o}C, \ CS = 8.2 \ mg/l$$

2.41.6.2.10.3 Detemine operating transfer efficiency

$$OTE = STE \left[\frac{(CS)(BETA)(P) - CL}{9.17} \right] ALPHA \ (1.02)^{TS-20}$$

where

OTE = Operating transfer efficiency, percent

STE = Standard transfer efficiency, 5 percent

CS = 8.2 mg/l at $25^{o}C$

BETA = 0.9

P = 1.0

CL = Minimum dissolved oxygen, 2.0 mg/l

ALPHA = 0.9

TS = $25^{o}C$, summer operating temperature

9.17 = O_2 saturation at $20^{o}C$

2.41.6.2.10.4 Calculate required airflow

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$CFM = \frac{O2 \ (7.48)}{OTE(0.0176)1440(V)(10^{3})}$$

where

CFM = Required air flow, cfm/1000 ft^3

OTE = Operating transfer efficiency, percent

V = Aeration tank volume, million gal

0.0176 = lb O_2 per ft^3 air

1440 = Min per day

02 = lb O_2/day

2.41.6.2.10.5 Check mixing requirement. To provide sufficient mixing in an activated sludge nitrification system, a minimum of 25 cfm/1000 ft^3 is required. If CFM $\leq$ 25 cfm, set

$$CFM = 25 \text{ cfm}/1000 \text{ ft}^3$$

If CFM > 25 cfm

$$CFM = CFM$$

2.41.6.2.11 Alkalinity Requirement

2.41.6.2.11.1 Alkalinity destroyed during nitrification

$$ALKD = 7.14 \ (NO\text{-}N1)$$

where

NO = Influent NH_3-N, mg/l

N1 = Effluent NH_3-N, mg/l

ALKD = Alkalinity destroyed, mg/l as $CaCO_3$

2.41.6.2.11.2 Required alkalinity addition. The amount of alkalinity which must be added, (ALKADD), is dependent upon the influent alkalinity (IALK) and the alkalinity destroyed (ALKD). It is recommended that a minimum value of 60 mg/l of alkalinity to be maintained in the aeration basin. Thus,

If (IALK - ALKD) $\geq$ 60 mg/l

then ALKADD = 0

IF (IALK - ALKD) < 60 mg/l

then ALKADD = 60 - (IALK - ALKD)

Total alkalinity added is:

$$ALKT = ALKADD \ (Q_{avg}) \ 8.34$$

where

ALKT = Total alkalinity added, lb/day ($CaCO_3$)

ALKADD = Amount of alkalinity which must be added, mg/l as $CaCO_3$

Q_{avg} = Average flow rate, mg/l

2.41.6.2.12 Effluent Quality

2.41.6.2.12.1 The effluent suspended solids (SS) concentration for a suspended growth nitrification system followed by a secondary clarifier should be in the range of 20 to 30 mg/l

ESS = 25 mg/l (or as specified by user)

2.41.6.2.12.2 The effluent BOD_5 concentration can be estimated from the effluent SS concentration.

$$EBOD_5 = S + 0.5\ (MLVSS/MLSS)\ ESS$$

where

$EBOD_5$ = Effluent BOD_5, mg/l

S = Effluent soluble BOD_5, mg/l

MLVSS = Mixed liquor volatile suspended solids in aeration tank, mg/l

MLSS = Mixed liquor suspended solids in aeration tank, mg/l

ESS = Effluent suspended solids, mg/l

2.41.6.2.12.3 COD.

$$CODE = 1.5\ EBOD_5$$
$$CODES = 1.5\ S$$

where

CODE = effluent COD concentration, mg/l.

$EBOD_5$ = effluent BOD_5 concentration, mg/l.

CODES = effluent soluble COD concentration, mg/l.

S = effluent soluble BOD_5 concentration, mg/l.

2.41.6.2.12.4 Nitrogen.

$$TKNE = (0.1)\ (ESS) + N1$$
$$NO3E = NO3 + (NO - TKNE)$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

ESS = effluent suspended solids concentration, mg/l.

N1 = effluent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO = influent Kjedahl nitrogen concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

NO2 = influent NO_2 concentration, mg/l.

2.41.6.2.12.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.41.6.2.12.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.41.6.2.12.7 pH.

$$PH = 7.2$$

where

PH = effluent pH.

2.41.6.2.12.8 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.41.6.3 Process Design Output Data.

2.41.6.3.1 SF, safety factor to account for flow and concentration variations.

2.41.6.3.2 SRT, design sludge retention time, days

2.41.6.3.3 HT, hydraulic detention time, days

2.41.6.3.4 V, aeration tank volume, million gallons

2.41.6.3.5 MLVSS, mixed liquor volatile suspended solids in aeration basin, mg/l

2.41.6.3.6 MLSS, mixed liquor suspended solids in aeration basin, mg/l.

2.41.6.3.7 SW, total sludge wasted per day, lb/day.

2.41.6.3.8 W, waste sludge flow rate, MGD

2.41.6.3.9 RSR, return sludge rate, percent

2.41.6.3.10 OR, oxygen requirement, mg/l/hr

2.41.6.3.11 02, oxygen requirement, lb/day

2.41.6.3.12 CFM, required airflow, diffuser system, scfm/1000 ft^3

2.41.6.3.13 ALKT, total added alkalinity, lb/day as $CaCO_3$

2.41.6.3.14 ESS, effluent suspended solids, mg/l

2.41.6.3.15 $EBOD_5$, effluent BOD_5, mg/l

2.41.6.3.16 N1, critical effluent ammonia concentration, mg/l as N

2.41.6.4 Quantities Calculations.

2.41.6.4.1 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT
0.5 - 2	2
2 - 4	3
4 - 10	4
10 - 20	6
20 - 30	8
30 - 40	10
40 - 50	12
50 - 70	14
70 - 100	16

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.41.6.4.2 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.41.6.4.2.1 When Q_{avg} 100 mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.41.6.4.2.2 When 100 Q_{avg} 200 mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in subsections (c) and (d) by using half the design flow as Q_{avg}. Thus

$$NB = 2$$

2.41.6.4.2.3 When Q_{avg} 200 mgd, the design will be perfomed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.41.6.4.3 Number of diffusers. The oxygen transfer rates used in the first-order design dictate the use of coarse bubble diffusers. These diffusers have an air flow from 10 - 15 scfm; for design purposes an average of 12 scfm will be used.

$$ND_t = \frac{CFM_d}{12\ (NT)\ \ (NB)}$$

ND_t must be an integer.

where

ND_t = number of diffusers per tank.

2.41.6.4.4 Number of swing arm diffuser headers. For ease of maintenance swing arm headers are usually used. The number of diffusers per header is dictated by the number of connections provided on each header by the manufacturer. This varies with manufacturer and header size from 8 to 30. For our purposes an average of 20 diffusers per header will be assumed.

$$NSA_t = \frac{ND_t}{20}$$

NSA_t must be an integer.

where

NSA_t = number of swing arm headers per tank.

2.41.6.4.5 Design of aeration tanks.

2.41.6.4.6 Volume of each tank would be

$$VN = \frac{V_d}{(NB)\ (NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.41.6.4.5.2 Depth and width of aeration tanks. The depth and width of the aeration tanks will be fixed at 15 ft. and 30 ft., respectively.

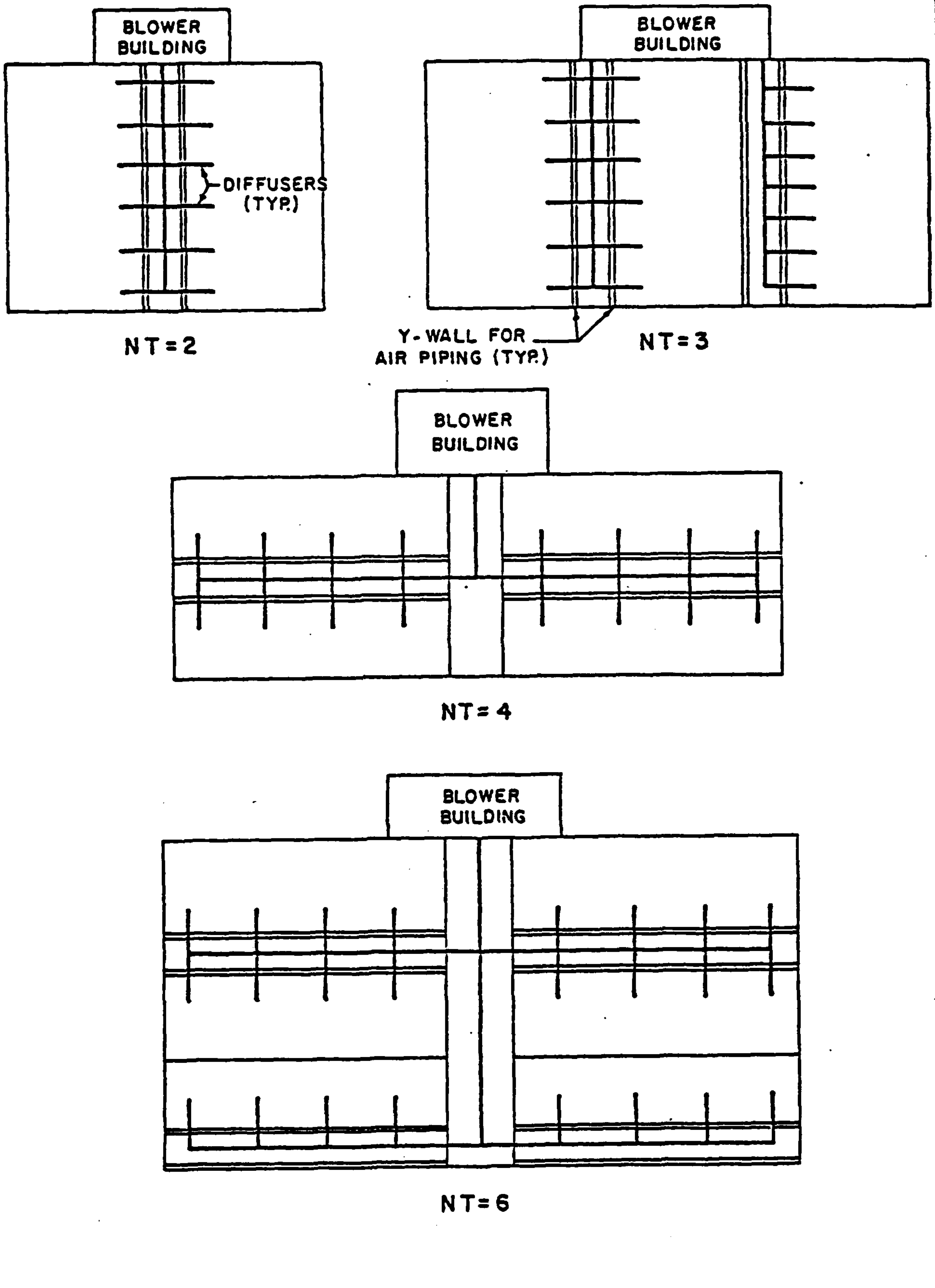


FIGURE 2.41-1. AERATION TANK ARRANGEMENT

2.41.6.4.5.3 Length of aeration tanks.

$$L = \frac{VN}{(15)\ (30)}$$

If L is greater than 400 ft., then recalculate VN using NT = NT + 1, then recalculate L.

2.41.6.4.6 Aeration tank arrangements.

2.41.6.4.6.1 Figure 2.41-1 shows the schematic diagram of the arrangements. A pipe gallery will be provided when the number of tanks is equal to or larger than four. The purpose of the pipe gallery is to house the various air and water piping systems and control equipment.

$$PGW = 20 + (0.4)\ \left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = pipe gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

2.41.6.4.7 Earthwork required for construction. It is assumed that the tank bottom will be 4 feet below ground level. The earthwork required can be estimated by the following equations:

2.41.6.4.7.1 When NT is less than 4, the earthwork required would be:

$$V_{ew} = 6\ NB\ \left[\frac{(NT(31.5) + (15.5)\ \ (L + 17) + \ \ (NT(31.5) + 23.5)\ (L + 25)}{2}\right]$$

where

V_{ew} = volume of earthwork required, cu. ft.

NT = number of tanks per battery.

L = length of aeration tanks, ft.

2.41.6.4.7.2 When NT is greater than or equal to 4, the earthwork required would be:

$$V_{ew} = 6\ NB\left[\frac{(15.75(NT) + (15.5)(2L + PGW + 20) + (15.75)(NT) + (23.5)(2L + PGW + 28}{2}\right.$$

2.41.6.4.8 Reinforced concrete slab quantity. It is assumed that a 1'-6" thick slab will be utilized regardless of the size of the system. The volume of reinforced concrete slab will be the same for both plug and complete mix flow.

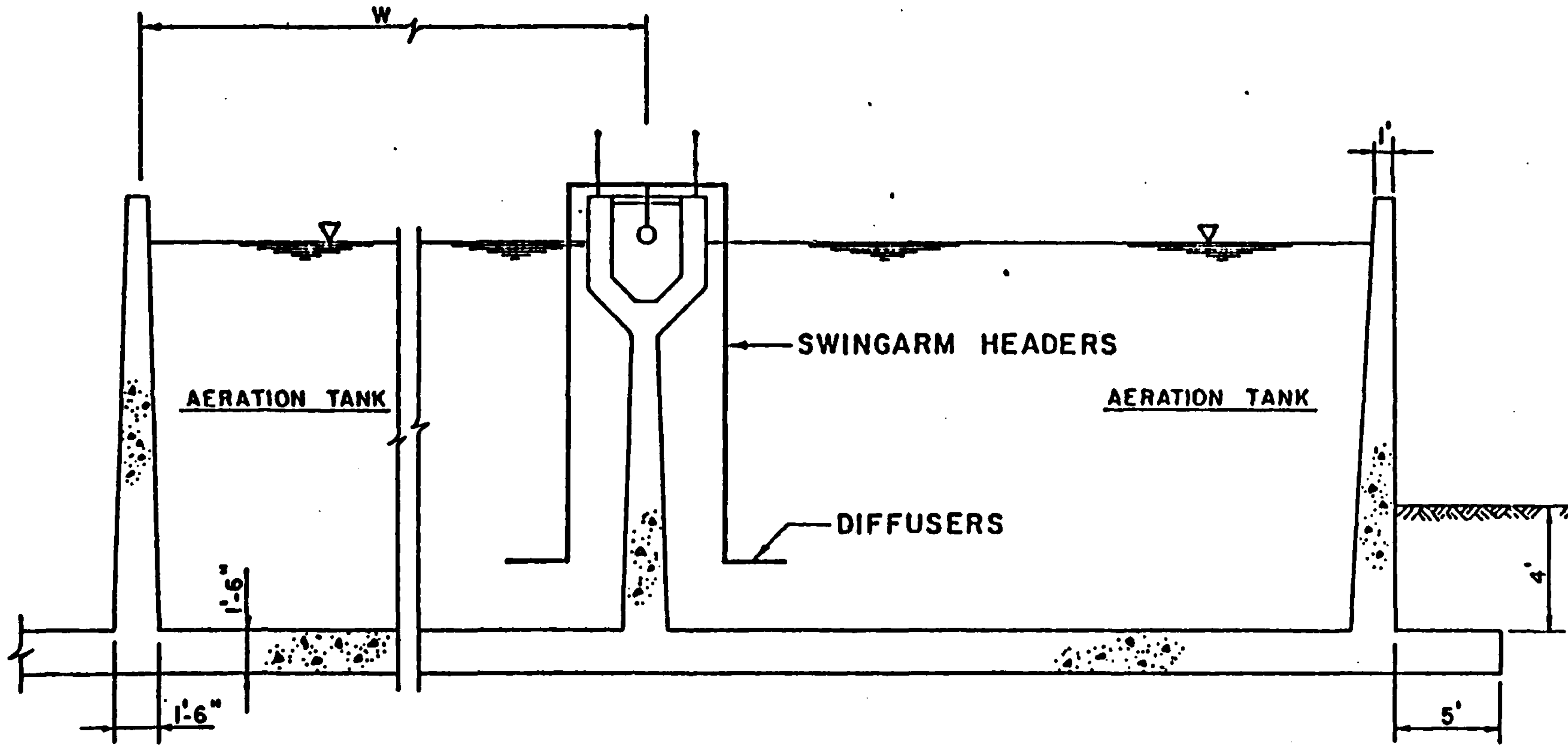


FIGURE 2.41-2. TYPICAL SECTION OF PLUG FLOW DIFFUSED AERATION TANKS

2.41.6.4.8.1 For NT less than 4:

$$V_{cs} = 1.5 \text{ NB } [(NT(31.5) + (15.5)(L + 17)]$$

where

V_{cs} = R.C. slab quantity required, cu. ft.

2.41.6.4.8.2 For NT greater than or equal to 4:

$$V_{cs} = 1.5 \text{ NB } [(15.75(NT)(2L + PGW + 200)]$$

2.41.6.4.9 Reinforced concrete wall quantity.

2.41.6.4.9.1 When NT is less than 4:

$$V_{cs} = (NB)[75.3(L) + [29.4(NT) - 58.8] (L) + 1383.75(NT) + 33.75]$$

2.41.6.4.9.2 When NT = 4, 8, 12, 16, 20, etc.:

$$V_{cw} = (NB)[29.35(NT)(L) + 56.8 (L) + 1350(NT) + 45(PGW) + 185.65]$$

2.41.6.4.9.3 When NT = 6, 10, 14, 18, 22, etc.:

$$V_{cw} = (NB)[15.15(NT)(L) + 28.5 (L) + 1367(NT) + 45 (PGW) + 168.75]$$

2.41.6.4.10 Quantity of handrail for safety. Handrail is required for safety protection of the operation personnel of wastewater treatment plants. Waterway walls and the top of the pipe gallery will require handrail. The quantity of handrail required may be estimated as follows:

2.41.6.4.10.1 For NT = 2

$$LHR = [(NT)(L) + 36.5 (NT) + 1.5] \text{ NB}$$

2.41.6.4.10.2 For NT = 3

$$LHR = [(NT + 1)L + 36.5(NT) + 6.5] \text{ NB}$$

2.41.6.4.10.3 For NT greater than or equal to 4:

$$LHR = [(NT + 4) (L) + 34(NT) + 2PGW + 3]\text{NB}$$

2.41.6.4.11 Calculate operation manpower requirements.

2.41.6.4.11.1 If CFM_d is less than or equal to 3000 scfm, the operation manpower can be calculated by:

$$OMH = 62.36 \, (CFM_d)^{0.3972}$$

where

OMH = operation manpower required, man-hours/yr

2.41.6.4.11.2 If CFM_d is greater than 3000 scfm, the operation manpower can be calculated by:

$$OMH = 26.56\ (CFM_d)^{0.5038}$$

2.41.6.4.12 Calculate maintenance manpower requirements.

2.41.6.4.12.1 If CFM_d is less than or equal to 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 22.82\ (CFM_d)^{0.4379}$$

2.41.6.4.12.2 If CFM_d 3000 scfm, the maintenance manpower can be calculated by:

$$MMH = 6.05\ (CFM_d)^{0.6037}$$

where

MMH = maintenance man-hours per year required.

2.41.6.4.13 Energy requirement for operation. The electrical energy required for operation is related to the average wastewater flow by the following equation:

$$KWH = 248{,}950.8\ (Q_{avg})^{0.9809}$$

where

KWH = electrical energy required for operation, kwhr/yr.

2.41.6.4.14 Operation and maintenance material and supply costs. Operation and maintenance material supply costs include items such as lubricant, paint, replacement parts, etc. These costs are estimated as a percent of the total bare construction costs.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = operation and maintenance material costs as percent of total bare construction cost, percent.

2.41.6.4.15 Other construction cost items. The majority of the costs of the diffused aeration activated sludge process have been accounted for. Other cost items, such as liquid piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent will be used.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.41.6.5 Quantities Calculations Output Data.

2.41.6.5.1 Number of aeration tanks, NT.

2.41.6.5.2 Number of diffusers per tank, ND_t.

2.41.6.5.3 Number of process batteries, NB.

2.41.6.5.4 Number of swing arm headers per tank, NSA_t.

2.41.6.5.5 Length of aeration tanks, L, ft.

2.41.6.5.6 Width of pipe gallery, PGW, ft.

2.41.6.5.7 Earthwork required for construction, V_{ew}, cu ft.

2.41.6.5.8 Quantity of R.C. slab required, V_{cs}, cu ft.

2.41.6.5.9 Quantity of R.C. wall required, V_{cw}, cu ft.

2.41.6.5.10 Quantity of handrail, LHR, ft.

2.41.6.5.11 Operation manpower requirement, OMH, man-hour/yr.

2.41.6.5.12 Maintenance manpower requirement, MMH, man-hour/yr.

2.41.6.5.13 Electrical energy for operation, KWH, kwhr/yr.

2.41.6.5.14 Operation and maintenance material and supply cost as percent of total bare construction cost, OMMP, percent.

2.41.6.5.15 Correction factor for minor construction costs, CF.

2.41.6.6 Unit Price Input Required.

2.41.6.6.1 Cost of earthwork, UPIEX, \$/cu yd.

2.41.6.6.2 Unit price input R.C. wall in-place, UPICW, \$/cu yd.

2.41.6.6.3 Unit price input R.C. slab in-place, UPICS, \$/cu yd.

2.41.6.6.4 Unit price input for handrails in-place, UPIHR, \$/ft.

2.41.6.6.5 Cost per diffuser, COSTPD, \$, (optional).

2.41.6.6.6 Cost per swing arm header, COSTPH, \$, (optional).

2.41.6.6.7 Current Marshall and Swift Equipment Cost Index, MSECI.

2.41.6.6.8 Current CE Plant Cost Index for pipe, valves, etc., CEPCIP.

2.41.6.6.9 Equipment installation labor rate, LABRI, \$/man-hour.

2.41.6.6.10 Unit price input for crane rental, UPICR, \$/hr.

2.41.6.7 Cost Calculations.

2.41.6.7.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, dollars.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.41.6.7.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. wall, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.41.6.7.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cw} = volume of concrete slab, cu yd.

UPICS = unit price of R.C. slab in-place, \$/cu yd.

2.41.6.7.4 Cost of handrails in-place.

$$COSTHR = LHR \times UPIHR$$

where

COSTHR = cost of handrails in-place, \$.

LHR = length of handrails, ft.

UPIHR = unit price input for handrails in-place, \$/ft.

2.41.6.7.5 Cost of diffusers.

2.41.6.7.5.1 The oxygen transfer values given in process design calculations indicate the use of coarse bubble diffusers. The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = 6.50 \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.41.6.7.5.2 Calculate COSTD.

$$COSTD = COSTPD \times ND_t \times NT \times NB$$

where

COSTD = cost of diffusers for system, \$.

ND_t = number of diffusers per tank.

NT = number of tanks.

2.41.6.7.6 Cost of swing arm diffuser headers.

2.41.6.7.6.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers. The cost of this header for the first quarter of 1977 is

$$COSTPH = \$5000$$

For a better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5000 \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.41.6.7.6.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA_t \times NT \times NB$$

where

COSTH = cost of swing arm headers for system, \$.

NSA_t = number of swing arm headers per tank.

NT = number of tanks.

NB = number of batteries.

2.41.6.7.7 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA_t \times NT \times NB$$

where

IMH = installation man-hour requirement, man-hours.

2.41.6.7.8 Crane requirement for installation.

$$CH = (.1)\ (IMH)$$

where

CH = crane time requirement for installation, hr.

2.41.6.7.9 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.41.6.7.9.1 If CFM_d is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (CFM_d)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

CFM_d = design capacity of blowers, scfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.41.6.7.9.2 If CFM_d is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (CFM_d)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.41.6.7.9.3 If CFM_d is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (CFM_d)^{0.8085}$$

2.41.6.7.10 Other costs associated with the installed equipment. This category includes the costs for weir installation, painting, inspection, etc., and can be added as a percentage of the purchased equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.41.6.7.11 Installed equipment costs.

$$IEC = (COSTD + COSTH)\left(1 + \frac{PMINC}{100}\right) + (IMH)(LABRI) + (CH)(UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/man-hour.

UPICR = crane rental rate, \$/hr.

2.41.6.7.12 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR)\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.41.6.7.13 Operation and maintenance material costs.

$$OMCC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material supply costs, \$/yr.

OMMP = operation and maintenance material and supply costs as percent of total bare construction cost, percent.

2.41.6.8 <u>Cost Calculations Output Data.</u>

2.41.6.8.1 Total bare construction cost TBCC, \$.

2.41.6.8.2 Operation and maintenance material and supply costs, OMMC, \$/yr.

2.41.7 Suspended Growth Nitrification (Mechanical Aeration).

2.41.7.1 Input Data.

2.41.7.1.1 TS, summer operating temperature, ^{o}C

2.41.7.1.2 TW, winter operating temperature, ^{o}C

2.41.7.1.3 NO, influent total kjeldahl nitrogen concentration, mg/l as N

2.41.7.1.4 N1S, effluent ammonia nitrogen concentration, summer effluent standard, mg/l as N

2.41.7.1.5 N1W, effluent ammonia nitrogen concentration, winter effluent standard, mg/l as N

2.41.7.1.6 DO, dissolved oxygen concentration, 20 mg/l

2.41.7.1.7 Qp, peak flow rate, mgd

2.41.7.1.8 Q_{avg}, averge flow rate, mgd

2.41.7.1.9 Kd, decay coefficient, 0.05 day^{-1}

2.41.7.1.10 Yb, heterotrophic yield coefficient, 0.65 lb VSS grown/lb BOD_5 removed

2.41.7.1.11 So, influent BOD_5 (must be 50 mg/10.

2.41.7.1.12 Km, metabolic factor, 15 hr^{-1} at 20°C.

2.41.7.1.13 SSI, influent inert suspended solids, mg/l.

2.41.7.1.14 XW, sludge solids concentration, mg/l.

2.41.7.1.15 STE, standard transfer efficiency of diffusers, 5 percent.

2.41.7.1.16 ALPHA, oxygen transfer coefficient, 0.9.

2.41.7.1.17 BETA, oxygent transfer coefficient, 0.9.

2.41.7.1.18 P, pressure correction factor, 1.0.

2.41.7.1.19 CS, 0_2 saturation concentration at summer operating temperature.

2.41.7.1.20 CL, minimum 0_2 concentration, 2.0 mg/l.

2.41.7.1.21 IALK, influent alkalinity, mg/l as $CaCO_3^-$.

2.41.7.1.22 SLR, solids loading rate to clarifier, 25 lb/ft^2/day.

2.41.7.1.23 OFR, maximum surface overflow rate to clarifier, 450 gpd/ft^2.

2.41.7.1.24 SAC, standard rated aerator capacity, 4.0 lb O_2 transfer/HP-hr mechanical systems.

2.41.7.2 <u>Process Design Calculations</u>.

2.41.7.2.1 Determine Sludge Retention Time (SRT). The sludge retention time is calculated from the temperature of the waste, the pH of the waste, the effluent concentration of ammonia nitrogen required and the dissolved oxygen concentration in the waste. In many states the effluent concentration standards differ for summer and winter operation. The design sludge retention time must be calculated for both summer temperature and effluent concentration and winter temperature and effluent concentration. The longer retention time should be used in the design of the nitrification system.

2.41.7.2.1.1 The first step required to calculate the desired sludge retention time in a plug flow nitrification system is to determine the growth rate of the nitrifying bacteria. Since the maximum growth rate of nitrobactor is considerably larger than the maximum growth rate of nitrosomonas and the value of substrate concentration at half maximum growth rate is less than 1 mg/l-N for both organisms, nitrite does not accumulate in large amounts in biological treatment systems. For this reason nitrifier growth can be modeled using the rate limiting step, ammonia conversion to nitrite, and the growth rate is of the nitrosomonas bacteria.

2.41.7.2.1.2 The growth rate of the nitrosomonas bacteria in plug flow is given by:

for pH < 7.2

$$MUN = 0.47\ [\ e^{0.098(T-15)}] \times [1-0.833\ (7.2-ph)]$$

$$\times \left[\frac{NO-N1}{(NO-N1) + 10^{0.051T-1.158} \ln \frac{NO}{N1}} \right] \times \left[\frac{DO}{DO + 1.3} \right] day^{-1}$$

For pH 7.2

$$MUN = 0.47\ [\ e^{0.098(T-15)}\ ] \times \left[\frac{NO-N1}{(NO-N1) + 10^{0.051T-1.158} \ln \frac{NO}{N1}} \right]$$

$$\times \left[\frac{DO}{DO + 1.3} \right]\ day^{-1}.$$

where

MUN = Growth rate, day^{-1}

T = Temperature, ^{o}C ($8^{o}C$ to $30^{o}C$)

NO = Influent TKN concentration (NH_4^+-N), mg/l

$N1$ = Effluent ammonia concentration (NH_4^+-N), mg/l

DO = Dissolved oxygen concentration, mg/l
default value of 2.0 mg/l

2.41.7.2.1.3 The minimum solids retention time is given by:

$$SRTMIN = \frac{1}{MUN}$$

where

SRTMIN = Minimum SRT required, days

2.41.7.2.1.4 The minimum solids retention time is the time that is theoretically required to obtain the desired effluent concentration of NH_3-N. Variation in flow may significantly affect the operation of a biological nitrification system. The degree of ammonia removal required may necessitate a longer SRT to accommodate the variations in flow. A safety factor must be multiplied by the minimum SRT to obtain the design SRT. The value of this safety factor is dependent on the allowable effluent ammonia concentration, the average flow rate, and the ratio of the peak flow rate to the average flow rate.

Estimates for the safety factor can be obtained for the following equations:

$$SF1 = 0.375 + 0.125 \frac{Q_p}{Q_{avg}}$$

$$SF2 = 0.935 - 0.271 \log (Q_{avg})$$

$$SF3 = 1.03 - 0.053\ N1$$

where

Q_p = Peak flow, mgd

Q_{avg} = Average flow, mgd

$N1$ = Effluent ammonia limitation, NH_3-N mg/l

The safety factor to be used is the sum of SF1, SF2, and SF3:

$$SF = SF1 + SF2 + SF3$$

2.41.7.2.1.5 The design sludge retention time is calculated by:

$$SRT = (SF)(SRTMIN)$$

where

SRT = Design sludge retention time, day

SF = Retention time safety factor

SRTMIN = Minimum sludge retention time

2.41.7.2.1.6 If the effluent concentration standards differ for summer and winter, repeat the design sludge retention time calculation for each season's parameters. The largest SRT is used in the design procedure below.

2.41.7.2.2 Calculate Organic Removal Rate

$$q_b = \frac{\frac{1}{SRT} + Kd}{Y_b}, \text{ (lb BOD removal/lb MLVSS/day)}$$

Y_b = 0.65 lb VSS/lb BOD removal (heterotrophic yield coef.)

K_d = 0.05 day $^{-1}$ (decay coefficient)

SRT = Sludge retention time

q_b = Organic removal rate, (lb BOD removal/lb MLVSS/day)

2.41.7.2.3 Hydraulic Detention Time

$$HT = \frac{(S_o - S)}{(q_b)(MLVSS)}$$

where

HT = Hydraulic detention time (days)

S_o = Influent BOD_5 (mg/l)

S = Effluent soluble BOD_5 (assume S = 0 mg/l)

q_b = Organic removal rate (lb BOD_5 removal/lb MLVSS/day)

MLVSS = Mixed liquor volatile suspended solids ($\approx$2000 mg/1).

2.41.7.2.4 Volume of Aeration Tank

$$V = Q_{avg}\ (HT)$$

where

V = Volume, million gallons

HT = Hydraulic detention time, days

Q_{avg} = Average flow rate, mgd

2.41.7.2.5 Calculate Effluent Soluble BOD_5

$$S = \frac{S_o}{K_m\ HT(24) + 1}$$

where

S = Effluent soluble BOD_5, mg/1

S_o = Influent BOD_5, mg/1

K_m = Metabolic factor = 15 hr^{-1} at 20°C

HT = Hydraulic detention time, days

2.41.7.2.6 Recalculate MLVSS

$$MLVSS = \frac{S_o - S}{(q_b)\ (HT)}$$

where

S_o = Influent BOD_5, mg/1

S = Effluent soluble BOD_5, mg/1

q_b = Organic removal rate, lb BOD_5 removal/lb MLSS/day

MLVSS = Mixed liquor volatile suspended solids, mg/1

HT = Hydraulic detention time, days

2.41.7.2.7 Calculate MLSS in Aeration Tank

MLSS = MLVSS + inert solids

$$\text{inert solids} = SSI \frac{SRT}{HT} + 0.1\ MLVSS$$

$$MLSS = 1.1\ MLVSS + SSI \frac{SRT}{HT}$$

where

MLSS = Mixed liquor suspended solids, mg/l

MLVSS = Mixed liquor volatile suspended solids, mg/l

SSI = Influent inert suspended solids, mg/l
(a default value of 20 mg/l)

SRT = Sludge retention time, days

HT = Hydraulic detention time, days

2.41.7.2.8 Sludge Waste Schedule

2.41.7.2.8.1 Sludge inventory

$$I = 8.34\ (MLSS)(V)$$

where

I = Sludge under aeration, lbs

MLSS = Mixed liquor suspended solids, mg/l

V = Aeration tank volume, million gallons

2.41.7.2.8.2 Total sludge wasted per day

$$SW = \frac{I}{SRT}$$

where

SW = Sludge wasted, lb/day

SRT = Sludge retention time, day

I = Sludge under aeration, lbs

2.41.7.2.8.3 Waste sludge flow rate. For a conservative estimate, assume that the effluent suspended solids is negligible compared to the sludge MLSS.

$$W = \frac{10^6\ SW}{8.34(XW)}$$

where

W = Sludge flow rate, gal/day

SW = Sludge wasted, lb/day

XW = Sludge suspended solids concentration, mg/l (12,000 mg/l)

2.41.7.2.8.4 Calculate return sludge rate

$$RSR = \frac{W}{Q_{avg}} \times 100$$

RSR = Return sludge rate, % of Q_{avg} (usually 100%)

W = Sludge flow rate, mgd

Q_{avg} = Average flow rate, mgd

2.41.7.2.9 Oxygen Requirement

2.41.7.2.9.1 Carbonaceous oxygen requirement

$$COR = \frac{1.5\ (S_o - S)}{(HT)(24)} - \frac{1.42\ (MLVSS)}{(SRT)(24)}\ ,\ (mg/l/hr)$$

where

S_o = Influent BOD mg/l

S = Effluent BOD mg/l

HT = Hydraulic detention time, days.

MLVSS = Mixed liquor volatile suspended solids mg/l

SRT = Design SRT, days.

2.41.7.2.9.2 Nitrogenous oxygen requirement

$$NOR = \frac{4.6\ (NO - N1)}{(HT)(24)}\ ,\ (mg/l/hr)$$

NOR = nitrogenous oxygen requirement, mg/l.

NO = Influent nitrogen concentration, mg/l.

N1 = Effluent nitrogen concentration, mg/l.

HT = Hydraulic detention time, days.

p. 2.41-45 not Included

OAC = operating aerator capacity, lbs of O_2 transfer/hr-HP.

SAC = standard rated aerator capacity, lbs of O_2 transfer/ hp-Hp use 4.0 as default value.

2.41.7.2.10.4 Calculate horsepower requirements.

$$HP = \frac{O2}{OAC}$$

where

HP = horsepower requirement for aerator, hp.

2.41.7.2.10.5 Check mixing requirement: The minimum horsepower requirement for mixing purposes should be 75 hp/MG of aeration basin. Thus,

$$HPM = 75 \times V$$

where

V = aeration tank volume, million gallons.

If HP $\geq$ HPM use HP = HP

If HP < HPM, use

HP = HPM

2.41.7.2.11 Alkalinity Requirement

2.41.7.2.11.1 Alkalinity destroyed during nitrification

$$ALKD = 7.14\ (NO-N1)$$

where

NO = Influent TKN, mg/l

N1 = Effluent NH_3-N, mg/l

ALKD = Alkalinity destroyed, mg/l as $CaCO_3$

2.41.7.2.11.2 Required alkalinity addition. The amount of alkalinity which must be added, (ALKADD), is dependent upon the influent alkalinity (IALK) and the alkalinity destroyed (ALKD). It is recommended that a minimum value of 60 mg/l of alkalinity to be maintained in the aeration basin. Thus,

If (IALK - ALKD) $\geq$ 60 mg/l

then ALKADD = 0

IF (IALK - ALKD) $<$ 60 mg/l

then ALKADD = 60 - (IALK - ALKD)

Total alkalinity added is:

$$ALKT = ALKADD\ (Q_{avg})\ 8.34$$

where

ALKT = Total alkalinity added, lb/day ($CaCO_3$)

ALKADD = Amount of alkalinity which must be added, mg/l as $CaCO_3$

Q_{avg} = Average flow rate, mg/l

2.41.7.2.12 Effluent Quality

2.41.7.2.12.1 The effluent suspended solids (SS) concentration for a suspended growth nitrification system followed by a secondary clarifier should be in the range of 20 to 30 mg/l

ESS = 25 mg/l

2.41.7.2.12.2 The effluent BOD_5 concentration can be estimated from the effluent SS concentration.

$$EBOD_5 = S + 0.8\ (MLVSS/MLSS)\ ESS$$

where

$EBOD_5$ = Effluent BOD_5, mg/l

S = Effluent soluble BOD_5, mg/l

MLVSS = Mixed liquor volatile suspended solids in aeration tank, mg/l

MLSS = Mixed liquor suspended solids in aeration tank, mg/l

ESS = Effluent suspended solids, mg/l

2.41.7.2.12.3 COD.

$$CODE = 1.5\ EBOD_5$$
$$CODES = 1.5\ S$$

where

CODE = effluent COD concentration, mg/l.

$EBOD_5$ = effluent BOD_5 concentration, mg/l.

CODES = effluent soluble COD concentration, mg/l.

S = effluent soluble BOD_5 concentration, mg/l.

2.41.7.2.12.4 Nitrogen.

$$TKNE = (0.1)\ (ESS) + N1$$
$$NO3E = NO3 + (NO - TKNE)$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

ESS = effluent suspended solids concentration, mg/l.

N1 = effluent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO = influent Kjedahl nitrogen concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

NO2 = influent NO_2 concentration, mg/l.

2.41.7.2.12.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.41.7.2.12.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.41.7.2.12.7 pH.

$$PH = 7.2$$

where

PH = effluent pH.

2.41.7.2.12.8 Settleable solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids, mg/l.

2.41.7.3 Process Design Output Data.

2.41.7.3.1 SF, safety factor to account for flow and concentration variations.

2.41.7.3.2 SRT, design sludge retention time, days

2.41.7.3.3 HT, hydraulic detention time, days

2.41.7.3.4 V, aeration tank volume, million gallons

2.41.7.3.5 MLVSS, mixed liquor volatile suspended solids in aeration basin, mg/l

2.41.7.3.6 MLSS, mixed liquor suspended solids in aeration basin, mg/l.

2.41.7.3.7 SW, total sludge wasted per day, lb/day.

2.41.7.3.8 W, waste sludge flow rate, MGD

2.41.7.3.9 RSR, return sludge rate, percent

2.41.7.3.10 OR, oxygen requirement, mg/l/hr

2.41.7.3.11 02, oxygen requirement, lb/day

2.41.7.3.12 CFM, required airflow, diffuser system, scfm/1000 ft^3

2.41.7.3.13 HP, horsepower requirement for mechanical aeration, hp.

2.41.7.3.14 ALKT, total added alkalinity, lb/day as $CaCO_3$

2.41.7.3.15 ESS, effluent suspended solids, mg/l

2.41.7.3.16 $EBOD_5$, effluent BOD_5, mg/l

2.41.7.3.17 N1, critical effluent ammonia concentration, mg/l as N

2.41.7.4 <u>Quantities Calculations</u>.

2.41.7.4.1 Selection of numbers of aeration tanks. The following rule will be utilized in the selection of numbers of aeration tanks.

Q_{avg} (mgd)	Number of Aeration Tanks NT	Number of Aerators Per Tank NA
0.5 - 2	2	1
2 - 4	3	1
4 - 10	4	1
10 - 20	6	2
20 - 30	8	2
30 - 40	10	3
40 - 50	12	3
50 - 70	14	3
70 - 100	16	4

When Q_{avg} is larger than 100 mgd, several batteries of aeration tanks will be used. See next section for details.

2.41.7.4.2 Selection of number of tanks and number of batteries of tanks when Q_{avg} is larger than 100 mgd. It is general practice in designing larger sewage treatment plants that several batteries of aeration tanks, instead of a single group of tanks, are used. This is due to land area availability and certain hydraulic limitations. To simplify the modeling process, the following rules will be used:

2.41.7.4.2.1 When $Q_{avg} \leq 100$ mgd, only one battery of aeration tanks will be used. Thus

$$NB = 1$$

where

NB = number of batteries of units.

2.41.7.4.2.2 When $100 < Q_{avg} \leq 200$ mgd, the system will be designed as two identical batteries of aeration basins. Each battery would handle half of the wastewater. The number of aeration tanks in each battery would be selected according to the rules established in Subsection 2.41.7.4.1 by using half the design flow as Q_{avg}. Thus,

$$NB = 2$$

2.41.7.4.2.3 When $Q_{avg} > 200$ mgd, the design will be performed to use three batteries of aeration basins, each handling one-third of the wastewater. Thus

$$NB = 3$$

2.41.7.4.3 Mechanical aeration equipment design.

2.41.7.4.3.1 Usually the slow-speed, fix-mounted mechanical surface aerators are used in domestic wastewater treatment plants. The available sizes of this type aerator are 5 HP, 7.5 HP, 10 HP, 15 HP, 20 HP, 25 HP, 30 HP, 40 HP, 50 HP, 60 HP, 75 HP, 100 HP, 125 HP and 150 HP.

2.41.7.4.3.2 Horsepower for each individual aerator:

$$HPN = \frac{HP_d}{(NB)\ (NT)\ (NA)}$$

If HPN > 150 HP and NT $=$ 2 or 3, repeat the calculation with NT $=$ NT $+$ 1.

If HPN > 150 HP and NT ≥ 4, repeat the calculation with NT $=$ NT $+$ 2.

where

HPN = horsepower of each unit, horsepower.

HP_d = design capacity of aeration equipment, horsepower.

NB = number of batteries.

NT = number of aeration tanks per battery.

NA = number of aerators per tank.

2.41.7.4.3.3 Sizing of aerators. Compare HPN with the available off-the-shelf sizes and select the smallest unit with capacity larger than HPN. The capacity of the selected unit would be designated as HPSN. Thus, the total capacity of the aeration units would be

$$HPT = (NB) \cdot (NT) \cdot (NA) \cdot (HPSN)$$

where

HPT = total capacity of selected aerators, horsepower.

2.41.7.4.4 Design of aeration tanks.

2.41.7.4.4.1 Volume of each individual tank would be

$$VN = \frac{V_d}{(NB)(NT)}$$

where

VN = volume of single aeration tank, cu ft.

2.41.7.4.4.2 Depth of aeration tanks. The depth of an aeration basin is controlled by the capacity of the aerators to be installed inside. If the water depth is too shallow, interference with the mixing current and oxygen transfer would occur. If the water depth is too deep, insufficient mixing would be extended at the bottom of the tank and sludge accumulation would occur. Thus proper selection of liquid depth of an aeration basin is important. The relationship between the recommended basin depth and the capacity of the aerators can be expressed as follows:

When $HPSN \leq 100$ HP

$$DW = 4.816\ (HPSN)^{0.2467}$$

When $HPSN > 100$ HP

$$DW = 15\ ft$$

where

DW = water depth of the aeration tanks, ft.

HPSN = capacity of the aerator, HP.

2.41.7.4.4.3 Width and length of aeration tank. The ratio between length and width of an aeration tank is dependent on the number of aerators to be installed in this tanks, NA.

If NA = 1. Square tank construction, L/W = 1

If NA = 2. Rectangular tank construction, L/W = 2

If NA = 3. Rectangular tank construction, L/W = 3

If NA = 4. Rectangular tank construction, L/W = 4

and

$$L/W = NA$$

where

NA = number of aerators per tank.

L = length of aeration tank, ft.

W = width of aeration tank, ft.

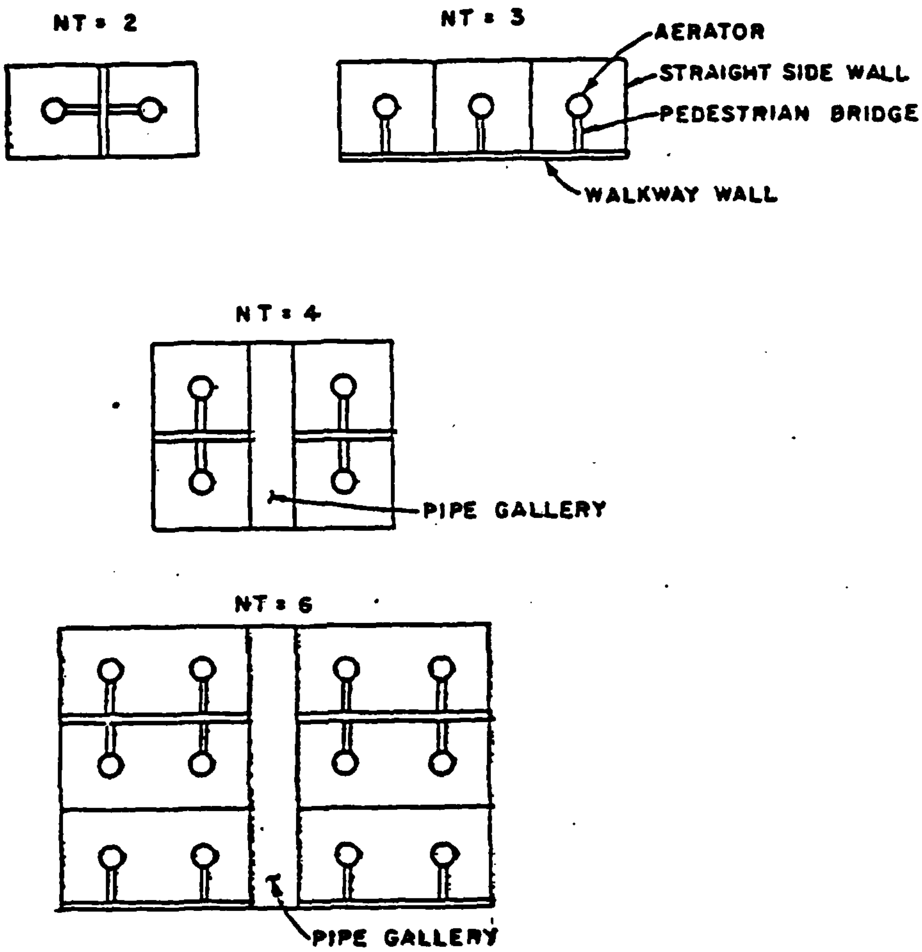
After the volume, depth and L/W ratio of the tank are determined, the width of the tank can be calculated by:

$$W = \left[\frac{VN}{(DW)\ (NA)}\right]^{0.5}$$

The length of the aeration tank would be:

$$L = (NA)\ (W)$$

2.41.7.4.5 Aeration tank arrangements.



FOR LARGER NT'S THE ARRANGEMENT WOULD BE SIMILAR TO THOSE WHEN NT = 4 AND NT = 6.

FIGURE 2.41-3. MECHANICAL AERATION TANK ARRANGEMENT

2.41.7.4.5.1 Figure 2.41-3 shows the schematic diagram of the tank arrangements. Piping gallery will be provided when the number of tanks is equal or larger than four. The purpose of piping gallery is to house various piping systems and control equipment.

2.41.7.4.5.2 Size of pipe gallery. The width of this gallery is dependent on the complexity and capacity of the piping system to be housed. An experience curve is provided to approximately estimate this width. It is expressed as:

$$PGW = 20 + (0.3)\left(\frac{Q_{avg}}{NB}\right)$$

where

PGW = piping gallery width, ft.

Q_{avg} = average influent wastewater flow, mgd.

NB = number of batteries.

2.41.7.4.6 Earthwork required for construction. It is assumed that tank bottom would be 4 feet below ground level. Thus the earthwork required would be estimated by the following equations:

2.41.7.4.6.1 When NT = 2, earthwork required would be:

$$V_{ew} = 3\,[(2\,W + 18.5)\,(W + 17) + (2\,W + 26.5)\,(W + 25)]$$

where

V_{ew} = quantity of earthwork required, cu ft.

W = width of aeration tank, ft.

2.41.7.4.6.2 When NT = 3, earthwork required would be:

$$V_{ew} = 3\,[(3\,W + 28)\,(W + 25) + (3\,W + 20)\,(W + 17)]$$

2.41.7.4.6.3 When NT $\geq$ 4, the width and length of the concrete slab for the whole aeration tank battery can be calculated by:

$$L_s = 2\,L + PGW + 16$$

$$W_s = \tfrac{1}{2}\,(NT)\,(W) + 14.5$$

where

L_s = length of the basin slab, ft.

L = length of one aeration tank, ft.

PGW = piping gallery width, ft.

W_s = width of the basin slab, ft.

NT = number of tanks per battery.

Thus the earthwork can be estimated by:

$$V_{ew} = 3 \cdot (NB) [(L_s + 4) (W_s + 4) + (L_s + 12) (W_s + 12)]$$

where

V_{ew} = volume of earthwork, cu ft.

2.41.7.4.7 Reinforced concrete slab quantity.

2.41.7.4.7.1 It is assumed that a 1'-6" thick slab will be utilized in this program regardless of the size of the system.

2.41.7.4.7.2 For NT = 2,

$$V_{cs} = 1.5 (2 W + 14.5) (W + 13)$$

where

V_{cs} = R.C. slab quantity, cu ft.

2.41.7.4.7.3 NT = 3,

$$V_{cs} = 1.5 (3 W + 16) (W + 13)$$

2.41.7.4.7.4 When NT ≥ 4,

$$V_{cs} = 1.5 (L_s) (W_s)$$

where

L_s = length of slab, ft.

W_s = width of slab, ft.

2.41.7.4.8 Reinforced concrete wall quantity.

2.41.7.4.8.1 When NT = 2,

$$V_{cw} = W (1.25 DW + 11) + (6 W + 9) (1.25 DW + 3.75)$$

2.41.7.4.8.2 When NT = 3,

$$V_{cw} = (1.25 DW + 11) (3 W + 6) + (1.25 DW + 3.75) (7 W + 6)$$

2.41.7.4.8.3 When NT ≥ 4,

$$V_{cw} = \frac{NT}{2} (L + 3) (1.25 DW + 11) + [(0.5 NT + 2) (L + 3) + 2 (NT) (W)] \cdot (1.25 DW + 3.75) \cdot (NB)$$

2.41.7.4.9 Reinforced concrete required for piping gallery construction. The quantity of piping gallery slab has been estimated with the aeration tanks slab calculations. Only the quantity of reinforced concrete for ceilings and end wall is necessary.

2.41.7.4.9.1 When NT < 4,

$$V_{cg} = 0$$

where

V_{cg} = quantity of R.C. for gallery construction, cu ft.

2.41.7.4.9.2 When NT ≥ 4, assuming the ceiling thickness is 1.5 feet, then the quantity of reinforced concrete would be:

$$V_{cgc} = (NB) \cdot (1.5) (PGW) [\frac{(NT) (W)}{2} + 0.75 (NT) + 1.5]$$

where

V_{cgc} = volume of R.C. ceiling for piping gallery construction, cu ft.

and for two end walls:

$$V_{cgw} = 2 (PGW) (NB) (DW + 3)$$

where

V_{cgw} = volume of R.C. walls for piping gallery construction, cu ft.

Thus total R.C. volume for piping gallery construction would be:

$$V_{cg} = V_{cgc} + V_{cgw}$$

2.41.7.4.10 Reinforced concrete quantity for aerator supporting platform construction.

2.41.7.4.10.1 Number of aerator-supporting platforms. Each aerator will be supported by an individual platform.

2.41.7.4.10.2 Figure 2.41-5 shows a typical supporting platform for the aeration equipment. The width of the platform would be a function of the capacity of the aerator to be supported. The following experience formula is given to approximate this relationship.

$$X = 5 + 0.078 (HPSN)$$

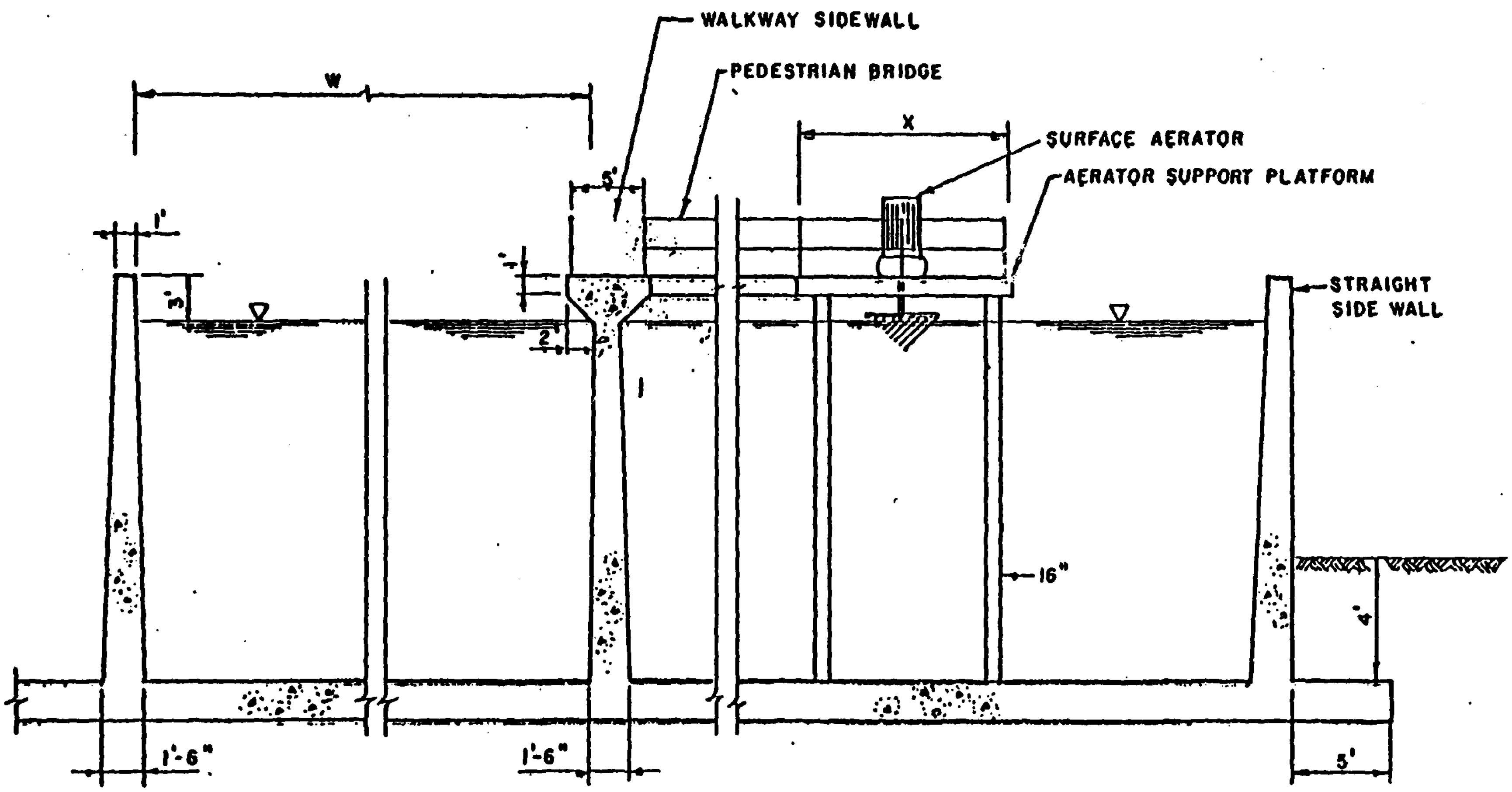


FIGURE 2.41-4 TYPICAL SECTION OF PLUG FLOW MECHANICAL AERATION TANKS

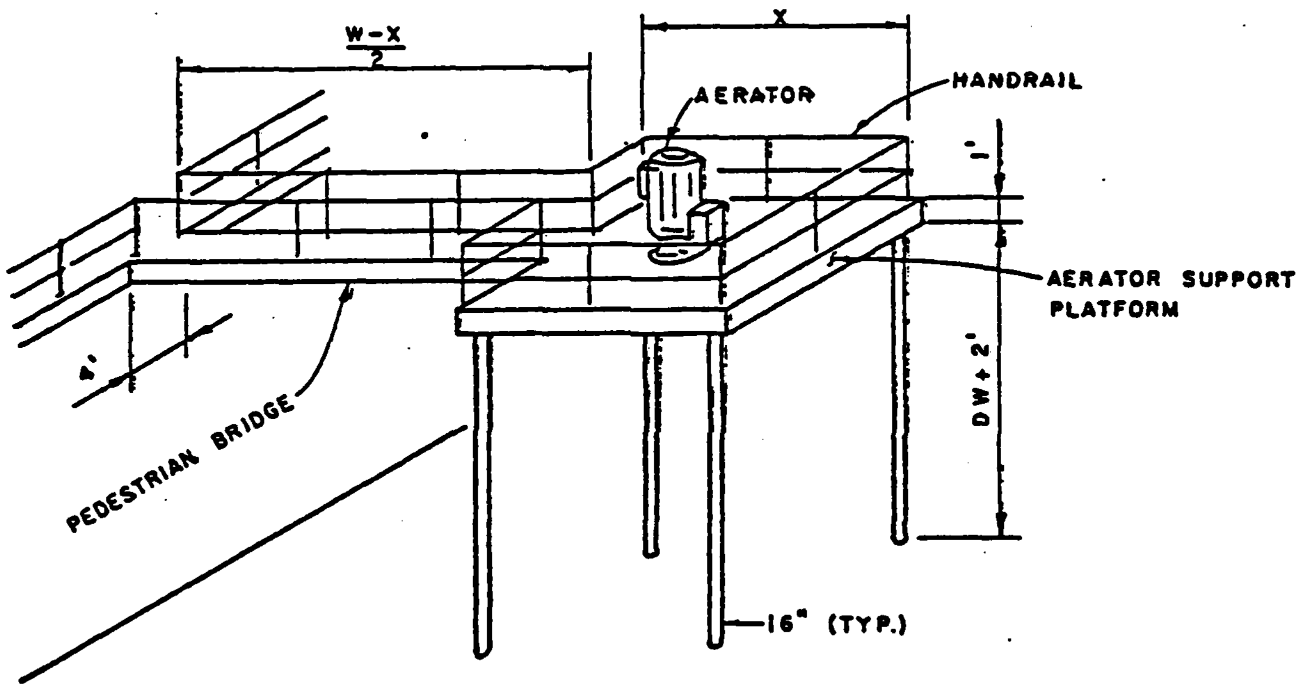


FIGURE 2.41-5. AERATOR SUPPORT PLATFORM

where

X = width of the platform, ft.

HPSN = horsepower of the mechanical aerator, HP.

2.41.7.4.10.3 Volume of reinforced concrete for the construction of the platforms would be:

$$V_{cp} = [X^2 + 5.6\ (DW + 2)]\ (NT)\ (NA)\ (NB)$$

where

V_{cp} = volume of R.C. for the platform construction, cu ft.

DW = water depth of the aeration basin, ft.

2.41.7.4.10.4 Volume of reinforced concrete for pedestrian bridges. The pedestrian bridge links the aerator platform to the walkway-sidewalls for ease of operation and maintenance. By using a width of 4 feet and slab thickness of 1 foot, the quantity of reinforced concrete can be calculated by:

$$V_{cwb} = [2\ (W - X)]\ (NB)\ (NT)\ (NA)$$

where

V_{cwb} = quantity of concrete for pedestrian bridge construction, cu ft.

2.41.7.4.11 Summary of reinforced concrete structures.

2.41.7.4.11.1 Quantity of concrete slab.

$$V_{cst} = V_{cs}$$

where

V_{cst} = total quantity of R.C. slab for the construction of aeration tanks, cu ft.

2.41.7.4.11.2 Quantity of concrete wall.

$$V_{cwt} = V_{cw} + V_{cg} + V_{cp} + V_{cwb}$$

where

V_{cwt} = quantity of R.C. wall for the construction of aeration tanks, cu ft.

V_{cw} = quantity of aeration tank R.C. walls, cu ft.

V_{cg} = quantity of R.C. for the construction of piping gallery, cu ft.

V_{cp} = quantity of R.C. for the construction of pedestrian bridges.

2.41.7.4.12 Quantity of handrail for safety. Handrail is required for the safety protection of the operation personnel of wastewater treatment plants. Waterway walls, aerator platforms and bridges, and the top of the piping gallery will require handrail. Quantity of handrail can be estimated thus:

2.41.7.4.12.1 When NT = 2

$$LHR = 4\ W + 11 + 2 \cdot (3X + W - 4)$$

where

LHR = handrail length, ft.

W = aeration tank width, ft.

X = width of aerator-supporting platform, ft.

2.41.7.4.12.2 When NT = 3

$$LHR = 6\ W + 10 + 3 \cdot (3X + W - 4)$$

2.41.7.4.12.3 When NT $\geq$ 4

If $\frac{NT}{2}$ is an even number

$$LHR = PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT) + (NA) \cdot (NT) \cdot (3X + W - 4) \cdot (NB)$$

If $\frac{NT}{2}$ is an odd number

$$LHR = PGW + (NT)\ (W) + [L + 3 - 4\ (NA)]\ (NT + 2) + (NA)\ (NT)\ (3X + W - 4) \cdot (NB)$$

where

PGW = width of the piping gallery, ft.

2.41.7.4.13 Operation and maintenance manpower requirements. Patterson and Bunker's data will be utilized to project the operation and maintenance manpower requirements. The man-hour per year requirement is presented as a function of the total horsepower of the aeration equipment.

2.41.7.4.13.1 Calculate the total installed capacity of the aeration equipment.

$$TICA = (NB)\ (NT)\ (NA)\ (HPSN)$$

where

TICA = total installed capacity of the aeration equipment, horsepower.

HPSN = capacity of one individual aerator, horsepower.

2.41.7.4.13.2 The operation manpower requirement can be estimated as follows:

When TICA < 200 HP

$$OMH = 242.4\ (TICA)^{0.3731}$$

When TICA $\geq$ 200 HP

$$OMH = 100\ (TICA)^{0.5425}$$

where

OMH = operational man-hour requirement, man-hour/yr.

2.41.7.4.13.3 The maintenance manpower requirement can be estimated as follows:

When TICA $\leq$ 100 HP

$$MMH = 106.3\ (TICA)^{0.4031}$$

When TICA > 100 HP

$$MMH = 42.6\ (TICA)^{0.5956}$$

where

MMH = maintenance manpower requirement, man-hour/yr.

2.41.7.4.14 Energy requirement for operation. By assuming that all the aerators will be operated 90 percent of the time year-round, the electrical energy consumption would be:

$$KWH = 0.85 \times 0.9 \times 24 \times 365 \times (TICA)$$

where

KWH = electrical energy required for operation, kwhr/yr.

0.85 = conversion factor from hp-hr to kwhr.

2.41.7.4.15 Material and supply costs for operation and maintenance. Material and supply costs for operation and maintenance include such items as lubrication oil, paint and repair material, etc. These costs are estimated as a percent of installed costs for the aeration equipment and are expressed as follows:

$$OMMP = 4.225 - 0.975 \log (TICA)$$

where

OMMP = percent of the installed equipment cost as O&M material costs, percent.

TICA = total installed capacity of aeration equipment, horsepower.

2.41.7.4.16 Other construction cost items. Using the above calculation, the majority of cost items of the activated sludge process have been accounted for. Other cost items, such as piping system, control equipment, painting, site cleaning and preparation, etc., can be estimated as a percent of the total bare construction cost. This percentage value has been shown to vary from 4 to 15 percent of the total construction cost of the aeration tank system. The value depends greatly on site conditions and complexity of the process. For a generalized model, an average value of 10 percent would be adequate, Thus,

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor to account for the minor cost items.

2.41.7.5 <u>Quantities Calculations Output Data.</u>

2.41.7.5.1 Number of aeration tanks, NT.

2.41.7.5.2 Number of aerators per tank, NA.

2.41.7.5.3 Number of process batteries, NB.

2.41.7.5.4 Capacity of each individual aerator, HPSN, hp.

2.41.7.5.5 Depth of aeration tanks, DW, ft.

2.41.7.5.6 Length of aeration tanks, L, ft.

2.41.7.5.7 Width of aeration tanks, W, ft.

2.41.7.5.8 Width of pipe gallery, PGW, ft.

2.41.7.5.9 Earthwork required for construction, V_{ew}, cu ft.

2.41.7.5.10 Total quantity of R.C. slab, V_{cst}, cu ft.

2.41.7.5.11 Total quantity of R.C. wall, V_{cwt}, cu ft.

2.41.7.5.12 Quantity of handrail, LHR, ft.

2.41.7.5.13 Operation manpower requirement, OMH, man-hour/yr.

2.41.7.5.14 Maintenance, manpower requirement, MMH, man-hour/yr.

2.41.7.5.15 Electrical energy for operation, KWH, kwhr/yr.

2.41.7.5.16 Percentage for O&M material and supply cost, OMMP, percent.

2.41.7.5.17 Correction factor for minor capital cost items, CF.

2.41.7.6 Unit Price Input Required.

2.41.7.6.1 Cost of earthwork, UPIEX, \$/cu yd.

2.41.7.6.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.41.7.6.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.41.7.6.4 Standard size low speed surface aerator cost (20 HP), SSXSA, \$, optional.

2.41.7.6.5 Marshall and Swift Equipment Cost Index, MSECI.

2.41.7.6.6 Equipment installation labor rate, \$/man-hour.

2.41.7.6.7 Crane rental rate, UPICR, \$/hr.

2.41.7.6.8 Unit price of handrail, UPIHR, \$/L.F.

2.41.7.7 Cost Calculations.

2.41.7.7.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input of earthwork, \$/cu yd.

2.41.7.7.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cwt}}{27} \cdot UPICW$$

where

COSTCW = cost of concrete wall in-place, \$.

V_{cwt} = quantity of R.C. wall, cu ft.

UPICW = unit price input of concrete wall in-place, \$/ cu yd.

2.41.7.7.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = \frac{V_{cst}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = quantity of concrete, \$/cu yd.

UPICS = unit price input of R.C. slab in-place, \$/cu yd.

2.41.7.7.4 Cost of installed aeration equipment.

2.41.7.7.4.1 Purchase cost of slow speed pier-mounted surface aerators. The purchase cost of aerators can be obtained by using the following equation:

$$CSXSA = SSXSA \cdot RSXSA$$

where

CSXSA = purchase cost of surface aerator, \$.

SSXSA = purchase cost of a standard size slow speed pier-mounted aerator. Motor horsepower is 20 HP.

RSXSA = ratio of cost of aerators with capacity of HPSN HP and that of the standard size aerator with 20 HP.

2.41.7.7.4.2 RSXSA. The cost ratio can be expressed as

$$RSXSA = 0.2148\ (HPSN)^{0.513}$$

where

HPSN = capacity of each individual aerator, HP.

2.41.7.7.4.3 Cost of standard size aerator. The cost of pier-mounted slow speed surface aerator for the first quarter of 1977 is

$$SSXSA = \$16{,}300$$

For a better estimate, SSXSA should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$SSXSA = 16{,}300 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index, First quarter 1977.

2.41.7.7.4.4 Equipment installation man-hour requirement. The man-hour requirement for field installation of fixed-mounted surface aerator can be estimated as:

When $HPSN \leq 60$ HP

$$IMH = 39 + 0.55 \, (HPSN)$$

When $HPSN > 60$ HP

$$IMH = 61.3 + 0.18 \, (HPSN)$$

where

IMH = installation man-hour requirement, man-hour.

2.41.7.7.4.5 Crane requirement for installation.

$$CH = (0.1) \cdot IMH$$

where

CH = crane time requ'rement for installation, hr.

2.41.7.7.4.6 Other costs associated with the installed equipment. This category includes the costs for electric wiring and setting, painting, inspection, etc., and can be added as a percentage of purchased equipment cost:

$$PMINC = 23\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.41.7.7.4.7 Installed equipment cost, IEC.

$$IEC = [CSXSA \, (1 + \frac{PMINC}{100}) + IMH \cdot LABRI + CH \cdot UPICR] \cdot (NB) \cdot (NT) \cdot (NA)$$

where

IEC = installed equipment costs, \$.

LABRI = labor rate, \$/man-hour.

UPICR = crane rental rate, \$/hr.

2.41.7.7.5 Cost of handrail. The cost of installed handrail system can be estimated as:

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, ft.

UPIHR = unit price input for handrail cost, \$ per linear foot. A value of \$25.20 per foot for the first quarter of 1977 is suggested.

2.41.7.7.6 Other cost items. This category includes cost of process piping system, control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.41.7.7.7 Total bare construction costs, TBCC, dollars.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \cdot CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

2.41.7.7.8 Operation and maintenance material costs. Since this item of the O&M expenses is expressed as a percentage of the installed equipment costs, it can be calculated by:

$$OMMC = IEC \cdot \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material and supply costs \$/yr.

OMMP = percent of the installed aerator cost as O&M material and supply expenses.

2.41.7.8 <u>Cost Calculations Output Data.</u>

2.41.7.8.1 Total bare construction cost of the mechanical aerated activated sludge process, TBCC, \$.

2.41.7.8.2 Operation and maintenance supply and material costs, OMMC, \$.

2.41.8 Trickling Filter Nitrification.

2.41.8.1 Input Data.

2.41.8.1.1 TS, operating temperature, summer ^{o}C

2.41.8.1.2 TW, operating temperature, winter ^{o}C

2.41.8.1.3 N1S, summer effluent standard, mg/1 as NH_4^+-N

2.41.8.1.4 N1W, winter effluent standard, mg/1 as NH_4^+-N

2.41.8.2 Process Design Calculations.

2.41.8.2.1 Calculate media surface area required.

2.41.8.2.1.1 The following equations can be used to calculate the required surface area based upon effluent concentration and operating temperature. If different concentration standards are given for summer and winter, determine the surface area required for both summer and winter operating conditions and base the design on the larger area required.

2.41.8.2.1.2 The critical ammonia effluent concentration is given by:

$$LNHE = 4.5 - 0.115T$$

where

LNHE = Critical ammonia effluent concentration, mg/1 NH_4^+-N

T = Operating temperature, ^{o}C

TS for summer

TW for winter

2.41.8.2.1.3 The required surface area is calculated:

when NHE is larger than LNHE

$$SRNO = 6900 - 190T$$

when NHE is smaller than LNHE

$$SRNO = (21{,}246 - 52.733T) - 3409\ NHE$$

where

LNHE = Critical ammonia effluent concentration, mg/l as NH_3^+-N

NHE = Desired effluent ammonia concentration, mg/l as NH_3^+-N

T = Operating temperature, ^oC

SRNO = Required surface area, ft^2

NHE = N1S, summer effluent standard

NHE = N1W, winter effluent standard

2.41.8.2.2 Calculate volume of media required

$$V = \frac{SRNO}{n} \times (NO - NHE)\ (8.34)\ (Q_{avg})$$

where

NO = Influent NH_3^+-N

V = Volume of media, ft^3

SRNO = Required surface area, ft^2

n = Specific surface area, ft^2/ft^3

The value of n, the specific surface area, depends on the type of media used. For nitrification process, high specific surface media can be used in order to minimize the cost of structure. A typical value is 41 ft^2/ft^3.

2.41.8.2.3 Calculate the surface area of the filter. A surface loading rate of 0.75 gpm/ft^2 at design average flow is recommended.

$$SA = \frac{694\ Q_{AVG}}{SLR}$$

where

SA = Cross-sectional area of filter, ft^2

SLR = Surface loading rate, 0.75 gpm/ft^2

Q_{AVG} = Average design flow rate, mgd

2.41.8.2.4 Calculate depth of filter

$$D = \frac{V}{SA}$$

where

D = Depth, ft

V = Volume, ft^3

SA = Cross-sectional area, ft^2

2.41.8.2.5 Check depth of filter. The maximum depth of filter is 28 ft

If $D \leq 28$ ft, the design is acceptable

RCY = 0
RCR = 0

where

RCY = Recycle flow, mgd

RCR = Recycle ratio

2.41.8.2.5.1 If $D > 28$ ft; set depth at 28 ft

2.41.8.2.5.2 Calculate new surface area

$$SA1 = \frac{V}{28\ ft}$$

where

V = Volume of media, cu.ft.

SA1 = New cross-sectional area, sq.ft.

2.41.8.2.5.3 Determine the amount of recycle necessary to obtain a surface loading of 0.75 gpm/ft^2

$$RCY = \frac{0.75\ SA1}{694} - Q_{AVG}$$

where

RCY = Amount of recycle, MGD

Q_{AVG} = Average flow, MGD

SA1 = New surface area required, ft^2

2.41.8.2.5.4 Determine recycle ratio of recycled flow over Q_{AVG}

$$RCR = \frac{RCY}{Q_{AVG}}$$

where

RCR = Recycle ratio

RCY = Recycle flow, MGD

Q_{AVG} = Average flow, MGD

2.41.8.2.5.5 Set SA = SA1, this becomes the design filter cross-sectional area.

2.41.8.2.6 Effluent Quality.

2.41.8.2.6.1 BOD_5.

$$BODE = BOD$$

where

BODE = effluent BOD_5 concentration, mg/l.

2.41.8.2.6.2 COD.

$$CODE = 1.5\ BODE$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

2.41.8.2.6.3 Suspended solids.

$$SSE = SS$$

where

SSE = effluent suspended solids concentration, mg/l.

SS = influent suspended solids concentration, mg/l.

2.41.8.2.6.4 Nitrogen.

$$TKNE = (0.1)\ (SSE) + NHE$$
$$NO3E = NO3 + (TKN - TKNE)$$
$$NO2E = NO2$$

where

$TKNE$ = effluent Kjedahl nitrogen concentration, mg/l.

SSE = effluent suspended solids concentration, mg/l.

NHE = effluent ammonia concentration, mg/l.

$NO3E$ = effluent NO_3 concentration, mg/l.

$NO3$ = influent NO_3 concentration, mg/l.

$NO2E$ = effluent NO_2 concentration, mg/l.

$NO2$ = influent NO_2 concentration, mg/l.

2.41.8.2.6.5 pH.

$$PH = 7.2$$

where

PH = effluent pH.

2.41.8.2.6.6 Oil and Grease.

$$OAGE = 0.0$$

where

$OAGE$ = effluent oil and grease concentration, mg/l.

2.41.8.2.6.7 Settleable solids.

$$SETSO = 0.0$$

where

$SETSO$ = effluent settleable solids concentration, mg/l.

2.41.8.2.6.8 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

$PO4E$ = effluent phosphorus concentration, mg/l.

$PO4$ = influent phosphorus concentration, mg/l.

2.41.8.3 Process Design Output Data.

2.41.8.3.1 SRNO, required media surface area, ft^2

2.41.8.3.2 V, volume of media, ft^3

2.41.8.3.3 SA, filter cross-sectional area, ft^2

2.41.8.3.4 D, depth of filter, ft (maximum depth of 28 ft)

2.41.8.3.5 RCY, recycle flow necessary to maintain total surface wetting

2.41.8.3.6 RCR, recycle ratio necessary to maintain total surface wetting

2.41.8.4 Quantities Calculations.

2.41.8.4.1 Select Number of Filters. Field experience indicates that a single filter will be used when the total media volume is less than approximately 50,000 cu ft. Above this value two towers are generally utilized. The surface area of the filter tower is limited by the available sizes of the distribution arms. The distribution arms are generally in the range of 20 to 200 feet in diameter. Thus, the maximum volume of a single trickling filter tower is limited by the diameter of the distributor arms and the depth of the media. Using the following values as the maximum

Diameter of tower = 150 ft

The maximum surface area of a filter tower would be:

$$SA_{max} = (150)^2 \times \frac{1}{4} \times \quad = 17,700 \text{ sq ft}$$

The number of towers per stage, N, would be determined by the following rule:

$\underline{N}$	Surface Area of Single Stage (sq ft)
1	2,000
2	2,000 - 17,700
3	17,700 - 35,400
4	35,400 - 53,100
6	53,100 - 88,500
8	88,500 - 123,900
10	123,900 - 159,300
12	159,300 - 194,700
14	194,700 - 247,800
15	247,800 - 265,500
16	265,500 - 283,200
18	283,200 - 300,900
20	300,900 - Above

2.41.8.4.2 Calculate the volume of the filter tower.

The volume to be handled by one filter, V_N, would be

$$V_N = \frac{V}{N}$$

where

V_N = volume of each individual filter tower, cu ft.

2.41.8.4.3 Calculate the diameter of the filter tower.

$$DIA = 1.128 \left(\frac{V_N}{D}\right)^{0.5}$$

where

DIA = diameter of the filter tower, ft.

D = depth of the tower, given by first order design, ft.

2.41.8.4.4 Trickling Filter Construction.

2.41.8.4.4.1 A typical section of a plastic media trickling filter is shown in Figure 2.41-6. It can generally be divided into four components: the minimum and the outside wall, the distributor arms, the medium support system and the underdrain.

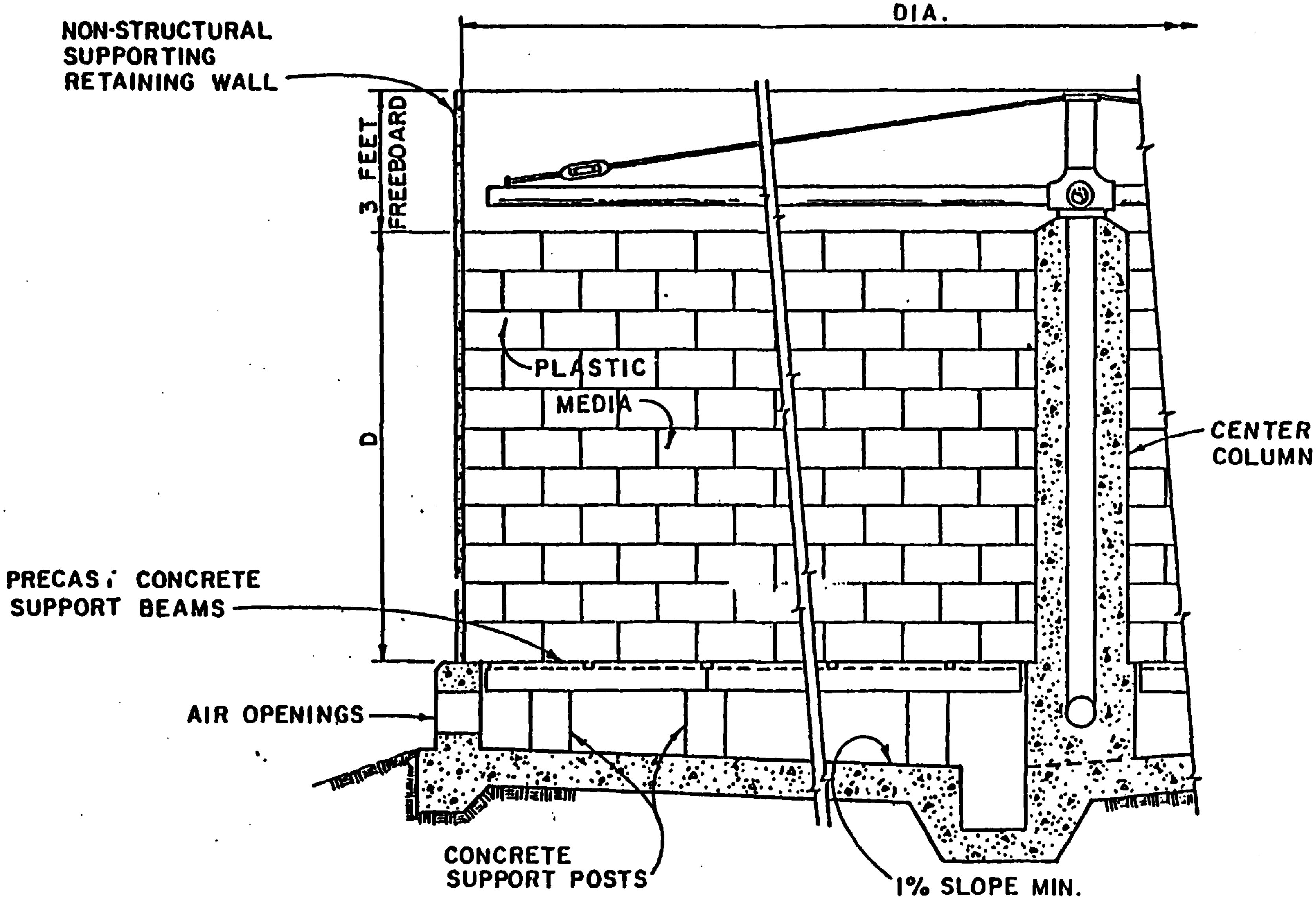


FIGURE 2.41-6 TYPICAL SECTION OF A PLASTIC MEDIA TRICKLING FILTER

2.41.8.4.4.2 The plastic medium usually is supplied and installed by the manufacturers and the cost of the installed system is estimated as dollars/cu ft. Polyester fiberglass, lightweight steel, and precast double-tee constructions have been utilized as the medium containment structure. In this design, it is assumed a 6-inch reinforced concrete wall will be utilized. The distributor arms include the center column and rotary distributors and their support. They are available from several manufacturers with size ranging from 20-feet to 200-feet diameter.

2.41.8.4.4.3 The medium support system consists of precast beams and concrete support posts as shown in Figure 2.41-7.

2.41.8.4.4.4 The underdrain system includes the drainage floor and channel, sidewall with air openings, and center column for distributor support.

2.41.8.4.5 External Wall Construction. A reinforced wall with 6-inch thickness will be assumed for the wall. Thus, the reinforced concrete wall quantity would be

$$V_{cwe} = (D + 3') \times \quad (DIA) \times \frac{6}{12}$$

$$= 1.57\ (D + 3)\ (DIA)$$

where

V_{cwe} = quantity of R.C. wall, cu ft.

D = depth of filter media, from first-order design, ft.

DIA = diameter of the filter tower, ft.

2.41.8.4.6 The Media Support System.

2.41.8.4.6.1 The supporting system consists of concrete supporting posts and precast concrete beams. The number of posts depend on the size of the filter and can be approximated by the following equations:

When DIA $\leq$ 40 ft

$$NCP = 0.00106\ (DIA)^{3.09}$$

When DIA > 40 ft

$$NCP = 0.1739\ (DIA)^{1.935}$$

where

NCP = total number of posts.

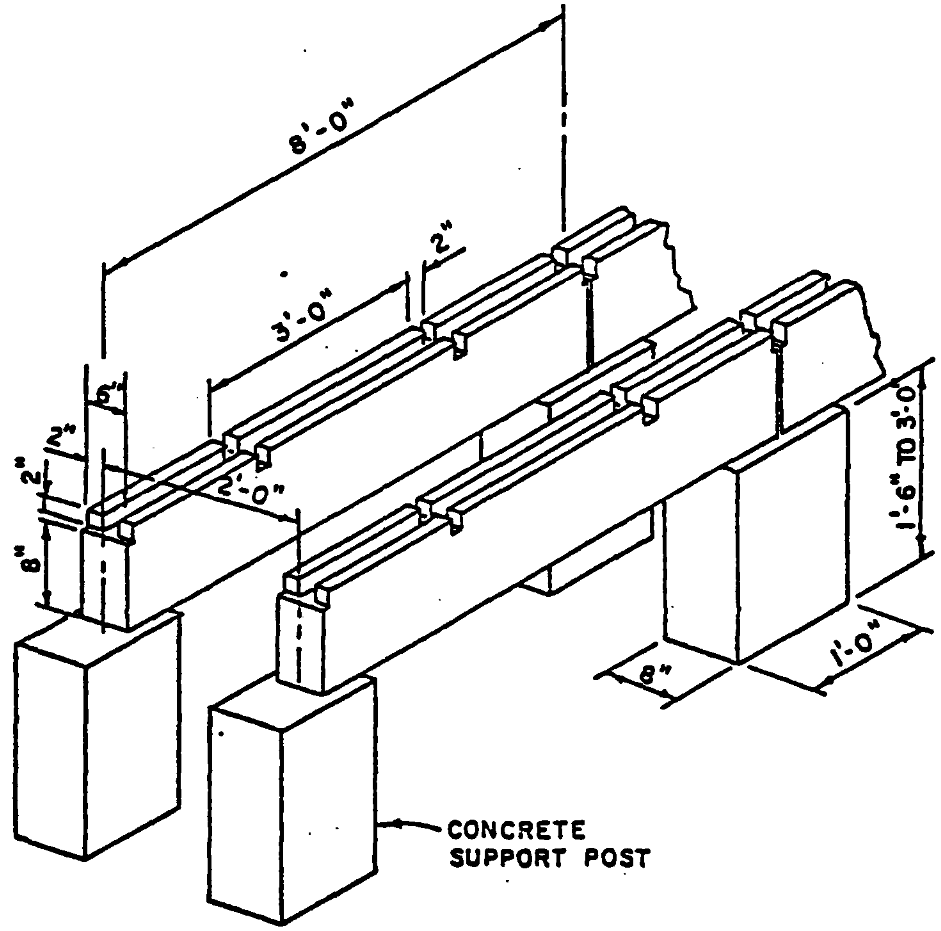


FIGURE 2.41-7. PRECAST CONCRETE BEAM SUPPORT SYSTEM FOR TRICKLING FILTER

2.41.8.4.6.2 With the dimensions shown in Figure 2.41-7 and using an average depth of two feet, the volume of concrete required for the posts would be

$$V_{cwp} = (NCP)\ (\frac{8}{12})\ (\frac{12}{12})\ 2$$
$$= 1.333\ (NCP)$$

where

V_{cwp} = volume of R.C. wall for the supporting post, cu ft.

2.41.8.4.6.3 The precast concrete beam quantity, expressed as total length, can be related to the size of the filter as follows:

When DIA $\leq$ 40 ft

$$LCB = (0.0119)\ (N)\ (DIA)^{2.963}$$

When DIA $>$ 40 ft

$$LCB = (0.364)\ (N)\ (DIA)^{2.035}$$

where

LCB = total length of the precast beams, ft.

2.41.8.4.7 The Underdrain System.

2.41.8.4.7.1 Floor concrete volume, assuming a thickness of 8"

$$V_{csf} = \pi(\frac{DIA}{2})^2\ (\frac{8}{12}) = 0.524\ (DIA)^2$$

where

V_{csf} = volume of R.C. slab, cu ft.

2.41.8.4.7.2 Drainage Channel. The quantity of concrete for the drainage channel can be approximated by:

When DIA $<$ 70 ft

$$V_{cwd} = (10)\ (DIA)$$

When DIA $\geq$ 70 ft

$$V_{cwd} = (17)\ (DIA)$$

where

V_{cwd} = volume of R.C. wall, cu ft.

2.41.8.4.7.3 The center column for the distributor arms. The quantity of concrete for the column can be estimated by:

When DIA < 70 ft

$$V_{cwc} = 4\ (D+2)$$

When DIA $\geq$ 70 ft

$$V_{cwc} = 16\ (D+2)$$

where

V_{cwc} = volume of concrete for the center column, cu ft.

2.41.8.4.7.4 The outside ring wall quantity.

$$V_{cwr} = 4 \cdot \pi \cdot (DIA) \cdot (1)$$
$$= 12.6\ (DIA)$$

where

V_{cwr} = volume of concrete for outside ring wall, cu ft.

2.41.8.4.8 Earthwork. The volume of earthwork can be estimated by:

$$V_{ewn} = (1.15) \times [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ewn} = earthwork required for the construction of a single tower, cu ft.

1.15 = safety factor for conservative design.

2.41.8.4.9 Total reinforced concrete wall quantity, V_{cw}.

$$V_{cw} = N\ [V_{cwe} + V_{cwp} + V_{cwd} + V_{cwc} + V_{cwr}]$$

where

V_{cw} = total quantity of R.C. wall in place, cu ft.

N = number of trickling filter towers.

2.41.8.4.10 Total reinforced concrete slab in place, V_{cs}.

$$V_{cw} = N \cdot V_{csf}$$

where

V_{cs} = total quantity of R.C. slab in place, cu ft.

2.41.8.4.11 Total earthwork required, V_{ew}.

$$V_{ew} = N \cdot V_{ewn}$$

where

V_{en} = total earthwork required, cu ft.

2.41.8.4.12 Electrical energy required for operation. The electric energy required for operation depends on the flow to be pumped and the total dynamic head. It can be estimated by the following equation.

$$KWH = 21{,}000\ (Q_{avg})^{0.961}$$

where

KWH = electric energy required, kwhr/yr.

Q_{avg} = average daily flow, mgd.

2.41.8.4.13 Operation and maintenance manpower requirement.

2.41.8.4.13.1 Operation man-hours required, OMH.

When $Q_{avg} \leq 1.0$ mgd

$$OMH = 128\ (Q_{avg})^{0.301}$$

When $1.0 < Q_{avg} \leq 10$ mgd

$$OMH = 128\ (Q_{avg})^{0.6088}$$

When $Q_{avg} > 10$ mgd

$$OMH = 68\ (Q_{avg})^{0.8861}$$

where

OMH = operational manpower requirement, man-hour/yr.

Q_{avg} = average daily flow, mgd.

2.41.8.4.13.2 Maintenance man-hour requirement, MMH.

When $Q_{avg} \leq 1.0$ mgd

$$MMH = 112\ (Q_{avg})^{0.2430}$$

When $1.0 < Q_{avg} \leq 10$ mgd

$$MMH = 112\ (Q_{avg})^{0.6021}$$

When $Q_{avg} > 10$ mgd

$$MMH = 80\ (Q_{avg})^{0.7066}$$

where

MMH = maintenance man-hours required, man-hour/yr.

2.41.8.4.14 Other operation and maintenance material costs. This item includes repair and replacement material costs. It is expressed as a percent of total installed cost of equipment which includes the plastic media and distributor arms.

$$OMMP = 1\%$$

OMMP = percent of the installed equipment costs for the operation and maintenance material costs.

2.41.8.4.15 Other minor construction items. Items such as piping, walkways around the towers, and site cleaning would be approximately 20 percent of the total construction costs.

CF, the correction factor for this minor cost would be

$$\frac{1}{0.8} = 1.25$$

2.41.8.5 <u>Quantities Calculations Output Data.</u>

2.41.8.5.1 Total volume of media, V_d, cu ft.

2.41.8.5.2 Number of filter towers, N.

2.41.8.5.3 Total R.C. wall in-place, V_{cw}, cu ft.

2.41.8.5.4 Total R.C. slab in-place, V_{cs}, cu ft.

2.41.8.5.5 Total earthwork, V_{ew}, cu ft.

2.41.8.5.6 Total length of precast concrete media support beam, LCB, ft.

2.41.8.5.7 Operational manpower requirement, OMH, man-hour/yr.

2.41.8.5.8 Maintenance manpower requirement, MMH, man-hour/yr.

2.41.8.5.9 Electric energy requirement, KWH, kwhr/yr.

2.41.8.5.10 Other operation and maintenance material costs, OMMP, percent.

2.41.8.5.11 Correction factor for other capital costs, CF.

2.41.8.6 Unit Price Input Required.

2.41.8.6.1 Cost of earthwork, UPIEX, $/cu yd.

2.41.8.6.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.41.8.6.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.41.8.6.4 Unit price of filter media, UPFM, $/cu yd.

2.41.8.6.5 Unit price for a 50-foot diameter distributor system, CODAS, $ (optional).

2.41.8.7 Cost Calculations.

2.41.8.7.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \times UPIEX$$

where

COSTE = costs of earthwork, $.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, $/cu yd.

2.41.8.7.2 Cost of reinforced concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cw}}{27} \times UPICW$$

where

COSTCW = cost of R.C. wall in-place, $.

V_{cw} = total quantity of R.C. wall, cu ft.

UPICW = unit price input of R.C. wall in-place, $/cu yd.

2.41.8.7.3 Cost of R.C. slab in-place, COSTCS.

$$COSTCS = \frac{V_{cs}}{27} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, $.

V_{cs} = volume of R.C. slab, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.41.8.7.4 Cost of the filter media, COSTFM.

$$COSTFM = V_d \cdot UPFM$$

where

COSTFM = cost of installed filter media, \$.

V_d = volume of plastic medium, cu ft.

UPFM = unit price of plastic media installed, \$/cu ft.

This media will be a high specific area media. (specific surface area must be larger than 41 sq ft/cu ft).

The 1st quarter 1977 price for UPFM is \$3.00/cu ft. For a better estimate this unit price should be obtained from equipment vendor.

2.41.8.7.5 Cost of distributor arms.

2.41.8.7.5.1 Purchase cost of distributor arms. The purchase cost of distributor arms can be obtained by using the following equation:

$$CODA = CODAS \cdot CRIODA$$

where

CODA = purchase cost of distributor arm with diameter of DIA ft, \$.

CODAS = purchase cost of a standard sized distributor arm with diameter of 50 feet.

2.41.8.7.5.2 The 1st quarter 1977 price for CODAS is \$39,000. However, for a better estimate, CODAS should be obtaiend from equipment vendors and treated as a unit price input. Otherwise, the following equation will be utilized for cost escalation purposes:

$$CODAS = 39{,}000 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment cost Index from input.

491.6 = MSECI value, 1st quarter 1977.

CROIDA = ratio of cost of distributor arm with diameter of DIA feet to that of the standard size arm.

and CROIDA can be estimated by:

$$CROIDA = 0.367 + 0.01265\ (DIA)$$

2.41.8.7.5.3 Installation costs. An additional 32 percent of the purchase costs would be added for the installation costs.

2.41.8.7.5.4 Total installed costs for the distributor arms, ICODA.

$$ICODA = (N)\ (CODA)\ (1.32)$$

where

ICODA = total installed cost for the distributor arm system, \$.

2.41.8.7.6 Cost of the precast concrete media support beams:

2.41.8.7.6.1 The unit cost of the precast concrete beams can be approximated by using five times the unit cost of reinforced concrete wall in-place. The quantity of precast beam is V_{pcb}.

$$V_{pcb} = \left(\frac{6}{12}\right)\ \left(\frac{8}{12}\right)\ (LCB)$$

where

LCB = total length of beams from second order design output, ft.

$\frac{6}{12}$ = width of the beam, ft.

$\frac{8}{12}$ = length of the beam, ft.

2.41.8.7.6.2 Cost of precast concrete beam, CPCB.

$$CPCB = \frac{5}{27}\ (V_{pcb})\ (UPICW)$$

where

CPCB = cost of precast concrete beams, \$.

UPICW = unit price input of R.C. wall in-place, \$/cu yd.

2.41.8.7.7 Total bare construction cost of the trickling filter system, \$.

$$TBCC = CF\ (COSTE + COSTCW + COSTCS + COSTFM + ICODA + CPCB)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor construction cost, from second order design output.

2.41.8.7.8 Operation and maintenance material cost, OMMC. Since O&M material cost is estimated as a percent of the installed equipment cost, it is thus:

$$OMMC = \frac{OMMP}{100} (COSTFM + ICODA)$$

where

OMMC = operation and maintenance material cost, \$/yr.

OMMP = O&M material cost as percent of equipment cost, %.

2.41.8.8 Cost Calculations Output Data.

2.41.8.8.1 Total bare construction cost of the trickling filter system, TBCC, \$.

2.41.8.8.2 Operation and maintenance material cost, OMMC, \$/yr.

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2.43 OXIDATION DITCH

2.43.1 Background. The oxidation ditch, developed in the Netherlands, is a variation of the extended aeration process that has been used in small towns, isolated communities, and institutions in Europe and the United States. The typical oxidation ditch (Figure 2.43-1) is equipped with aeration rotors or brushes that provide aeration and circulation. The sewage moves through the ditch at 1 to 2 fps. The ditch may be designed for continuous or intermittent operation. Because of this feature, this process may be adaptable to the fluctuations in flows and loadings associated with recreation area wastewater production.

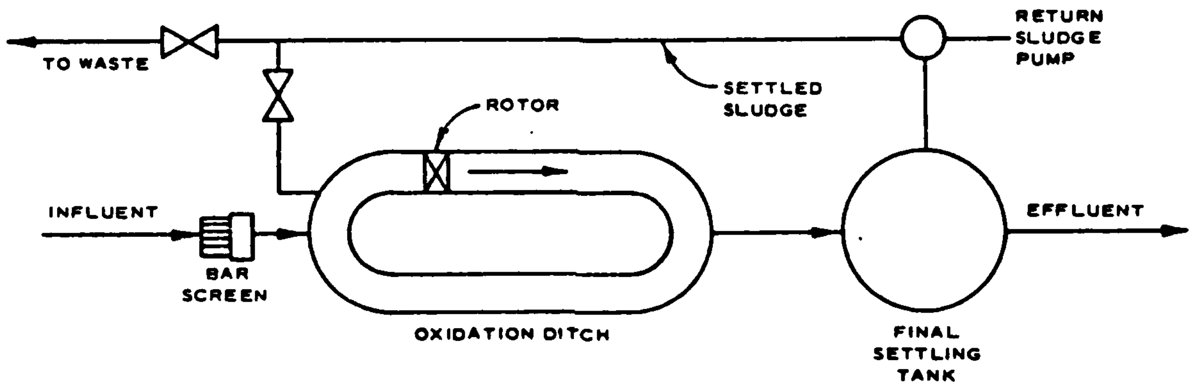


Figure 2.43-1. Typical Oxidation Ditch

2.43.2 Input Data.

2.43.2.1 Wastewater flow.

2.43.2.1.1 Average, mgd.

2.43.2.1.2 Peak hourly, mgd.

2.43.2.2 Wastewater characteristics.

2.43.2.2.1 BOD_5 (average and peak).

2.43.2.2.2 COD (average and peak).

2.43.2.2.3 SS (average and peak).

2.43.2.2.4 VSS (average and peak).

2.43.2.2.5 Nondegradable VSS (average and peak).

2.43.3 Design Parameters.

2.43.3.1 MLSS, mg/l, (Range 4000-8000; mean 5000).

2.43.3.2 Organic loading (F/M ratio), lb BOD_5/lb MLVSS/day, (0.067).

2.43.3.3 Volumetric loading, lb BOD_5/1000 ft^3/day, (12.5).

2.43.3.4 Recycle ratio, (50-100 percent).

2.43.3.5 Oxygen requirement, lb O_2 removed, (2.35).

2.43.3.6 Wasted sludge, lb/lb BOD_5 removed, (0.68).

2.43.3.7 Effluent quality, excellent, approximately 90-95% BOD and S.S. reduction.

2.43.4 <u>Process Design Calculations</u>.

2.43.4.1 Calculate Ditch Volume.

$$V = \frac{Q_{avg} \times S_o \times 8.34 \times 1000}{12.5}$$

where

V = volume of ditch, cu ft.

Q_{avg} = average daily flow, mgd.

S_o = BOD_5 in influent, mg/l.

12.5 = loading rate, lb BOD_5/1000 cu ft/day.

2.43.4.2 Calculate Oxygen Requirements.

$$O_2 = 2.35 \times Q_{avg} \times S_o \times 8.34$$

where

O_2 = oxygen required, lb/day.

2.35 = oxygen utilization, lb O_2/lb BOD_5 applied.

2.43.4.3 Effluent Quality.

2.43.4.3.1 Suspended solids. The effluent suspended solids are specified by the user.

2.43.4.3.2 BOD_5.

$$BODE = Se + .84 f' (X_V)$$

where

BODE = effluent BOD_5 concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

f' = degradable fraction of MLVSS.

X_V = effluent volatile suspended solids concentration, mg/l.

2.43.4.3.3 COD.

$$CODE = 1.5\ BODE$$

$$CODES = 1.5\ Se$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODES = effluent soluble COD concentration, mg/l.

Se = effluent soluble BOD_5 concentration, mg/l.

2.43.4.3.4 Nitrogen.

$$TKNE = (0.4)\ TKN$$
$$NH3E = TKNE$$
$$NO3E = (0.4)\ (TKNE)$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

NO2 = influent NO_2 concentration, mg/l.

2.43.4.3.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.43.4.3.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.43.4.4 Sludge Production. For municipal wastewater only.

biomass: 0.18 lb per lb of BOD_5 removed

$$X_v = 0.18 \times (S_o - S_e) \times Q_{avg} \times 8.34$$

Inert Mass: 0.50 lb per lb of BOD_5 removed

$$X_o = 0.50 \times (S_o - S_e) \times Q_{avg} \times 8.34$$

Total sludge produced

$$X_a = X_v + X_o = 0.68 \times (S_o - S_e) \times Q_{avg} \times 8.34$$

where

X_a = total sludge production, lb/day.

X_v = biomass wasted per day, lb/day.

X_o = inert mass wasted per day, lb/day.

S_o = influent BOD_5, mg/l

S_e = effluent BOD_5, mg/l

Q_{avg} = averaged daily flow, mgd.

2.43.5 Process Design Output Data.

2.43.5.1 Volume of aeration basin required, V, cu ft.

2.43.5.2 Oxygen requirement, O_2, lb/day.

2.43.5.3 Effluent BOD_5, S_e, mg/l.

2.43.5.4 Effluent suspended solids, (SS), eff., mg/l.

2.43.5.5 Sludge production, X_a, lb/day.

2.43.6 Quantities Calculations.

2.43.6.1 Assumptions for quantities calculations.

2.43.6.1.1 The applicable flow range of the oxidation ditch for the treatment of domestic waste is considered to be 0.5 to 10.0 mgd in this manual.

2.43.6.1.2 Based on field experience, it will be assumed that for flows of 0.5 mgd to 1.0 mgd a single ditch will be utilized, while for flows 1.0 mgd to 10.0 mgd two ditches will be used. The shape and construction of the single and double ditches are sketched in Figure 2.43-2 and 2.43-3, respectively.

2.43.6.1.3 For the range of applicability assumed, a 42-inch diameter rotor will be appropriate over the entire range. Figure 2.43-4 shows the oxygen transfer efficiency and power consumption rate for the 42-inch diameter rotor. For design purposes, an 8-inch submergence is assumed for oxygenation.

2.43.6.2 Rotor selection.

2.43.6.2.1 For oxygenation purposes, length required.

$$LRTO = \frac{O_2}{(3.74)\ (24)}$$

where

LRTO = length of rotor required for oxygenation, ft.

O_2 = oxygen required, lb/day.

3.74 = oxygen transfer of 42 inch diameter rotor with 8 inch submergence, $\frac{O_2\ \text{lb/hr}}{\text{ft}}$.

24 = conversion, hrs/day.

2.43.6.2.2 For complete mix purposes, length required.

$$LRTM = \frac{(7.48)(V)}{21{,}000}$$

where

LRTM = length of rotor required for mixing, ft.

V = volume of basin, cu ft.

7.48 = conversion factor, gal/cu ft.

21,000 = mixing capacity, gal/ft of rotor.

2.43.6.2.3 Design length of rotor, LRT, is the larger of the two LRTO and LRTM.

2.43.6.2.4 For selection of the number of rotors per oxidation ditch, assume the following design procedure:

Total Volume of Each Basin, cu ft	Number of Rotors per basin, K
0 - 332,000	2
332,000 - 498,000	3
498,000	4

2.43.6.2.5 Selection of length of individual rotor, LRTK.

$$LRTK = \frac{LRT}{N \times K}$$

where

LRTK = individual rotor length, ft.

LRT = total design length of rotor required, ft.

N = number of basins.

K = number of rotors per basin.

If LRTK > 50 feet, use one more rotor than the table suggests and recalculate LRTK.

2.43.6.2.6 Motor horsepower required for rotor. The following table gives the motor horsepower required for the operation of the rotor.

LRTK	HPK
7 - 10	15
11 - 13	20
14 - 16	30
17 - 22	40
23 - 27	50
28 - 30	60
> 30	75

HPK: HP of electrical motor for each individual rotor.

2.43.6.3 **Basin Design and Calculations.**

2.43.6.3.1 The ditch bottom width W_b is determined by the length of the rotor.

$$LRTK \le 15.5', \ W_b = LRTK + 1$$

$$LRTK > 15.5', \ W_b = LRTK + 4$$

2.43.6.3.2 Single Basin Design, Figure 2.21-2 configuration will be assumed.

Assume:

Basin water depth, D_s = 6 ft
Basin bottom width, W_b = (as described above)
45 degree side walls
Basin water surface width, $W_s = W_b + 6$
Freeboard = 1.0 ft
Median strip width = 1.0 ft

Volume of circular ends, V_e

$$V_e = 18.85\ W_b^2 + 150.8\ W_b + 282.7$$

Volume of straight sections, V_s

$$V_s = 36L_s + 12\ L_s W_b$$

where

L_s = length of straight, ft.

Length of straight section, L_s

$$L_s = \frac{V - V_e}{36 + 12\ W_b}$$

where

V = volume required, cu ft.

Total length, L_t, and width, W_t, including freeboard

$$L_t = L_s + 2\ W_b + 14$$

$$W_t = 2\ W_b + 15$$

2.43.6.3.3 Two basin design, Figure 2.43-3 configuration will be assumed.

Assume:

Basin water depth, D_s = 12 ft
Basin top width = W_b ft
Freeboard = 1.5 ft
Volume of each ditch, $V_d = \frac{V}{2}$

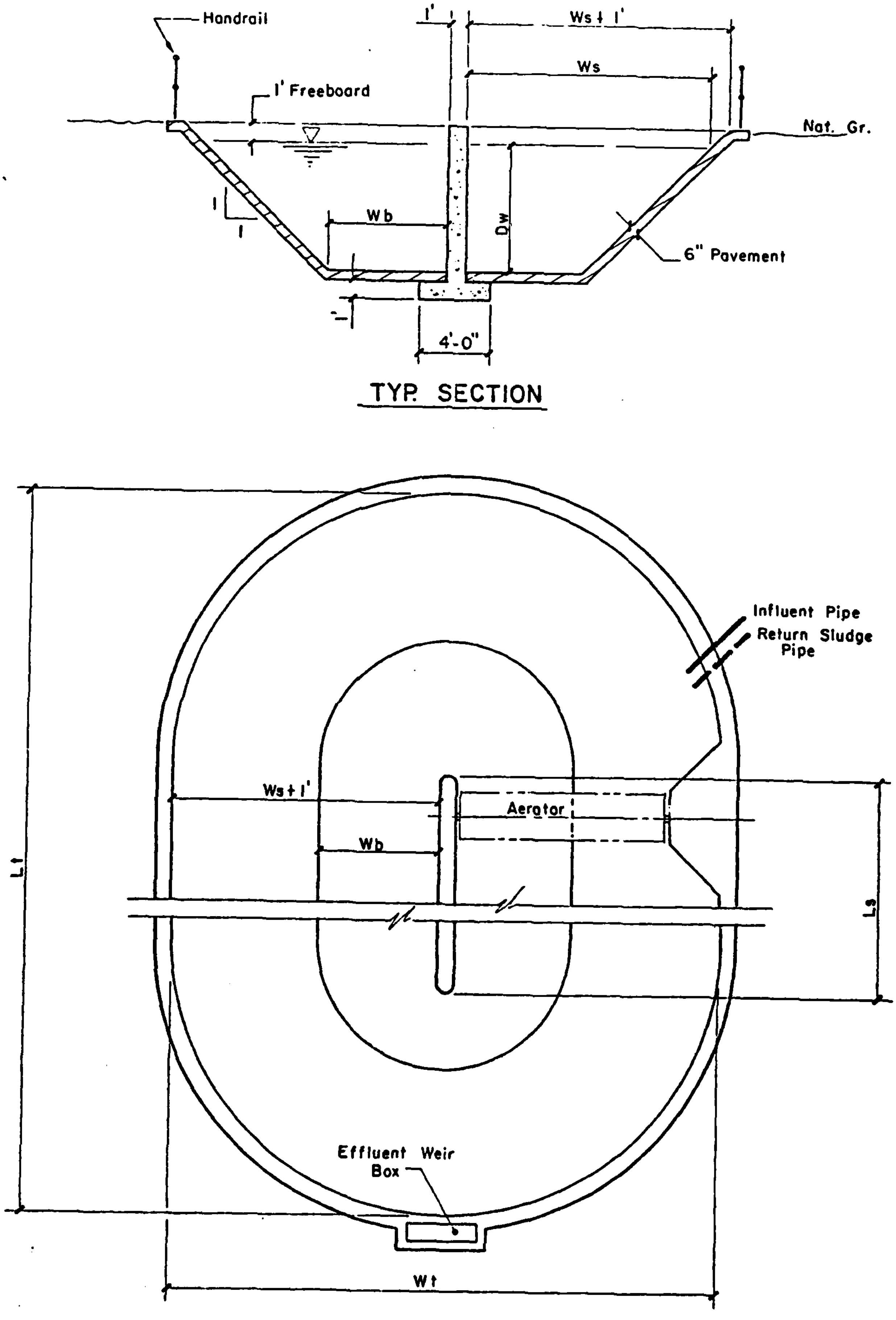


FIGURE 2.43-2. SINGLE OXIDATION DITCH

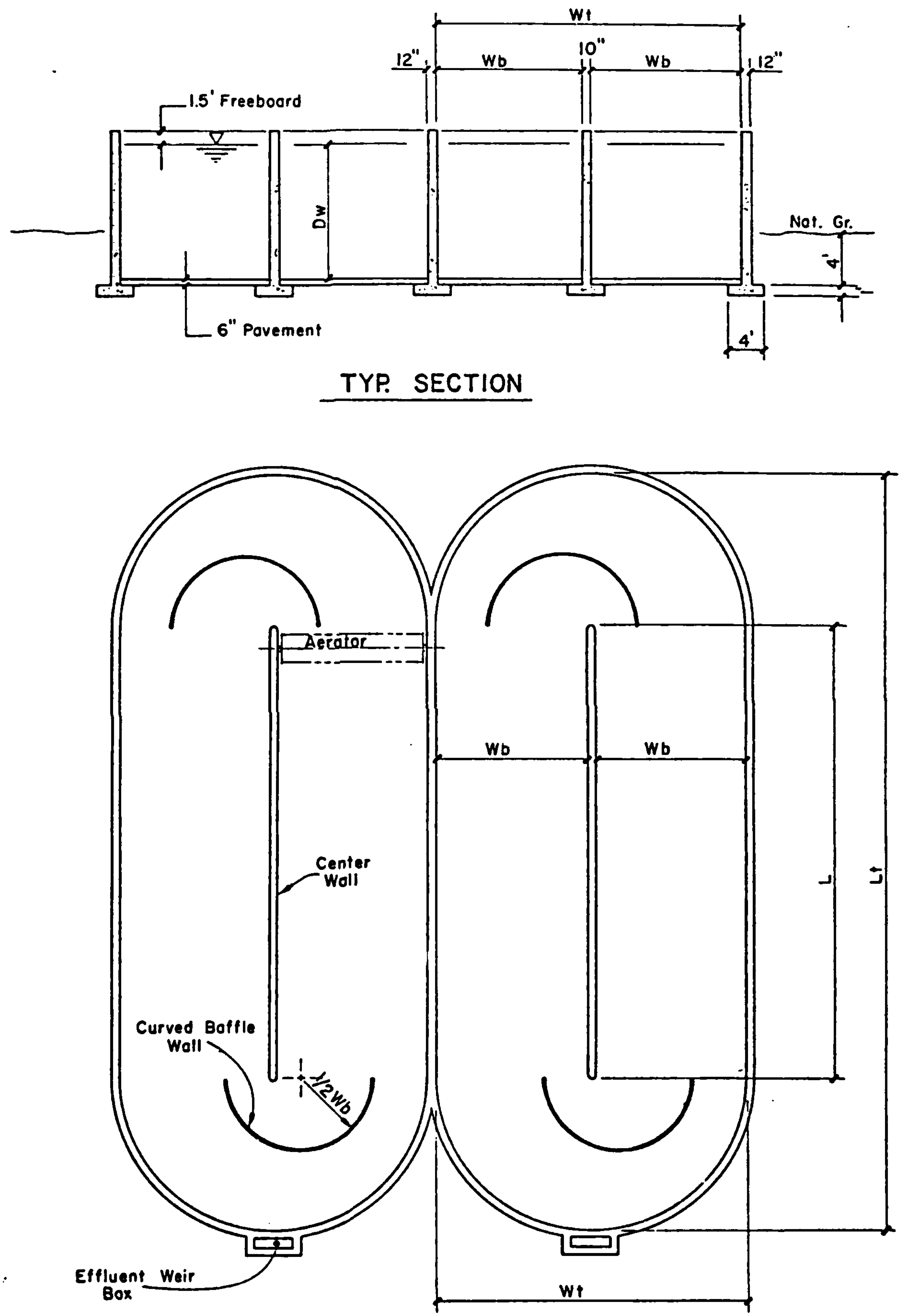
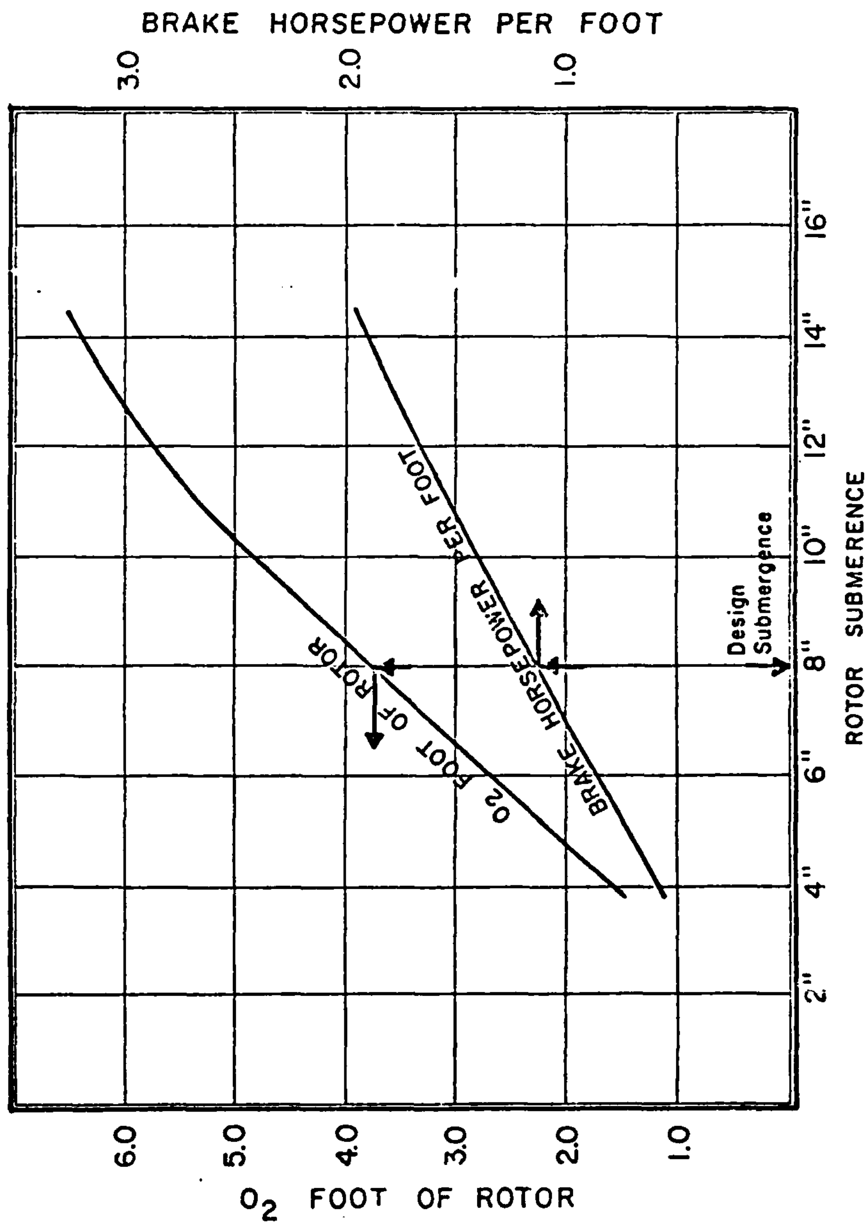


FIGURE 2.43-3. DOUBLE OXI'ATION DITCH



(The curves reflect a saftey factor added for design purposes)

FIGURE 2.43-4. 42" ∅ ROTOR CXYGENATION CURVES
72 RPM ROTATING SPEED

Volume of circular ends, V_e

$$V_e = 37.7\ W_b^{\ 2} + 18.9\ W_b$$

Volume of straight section, V_s

$$V_s = V_b - V_e$$

Straight volume length, L_s

$$L_s = \frac{V_s}{2 \times 12\ W_b}$$

Total ditch length $L_t = L_s + 2\ W_b$

Total ditch width $W_t = 2\ W_b + 1$

2.43.6.4 Quantity of earthwork.

2.43.6.4.1 When N = 1, earth excavation would be the volume of the ditch.

$$V_{ex} = 1.1\ (14\ L_s W_b + 56\ L_s + 44\ W_b^{\ 2} + 426.6\ W_b + 1002.8)$$

where

V_{ex} = volume of excavation, cu ft.

1.1 = add 10% for contingency.

2.43.6.4.2 When N = 2, more earth than the basin volume is to be excavated for ease of concrete forming and safety of workers. Backfill is also required.

$$V_{ex} = 17.6\ (2\ W_t + L_t + 20) + 5\ (2\ W_t + 10)\ (L_t + 10)$$

$$V_{bf} = 16\ (2\ W_t + L_t + 20)$$

where

V_{ex} = volume of earth excavated, cu ft.

V_{bf} = volume of earth to be backfilled, cu ft.

2.43.6.5 Quantity of concrete in-place.

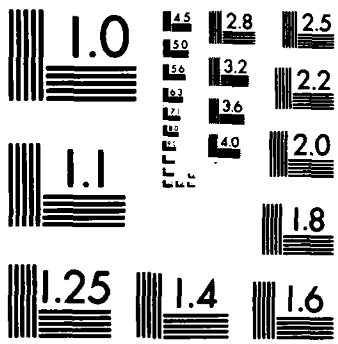
2.43.6.5.1 When N = 1

Concrete wall in-place, V_{cw}, cu ft

$$V_{cw} = 7\ L_s$$

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL. 13/18

UNCLASSIFIED JAN 82 F/G 5/3 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Concrete slab in-place, V_{cs}, cu ft

$$V_{cs} = 4\ L_s + (W_bL_s + 10.5\ L_s + 1.57\ W_b^2 + 21.6\ W_b + 45.4)$$

2.43.6.5.2 When N = 2

Concrete wall in-place, V_{cw}, cu ft.

$$V_{cw} = 63\ L_s + 340.1\ W_b + 85.1$$

Concrete slab in-place, V_{cs}, cu ft

$$V_{cs} = 1.33\ W_bL_s + 20.7\ L_s + 2.1\ W_b^2 + 52.5\ W_b + 25.72$$

2.43.6.6 Adjustable effluent weir.

2.43.6.6.1 Length of effluent weir, LW, ft.

$$LW = \frac{3 \times Q_{avg} \times 10^6}{N \times 66.1 \times 1440}$$

where

LW = length of adjustable weir, ft.

Q_{avg} = average daily flow, mgd.

66.1 = flow can be handled by one foot of weir, gpm.

2.43.6.6.2 Number of adjustable weirs is the same as number of ditches, N.

2.43.6.6.3 Concrete work associated with weir, V_{cew}, cu ft.

$$V_{cew} = 20 + 4\ (LW)$$

2.43.6.7 Quantity of Handrailing for Safety. It is assumed that only the slope side wall construction requires handrailing for safety; the straight side wall construction does not due to the fact that the above-ground wall construction provides enough barricade for safety of workers.

2.43.6.7.1 N = 1. Handrail length LHR; feet is the length of outside wall.

$$LHR = 2\ L_s + 6.28\ W_b + 47.1$$

2.43.6.7.2 When N = 2,

$$LHR = 0$$

2.43.6.8 Operation and Maintenance Manpower Requirement. Limited information is available in the literature. Data are especially rare for larger installations. The curve provided by "Estimating Staffing for Municipal Treatment Facilities", published by EPA, March 1973, is used here.

$$OMMH = 1000\ Q_{avg}^{0.544}$$

where

OMMH = operation and maintenance man-hours required per year, MH/yr.

Q_{avg} = average daily flow of wastewater, mgd.

2.43.6.9 Energy Requirement for Operation. The operation energy for the rotor with 8-inch submergence is 1.15 brake hp per foot of rotor. The operation energy for the total system is:

$$KWH = N \times K \times LRTK \times 1.15 \times 0.85 \times 24 \times 365$$

where

N = number of ditches.

K = number of rotors per ditch.

LRTK = length of each individual rotor, ft.

KWH = KWH per year, electrical energy requirement for operation.

1.15 = brake hp set foot of rotor.

0.85 = conversion factor from hp-hr to kwhr.

2.43.6.10 Material and Supply Costs for Operation and Maintenance.

2.43.6.10.1 Material and supply costs include such items as lubrication oil, paint, and repair materials, etc. These costs are estimated as a percent of installed costs for the aeration equipment. (No information is available specifically for rotor; it is assumed that material supply costs of operation and maintenance for rotor can be approximate by the equation for mechanical surface aerators):

$$OMMP = 4.225 - 0.975 \log (THP)$$

where

OMMP = percent of the installed equipment costs such as O&M material.

THP = total installed hp = (N) (K) (hpK).

2.43.6.11 Other construction cost items.

2.43.6.11.1 From the above calculation approximately 95 percent of the construction cost has been accounted for.

2.43.6.11.2 Other minor items such as piping, site cleaning, effluent weir, etc., would be approximately 5 percent.

2.43.6.11.3 (CF) correction factor would be $\frac{1}{0.95} = 1.052$.

2.43.7 <u>Quantities Calculations Output Data.</u>

2.43.7.1 Number of ditches, N.

2.43.7.2 Number of rotors per ditch, K.

2.43.7.3 Length of each individual rotor, LRTK, ft.

2.43.7.4 Sizes of each individual rotor, hpK.

2.43.7.5 Basin water depth, D_s, ft.

2.43.7.6 Basin bottom width, W_b, ft.

2.43.7.7 Total ditch length, L_T, ft.

2.43.7.8 Total ditch width, W_T, ft.

2.43.7.9 Earthwork volume for excavation, V_{ex}, cu ft.

2.43.7.10 Earthwork volume for backfill, V_{bf}, cu ft.

2.43.7.11 Quantity of concrete wall in-place, V_{cw}, cu ft.

2.43.7.12 Quantity of concrete slab in-place, V_{cs}, cu ft.

2.43.7.13 Quantity of concrete wall for weir, V_{cew}, cu ft.

2.43.7.14 Lenght of effluent adjustable weir, LW, ft.

2.43.7.15 Operation and maintenance man-hour per year, OMMH.

2.43.7.16 Handrail quantity, LHR, ft.

2.43.7.17 Electrical energy requirement for operation, KWH, kwhr/yr.

2.43.7.18 Percentage of installed equipment costs for O&M, percent. OMMP, percent.

2.43.7.19 Correction factor for other minor costs, CF.

2.43.8 <u>Unit Price Input Required</u>.

2.43.8.1 Cost of earthwork, UPIEX, $/cu yd.

2.43.8.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

2.43.8.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

2.43.8.4 Standard size rotor cost, COSTDS, $ (optional).

2.43.8.5 Equipment installation labor rate, $/man-hour.

2.43.8.6 Cost of handrail per lineal foot, UPIHR, $/L.F.

2.43.8.7 Current Marshall & Swift Equipment Cost Index, MSECI.

2.43.9 <u>Cost Calculations</u>.

2.43.9.1 Cost of earthwork.

2.43.9.1.1 Total earthwork, V_{et}.

$$V_{et} = V_{ex} + V_{bf}$$

2.43.9.1.2 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{et}}{27} \times UPIEX$$

2.43.9.2 Cost of rienforced concrete wall in-place.

2.43.9.2.1 Total quantity of concrete wall in-place, V_{cwt}, cu ft.

$$V_{cwt} = V_{cw} + V_{cew}$$

2.43.9.2.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = UPICW \times \frac{V_{cwt}}{27}$$

2.43.9.3 Cost of rienforced concrete slab in-place.

$$COSTCS = UPICS \times \frac{V_{cs}}{27}$$

2.43.9.4 Cost of installed equipment.

2.43.9.4.1 Purchase cost of rotors. The purchase cost of rotors can be obtained by using the following equation:

$$COSTRK = COSTDS \cdot COSTRO$$

where

COSTRK = purchase cost of rotor with length of LRTK feet, \$.

COSTDS = purchase cost of standard size rotor. Rotor with 42-inch diameter and 20-foot length, \$.

COSTRO = ratio of cost of rotor with length of LRTK, feet, and the cost of the standard size rotor.

2.43.9.4.2 COSTRO. The relationship between COSTRO vs. LRTK can be obtained by:

2.43.9.4.2.1 If 6 ft $\leq$ LRTK $\leq$ 20 ft.

$$COSTRO = 10^{(-.2672 + 0.001336 \times LRTK)}$$

2.43.9.4.2.2 If 20 ft $<$ LRTK $\leq$ 30 ft.

$$COSTRO = 10^{0.02228\ (LRTK-20)}$$

2.43.9.4.2.3 If LRTK > 30, divide the length by two.

$$LRTK = \frac{LRTK}{2}$$

Substitute into the other equations to find the ratio. The actual cost ratio would be:

$$COSTRO = COSTRO \text{ (for LRTK/2)} \times 1.755$$

2.43.9.4.3 Cost of standard size rotor. The approximate cost of a 42-inch diameter rotor with 20-ft length for the first quarter of 1977 is $15,340. For a better estimation, COSTDS should be obtained from equipment vendor and treated as a unit price input. Otherwise the following equation will be utilized for cost escalation purposes:

$$COSTDS = \$15,340 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = MSECI value, 1st quarter, 1977.

2.43.9.4.4 Equipment installation man-hour requirement. The man-hour requirement for field erection of rotor is a function of the size of rotor and is expressed as:

$$IMH = 10 + 2.667 \times LRTK$$

where

IMH = installation man-hour requirement, man-hours.

Installation cost would be:

$$ICOST = IMH \times LABRI$$

where

ICOST = installation cost, $.

LABRI = installation labor rate, $/man-hour.

2.43.9.4.5 Other minor costs associated with the installed equipment. This category includes the costs for electric works, foundation, bridge, painting and other minor costs. It is expressed as percentage of the equipment purchase cost:

$$PMINC = 52.04 - 0.34 \times LRTK$$

and PMINC is always larger than 40.

PMINC = percentage of purchasing cost of equipment as minor cost, %.

2.43.9.4.6 Installed equipment cost, IEC.

$$IEC = [COSTRK \times (1 + \frac{PMINC}{100}) + ICOST] (N) (K)$$

2.43.9.5 <u>Other Cost Items</u>.

2.43.9.5.1 Cost of handrail can be a major portion of the capital costs of oxidation ditch process.

If N > 1. No handrail is required due to the straight sidewall construction.

$$COSTHR = 0.0$$

If N = 1

$$COSTHR = LHR \times UPIHR$$

where

LHR = handrail quantity, feet.

UPIHR = unit price input for handrail cost, $/L.F.

UPIHR = unit price for aluminum pipe rail anodized UPIHR = $25.20 per linear foot for first quarter, 1977.

2.43.9.5.2 Other cost items such as effluent weir, piping, site work, etc., can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.43.9.6 <u>Total Bare Construction Costs</u>.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR) \times CF$$

where

TBCC = total bare construction cost, $.

2.43.9.7 Operation and maintenance material costs. Since this item of the operation and maintenance costs is expressed as a percentage of installed equipment costs, it can be calculated by:

$$OMMC = IEC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

IEC = installed equipment costs, \$.

OMMP = percentage of installed equipment costs as operation and maitenance material, %.

2.43.10 Cost Calculations Output Data.

2.43.10.1 Total bare construction costs, TBCC, \$.

2.43.10.2 O&M material costs, OMMC, \$/yr.

2.43.11 Bibliography.

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2.43.11.6 Metcalf and Eddy, Inc., Wastewater Engineering: Collection, Treatment, and Disposal, McGraw Hill, New York, 1972.

2.43.11.7 Parker, H.W., Wastewater Systems Engineering, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1975.

2.43.11.8 Patterson and Bunker, "Estimating Cost and Manpower Requirements for Conventional Wastewater Treatment Facilities", EPA Report 17090 DAN. 10/71.

2.43.11.9 Personal Communication, Mr. Arthur P. Malm, Lakeside Equipment Corporation, March 1977.

2.45 POSTAERATION

2.45.1 Background.

2.45.1.1 Many states have established standards which require minimum levels of dissolved oxygen in the effluent from sewage treatment plants. To maintain this desired oxygen level postaeration is often used. Postaeration may be accomplished by diffused or mechanical aeration in separate basins or by cascade aeration.

2.45.2 Cascade Aeration.

2.45.2.1 Input Data.

2.45.2.1.1 Wastewater flow.

2.45.2.1.1.1 Average daily flow, mgd.

2.45.2.1.1.2 Peak flow, mgd.

2.45.2.1.2 Dissolved oxygen concentration of the postaeration influent, mg/l.

2.45.2.1.3 Desired dissolved oxygen concentration in effluent, mg/l.

2.45.2.2 Design parameters.

2.45.2.2.1 O_2 saturation at selected summer temperature, C_s, mg/l.

2.45.2.2.2 Temperature, T, °C.

2.45.2.2.3 Water quality parameter equal to 0.8 for a wastewater treatment plant effluent, n.

2.45.2.2.4 Weir geometry parameter equal to unity for a free wier and 1.3 for step weirs, m.

2.45.2.3 Process Design Calculations.

2.45.2.3.1 Calculate deficit ratio.

$$r = \frac{C_s - C_i}{C_s - C_e}$$

where

r = deficit ratio.

C_s = O_2 saturation at selected summer temperature, mg/l.

C_i = dissolved oxygen concentration of the postaeration influent, mg/l.

C_e = desired dissolved oxygen concentration in effluent, mg/l.

2.45.2.3.2 Calculate head required.

$$h = \frac{r-1}{0.11\ (n)\ (m)\ (1 + 0.046T)}$$

where

h = required head, ft.

r = deficit ratio.

n = water quality parameter equal to 0.8 for wastewater treatment plant effluent.

m = weir geometry parameter equal to unity for free weir and 1.3 for step weirs.

T = temperature, °C.

2.45.2.4 Process Design Output Data.

2.45.2.4.1 Required head, h, ft.

2.45.2.5 Quantities Calculations.

2.45.2.5.1 Assumptions. Several assumptions must be made concerning the construction of the step aerator.

2.45.2.5.1.1 The steps will have a 6" drop with an 18" tread.

2.45.2.5.1.2 There is sufficient relief to provide for the head required.

2.45.2.5.1.3 The step aerator will provide 40 sq ft of surface area per mgd.

2.45.2.5.2 Calculate the number of steps required.

$$NS = \frac{h}{0.5}$$

NS must be an integer.

where

NS = number of steps.

h = required head, ft.

2.45.2.5.3 Calculate width of cascade aerator. Using the surface area criteria of 40 sq ft/mgd the width may be calculated by:

$$W = \frac{(40)\ (Q_{avg})}{(NS-1)\ (1.5) + (NS)\ (0.5)}$$

where

W = width of cascade aerator, ft.

Q_{avg} = average daily flow, mgd.

NS = number of steps.

2.45.2.5.4 Calculate volume of R.C. slab required.

$$V_{cs} = (W)\ (1.5)\ (NS)$$

where

V_{cs} = volume of R.C. slab required, cu ft.

W = width of cascade aerator, ft.

NS = number of steps.

2.45.2.5.5 There are virtually no power, operation, or maintenance costs associated with cascade aeration where sufficient head is available.

2.45.2.5.6 Other minor construction costs. The above calculations account for approximately 90% of the construction cost. The other items such as earthwork special piping, etc., would be about 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs.

2.45.2.6 <u>Quantity Calculations Output Data.</u>

2.45.2.6.1 Number of steps, NS.

2.45.2.6.2 Width of cascade aerator, ft.

2.45.2.6.3 Volume of R.C. slab required, V_{cs}, cu ft.

2.45.2.6.4 Correction factor for other construction costs, CF.

2.45.2.7 Unit Price Input Required.

2.45.2.7.1 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

2.45.2.8 Cost Calculations.

2.45.2.8.1 Calculate cost of R.C. slab.

$$COSTCS = \left(\frac{V_{cs}}{27}\right) UPICS$$

where

COSTCS = cost of R.C. slab in-place, $.

V_{cs} = volume of R.C. slab required.

UPICS = unit price input for R.C. slab in-place, $/cu yd.

2.45.2.8.2 Calculate total bare construction cost.

$$TBCC = (COSTCS)\ (CF)$$

where

TBCC = total bare construction cost, $.

COSTCS = cost of R.C. slab in-place, $.

CF = correction factor for other construction costs, $.

2.45.2.9 Cost Calculations Output Data.

2.45.2.9.1 Total bare construction cost, TBCC, $.

2.45.3 Diffused Aeration.

2.45.3.1 Input Data.

2.45.3.1.1 Wastewater flow.

2.45.3.1.1.1 Average daily flow, mgd.

2.45.3.1.1.2 Peak flow, mgd.

2.45.3.1.2 Dissolved oxygen concentration of the postaeration influent, mg/l.

2.45.3.1.3 Desired dissolved oxygen concentration in effluent, mg/l.

2.45.3.1.4 Detention time, min.

2.45.3.2 Design Parameters.

2.45.3.2.1 Standard transfer efficiency, STE, %, (from manufacturer 5-8 percent).

2.45.3.2.2 O_2 transfer in waste/O_2 transfer in water, α, $\approx$ 0.9.

2.45.3.2.3 O_2 saturation in waste/O_2 saturation in water, β $\approx$ 0.9.

2.45.3.2.4 Correction factor for pressure, P, 1.0.

2.45.3.2.5 Temperature, T, °C.

2.45.3.2.6 O_2 saturation at selected summer temperature, C_s, mg/l.

2.45.3.2.7 Minimum dissolved oxygen to be maintained in the basin, C_L, mg/l.

2.45.3.3 Process Design Calculations.

2.45.3.3.1 Calculate oxygen required.

$$O_2 = (C_e - C_i)\ (Q_{avg})\ (8.34)$$

where

O_2 = oxygen required, lb O_2/day.

C_e = desired dissolved oxygen concentration in effluent, mg/l.

C_i = dissolved oxygen concentration of the postaeration influent, mg/l.

2.45.3.3.2 Calculate operating transfer efficiency.

$$OTE = STE \frac{[(C_s)\ (\beta)\ (p) - C_L]}{9.17} \propto (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, %.

STE = standard transfer efficiency, %.

C_s = 0_2 saturation at selected summer temperature, mg/l.

β = 0_2 saturation in waste/0_2 saturation in water.

$\propto$ = 0_2 transfer in waste/0_2 transfer in water.

p = correction factor for pressure.

C_L = minimum dissolved oxygen to be maintained in the basin, mg/l.

T = temperature, °C.

2.45.3.3.3 Calculate tank volume.

$$V = \frac{(Q_{avg})\ (t)\ (10^6)}{1440}$$

where

V = volume of basin, gal.

Q_{avg} = average daily flow, mgd.

t = detention time, min.

2.45.3.3.4 Calculate required air flow.

Blowers are treated as a separate unit process since several unit processes in a single plant may require air from the blowers. The air requirements from all unit processes in a treatment train which require air are summed and the total air requirement is used to size the blower facility. The unit process design for the blower facility is found in subsection 2.3.

$$R_a = \frac{O_2\ (100)\ (7.48)}{(OTE)\ (0.0176)\ (1440)\ (V)}$$

where

R_a = required air flow, cfm.

O_2 = oxygen required, lb O_2/day.

OTE = operating transfer efficiency, %.

V = volume of basin, gal.

2.45.3.4 Process Design Output Data.

2.45.3.4.1 Oxygen required, O_2, lb O_2/day.

2.45.3.4.2 Operating transfer efficiency, OTE, %.

2.45.3.4.3 Volume of basin, V, gal.

2.45.3.4.4 Required air flow, R_a, cfm.

2.45.3.5 Quantities Calculations.

2.45.3.5.1 Calculate dimensions of basin. Assume for diffused air the basin will be 15 ft deep and 30 ft wide.

$$L = \frac{V}{(7.48)\ (450)}$$

where

L = length of basin, ft.

V = volume of basin, gal.

2.45.3.5.2 Calculate number of diffusers. The flow per diffuser ranges from 10-15 cfm, we will assume a flow of 12 cfm.

$$ND = \frac{R_a}{12}$$

where

ND = number of diffusers.

R_a = required air flow, cfm.

2.45.3.5.3 Calculate number of diffuser headers. It is assumed that swing arm diffuser headers will be used. These headers have 8 to 30 diffuser connections depending upon the manufacturer. An average of 20 diffusers per header will be used.

$$NSA = \frac{ND}{20}$$

where

NSA = number of swing arm diffuser headers.

ND = number of diffusers.

2.45.3.5.4 Calculate volume of reinforced concrete for tank.

Assume the walls and slabs are 9" thick.

2.45.3.5.4.1 Volume of R.C. wall.

$$V_{cw} = (22.5)\ (L) + 675$$

where

V_{cw} = volume of R.C. wall required, cu ft.

L = length of basin, ft.

2.45.3.5.4.2 Volume of R.C. slab required.

$$V_{cs} = (22.5)\ (L)$$

where

V_{cs} = volume of R.C. slab required, cu ft.

L = length of basin, ft.

2.45.3.5.5 Calculate electrical energy required.

$$KWH = (R_a)\ (248.9)$$

where

KWH = electrical energy required, kwhr/yr.

R_a = required air flow, cfm.

2.45.3.5.6 Calculate O&M manpower required.

2.45.3.5.6.1 Operation manpower required.

$$OMH = 509\ (Q_{avg})^{0.1651}$$

where

OMH = operation manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

2.45.3.5.6.2 Maintenance manpower required.

$$MMH = 160\ (Q_{avg})^{0.301}$$

where

MMH = maintenance manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

2.45.3.5.7 Other operating and maintenance material and supply costs. This includes such items as lubrication oil, paint, repair parts, etc.

$$OMMP = 3.57\ (Q_{avg})^{-0.2602}$$

where

OMMP = O&M material and supply costs as percent of total bare construction cost, %.

Q_{avg} = average daily flow, mgd.

2.45.3.5.8 Other minor construction costs. From the calculations approximately 90% of the construction cost has been accounted for. Other items such as miscellaneous piping, earthwork, painting, etc. would be approximately 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs.

2.45.3.6 <u>Quantities Calculations Output Data</u>

2.45.3.6.1 Length of basin, L, ft.

2.45.3.6.2 Number of diffusers, ND.

2.45.3.6.3 Number of swing arm diffuser headers, NSA.

2.45.3.6.4 Volume of R.C. wall required, V_{cw}, cu ft.

2.45.3.6.5 Volume of R.C. slab required, V_{cs}, cu ft.

2.45.3.6.6 Electrical energy required, KWH, kwhr/yr.

2.45.3.6.7 Operating manpower required, OMH, MH/yr.

2.45.3.6.8 Maintenance manpower required, MMH, MH/yr.

2.45.3.6.9 O&M material and supply costs as percent of total bare construction cost, OMMP, %.

2.45.3.6.10 Correction factor for other construction costs, CF.

2.45.3.7 Unit Price Input Required.

2.45.3.7.1 Unit price input for R.C. wall in-place, UPICW, $/cu yd.

2.45.3.7.2 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

2.45.3.7.3 Cost per diffuser, COSTPD, $ (optional).

2.45.3.7.4 Cost per swing arm header, COSTPH, $ (optional).

2.45.3.7.5 Current Marshall and Swift Equipment Cost Index, MSECI.

2.45.3.7.6 Current CE plant cost index for pipe, valves, etc., CEPCIP.

2.45.3.7.7 Equipment installation labor rate, LABRI, $/MH.

2.45.3.7.8 Unit price input for crane rental, UPICR, $/hr.

2.45.3.8 Cost Calculations.

2.45.3.8.1 Calculate cost of R.C. wall.

$$\text{COSTCW} = \left(\frac{V_{cw}}{27}\right) \text{UPICW}$$

where

COSTCW = cost of R.C. wall in-place, $.

V_{cw} = volume of R.C. wall required, cu ft.

UPICW = unit price input for R.C. wall in-place, $/cu yd.

2.45.3.8.2 Calculate cost of R.C. slab.

$$COSTCS = (\frac{V_{cs}}{27})\ UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.45.3.8.3 Cost of diffusers.

2.45.3.8.3.1 The cost of a coarse bubble diffuser with a capacity of 12 scfm for the first quarter of 1977 is:

$$COSTPD = \$6.50$$

For a better estimate COSTPD should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPD = \$6.50\ \frac{MSECI}{491.6}$$

where

COSTPD = cost per diffuser, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter 1977.

2.45.3.8.3.2 Calculate COSTD.

$$COSTD = COSTPD \times ND$$

where

COSTD = cost of diffusers for system, $.

ND = number of diffusers.

2.45.3.8.4 Cost of swing arm diffuser headers.

2.45.3.8.4.1 Swing arm diffuser headers come in several sizes. The cost used is for a header which will handle 550 scfm and up to 37 diffusers.

The cost of this header for the first quarter of 1977 is:

$$COSTPH = \$5{,}000$$

For better estimate COSTPH should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTPH = \$5{,}000 \, \frac{MSECI}{491.6}$$

where

COSTPH = cost per swing arm header, $.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index, first quarter of 1977.

2.45.3.8.4.2 Calculate COSTH.

$$COSTH = COSTPH \times NSA$$

where

COSTH = cost of swing arm headers for system, $.

NSA = number of swing arm headers.

2.45.3.8.5 Equipment installation man-hour requirement. The labor requirement for field installation of the swing arm headers, including mounting the diffusers, is approximately 25 man-hours per header.

$$IMH = 25\ NSA$$

where

IMH = installation man-hour requirement, MH.

2.45.3.8.6 Crane requirement for installation.

$$CH = (.1)(IMH)$$

where

CH = crane time requirement for installation, hr.

2.45.3.8.7 Cost of air piping. The air piping for the diffused aeration system is very complex and includes many valves and fittings of different sizes. This causes cost estimation by material take-off to be very difficult for a wide range of flow. In this case we feel the use of parametric costing is justified as the overall accuracy of the estimate will not be affected to a great extent.

2.45.3.8.7.1 If R_a is between 100 scfm and 1000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 617.2\ (R_a)^{0.2553} \times \frac{CEPCIP}{241.0}$$

where

COSTAP = cost of air piping, \$.

R_a = required air flow, cfm.

CEPCIP = current CE Plant Cost Index for pipe, valves, etc.

241.0 = CE Plant Cost Index for pipe, valves, etc., for first quarter of 1977.

2.45.3.8.7.2 If R_a is between 1000 scfm and 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 1.43\ (R_a)^{1.1337} \times \frac{CEPCIP}{241.0}$$

2.45.3.8.7.3 If R_a is greater than 10,000 scfm, the cost of air piping can be calculated by:

$$COSTAP = 28.59\ (R_a)^{0.8085} \times \frac{CEPCIP}{241.0}$$

2.45.3.8.8 Other costs associated with the installed equipment. This category includes the costs for weir installation, painting, inspection, etc., and can be added as a percentage of the purchase equipment cost:

$$PMINC = 10\%$$

where

PMINC = percentage of purchase costs of equipment as minor installation cost, percent.

2.45.3.8.9 Installed equipment costs.

$$IEC = (COSTD + COSTH + COSTAP)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)$$

where

IEC = installed equipment cost, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/hr.

2.45.3.8.10 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor cost items.

2.45.3.8.11 Operation and maintenance material costs.

$$OMCC = TBCC\ \frac{OMMP}{100}$$

where

OMMC = operation and maintenace material supply costs, \$/yr.

OMMP = operation and maintenance material and supply costs as percent of total bare construction cost, percent.

2.45.3.9 Cost Calculations Output Data.

2.45.3.9.1 Total bare construction cost, TBCC, \$.

2.45.3.9.2 O&M material and supply costs, OMMC, \$/yr.

2.45.4 Mechanical Aeration.

2.45.4.1 Input Data.

2.45.4.1.1 Wastewater flow.

2.45.4.1.1.1 Average daily flow, mgd.

2.45.4.1.1.2 Peak flow, mgd.

2.45.4.1.2 Dissolved oxygen concentration of the postaeration influent, mg/l.

2.45.4.1.3 Desired dissolved oxygen concentration in effluent, mg/l.

2.45.4.1.4 Detention time, min.

2.45.4.2 Design Parameters.

2.45.4.2.1 Standard transfer efficiency, STE, lb/hp-hr.

2.45.4.2.2 O_2 transfer in waste/O_2 transfer in water, α, 0.9.

2.45.4.2.3 O_2 saturation in waste/O_2 saturation in water, β, 0.9.

2.45.4.2.4 Correction factor for pressure, P, 1.0.

2.45.4.2.5 O_2 saturation at summer temperature, C_s, mg/l.

2.45.4.2.6 Minimum dissolved oxygen to be maintained in the basin, C_L, mg/l.

2.45.4.2.7 Temperature, T, °C.

2.45.4.3 Process Design Calculations.

2.45.4.3.1 Calculate oxygen required.

$$O_2 = (C_e - C_1)\ (Q_{avg})\ (8.34)$$

where

O_2 = oxygen required, lb O_2/day.

C_e = desired dissolved oxygen concentration in effluent, mg/l.

C_1 = dissolved oxygen concentration of the postaeration influent, mg/l.

2.45.4.3.2 Calculate operating transfer efficiency.

$$OTE = STE \frac{[(C_s)\ \beta\ (P) - C_L]}{9.17}\ \alpha\ (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

α = O_2 transfer in waste/O_2 transfer in water, 0.9.

β = O_2 saturation in waste/O_2 saturation in water, 0.9.

C_s = O_2 saturation of selected summer temperature, mg/l.

C_L = minimum dissolved oxygen to be maintained in basin, mg/l.

2.45.4.3.3 Calculate volume of basin.

$$V = \frac{(Q_{avg})\ (t)\ (10^6)}{1440}$$

where

V = volume of basin, gal.

Q_{avg} = average daily flow, mgd.

t = detention time, min.

2.45.4.3.4 Calculate horsepower required.

$$HP = \frac{O_2}{OTE\ (24)}$$

where

HP = horsepower required, hp.

O_2 = oxygen required, lb O_2/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

2.45.4.4 <u>Process Design Output Data</u>.

2.45.4.4.1 Oxygen required, O_2, lb O_2/day.

2.45.4.4.2 Volume of basin, V, gal.

2.45.4.4.3 Average daily flow, Q_{avg}, mgd.

2.45.4.4.4 Horsepower required, HP, hp.

2.45.4.5 Quantities Calculations.

2.45.4.5.1 Determine number of aerators required. The number of aerators must be one of the following: 1, 2, 3, 4, 6, 8. Also the aerators must be one of the following sizes, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100, or 150. The selection process will be trial and error.

Assume number of aerators per basin (K) is 1. If $\frac{HP}{K} > 150$, go to next trials K=2, 3, 4, 6, 8 until $\frac{HP}{K} \leq 150$, then compare $\frac{HP}{K}$ with values for individual aerators (HP_a) given above. Select the smallest value of HP_a that is greater than $\frac{HP}{K}$.

2.45.4.5.2 Calculate basin depth. The basin water depth (Dw) is controlled by the aerator sizes used. This is true because each size aerator has a maximum depth at which it can be used and still achieve mixing without the use of a draft tube.

2.45.4.5.2.1 If $HP_a < 100$ Hp

$$DW = 4.82\ (HP_a)^{0.2467}$$

2.45.4.5.2.2 If $100 \leq HP_a \leq 150$

$$DW = 15 \text{ ft}$$

where

DW = basin water depth, ft.

HP_a = horsepower of individual aerators, hp.

2.45.4.5.3 Calculate basin dimensions.

2.45.4.5.3.1 Calculate length to width ratio.

If $K \leq 4$, $r = K$

If $K > 4$, $r = k/2$

where

K = number of aerators.

r = basin length to width ratio.

2.45.4.5.3.2 Calculate length of basin.

$$L = \left[\frac{(V)\ (r)}{(7.48)\ (DW)}\right]^{0.5}$$

where

L = length of basin, ft.

V = volume of basin, gal.

DW = basin water depth, ft.

r = length to width ratio.

2.45.4.5.3.3 Calculate width of basin.

$$W = \frac{L}{r}$$

where

W = width of basin, ft.

L = length of basin, ft.

r = length to width ratio.

2.45.4.5.4 Calculate volume of reinforced concrete required. Assume walls and slab will be 0.75 ft thick.

2.45.4.5.4.1 Calculate volume of R.C. wall required.

$$V_{cw} = [(2)\ (DW + 2)\ (L + W)]\ (0.75)$$

where

V_{cw} = volume of R.C. wall required, cu ft.

DW = basin water depth, ft.

L = length of basin, ft.

W = width of basin, ft.

2.45.4.5.4.2 Calculate volume of R.C. slab required.

$$V_{cs} = (L)\ (W)\ (0.75)$$

where

V_{cs} = volume of R.C. slab required, cu ft.

L = length of basin, ft.

W = width of basin, ft.

2.45.4.5.5 Calculate total horsepower installed.

$$HP_T = (K)\ (HP_a)$$

where

HP_T = total horsepower installed, hp.

HP_a = horsepower of individual aerators, hp.

K = number of aerators required.

2.45.4.5.6 Calculate electrical energy required.

$$KWH = (HP_T)\ (365)\ (24)\ (.9)\ (.85)\ (.877)$$

where

KWH = electrical energy required, kwhr/yr.

HP_T = total horsepower installed, hp.

2.45.4.5.7 Calculate O&M manpower required.

2.45.4.5.7.1 Operation manpower required.

$$OMH = 509\ (Q_{avg})^{0.1651}$$

where

OMH = operation manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

2.45.4.5.7.2 Maintenance manpower required.

$$MMH = 160\ (Q_{avg})^{0.301}$$

where

MMH = maintenance manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

2.45.4.5.8 Other operating and maintenance material and supply cost. This item includes such items as lubrication oil, paint, repair and replacement parts. These costs are estimated as a percent of the installed equipment costs.

$$OMMP = 4.93\ (HP_T)^{-0.1827}$$

where

$OMMP$ = O&M material and supply costs as percent of the installed equipment cost, %.

HP_T = total horsepower installed, hp.

2.45.4.5.9 Other minor construction cost items. From the calculations above approximately 90% of the construction costs have been accounted for. Other minor items such as handrails, piping, paint, etc., would be 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction cost items.

2.45.4.6 Quantities Calculations Output Data.

2.45.4.6.1 Horsepower of individual aerators, HP_a, hp.

2.45.4.6.2 Number of aerators, k.

2.45.4.6.3 Volume of R.C. wall required, V_{cw}, cu ft.

2.45.4.6.4 Volume of R.C. slab required, V_{cs}, cu ft.

2.45.4.6.5 Total horsepower installed, HP_T, hp.

2.45.4.6.6 Electrical energy required, KWH, kwhr/yr.

2.45.4.6.7 Operation manpower required, OMH, MH/yr.

2.45.4.6.8 Maintenance manpower required, MMH, MH/yr.

2.45.4.6.9 O&M material and supply costs as percent of the installed equipment cost, OMMP, %.

2.45.4.6.10 Correction factor for other minor construction costs, CF.

2.45.4.7 <u>Unit Price Input Required</u>.

2.45.4.7.1 Unit price input for R.C. wall, UPICW, \$/cu yd.

2.45.4.7.2 Unit price input for R.C. slab, UPICS, \$/cu yd.

2.45.4.7.3 Equipment installation labor rate, LABRI, \$/MH.

2.45.4.7.4 Current Marshall and Swift Equipment Cost Index, MSECI.

2.45.4.7.5 Cost of standard size aerator, COSTSA, \$ (optional).

2.45.4.8 <u>Cost Calculations</u>.

2.45.4.8.1 Calculate cost of reinforced concrete.

2.45.4.8.1.1 Cost of R.C. wall.

$$COSTCW = \left(\frac{V_{cw}}{27}\right) UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.45.4.8.1.2 Cost of R.C. slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.45.4.8.2 Calculate purchase cost of aerators.

$$COSTA = \frac{(COSTSA)\ (COSTR)\ (K)}{100}$$

where

COSTA = purchase cost of aerators, \$.

COSTSA = cost of standard size aerator (50 hp), \$.

COSTR = cost of aerator of horsepower HP_a as a percent of the cost of the standard size aerators, %.

K = number of aerators per basin.

2.45.4.8.2.1 Calculate COSTR.

If $HP_a \leq 25$ hp; COSTR is calculated by:

$$COSTR = 20.7\ (HP_a)^{0.2686}$$

If $HP_a > 25$ hp; COSTR is calculated by:

$$COSTR = 4.12\ (HP_a)^{0.7878}$$

2.45.4.8.2.2 Purchase cost of standard size aerator. The standard size aerator is a 50 hp, high-speed floating aerator. The cost of the 50 hp aerator in the first quarter of 1977 is:

$$COSTSA = \$13,960$$

For better cost estimation, COSTSA should be obtained from the equipment vendor and treated as a unit price input. However, if COSTSA is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSA = \$13,960\ \frac{MSECI}{491.6}$$

where

COSTSA = cost of standard size aerator (50 hp), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

2.45.4.8.3 Calculate total installed equipment cost.

2.45.4.8.3.1 Calculate aerator installation labor.

$$IMH = 0.633 \ (HP_a) + 40$$

where

IMH = aerator installation labor, MH.

HP_a = horsepower of individual aerators, hp.

2.45.4.8.3.2 Calculate aerator installation cost.

$$AIC = (IMH) \ (K) \ (LABRI)$$

where

AIC = aerator installation cost, \$.

IMH = aerator installation labor, MH.

K = number of aerators per basin.

LABRI = installation labor rate, \$/MH.

2.45.4.8.3.3 Calculate installed cost for electrical/mechanical.

$$EMC = 0.589 \ (HP_a)^{-0.1465} \ (COSTA)$$

where

EMC = installed cost for electrical/mechanical, \$.

HP_a = horsepower of individual aerators, hp.

COSTA = purchase cost of aerators, \$.

2.45.4.8.3.4 Calculate total installed equipment cost.

$$IEC = COSTA + AIC + EMC$$

where

IEC = total installed equipment cost, \$.

AIC = aerator installation cost, \$.

EMC = installed cost of electrical/mechanical, \$.

2.45.4.8.4 Calculate O&M material and supply costs.

$$OMMC = \left(\frac{OMMP}{100}\right) IEC$$

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as percent of installed cost, %.

IEC = total installed equipment cost, \$.

2.45.4.8.5 Calculate total bare construction cost.

$$TBCC = (COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

IEC = total installed equipment cost, \$.

CF = correction factor for other minor construction costs.

2.45.4.9 Cost Calculations Output Data.

2.45.4.9.1 Total bare construction cost, TBCC, \$.

2.45.4.9.2 O&M material and supply costs, OMMC, \$/yr.

2.45.5 Bibliography.

2.45.5.1 Barrett, M.J. et al., "Aeration Studies for Four Weir Systems," Water and Wastewater Engineering, 64, No. 9, 1960, pp 407-413.

2.45.5.2 Metcalf and Eddy, "Water Pollution Abatement Technology: Capabilities and Costs, Public Owned Treatment Works", 1975, PB 250 690-03 NTIS, Springfield, VA.

2.45.5.3 Roy F. Weston, Inc., "Process Design Manual for Upgrading Existing Wastewater Treatment Plants", prepared for USEPA, Technology Transfer, October, 1971, Washington, D.C.

2.45.5.4 "Water Treatment Plant Design", American Waterworks Association, Inc., 1969.

2.47 PRELIMINARY TREATMENT

2.47.1 Background. This section combines three processes: grit chambers, comminution and screening. This has been done because most of the cost information available combines these processes. The cost estimates will be parametric in nature for this same reason and also because the costs for these processes are small in comparison with the entire facility so that large errors in estimating the costs of the preliminary treatment would result in a very small error in the total treatment facility cost.

2.47.2 General Description Comminution.

2.47.2.1 Comminutors are screens equipped with a device that cuts and shreds the screenings without removing them from the waste stream. Thus, comminuting devices eliminate odors, flies, and other nuisances associated with other screening devices. A variety of comminuting devices are available commercially.

2.47.2.2 Comminutors are usually located behind grit removal facilities in order to reduce wear on the cutting surfaces. They are frequently installed in front of pumping stations to protect the pumps against clogging by large floating objects.

2.47.2.3 The comminutor size is based usually on the volume of waste to be treated. Treatment plants with a wastewater flow below 1 mgd normally use one comminutor. Table 2.47-1 summarizes design characteristics of comminutors.

2.47.2.4 In wastewater treatment facilities for recreation areas, one comminutor may be installed in the wet well to protect the pump from large floating objects. In the treatment of vault waste, a comminutor may be included as an integral part of a vault waste holding station. Figure 2.47-1 shows a comminutor.

2.47.3 General Description Grit Removal.

2.47.3.1 Grit removal is classified as a protective or a preventive measure. The process does not contribute materially to the reduction in the pollutional load applied to the wastewater treatment facility. Grit chambers are designed to remove grit which may include sand, gravel, cinder, and other inorganic abrasive matter. Grit causes wear on pumps, fills pump sumps and sludge hoppers, clogs pipes and channels, and occupies valuable space in sludge digestion tanks. Grit removal, therefore, results in the reduction of maintenance costs of mechanical equipment and the elimination of operational difficulties caused by grit. Grit removal is recommended for small as well as large treatment facilities and for those served by combined as well as separate sewer systems. Bar screens are usually installed ahead of grit chambers to remove large floating objects.

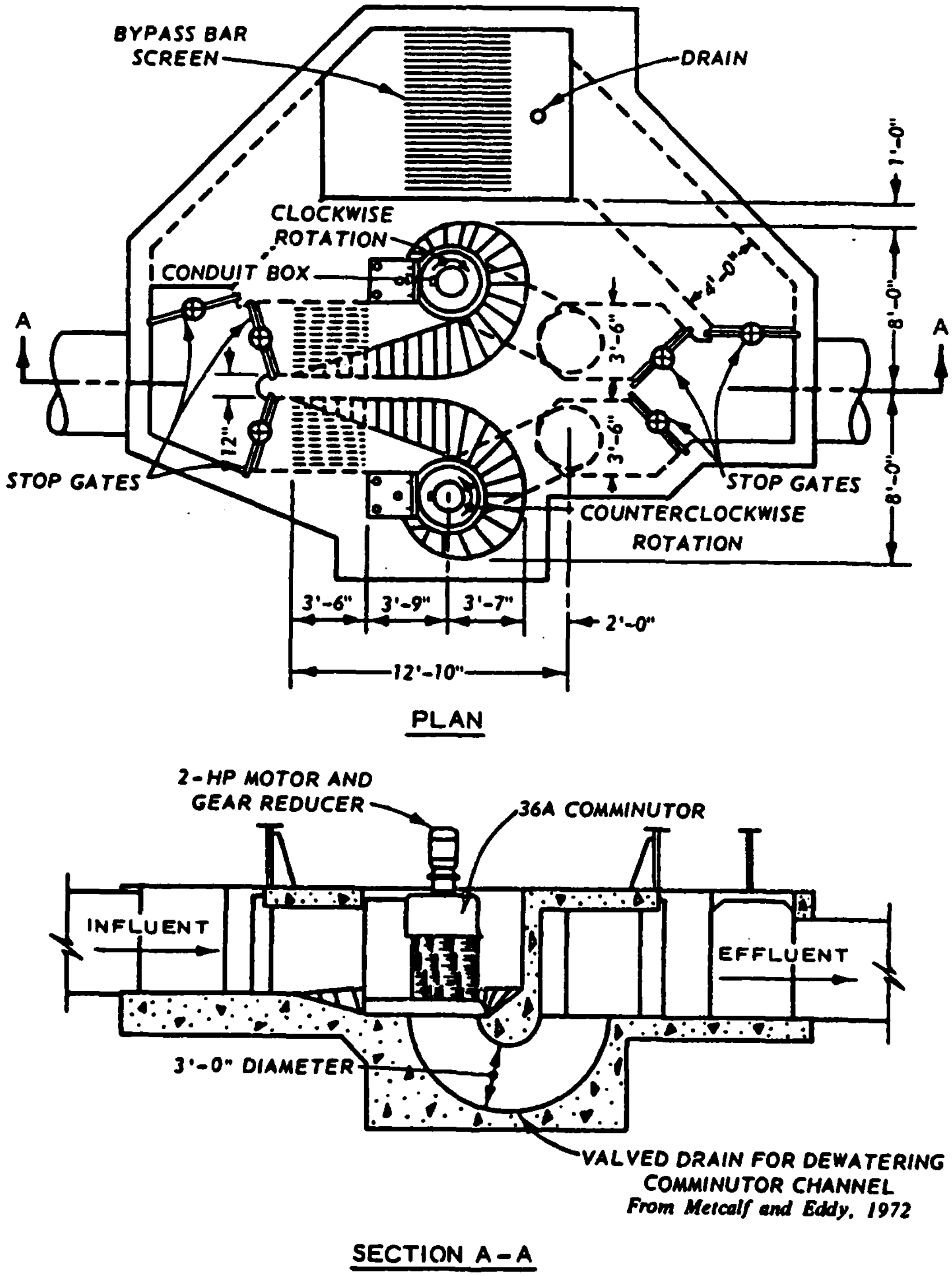

Figure 2.47-1. Schematic of a typical comminutor.

2.47.3.2 Grit removal is normally accomplished through control of velocity and settling time. The objective is to settle the grit particles while keeping the putrescible matter in suspension. Theoretically, it is desirable to remove all grit; however, experience indicates that removing 65-mesh grit, i.e. grit that is retained on a 65-mesh screen, provides sufficient protection to mechanical equipment and eliminates the majority of operational troubles caused by the grit. To remove the 65-mesh grit with a minimum of putrescible matter, a flow-through velocity of 0.75-1.25 fps must be provided at all flows.

2.47.3.3 The type of settling that normally takes place in a grit chamber is classified as discrete settling, since each particle retains its identity while settling at a constant rate. The design of such a chamber is usually based on an overflow rate that exceeds the settling velocity of the smallest particle desired to be removed. Smaller particles are removed in proportion, according to the ratio of their settling velocities to the settling velocity of the smallest particle that is, theoretically, 100 percent removed. The overflow rates selected in the design of a grit chamber must, therefore, exceed the settling velocity of the 65-mesh particle. Grit settling velocities are summarized in Table 2.47-2.

2.47.3.4 Grit chambers may be classified generally as either horizontal flow or aerated. In the horizontal flow type, the velocity is controlled by the dimensions of the chamber or by the use of a proportional weir or a Parshall flume at the effluent end of the chamber. Aerated grit chambers consist of a spiral flow aeration tank with the spiral flow velocity controlled by the dimensions and the quantity of air supplied to the chamber. These chambers are very efficient and the grit will be washed and easy to handle. Aerated grit chambers provide a detention time of 3 min at the maximum rate of flow. Mechanical grit removal equipment is usually recommended.

2.47.3.5 In summary, the design of grit chambers depends on the type selected, type of grit removal equipment, specification of the selected grit removal equipment, and the quantity and quality of the grit to be handled.

2.47.3.6 As the trend toward mechanization of wastewater treatment facilities continues to increase at a rapid rate, it is becoming a common practice to include grit removal facilities in the design of treatment systems serving small, as well as large, communities. Picnickers and campers in some recreation areas either maliciously or accidentally drop cans, bags, bottles, sticks and rocks in vault toilets, trailer dump stations and other facilities. Hence, wastewater characteristics should be thoroughly examined to determine the need for grit removal facilities in the design of systems for recreation areas. Figure 2.47-2 shows a typical aerated grit chamber. Figure 2.47-3 shows a typical horizontal flow grit chamber.

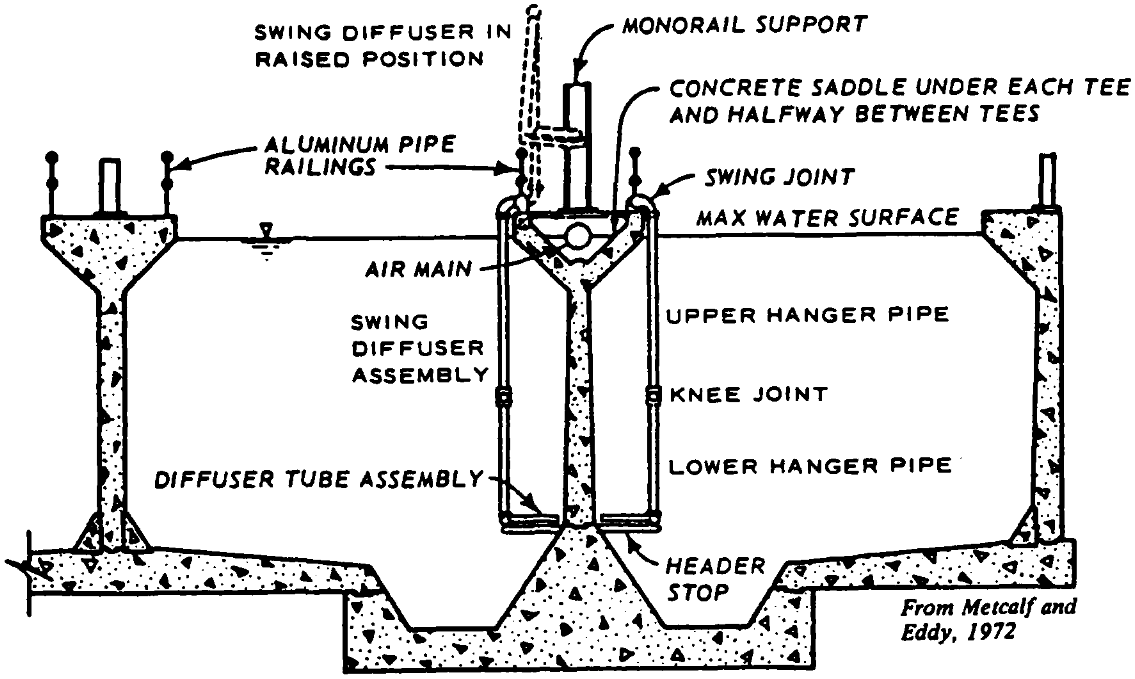

Figure 2.47-2. Schematic of a typical aerated grit chamber.

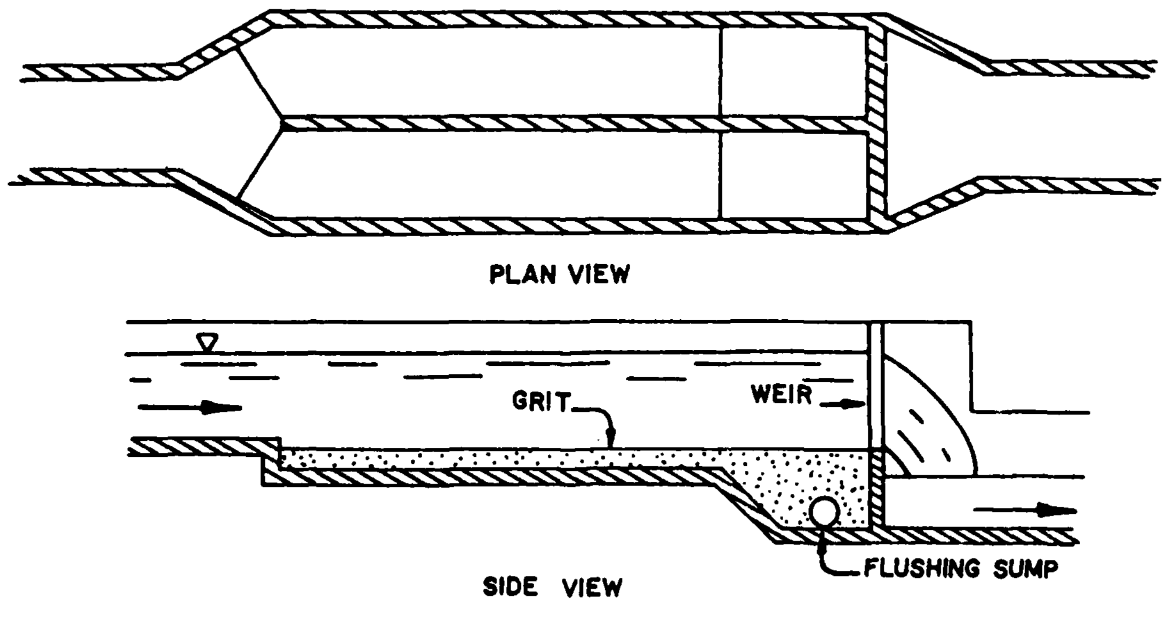

Figure 2.47-3. Schematic of a typical horizontal flow grit chamber.

2.47.4 General Description Screening.

2.47.4.1 Screening devices are used normally to remove large floating objects that otherwise may damage pumps and other equipment, obstruct pipelines, and interfere with the normal operation of the treatment facilities. Screens used in wastewater treatment facilities or in pumping stations are generally classified as fine screens or bar screens.

2.47.4.2 Fine screens are those with openings of less than 1/4 in. These screens have been used as a substitute for sedimentation tanks to remove suspended solids prior to biological treatment. However, few plants today use this concept of solids removal. Fine screens may be of the disk, drum or bar type. Bar-type screens are available with openings of 0.005 to 0.10 in.

2.47.4.3 Bar screens are used mainly to protect pumps, valves, pipelines, and other devices from being damaged or clogged by large floating objects. Bar screens are sometimes used in conjunction with comminuting devices. Bar screens consist of vertical or inclined bars spaced at equal intervals (usually 3/4 to 3 in.) across the channel where wastewater flows. These devices may be cleaned manually or mechanically. Bar screens with openings exceeding 2-½ in are termed trashracks.

2.47.4.4 The quantity of screenings removed by bar screens usually depends on the size of the bar spacings. Since the handling and disposal of screenings is one of the most disagreeable jobs in wastewater treatment, it is usually recommended that the quantity of screenings be kept at a minimum. Amounts of screenings from normal domestic wastes have been reported from 0.5 to 5 ft^3/million gal of wastewater treated. Screenings may be disposed of by burial, incineration, grinding, and digestion.

2.47.4.5 Design of bar screens is based mainly on average and peak wastewater flow. Normal design and operating parameters are usually presented in the manufacturer's specifications. General characteristics of bar and fine screens are presented in Tables 2.47-4 and 2.47-5, respectively. Figure 2.47-4 shows a mechanically cleaned bar rack.

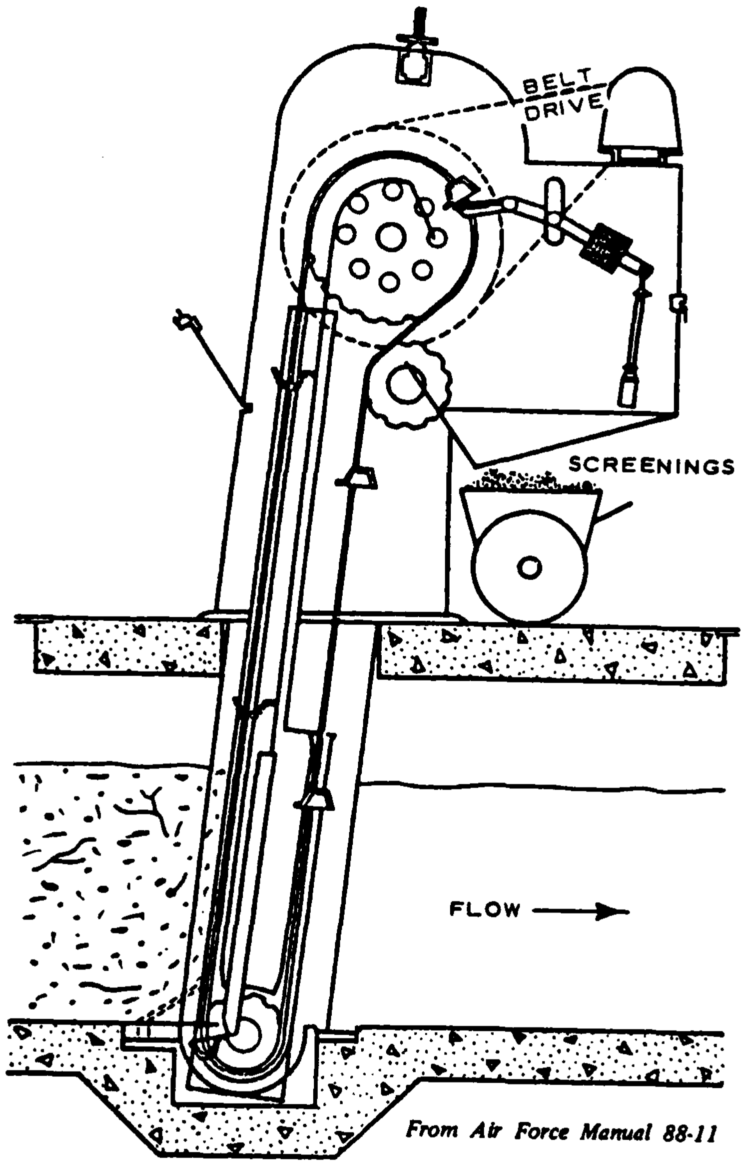

Figure 2.47-4. Schematic of a mechanically cleaned bar rack.

2.47.5 Comminution.

2.47.5.1 Input Data.

2.47.5.1.1 Wastewater flow, mgd.

2.47.5.1.1.1 Average daily flow, mgd.

2.47.5.1.1.2 Maximum flow, mgd.

2.47.5.2 Design Parameters. None.

2.47.5.3 Process Design Calculations.

2.47.5.3.1 Select size and number of comminutors from equipment manufacturer's catalog or Table 2.47-1 to correspond to maximum wastewater flows.

2.47.5.4 Process Design Output Data.

2.47.5.4.1 Comminutor specifications.

2.47.5.4.2 Number of comminutors.

Table 2.47-1. Comminutor Size Selection

Drum Diameter in	Drum rpm	Avg Slot Width in	Horse-power	Standard Sizes: Height	Standard Sizes: Net Weight lb	Rates of Flow Avg 12-hr Day Time, mgd	Maximum Hourly Rates of Flow mgd
4	56	¼	¼	2 ft 3¼ in	175	0 to 0.035	0.09
7	56	¼	¼	4 ft 3 in	450	0.03 to 0.113	0.24
7	56	¼	¼	4 ft 3 in	450	0.06 to 0.200	0.36
10	45	¼	½	4 ft 5 in	650	0.17 to 0.720	1.08
15	37	¼	3/4	4 ft 11½ in	1100	0.25 to 1.820	2.40
25	25	3/8	1½	5 ft 9½ in	2100	0.97 to 5.100	6.10
25	25	3/8	1½	6 ft 11½ in	3500	1.00 to 9.400	11.10
36	15	3/8	2	9 ft 4½ in	8500	1.30 to 20.00	24.00

2.47-8

2.47.6 Grit Removal.

2.47.6.1 Input Data.

2.47.6.1.1 Wastewater Flow.

2.47.6.1.1.1 Minimum and peak flows, mgd.

2.47.6.1.1.2 Average daily flow, mgd.

2.47.6.2 Design Parameters.

2.47.6.2.1 Horizontal Flow Grit Chamber.

2.47.6.2.1.1 Particle size, mm.

2.47.6.2.1.2 Specific gravity.

2.47.6.2.1.3 Maximum and average velocities, fps.

2.47.6.2.1.4 Current allowance (1.7).

2.47.6.2.1.5 Volume of grit, ft^3.

2.47.6.2.1.6 Number of units.

2.47.6.2.2 Aerated Grit Chamber.

2.47.6.2.2.1 Detention time, min.

2.47.6.2.2.2 Air supply, cfm/ft.

2.47.6.2.2.3 Maximum and average veolcities, fps.

2.47.6.2.2.4 Volume of grit, ft^3.

2.47.6.2.2.5 Number of units.

2.47.6.3 Process Design Calculations.

2.47.6.3.1 Horizontal flow grit chamber.

2.47.6.3.1.1 Select the number of units and calculate flow per unit.

$$Q/\text{unit} = \frac{Q_T}{N}$$

where

Q_T = total flow to plant, mgd.

N = number of units.

2.47.6.3.1.2 Select maximum controlled velocity, v $\approx$ 1.25 fps.

2.47.6.3.1.3 Calculate cross-sectional area.

$$A = \frac{Q_{max}}{v_{max}} \times 1.54$$

where

A = cross-sectional area, ft^2.

Q_{max} = maximum flow/unit, mgd.

v_{max} = maximum controlled velocity, fps.

2.47.6.3.1.4 Assume depth and calculate width or vice versa.

$$A = WD$$

where

W = channel width, ft.

D = channel depth, ft.

2.47.6.3.1.5 Calculate settling velocity of the smallest particle desired to be 100% removed (according to Table 2.47-2 or calculations).

Size $\approx$ 0.2 mm

Specific gravity $\approx$ 2.65

$$v_s = \left[\frac{(4/3)(g/c_d)(P_s - 1)\ (d_p)}{3.28 \times 10^{-3}}\right]^{1/2}$$

$$c_d = \left(\frac{24}{R_n}\right) + \left(\frac{3}{\sqrt{R_n}}\right) + (0.34)$$

where

v_s = settling velocity of smallest particle that is 100% removed, fps.

g = gravitational acceleration, 32.2 fps^2.

c_d = drag coefficient.

P_s = specific gravity of the particle.

d_p = diameter of the particle, mm.

$$R_n = \text{Reynolds number} = \frac{(v_s)\ (d_p)}{\nu \times 3.28 \times 10^{-3}}$$

ν = kinematic viscosity of the liquid, ft^2/sec.

3.28×10^{-3} = conversion factor, mm to ft.

2.47.6.3.1.6 Calculate the length of each channel.

$$L = D_{max} \ \frac{v_{max}}{v_s} \quad (1.7)$$

where

L = length of the channel, ft.

D_{max} = depth at maximum flow, ft.

v_{max} = velocity at maximum discharge, fps.

v_s = settling velocity of the smallest particle that is 100% removed, fps.

1.7 = allowance for current.

2.47.6.3.1.7 Calculate detention time.

$$t = \frac{(D)(W)(L)}{Q \times 1.54}$$

where

t = detention time, sec.

D = channel depth, ft.

W = channel width, ft.

L = length of channel, ft.

Q = flow, mgd.

1.54 = conversion factor, mgd to cfs.

2.47.6.3.1.8 Calculate bottom slope of channel.

$$v_{avg} = \frac{1.49}{n} R^{2/3} S^{1/2}$$

where

v_{avg} = velocity at average discharge, fps.

n = Manning's coefficient (a value of 0.03 may be used).

R = hydraulic radius of cross section = $\frac{A}{2D + w}$ for rectangular channel.

S = slope.

2.47.6.3.1.9 Calculate approximate volume of grit. The volume of grit may vary from less than 1 ft^3 to more than 12 ft^3 per million gal of wastewater. It must also be noted that the grit collected from a horizontal flow grit chamber will contain a high amount of putrescible matter and should be washed before disposal.

Assume 4 ft^3/million gal (approximate)

$$V_g = 4Q$$

where

V_g = volume of grit, ft^3/day.

Q = flow, mgd.

2.47.6.3.2 Aerated Grit Chamber.

2.47.6.3.2.1 Calculate the length of channel.

$$L = \frac{(t)(Q_p)(1.54)(1.7)(60)}{A}$$

where

L = length of channel, ft.

t = detention time, min.

Q_p = peak flow, mgd.

A = cross-sectional area, ft^2.

2.47.6.3.2.2 Calculate air requirements, cfm.

$$\text{Total air} = L \times O_2/ft$$

where

L = length of chamber.

O_2/ft = air supply per ft of length (Table 2.47-3).

2.47.6.4 Process Design Output Data.

2.47.6.4.1 Horizontal Flow Grit Chamber.

2.47.6.4.1.1 Maximum flow, cfs.

2.47.6.4.1.2 Average flow, cfs.

2.47.6.4.1.3 Minimum flow, cfs.

2.47.6.4.1.4 Temperature, °C.

2.47.6.4.1.5 Maximum flow-through velocity, fps.

2.47.6.4.1.6 Average flow-through velocity, fps.

2.47.6.4.1.7 Size smallest particle 100% removed, mm.

2.47.6.4.1.8 Specific gravity of particle.

2.47.6.4.1.9 Number of units.

2.47.6.4.1.10 Maximum flow/unit, cfs.

2.47.6.4.1.11 Width of channel, ft.

2.47.6.4.1.12 Depth of channel, ft.

2.47.6.4.1.13 Length of channel, ft.

2.47.6.4.1.14 Settling velocity of particle, fps.

2.47.6.4.1.15 Slope of channel bottom.

2.47.6.4.1.16 Allowance for currents.

2.47.6.4.1.17 Detention time, sec.

2.47.6.4.1.18 Manning's coefficient.

2.47.6.4.1.19 Volume of grit, ft^3/day.

2.47.6.4.1.20 Design outlet control section.

2.47.6.4.2 <u>Aerated Grit Chamber</u>.

2.47.6.4.2.1 Detention time, min.

2.47.6.4.2.2 Air supply, cfm/ft.

2.47.6.4.2.3 Length, ft.

2.47.6.4.2.4 Total air requirement, cfm.

Table 2.47-2. Grit Settling Velocities

Particle Size		Settling Velocity			Area Required
Mesh	mm	ft/min	gpd/ft^2	mgd/ft^2	ft^2/million gal
18	0.833	14.7	160,000	0.1600	6.3
20	0.595	10.5	114,500	0.1145	8.7
35	0.417	7.4	80,100	0.0801	12.5
48	0.295	5.2	56,700	0.0567	17.7
65[a]	0.208	3.7	40,000	0.0400	25.0
100	0.147	2.6	28,200	0.0282	35.5
150	0.105	1.8	20,200	0.0202	49.5

[a] Minimum particle size desirable for removal.

Table 2.47-3. Design Parameters for Aerated Grit Chambers

1. Air supply - 3 cfm/ft of tank length.
2. Air diffusers - located 2 to 3 ft above tank bottom on one side of tank.
3. Surface velocity - 1.5 to 2 fps.
4. Tank floor velocity - 1 to 1.5 fps.
5. Grit collectors - air lift pumps to decanting channels, grit conveyors or grit pumps.
6. Detention time - 2 to 3 min.
7. Efficiency - 100% removal of 65-mesh grit.

2.47.7 Screening.

2.47.7.1 Input Data.

2.47.7.1.1. Wastewater flow.

2.47.7.1.1.1 Average daily flow, mgd.

2.47.7.1.1.2 Maximum daily flow, mgd.

2.47.7.1.1.3 Peak wet weather flow, mgd.

2.47.7.2 Design Parameters.

2.47.7.2.1 Type of bar screen.

2.47.7.2.1.1 Manually cleaned.

2.47.7.2.1.2 Mechanically cleaned.

2.47.7.2.2 Velocity through bar screen, fps (Table 2.47-4).

2.47.7.2.3 Approach velocity, fps (Table 2.47-4).

2.47.7.2.4 Maximum head loss through screen, in (Table 2.47-4).

2.47.7.2.5 Bar spacings, in (Table 2.47-4).

2.47.7.2.6 Slope of bars, deg (Table 2.47-4).

2.47.7.2.7 Channel width, ft.

2.47.7.2.8 Width of bar, in.

2.47.7.2.9 Shape factor.

2.47.7.3 Process Design Calculations.

2.47.7.3.1 Consult equipment manufacturer's specifications and select a bar screen which meets design requirements.

2.47.7.3.2 Calculate head loss through the screen. It should be noted that when screens start to become clogged between cleanings in manually cleaned screens head loss will increase.

$$h_e = \beta \left(\frac{W}{b}\right)^{4/3} \frac{v^2 \sin^2 \theta}{2g}$$

where

h_e = head loss through the screen, ft.

β = bar shape factor.

= 2.42 for sharp-edged rectangular bars.

= 1.83 for rectangular bars with semicircular upstream faces.

= 1.79 for circular bars.

= 1.67 for rectangular bars with semicircular upstream and downstream faces.

= 0.76 for rectangular bars with semicircular upstream faces and tapering in a symmetrical curve to a small circular downstream face (teardrop).

W = maximum width of bars facing the flow, in.

b = minimum width of the clear spacings between pairs of bars, in.

v = longitudinal approach velocity, fps.

θ = angle of the rack with horizontal, deg.

g = gravitational acceleration.

2.47.7.3.3 Calculate average depth.

$$D = \frac{(Q_{avg})(1.54)}{(W_c)(V)}$$

where

D = average depth, ft.

Q_{avg} = average flow, mgd.

W_c = channel width, ft.

V = average velocity, fps.

2.47.7.3.4 Calculate maximum depth.

$$D_{max} = D \quad \frac{Q_p}{Q_{avg}}$$

where

D_{ax} = maximum depth, ft.

D = average depth, ft.

Q_p = peak flow, mgd.

Q_{avg} = average flow, mgd.

2.47.7.4 Process Design Output Data.

2.47.7.4.1 Bar size, in.

2.47.7.4.2 Bar spacing, in.

2.47.7.4.3 Slope of bars from horizontal, deg.

2.47.7.4.4 Head loss through screen, ft.

2.47.7.4.5 Approach velocity, fps.

2.47.7.4.6 Average flow-through velocity, fps.

2.47.7.4.7 Maximum flow-through velocity, fps.

2.47.7.4.8 Screen channel width, ft.

2.47.7.4.9 Channel depth, ft.

Table 2.47-4. General Characteristics of Bar Screens

Item	Hand Cleaned	Mechanically Cleaned
Bar screen size		
Width, in.	1/4 to 5/8	1/4 to 5/8
Depth, in.	1 to 3	1 to 3
Spacing, in.	1 to 2	5/8 to 3
Slope from vertical, deg.	30 to 45	0 to 30
Approach velocity, fps	1 to 2	2 to 3
Allowable head loss, in.	6	6

Table 2.47-5. General Characteristics of Fine Screens

Item	Disk	Drum
Fine Screen		
Openings, in.	0.126 to 0.009 (6 to 60 mesh)	0.126 to 0.009 (6 to 60 mesh)
Diameter, ft.	4 to 18	3 to 5
Length, ft.		4 to 12
rpm		4

2.47.8 Quantities Calculation.

2.47.8.1 Calculate operation manpower required.

2.47.8.1.1 If $0 < Q_{avg} \leq 3$ mgd, the operation manpower is calculated by:

$$OMH = 600 \ (Q_{avg})^{0.3382}$$

where

Q_{avg} = average daily wastewater flow, mgd.

OMH = operation manpower required, man-hours/yr.

2.47.8.1.2 If $3 < Q_{avg} \leq 7$ mgd, the operation manpower is calculated by:

$$OMH = 469.3 \ (Q_{avg})^{0.5618}$$

2.47.8.1.3 If $Q_{avg} > 7$ mgd, the operation manpower is calculated by:

$$OMH = 255.3 \ (Q_{avg})^{0.8746}$$

2.47.8.2 Calculate maintenance manpower required.

2.47.8.2.1 If $0 < Q_{avg} \leq 3$ mgd, the maintenance manpower is calculated by:

$$MMH = 340 \ (Q_{avg})^{0.2946}$$

where

MMH = maintenance manpower required, man-hours/yr.

2.47.8.2.2 If $3 < Q_{avg} \leq 7$ mgd, the maintenance manpower is calculated by:

$$MMH = 265.5\ (Q_{avg})^{0.5197}$$

2.47.8.2.3 If $Q_{avg} > 7$ mgd, the maintenance manpower is calculated by:

$$MMH = 168.5\ (Q_{avg})^{0.7534}$$

2.47.8.3 Calculate the energy required for operation.

$$KWH = 16{,}000\ (Q_{avg})^{0.4631}$$

where

KWH = energy required for operation, kwhr/yr.

2.47.8.4 Calculate the operation and maintenance material and supply costs. The operation and maintenance material and supply costs can be expressed as a percent of the total bare construction cost.

$$OMMP = 2.5$$

where

OMMP = operation and maintenance material and supply costs as percent of total bare construction cost, percent.

2.47.9 <u>Quantities Calculations Output Data</u>.

2.47.9.1 Peak wastewater flow, Q_p, mgd.

2.47.9.2 Average daily wastewater flow, Q_{avg}, mgd.

2.47.9.3 Operation manpower required, OMH, man-hours/yr.

2.47.9.4 Maintenance manpower required, MMH, man-hours/yr.

2.47.9.5 Energy required for operation, KWH, kwhr/yr.

2.47.9.6 Operation and maintenance material and supply costs, as percent of total bare construction cost, OMMP, percent.

2.47.10 <u>Unit Price Input Required</u>. (None parametric costing used).

2.47.11 <u>Cost Calculations</u>.

2.47.11.1 Total bare construction cost.

$$TBCC = 40,000\ (Q_p)^{0.6233}$$

where

TBCC = total bare construction cost, \$.

Q_p = peak wastewater flow, mgd.

2.47.11.2 Operation and maintenance material and supply costs.

$$OMMC = \frac{(OMMP)}{100} \cdot TBCC$$

where

OMMC = operation and maintenance material and supply costs, \$/yr.

2.47.12 <u>Cost Calculations Output Data</u>.

2.47.12.1 Total bare construction cost, TBCC, \$.

2.47.12.2 Operation and maintenance material and supply costs, OMMC, \$/yr.

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2.49 PUMPING

2.49.1 Background. There are several situations throughout a sewage treatment facility which require pumping. Typically at the head of the treatment facility pumping of the raw waste is required. Other points in the treatment facility which might require pumping are prior to trickling filters, tertiary filters, carbon adsorption units, or any treatment process which creates relatively high head losses. Generally speaking two different type pumps are used for raw waste pumping and pumping for other processes in the treatment facility. For this reason the pumping has been divided into raw waste pumping and intermediate pumping.

2.49.2 General Description Intermediate Pumping.

2.49.2.1 In intermediate pumping the wastewater is relatively clean and free from large solids so that more efficient pumps can be used for these processes than for raw waste pumping.

2.49.3 General Description Raw Waste Pumping.

2.49.3.1 Pumping of raw sewage at the head of a treatment facility is often required to produce the head required for sewage to flow through the plant. This pumping is accomplished with relatively inefficient low-head pumps which are capable of passing large solids without damage to the pumps.

2.49.4 Intermediate Pumping.

2.49.4.1 Input Data.

2.49.4.1.1 Average daily wastewater flow, Q_{avg}, mgd.

2.49.4.1.2 Peak wastewater flow, Q_{pk}, mgd.

2.49.4.2 Design Parameters.

2.49.4.2.1 Wastewater flow, mgd.

2.49.4.2.2 Number of pumps.

2.49.4.2.3 Total head on pumps, ft.

2.49.4.3 Process Design Calculations.

2.49.4.3.1 Calculate design capacity of pumps.

2.49.4.3.1.1 Intemediate pumping.

$$GPM = \frac{(Q_{avg})\ (2)\ (10^6)}{1440}$$

where

GPM = design capacity of pumps, gpm.

2 = excess capacity factor to handle peak flows.

2.49.4.3.1.2 Return sludge pumping.

$$GPM = \frac{(Q_{avg})\ (RSR)\ (10^6)}{1440}$$

where

GPM = design capacity of pumps, gpm.

RSR = return sludge ratio to average wastewater flow, from Table 2.49-1.

TABLE 2.49-1

Activated Sludge Processes	RSR
Conventional	1.0
Complete-Mix	1.0
Step-Aeration	1.0
Modified-Aeration	1.0
Contact-Stabilization	1.0
Extended-Aeration	1.5
Kraus-Process	1.0
High-Rate Aeration	5.0
Pure-Oxygen System	1.0

2.49.4.3.2 Determine the type, number and size of pumps required. For the purposes of this program it has been assumed the pumps will be horizontal single-stage, single-suction, split casing centrifugal pumps designed for sewage applications. Also the head required is assumed to be 40 ft for all applications. All pumps will be assumed to be the same size with variable speed drives and the convention of sparing the largest pump will be adhered to. The pumps will be arranged in identical batteries, with each battery handling a maximum flow of 80,000 gpm.

2.49.4.3.2.1 Number of batteries. The number of batteries will be calculated by trial and error; begin with NB = 1. If GPM/NB > 80,000, then go the NB = NB + 1 and repeat until GPM/NB ≤ 80,000. Then:

$$GPMB = \frac{GPM}{NB}$$

where

GPMB = design flow per battery,gpm.

NB = number of batteries.

2.49.4.3.2.2 Number of pumps per battery. The number of pumps per battery will be calculated by trial and error. Start with N = 2. If GPMB/N > 20,000 gpm, go to N = N + H and repeat until GPMB/N ≤ 20,000 gpm.

$$GPMP = \frac{GPMB}{N}$$

$$NP = N + 1$$

where

GPMP = design capacity of the individual pumps, gpm.

N = number of pumps required to handle design flow.

NP = total number of pumps per battery, including spare.

2.49.4.4 Process Design Output Data.

2.49.4.4.1 Design capacity of pumps, GPM, gpm.

2.49.4.4.2 Number of batteries, NB.

2.49.4.4.3 Design flow per battery, GPMB, gpm.

2.49.4.4.4 Number of pumps required to handle design flow, N.

2.49.4.4.5 Total number of pumps per battery, including spare, NP.

2.49.4.4.6 Design capacity of the individual pumps, GPMP, gpm.

2.49.4.5 Quantities Calculations.

2.49.4.5.1 Determine area of pump building.

$$PBA = [0.0284\ (GPMB) + 640]\ NB$$

where

PBA = pump building area, sq ft.

2.49.4.5.2 Calculate volume of earthwork required. The pumping building is usually a bilevel building with the pumps below ground and all electrical and control facilities above ground. It is assumed that the average depth of excavation would be 8 ft. The volume of earthwork will be estimated by:

$$V_{ew} = (8)(PBA)$$

where

V_{ew} = volume of earthwork required, cu ft.

2.49.4.5.3 Calculate operation manpower required. The operation manpower can be related to the firm pumping capacity.

2.49.4.5.3.1 Calculate firm pumping capacity.

$$FPC = \frac{(GPM)(1440)}{10^6}$$

where

FPC = firm pumping capacity, mgd.

2.49.4.5.3.2 If $0 < FPC \leq 7$ mgd:

$$OMH = 440\ (FPC)^{0.1285}$$

2.49.4.5.3.3 If $7 < FPC \leq 30$ mgd:

$$OMH = 294.4\ (FPC)^{0.3350}$$

2.49.4.5.3.4 If $30 < FPC \leq 80$ mgd:

$$OMH = 40.5\ (FPC)^{0.8661}$$

2.49.4.5.3.5 If $FPC > 80$ mgd:

$$OMH = 21.3\ (FPC)^{1.012}$$

where

OMH = operating manpower required, man-hours/yr.

2.49.4.5.4 Calculate maintenance manpower.

2.49.4.5.4.1 If $0 < FPC \leq 7$ mgd:

$$MMH = 360\ (FPC)^{0.1478}$$

2.49.4.5.4.2 If $7 < FPC \leq 30$ mgd:

$$MMH = 255.2\ (FPC)^{0.3247}$$

2.49.4.5.4.3 If $30 < FPC \leq 80$ mgd:

$$MMH = 85.7\ (FPC)^{0.6456}$$

2.49.4.5.4.4 If $FPC < 80$ mgd:

$$MMH = 30.6\ (FPC)^{0.8806}$$

where

MMH = maintenance manpower requirement, man-hours/yr.

2.49.4.5.5 Calculate electrical energy required.

$$KWH = 67{,}000\ (Q_{avg})^{0.9976}$$

where

KWH = electric energy required, kwhr/yr.

2.49.4.5.6 Calculate operation and maintenance material and supply costs. This item covers the cost of lubrication oils, paint, repair and replacement parts, etc. It is expressed as a percent of the total bare construction costs.

$$OMMP = 0.7\%$$

where

OMMP = operation and maintenance material and supply costs, as percent of the total bare construction cost, percent.

2.49.4.5.7 Other minor construction cost items. From the calculations approximately 85 percent of the construction costs have been accounted for. Other minor items such as piping, overhead crane, site cleaning, seeding, etc., would be 15 percent.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor construction costs.

2.49.4.6 Quantities Calculations Output Data.

2.49.4.6.1 Pump building area, PBA, sq ft.

2.49.4.6.2 Volume of earthwork required, V_{ew}, cu ft.

2.49.4.6.3 Firm pumping capacity, FPC, mgd.

2.49.4.6.4 Operating manpower required, OMH, man-hours/yr.

2.49.4.6.5 Maintenance manpower required, MMH, man-hours/yr.

2.49.4.6.6 Electrical energy required, KWH/kwhr/yr.

2.49.4.6.7 Operation and maintenance material and supply costs, OMMP, percent.

2.49.4.6.8 Correction factor for other minor construction costs, CF.

2.49.4.6.11 Design capacity of the individual pumps, GPMP, gpm.

2.49.4.7 Unit Price Input Required.

2.49.4.7.1 Unit price input for building cost, UPIBC, \$/sq ft.

2.49.4.7.2 Unit price input for earthwork, UPIEX, \$/cu ft.

2.49.4.7.3 Cost of standard size pump equipment, COSTPS, \$ (optional).

2.49.4.7.4 Marshall and Swift Equipment Cost Index, MSECI.

2.49.4.8 Cost Calculations.

2.49.4.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.49.4.8.2 Cost of pump building.

$$COSTPB = (PBA)(UPIBC)$$

where

COSTPB = cost of pump building, \$.

PBA = pump building area, sq ft.

UPIBC = unit price input for building cost, \$/sq ft.

2.49.4.8.3 Purchase cost of pumps and drivers.

2.49.4.8.3.1 Calculate COSTP.

$$COSTP = \frac{COSTRO}{100} (COSTPS)(NP)(NB)$$

where

COSTP = cost of pumps and drivers, \$.

COSTRO = cost of pumps and drivers of capacity GPMP, as percent of cost of standard size pump, percent.

COSTPS = cost of standard size pump (3000 gpm), \$.

NP = total number of pumps per battery.

NB = number of batteries.

2.49.4.8.3.2 Calculate COSTRO.

If $0 < GPMP \leq 5000$ gpm, COSTRO is calculated by:

$$COSTRO = 2.93\ (GPMP)^{0.4404}$$

If $GPMP > 5000$ gpm, COSTRO is calculated by:

$$COSTRO = .0064\ (GPMP)^{1.16}$$

2.49.4.8.3.3 Purchase cost of standard size pump and driver. A 3000 gpm pump was selected as a standard. The cost of a 3000 gpm pump and driver for the first quarter of 1977 is:

$$COSTPS = \$17,250$$

For better estimation, COSTPS should be obtained from the vendor and treated as a unit price input. If this is not done, the cost will be adjusted using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index.

2.49.4.8.4 Installed equipment costs. Typically, the installation cost of pumps is approximately 100 percent of the equipment cost. This includes cost of piping, concrete, steel, electrical, paint, and installation labor.

$$IPC = (COSTP)(2.0)$$

where

IPC = installed pumping equipment cost, \$.

2.49.4.8.5 Total bare construction cost.

$$TBCC = [COSTE + COSTPB + IPC]\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor minor construction costs

2.49.4.8.6 Operation and maintenance material and supply costs.

$$OMMC = (TBCC) \left(\frac{OMMP}{100}\right)$$

where

OMMC = operation and maintenance material and supply cost, \$.

OMMP = operation and maintenance material and supply costs, as a percent of total bare construction cost, percent.

2.49.4.9 Cost Calculations Output Data.

2.49.4.9.1 Total bare construction cost, TBCC, \$.

2.49.4.9.2 Operation and maintenance material and supply costs, OMMC, \$/yr.

2.49.5 Raw Waste Pumping.

2.49.5.1 Input Data.

2.49.5.1.1 Average daily wastewater flow, Q_{avg}, mgd.

2.49.5.1.2 Peak wastewater flow, Q_{pk}, mgd.

2.49.5.1.3 Depth to the influent sewer, DS, ft.

2.49.5.1.4 Static head to be pumped, SH, ft. Static head is defined as the vertical distance between the invert of the incoming sewer and the maximum water level in the next unit process.

2.49.5.1.5 Number of pumps, N, (include standby).

2.49.5.1.6 Variable speed or constant speed pumps.

2.49.5.2 Design Parameters.

2.49.5.2.1 Wastewater flow, mgd.

2.49.5.2.2 Number of pumps.

2.49.5.2.3 Total head on pumps, ft.

2.49.5.3 Process Design Calculations.

2.49.5.3.1 Calculate peak capacity of raw sewage pumps if Q_{pk} is not given in the input.

$$Q_{pk} = 3.84\ (Q_{avg})^{0.9098}$$

where

Q_{pk} = peak flow, mgd.

Q_{avg} = average daily flow, mgd.

2.49.5.3.2 Calculate number of pumps, N. If the number of pumps, N, is not given in the input, the following rule will be used to decide the number of pumps.

N	Q_p
2	Less than 2 mgd
3	2 mgd to 10 mgd
4	10 mgd to 150 mgd
5	Larger than 150 mgd

N includes one standby pump. In this model all the pumps will be the same size.

Thus each individual pump would handle:

$$q = \frac{Q_{pk} \times 10^6}{1440(N-1)} = 694.4 \left(\frac{Q_{pk}}{N-1}\right)$$

q = capacity of each individual pump, gpm.

2.49.5.3.3 Variable speed or constant speed pumps.

2.49.5.3.3.1 If no variable speed pump is specified in the input, all constant speed pumps will be supplied.

$$NCS = N$$

where

NCS = number of constant speed pumps

N = total number of pumps.

2.49.5.3.3.2 If variable speed pump selection is made by the user, the following rule will be used in deciding the number of various pumps.

$$NVS = 2$$

$$NCS = N-2$$

where

NVS = number of variable speed pumps.

NCS = number of constant speed pumps.

2.49.5.4 Process Design Output Data.

2.49.5.4.1 Peak flow, Q_{pk}, mgd.

2.49.5.4.2 Number of pumps, N.

2.49.5.4.3 Number of constant speed pumps, NCS.

2.49.5.4.4. Number of variable speed pumps, NVS.

2.49.5.5 Quantities Calculations.

2.49.5.5.1 Sizing the discharge header pipe. Based on experience, for a municipal wastewater treatment plant, the most economical pumping system is when the peak velocity in the discharge header is approximately 8 fps. Thus, the diameter would be:

$$DIAP = 5.955 \; (Q_{pk})^{0.5}$$

where

DIAP = header pipe diameter, inches.

Q_{pk} = peak flow, mgd.

2.49.5.5.2 Calculation of total dynamic head.

2.49.5.5.2.1 If the static head, SH, is not given by the user, the total dynamic head (TDH) would be assumed:

If $Q_{pk} < 1$ mgd

$$TDH = 70$$

If 1 mgd $\leq Q_{pk} \leq$ 100 mgd

$$TDH = 70 - (15 \; \mathrm{Log} \; Q_{pk})$$

If $Q_{pk} > 100$ mgd

$$TDH = 40$$

where

TDH = total dynamic head, ft.

Q_{pk} = peak flow, mgd.

2.49.5.5.2.2 If the static head is given, the total dynamic head would take the following form:

$$TDH = SH + 6 + \frac{376,460 \; (Q_{pk})^{1.85}}{(DIAP)^{4.8655}}$$

where

SH = static head from input data, ft.

6 = drop from the sewer invert to the low level point in the wet well, ft.

Q_{pk} = peak flow, mgd.

DIAP = diameter of pipe, inches.

376,400 = Hazen-Williams coefficient with pipe length assumed to be 500 ft.

2.49.5.5.3 Pump Sizing. It is indicated that the pump discharge nozzle velocity should be in the range of from 11 to 14 fps. Thus, the pump sizing procedure would be to calculate the pump nozzle diameter.

$$Dpump = 0.1927\ (q)^{0.5}$$

The available pump sizes are 4", 6", 8", 10", 12", 14", 16", 18", 20", 24", 30", 36", 42", 48", 54", 60" and 72". The actual pump size would be the smallest size available which is larger than the value given from the above equation.

where

Dpump = pump size, inches.

q = pump capacity, gpm.

2.49.5.5.4 Pump speed. One of the most important characteristics of centrifugal pumps is the parameter specific speed, NS. It is defined by the following equation.

$$NS = \frac{(RPM)\quad (q)^{0.5}}{(TDH)^{0.75}}$$

where

NS = specific speed.

RPM = pump rotating speed, rpm.

For any pump operating at any given speed, q and TDH are taken at the point of maximum efficiency. The computed value of specific speed has no usable physical meaning, except as a type number, but it is extremely useful because it is constant for all similar pumps. It is usually considered a shape factor. For the most often used centrifugal pumps in the raw sewage pump station, a NS value of 4000 is a typical value. The rotating speed for the pump selected would be:

$$RPM = \frac{4000\ (TDH)^{0.75}}{(q)^{0.5}}$$

However, the calculated RPM should match the available RPM of synchronous motor. It is available in 600, 900, 1200 and 1800 RPM. The final selected RPM should be the closest available speed.

2.49.5.5.5 Motor Sizing. Brake horsepower of the motor can be calculated by the following equation:

$$BH_p = \frac{(q)\ (TDH)}{3959.7\ (Pumpe)}$$

where

BH_p = brake horsepower, hp.

q = pump capacity, gpm.

TDH = total dynamic head, ft.

Pumpe = Pump efficiency.

Pump efficiency can be calculated by the following equation:

$$Pumpe = 0.63 + (2.42 \times 10^{-5})\ (q)$$

The available motor sizes in the market are rated in hp: 1, 2, 3, 5, 7.5, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100, 125, 150, 200, 250, 300, 350, 400, 450, 500, 600, 700.

The actual motor size would be the smallest size available which is larger than the calculated value.

2.49.5.5.6 Pump dimension: The length and width of a pump with its' motor and other necessary valves and piping can be calculated by the following fomula:

The width of the pump system.

$$WP = 0.2\ Dpump + 0.6$$

The length of the pump system.

$$LP = 0.333\ [(0.084)\ Dpump + 6.38]^2$$

where

WP = width of pump system, ft.

LP = length of pump system, ft.

Dpump = pump size, inch.

2.49.5.5.7 Dry well dimensions: The total length of the dry well can be calculated by:

$$L = N\ (WP) + (N + 1)\ 4$$

where

L = length of the dry well, ft.

And the width of the dry well would be:

$$WD = LP + 9$$

where

WD = width of dry well, ft.

2.49.5.5.8 Wet Well Dimensions: The length of the wet well would be the same as that of the dry well. However, the width of the wet well is dependent on various factors. The following sections give the calculation procedures.

2.49.5.5.8.1 When constant speed pumps are used. The wet well size is determined by the cycle time of the pump and the cycle time, in turn, is dependent on the motor size. The following rule of thumb will be used in determining the cycle time, CT.

BHP, HP of Motor	CY, Cycle Time, In Minutes
0 - 15	10
15 - 50	15
50 - 200	30
200 - 600	60
600 and above	120

The wet well volume would be:

$$VOLW = \frac{(Q - Q_{min})\ CT}{7.48}$$

where

$VOLW$ = wet well volume, cu ft.

Q_{min} = minimum inflow, gpm.

q = pump capacity, gpm.

CT = cycle time, min.

Assuming a 6 ft operating wet well depth, the width of the wet well, WW, would be:

$$WW = \frac{VOLW}{6\ L}$$

where

WW = wet well width, ft.

L = drywell length.

In some cases it may be possible for the wet well width calculated in the above fashion to approach zero. In such cases, wet well width is calculated as follows based on the standards of the Hydraulic Institute:

$$WW = 0.32854\ q^{2.04809}$$

The larger WW will be utilized in design calculations. If WW is calculated to be less than 8 ft WW will be set to 8 ft.

2.49.5.5.8.2 When variable speed pumps are specified, the width of the wet well is determined by the distance required to prevent vortexing and can be calculated by:

$$WW = 10 + \frac{q}{4200}$$

where

WW = wet well width, ft.

q = pump capacity, gpm.

Since it still requires 6 ft of operating depth for control system to perform smoothly, the wet well volume is:

$$VOLW = (6)\ (WW)\ (L)$$

2.49.5.5.9 Depth of the pump station: The depth of the pump station is defined as the vertical distance between the bottom of the dry well to the ground level.

$$DP = DS + 6' + DM$$

where

DP = depth of the pump station, ft.

DS = depth of the sewer invert, ft.

DM = minimum depth of water in the wet well, ft.

2.49.5.5.9.1 Depth of the sewer, DS, if not in the input, a value of 15 ft will be used.

2.49.5.5.9.2 Minimum depth of water in the wet well, DM. It is related the pump capacity as:

$$DM = 0.0544\ (q)^{0.5} + 2.033$$

where

q = pump capacity in gpm.

2.49.5.5.10 Reinforced concrete quantity for pump station construction.

2.49.5.5.10.1 Top Slab:

$$V_{TS} = 0.833\ (L)\ (W)$$

$$W = WW + WD$$

where

V_{TS} = reinforced concrete slab, quantity, cu ft.

L = length of the pump station, ft.

W = width of the pump station, ft.

2.49.5.5.10.2 Walls.

$$V_{cw} = (1 + \frac{DP}{96})\ (DP)\ [2W + 3L + 4\ (1 + \frac{DP}{96})]$$

where

V_{cw} = volume of R.C. walls, cu ft.

DP = depth of pump station, ft.

2.49.5.5.10.3 Bottom Slab: It is assumed that the thickness of the bottom slab will vary to counter the uplift force of ground water. The thickness of the bottom slab would be:

$$t_{BS} = \frac{(0.416)\ (DP)\ (L + X)\ (W + X) - V_{TS} - V_{cw}}{(L + 8)\ (W + 8)}$$

where

$$X = 2\ (1 + \frac{DP}{96})$$

and

T_{BS} = thickness of the bottom slab, ft.

Thus, the volume of bottom slab is:

$$V_{BS} = (L + 8)\ (W + 8)\ (t_{BS})$$

where

V_{BS} = volume of R.C. slab, cu ft.

2.49.5.5.10.4 The total reinforced concrete slab quantity would be:

$$V_{CS} = V_{TS} + V_{BS}$$

where

V_{CS} = R.C. slab quantity, cu ft.

2.49.5.5.11 Earthwork required. The volume of earthwork would include excavation and backfill quantities and can be estimated by:

$$V_{exc} = V_{BS} + (DP)\ [(L + 8)\ (W + 8) + \frac{DP}{2}\ (W + L + DP + 16)]$$

where

V_{exc} = quantity of excavation, cu ft.

DP = depth of pump station, ft.

V_{BS} = volume of the bottom slab.

$$V_{bf} = V_{exc} - (L + X)\ (W + X)\ (DP)$$

where

V_{bf} = volume of backfill required, cu ft.

Thus, the total volume of earthwork would be:

$$V_{ew} = 1.10\ \ (V_{exc} + V_{bf})$$

where

V_{ew} = total earthwork volume, cu ft.

2.49.5.5.12 Determine area of pump building.

$$PBA = 1.05\ (WD)\ (L)$$

where

PBA = pump building area, sq ft.

2.49.5.5.13 Calculate operation manpower required. The operation manpower can be related to the firm pumping capacity.

2.49.5.5.13.1 Calculate fim pumping capacity.

$$FPC = (N-1)\ (q)\ (0.00144)$$

where

FPC = fim pumping capacity, mgd.

2.49.5.5.13.2 If $0 < FPC \leq 7$ mgd:

$$OMH = 440\ (FPC)^{0.1285}$$

2.49.5.5.13.3 If $7 < FPC \leq 30$ mgd:

$$OMH = 294.4\ (FPC)^{0.3350}$$

2.49.5.5.13.4 If $30 < FPC \leq 80$ mgd:

$$OMH = 40.5\ (FPC)^{0.8661}$$

2.49.5.5.13.5 If $FPC > 80$ mgd:

$$OMH = 21.3\ (FPC)^{1.012}$$

where

OMH = operating manpower required, MH/yr.

2.49.5.5.14 Calculate maintenance manpower.

2.49.5.5.14.1 If $0 < FPC \leq 7$ mgd:

$$MMH = 360\ (FPC)^{0.1478}$$

2.49.5.5.14.2 If $7 < FPC \leq 30$ mgd:

$$MMH = 255.2\ (FPC)^{0.3247}$$

2.49.5.5.14.3 If $30 < FPC \leq 80$ mgd:

$$MMH = 85.7\ (FPC)^{0.6456}$$

2.49.5.5.14.4 If $FPC > 80$ mgd:

$$MMH = 30.6\ (FPC)^{0.8806}$$

where

MMH = maintenance manpower requirement, MH/yr.

2.49.5.5.15 Electrical energy required for operation.

2.49.5.5.15.1 Calculate the total dynamic head at average conditions. If the static head, SH, is not given by the user, the averaged total dynamic head would be 75% of the peak total dynamic head.

$$ATDH = 0.75\ TDH$$

where

ATDH = total dynamic head under average condition, ft.

If the static head, SH, is given.

$$ATDH = SH + 3 + \frac{376,460\ (Q_{avg})^{1.85}}{(DIAP)^{4.8655}}$$

where

ATDH = total dynamic head under average condition, ft.

SH = static head, ft.

DIAP = diameter of header pipe, inches.

2.49.5.5.15.2 The total annual electrical energy requirement would be:

$$KWH = 1352.65 \times Q_{avg} \times ATDH \times \left(\frac{1}{pumpe}\right)$$

where

KWH = annual electrical energy required, kwhr/yr.

Q_{avg} = average flow, mgd.

Pumpe = pump efficiency

2.49.5.5.16 Calculate operation and maintenance material and supply costs. This item covers the cost of lubrication oils, paint, repair and replacement parts, etc. It is expressed as a percent of the total bare construction costs.

$$OMMP = 0.7\%$$

where

OMMP = operation and maintenance material and supply costs, as percent of the total bare construction cost, percent.

2.49.5.5.17 Other minor construction cost items. From the calculations approximately 85 percent of the construction costs have been accounted for. Other minor items such as overhead crane, heat, ventilation, site cleaning, etc., would be 15 percent.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor construction costs.

2.49.5.6 Quantities Calculations Output Data.

2.49.5.6.1 Design capacity of pump, q, gpm.

2.49.5.6.2 Pump size, DPump, inches.

2.49.5.6.3 Pump speed, RPM, rpm.

2.49.5.6.4 Motor size, BHP, horsepower.

2.49.5.6.5 Pump building area, PBA, sq ft.

2.49.5.6.6 Volume of earthwork required, V_{ew}, cu ft.

2.49.5.6.7 Volume of R.C. slab required, V_{cs}, cu ft.

2.49.5.6.8 Volume of R.C. wall required, V_{cw}, cu ft.

2.49.5.6.9 Firm pumping capacity, FPC, mgd.

2.49.5.6.10 Operating manpower required, OMH, MH/yr.

2.49.5.6.11 Maintenance manpower required, MMH, MH/yr.

2.49.5.6.12 Electrical energy required, KWH, kwhr/yr.

2.49.5.6.13 Operation and maintenance material and supply costs, OMMP, percent.

2.49.5.6.14 Correction factor for other minor construction costs, CF.

2.49.5.7 Unit Price Input Required.

2.49.5.7.1 Unit price input for building cost, UPIBC, \$/sq ft.

2.49.5.7.2 Unit price input for earthwork, UPIEX, \$/cu ft.

2.49.5.7.3 Unit price input for R.C. wall, UPICW, \$/cu yd.

2.49.5.7.4 Unit price input for R.C. Slab, UPICS, \$/cu yd.

2.49.5.7.5 Cost of standard size pump COSTPS, \$, (optional).

2.49.5.7.6 Cost of standard size motor, COSTMS, \$ (optional).

2.49.5.7.7 Cost of variable frequency motor control, \$, (optional).

2.49.5.7.8 Current Marshall and Swift Equipment Cost Index, MSECI.

2.49.5.8 Cost Calculations.

2.49.5.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.49.5.8.2 Cost of pump building.

$$COSTPB = (PBA)\ (UPIBC)$$

where

COSTPB = cost of pump building, \$.

PBA = pump building area, sq ft.

UPIBC = unit price input for building cost, \$/sq ft.

2.49.5.8.3 Calculate cost of reinforced concrete walls.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of concrete walls, \$.

V_{cw} = volume of reinforced concrete for walls, cu ft.

UPICW = unit price input for concrete walls, \$/cu yd.

2.49.5.8.4 Calculate cost of reinforced concrete for slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of concrete slab, \$.

V_{cs} = volume of reinforced concrete for slab, cu ft.

UPICS = unit price input for concrete slab, \$/cu ft.

2.49.5.8.5 Purchase cost of pumps.

2.49.5.8.5.1 Cost of a single pump.

$$COSTP = (COSTRO)\ (COSTPS)$$

where

COSTP = cost of pump, \$.

COSTRO = cost pump as fraction of cost of standard size pump.

COSTPS = cost of standard size pump (16 inches), \$.

2.49.5.8.5.2 Calculate COSTRO.

If PumpD is smaller than 16 inches:

$$COSTRO = 0.041\ (PumpD)^{1.152}$$

If PumpD is equal or greater than 16 inches

$$COSTRO = 0.01\ (PumpD)^{1.663}$$

where

PumpD = pump size, inches.

2.49.5.8.5.3 Purchase cost of standard size pump. A 16 inch pump was selected as a standard. The cost of a 16 inch pump for the first quarter of 1977 is:

$$COSTPS = \$13,630$$

For better cost estimation, COSTPS should be obtained from the vendor and treated as a unit price input. If this is not done, the cost will be adjusted using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$13,630\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index first quarter 1977.

2.49.5.8.6 Purchase cost of motor.

2.49.5.8.6.1 Cost of a single motor.

$$COSTM = (COTMS)\ (5.4)\ \frac{(BHp + 5)^{0.85}}{(RPM)^{0.75}}$$

where

COSTM = cost of motor, \$.

COSTMS = cost of standard size motor. (100 Hp, 1200 RPM), \$.

BHp = motor horsepower, hp.

RPM = speed of motor, rpm.

2.49.5.8.6.2 Purchase price of standard size motor. A 100 Hp, 1200 RPM synchronous motor was selected as a standard. The cost of this motor for the first quarter 1977 is:

$$COSTMS = \$2,350$$

For better cost estimation, COSTMS should be obtained from the vendor and treated as a unit price input. If this is not done, the cost will be adjusted using the Marshall and Swift Cost Index:

$$COSTMS = 2350 \cdot \frac{MSECI}{491.6}$$

2.49.5.8.7 Purchase price of pumps and driver.

$$COSTPM = COSTP + COSTM$$

where

COSTPM = cost of pump and drivers, \$.

2.49.5.8.8 Installed equipment cost:

2.49.5.8.8.1 Guthrie gives the following factors to be used to estimate field installation cost of pump and drivers.

Items	Percent of Equipment Cost
Equipment cost	100.0%
Field Material	
Piping	29.6%
Concrete	3.9%
Instruments	2.9%
Electrical	30.3%
Insulation	2.5%
Paint	0.8%
	70.0%
Field Labor	
Material erection	59.0%
Equipment setting	8.9%
	67.9%
Total Direct Cost	237.9%

2.49.5.8.8.2 The installed equipment cost.

$$IPC = 2.379 \times (COSTPM)\ N$$

where

IPC = installed pumping equipment cost, $.

N = total number of pumps.

2.49.5.8.9 Variable speed control module. It is assumed the variable frequency drive system will be used.

2.49.5.8.9.1 If no variable speed pump is specified:

$$COSTV = 0$$

2.49.5.8.9.2 If variable speed pump is specified.

$$COSTV = (COSTVS)\ \mathrm{Exp}\ (0.00432\ BHp)$$

where

COSTV = purchase price of variable frequency drive module, $.

COSTVS = cost of a standard size variable frequency drive, $, (20HP).

BHp = horsepower of the motor, hp.

2.49.5.8.9.3 Cost of the standard size control module for a 20 HP motor.

$$COSTVS = \$11{,}500 \cdot \frac{MSECI}{491.6}$$

2.49.5.8.9.4 Installed cost. The following multipliers will have to be added to the purchase price of the equipment for installation cost.

Logic Control	20%
Motor Starter	10%
Installation	15%
Sum	45%

Thus the installed control module cost would be:

$$COSTVI = 1.45\ (COSTV)\ (NVS)$$

2.49.5.8.10 Total bare construction cost.

$$TBCC = CF\ (COSTE + COSTPB + COSTCW + COSTCS + IPC + COSTV)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor minor construction costs.

2.49.5.8.11 Operation and maintenance material and supply costs.

$$OMMC = (TBCC)\ \left(\frac{OMMP}{100}\right)$$

where

OMMC = operation and maintenace material and supply cost, \$.

OMMP = operation and maintenance material and supply costs, as a percent of total bare construction cost, percent.

2.49.5.9 Cost Calculations Output Data.

2.49.5.9.1 Total bare construction cost, TBCC, \$.

2.49.5.9.2 Operation and maintenance material and supply costs, OMMC, \$/yr.

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2.51 PRESSURE FILTRATION

2.51.1 Background.

2.51.1.1 Belt pressure filters are marketed in the United States by Passavant and Ralph B. Carter Company. The system is widely used in Europe, where it was jointly developed by two firms, Degemont and Phillipe. It is known as the "Flocpress" in Europe. Most of the comments related to filter presses are also applicable to belt pressure filters.

2.51.1.2 In this design the sludge is spread across a woven synthetic fiber belt, which travels horizontally for a variable length where the action of both capillarity and gravity will allow a natural drainage. This belt, after running horizontally on supporting rollers, wraps around a rubber-covered drum provided with grooves for draining away the filtrate. The action of a continuous pressure belt of cloth reinforced rubber subjects a pressure by gradually decreasing the gap between the filter belt and the pressure belt as they move forward, so that the pressure applied increases over the whole length of the filtration zone. The dried cake is then removed from the filter belt by means of a flexible scraper. A second scraper is also needed to clean the pressure belt.

2.51.1.3 Chemical conditioning is required for belt pressure filters just as it is for filter presses. One notable exception is that, while filter presses usually reouire media conditioning, belt pressure filters do not generally require conditioning. Chemical addition, mixing, flocculation, and gravity thickening are required. In most instances a polymer addition is helpful.

2.51.1.4 There is a tendency for sludge to flow out the edges between the belts and to limit the applied pressure to slightly more than two atmospheres as pressure increases. Because of pressure limitation, the dryness of the cake never approaches that of filter presses. Table 2.51-1 indicates typical operating perfomances of the "Flocpress".

2.51.2 Input Data.

2.51.2.1 Average wastewater flow, mgd.

2.51.2.2 Sludge volume, gallons/million gallons.

2.51.2.3 Raw sludge solids concentration, %.

2.51.2.4 Sludge produced by conditioning chemicals.

Table 2.51-1. Operating Performance for the Flocpress

Type of Sludge	Feed Sludge Percent Dry Solids	Output in lbs Per ft of Belt Width Per Hr	Cake Percent Dry Solids	Polymer lbs per Dry Ton
Municipal Sewage				
Raw Primary	5-10	161-268	25-35	1.8-4
Anaerobic Digested - Primary	4-10	168-235	26-36	2.0-6
Mixed Primary and Waste Activated	4-9	87-168	20-28	1.5-6
Anaerobic Digested Primary and Waste Activated	3.5-9	134-268	18-23	3.4-8
Aerobic Digested	1.5-2.5	54-101	12-18	4.0-8
Raw Slaughter House Waste	2.5-3.5	134-302	20-30	5.0-8
Lime Softening Sludge	10-15	335-670	55-70	0.4-1
Alum Sludge	3-6	54-85	14-18	1.5-5
White Water Sludge				
Paper Mill	2.5-4	67-268	20-25	2.0-4
Manufacturer of Semi-Chem Pulp				
Fibers and Sawdust	4-7	134-335	25-32	0
Sawdust	8-15	402-670	30-35	0

2.51.2.5 Cake solids, %.

2.51.2.6 Cake density, lb/ft^3.

2.51.2.7 Belt speed, ft/hr.

2.51.2.8 Loading rate, lb/ft of width/hr.

2.51.2.9 Operating schedule, hr/day.

2.51.3 Design Parameters.

2.51.3.1 Sludge volume per million gallons treated.

2.51.3.2 Raw sludge solids concentration, %. 1.5-15%

2.51.3.3 Conditioning sludge, from laboratory studies. Use 0.2 of sludge solids as an estimate if laboratory data are not available.

2.51.3.4 Type and amount of conditioning chemicals, %, from laboratory studies.

2.51.3.5 Cake solids. (Table 2.51-1).

2.51.3.6 Cake density. Approximately 70-80 lb/ft^3.

2.51.3.7 Belt speed, from laboratory studies. If unknown, use 10-15 ft/hr, or an average of 12 ft/hr.

2.51.3.8 Loading rate, from laboratory studies. If unknown, use 5-8 $lb/ft^2/hr$, or an average of 6.5 $lb/ft^2/hr$.

2.51.3.9 Operating schedule, hours per day. (8-16 hr).

2.51.4 Process Design Calculations.

2.51.4.1 Calculate the pounds of dry solids in sludge flow per day.

$$DSS = \frac{(Q_{avg})(SF)(SS)(8.34)}{100}$$

where

DSS = pounds of dry sludge solids per day.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gallons/million gallons.

SS = suspended solids in flow to pressure filter, %.

2.51.4.2 Calculate the total pounds of dry solids produced per day.

$$PDSPD = DSS + (CS)(DSS)$$

where

PDSPD = dry solids produced daily, pounds.

CS = conditioning solids, expressed as a fraction of sludge solids. (0.2)

2.51.4.3 Calculate the weight of the filter cake produced.

$$PFC = \frac{PDSPD}{CSC} \times 100$$

where

PFC = pounds of filter cake produced per day, net weight.

CSC = cake solids content, %.

2.51.4.4 Determine the cake volume.

$$CV = \frac{PFC}{CD}$$

where

CV = cake volume, ft^3/day.

CD = cake density, lb/ft^3.

2.51.4.5 Calculate the total area of pressure filter needed to dewater the specific sludge at specified loading rate.

$$AF = \frac{PDSPD}{(HPD)(SLR)}$$

where

AF = area of pressure filter, ft^2.

HPD = operating schedule for unit, hours per day.

LR = solids loading rate, lb/ft^2/hr.

2.51.4.6 Consult manufacturer's literature to select appropriate filter, based on the area calculated and desired dewatering time (typically 3-6 hours) and the belt speed determined from laboratory studies.

2.51.5 Process Design Output Data.

2.51.5.1 Volume of filter cake, ft^3.

2.51.5.2 Cake solids content, %.

2.51.5.3 Weight of filter cake, lb/ft^3.

2.51.5.4 Area of pressure filter, ft^2.

2.51.6 Quantities Calculations. Not Used.

2.51.7 Quantities Calculations Output Data. Not Used.

2.51.8 Unit Price Input Required. Not Used.

2.51.9 Cost Calculations.

2.51.9.1 Calculate operation and maintenance cost.

$$X = \log (Q_{avg})$$

$$Z = 1.3906 - 0.73944(X) + 0.081625(X)^2$$

$$O\&M = \frac{e^Z \, Q_{avg} \times 10^3}{100}$$

where

O&M = operation and maintenance cost, \$/yr.

2.51.9.2 Calculate total bare construction cost.

$$Y = -0.69698 - 0.12594(X) + 0.095578(X)^2$$

$$TBCC = e^Y$$

where

TBCC = total bare construction cost, \$.

2.51.10 Cost Calculations Output Data.

2.51.10.1 Total bare construction cost, TBCC, \$.

2.51.10.2 Operation and maintenance cost, O&M, \$/yr.

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2.53 RECARBONATION

2.53.1 Background.

2.53.1.1 Recarbonation is a unit process that has long been used in lime-softening water treatment plants. In water treatment, recarbonation is usually practiced ahead of the filters to prevent calcium carbonate deposition on the grains which will result in shortening of the filter runs. Recarbonation is also used to lower the pH of the lime-treated water to the point of calcium carbonate stability to avoid deposition of calcium carbonate in pipelines.

2.53.1.2 More recently, with the increased use of lime treatment of wastewaters, recarbonation has been more widely used in wastewater treatment. Recarbonation, in wastewater treatment, is mainly used to adjust the pH following lime treatment for such applications as phosphorus removal, ammonia stripping, or chemical clarification.

2.53.1.3 Recarbonation may be practiced as either a two-stage or a single-stage system. Two-stage recarbonation consists of two separate treatment steps. In the first stage, sufficient carbon dioxide is added in the primary recarbonation stage to lower the pH of the wastewater to pH = 9.3, which is near the minimum solubility of calcium carbonate. The sludge produced, which is mainly calcium carbonate, is then removed through settling and recalcined if recovery of the lime is desired. The time required to complete the reaction is normally 15-30 min. In the second stage, carbon dioxide is added to lower the pH to a value of pH = 7. It is possible, however, to add sufficient carbon dioxide to lower the pH from 11 to 7 in a single stage. Single-stage recarbonation eliminates the need for an intermediate settling basin which is needed in the two-stage system. However, single-stage recarbonation normally results in an increase in the calcium hardness of the water.

2.53.1.4 The reactions involved in the recarbonation process may be simplified as follows:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

$$CaCO_3 + CO_2 + H_2O \rightarrow Ca(HCO_3)_2$$

2.53.1.5 The amount of CO_2 needed to complete the reactions, as calculated from the equations above is as follows:

Calcium hydroxide to calcium carbonate.

CO_2 (lb/million gal) = 3.7 (OH^- alkalinity in mg/l as $CaCO_3$)

Calcium carbonate to calcium bicarbonate.

CO_2 (lb/million gal) = 3.7 ($CO_3^=$ alkalinity in mg/l as $CaCO_3$).

2.53.2 Input Data.

2.53.2.1 Wastewater flow.

2.53.2.1.1 Average flow.

2.53.2.1.2 Peak flow.

2.53.2.2 Wastewater characteristics.

2.53.2.2.1 Alkalinity.

2.53.2.2.2 Hydroxide alkalinity, mg/l as $CaCO_3$.

2.53.2.2.3 pH.

2.53.3 Design Parameters.

2.53.3.1 Contact time, min (15-30).

2.53.3.2 Carbon dioxide dose, lb/million gal.

2.53.3.3 Desired effluent pH.

2.53.4 Process Design Calculations.

2.53.4.1 Two-stage.

2.53.4.1.1 Primary stage to pH = 9.3.

2.53.4.1.1.1 Calculate tank volume,

$$V = \frac{Q(t)10^6}{60(24)}$$

where

V = tank volume, gal.

Q = wastewater flow, mgd.

t = contact time, min (15-30).

2.53.4.1.1.2 Calculate CO_2 requirement.

$$CO_2 = (3.7)(OH^- (Q)/0.116/1440)$$

where

CO_2 = carbon dioxide requirement, cfm/mgd.

3.7 = stochiometric value to convert hydroxide to carbonate.

OH^- = hydroxide alkalinity, in mg/l as $CaCO_3$.

Q = wastewater flow, mgd.

0.116 = density of CO_2, lb/ft^3.

1440 = min/day.

2.53.4.1.2 Secondary stage to pH = 7.

2.53.4.1.2.1 Calculate tank volume,

$$V = \frac{Q(t)10^6}{24(60)}$$

where

V = volume, gal.

Q = wastewater flow, mgd.

t = contact time, min (15-30 min).

2.53.4.1.2.2 Calculate CO_2 requirement.

$$CO_2 = 3.7(CO_3^= + OH^-)(Q)/0.116/1440$$

where

$CO_3^=$ = carbonate alkalinity, mg/l as $CaCO_3$.

OH^- = hydroxide alkalinity, mg/l as $CaCO_3$.

Q = wastewater flow, mgd.

2.53.4.2 Single-stage recarbonation to pH = 7.

2.53.4.2.1 Calculate volume of tank.

$$V = (Q)(t)/60/1440$$

where

V = volume, million gal.

t = contact time, min (5-30 min).

2.53.4.2.2 Calculate CO_2 requirements.

$$CO_2\ (cfm/mgd) = 7.4(OH^-) + 3.7\ (CO_3^=)(Q)/0.116/1440$$

where

OH^- = hydroxide alkalinity, mg/l as $CaCO_3$.

$CO_3^=$ = carbonate alkalinity, mg/l as $CaCO_3$.

2.53.5 Process Design Output Data.

2.53.5.1 Volume of tank, million gal.

2.53.5.2 Carbon dioxide requirement, cfm/mgd.

2.53.5.3 Final pH.

2.53.5.4 Contact time, min.

2.53.6 Quantities Calculations. Not used.

2.53.7 Quantities Calculations Output Data. Not used.

2.53.8 Unit Price Input Required. Not used.

2.53.9 Cost Calculations. At present no cost data are available for recarbonation units. As costs are made available they will be included.

2.53.10 Cost Calculations Output Data. Not used.

2.53.11 Bibliography.

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2.55 ROTATING BIOLOGICAL CONTACTOR SYSTEM

2.55.1 Background.

2.55.1.1 The rotating biological contactor (RBC) system consists of a series of plastic discs mounted on a shaft. These discs are partially submerged in the wastewater and slowly rotated, thus exposing them alternately to air and wastewater. The discs act as supporting media for the biological growth which aerobically treats the wastes. Growth thickness is controlled by the shear forces produced by the water acting on the rotating discs. Covering the shafts is necessary to prevent sun-induced algae growth as well as to prevent freezing in colder climates. RBC shafts are available in lengths of from 10 to 25 feet and support from 40,000 to 150,000 square feet of media per shaft. In order to simplify the design procedure, a shaft of 100,000 square feet of surface area is used in this section.

2.55.1.2 The main advantages RBC has over activated sludge and trickling filters are reduced land requirements and lower power consumption. With increased concern in these areas, RBC has been receiving increased attention. Although this process is relatively new to the U.S. (since 1969), it has been used successfully in Europe for the past 20 years.

2.55.1.3 RBC is applicable to both carbonaceous removal and nitrification, however, this design procedure applies only to the former.

2.55.1.4 New developments in RBC drives, such as air drive system, are presently available from some manufacturers which reduce maintenance costs, however only mechanical drives will be considered here to represent a more accurate cross section of available units.

2.55.2 Input Data.

2.55.2.1 Influent soluble BOD, mg/l (optional).

2.55.2.2 Influent BOD, mg/l.

2.55.2.3 Design effluent total BOD, mg/l.

2.55.2.4 Design flow rate, mgd.

2.55.2.5 Peak design flow, mgd.

2.55.2.6 Wastewater temperature, ^{o}C.

2.55.2.7 Media surface area per shaft, $ft.^2$ (default value of 100,000).

2.55.3 Design Parameters.

2.55.3.1 Influent soluble, BOD, mg/l.

2.55.3.2 Hydraulic loading.

2.55.3.3 Media surface area per shaft, ft^2.

2.55.4 Process Design Calculations.

2.55.4.1 The governing factor in the design of RBC system has been shown to be the soluble BOD_5 loadings to the media surface. However, very few soluble BOD values of municipal sewage are available. In this design, if soluble BOD value is not given, the following empirical equation is utilized to synthesize an influent soluble BOD_5 value from the total BOD given.

$$I = 0.6\ IH$$

where

I = Influent Soluble BOD, mg/l

IH = Total Influent BOD, mg/l

2.55.4.2 Calculate design hydraulic loading. The following design procedure is suggested by one of the leading manufacturers of RBC systems.

2.55.4.2.1 If the design effluent total BOD is between 30 and 50 mg/l,

$$H = \frac{E - 0.06I + 50}{0.3I + 1.58}$$

2.55.4.2.2 If the design effluent total BOD is between 10 and 29 mg/l,

$$H = \frac{E + 9.94}{0.17I - 0.37}$$

where

H = hydraulic loading, $gpd/ft.^2$

E = design effluent total BOD, mg/l.

I = influent soluble BOD, mg/l.

2.55.4.3 Temperature Effect. Since the empirical equation is expressed with regard to temperature in °F, the influent temperature will be corrected by using the following equation:

$$TF = 1.8 \times T + 32$$

If the average wastewater temperature is expected to be below $55^{o}F$, the hydraulic loading must be corrected to achieve complete treatment. The following equation is provided for this correction:

$$C = 0.0225(TF) - 0.25$$

$$H_c = C \times H$$

where

C = correction factor.

TF = average wastewater temperature, ^{o}F.

H_c = corrected hydraulic loading, gpd/ft.^{2}.

H = hydraulic loading, gpd/ft.^{2}.

2.55.4.4 Safety Factor for Variation of Influent Flow. The design formula used in the above procedure was empirically obtained under the condition that no great variation of flow was received at the plant.

One of the RBC manufacturers suggested that when the peak flow to average flow ratio is more than 2.5, a safety factor should be used to give a conservative design. The safety factor is calculated by using the following equation.

$$SF = 1 + 0.143 \left(\frac{Qpk}{Qavg} - 2.5\right)$$

where

SF = safety factor

Qpk = peak flow in mgd

Qavg = average design flow, mgd

and the design hydraulic loading, Hd would be:

$$Hd = Hc/SF$$

where

Hd = design hydraulic loading, gpd/ft.^{2}.

Hc = corrected hydraulic loading.

2.55.4.5 Calculate the required media surface area.

$$A = \frac{Qavg \times 10^6}{H_d}$$

where

A = required media surface area, $ft.^2$.

Qavg = design flow rate, mgd.

H_d = designed hydraulic loading, $gpd/ft.^2$.

2.55.4.6 To ensure that the first stage is not overloaded, which creates anaerobic conditions, it must contain a certain percentage of the total media area.

The percentage is given as:

$$P = 2.09 \times 10^{-3}\, H_c I$$

$$A_{fs} = P \times A$$

where

P = percent of total media surface area required in first stage, decimal fraction.

H_c = corrected hydraulic loading, $gpd/ft.^2$.

I = influent soluble BOD, mg/l.

A_{fs} = media surface area required for first stage, $ft.^2$.

A = total media surface area, $ft.^2$.

2.55.4.7 Determine the recommended number of stages using the following table:

E (mg/1)	N_{st}
less than 30	4
between 30 and 50	3
between 50 and 120	2
greater than 120	1

where

E = design effluent BOD, mg/l.

N_{st} = recommended number of stages.

2.55.4.8 Secondary Clarifier Loading. Secondary clarifier following the rotating contactor can be designed in accordance with accepted standards. However, it is recommended that a surface overflow rate of 400 - 700 gpd/ft.^{2} should be used.

Because rotating contactor effluent will generally contain 100 to 150 mg/l of suspended solids prior to reaching the secondary clarifier, the solids loading on the clarifier is quite low. Therefore, surface overflow rate is the only important design factor.

2.55.4.9 Effluent Quality.

2.55.4.9.1 BOD_5.

Both effluent BOD_5 and effluent soluble BOD_5 are specified by the user.

2.55.4.9.2 Suspended solids.

The effluent suspended solids is specified by the user.

2.55.4.9.3 COD.

$$CODE = 1.5\ BODE$$
$$CODES = 1.5\ BODES$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODES = effluent soluble COD concentration, mg/l.

BODES = effluent soluble BOD_5 concentration, mg/l.

2.55.4.9.4 Nitrogen.

$$TKNE = 0.7\ TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3 + 0.3\ TKN$$
$$NO2E = 0.0$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitorgen concentration, mg/l.

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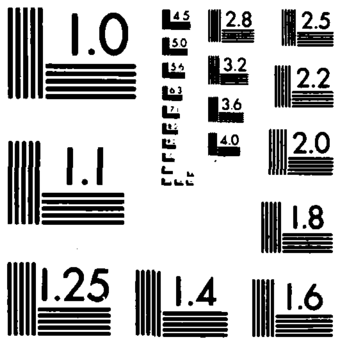

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

NH3E = effluent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

2.55.4.9.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.55.4.9.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

2.55.4.9.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.55.4.10 Sludge Production.

2.55.4.10.1 Sludge production by the rotating biological contactor process has been shown to be related to the percent BOD removal through the system.

The total dry solids produced can be estimated by using:

(INF. BOD − EFF. BOD)

$$SLDS = Y \times (IH-E) \times 8.34 \times Qavg$$

and the volatile portion is usually 80% of the total solids. Thus the volatile solids production rate is:

$$SLVDS = 0.8 \times SLDS$$

where

SLDS = sludge production rate, lbs. of dry solids/day.

SLVDS = volatile sludge product rate, lbs. of dry solids/day.

Y = sludge yield factor, lbs. of sludge/lb. BOD removed.

2.55.4.10.2 The RBC sludge from the bottom of clarifiers has been shown to be in the range of 1 - 2% solids. Thus the sludge flow rate is:

$$SLF = \frac{SLDS \times 10^6}{SLCON \times 8.34 \times S.G.}$$

SLF = sludge flow rate, gpd.

10,000 mg/l

SLCON = sludge concentration, use $\boxed{15,000}$ mg/l.

S.G. = Specific gravity

2.55.5 <u>Process Design Output Data.</u>

2.55.5.1 Average design flow, Qavg, mgd.

2.55.5.2 Influent BOD concentration, IH, mg/l

2.55.5.3 Influent soluble BOD concentration, I, mg/l

2.55.5.4 Design hydraulic loading, H_d, gpd/sq. ft.

2.55.5.5 Required media surface area, A, $ft.^2$

2.55.5.6 Media surface area required for first sgate, A_{fs}, $ft.^2$

2.55.5.7 Recommended number of stages, N_{st}

2.55.5.8 Recommended secondary clarifier overflow rate, $gpd/ft.^2$

2.55.5.9 Effluent suspended solids concentration, ESS, mg/l

2.55.5.10 Total sludge production, SLDS, lbs./day

2.55.5.11 Total volatile solids production, SLVDS, lbs./day

2.55.5.12 Total sludge flow, SLF, gpd

2.55.6 Quantities Calculations.

2.55.6.1 Calculate number of shafts necessary.

$$N_{sh} = \frac{A}{A_{sh}}$$

If N_{sh} is not an integer, use the next larger integer.

where

N_{sh} = number of shafts necessary.

A = total media surface area, ft.^{2}.

A_{sh} = media surface area per shaft, ft.^{2}.
(default value is 100,000)

2.55.6.2 Calculate number of shafts necessary in first stage.

$$N_{fs} = PN_{sh}$$

If N_{fs} is not an integer, use next larger integer.

where

N_{fs} = number of shafts in first stage.

P = percent of total media surface area required in first stage, decimal fraction.

N_{sh} = number of shafts necessary.

2.55.6.3 Calculate number of shafts in each subsequent stage, N_{ss}.

2.55.6.3.1 If the design effluent total BOD is greater than 120 mg/l, then only one stage is required and there are no subsequent stages.

$$\text{If } N_{st} = 1, \text{ then } N_{ss} = 0$$

2.55.6.3.2 If the design effluent total BOD is less than 120 mg/l, then the following formula should be used.

$$N_{ss} = \frac{N_{sh} - N_{fs}}{N_{st} - 1}$$

If N_{ss} is not an integer, use next larger integer.

where

N_{ss} = number of shafts in each stage following the first stage.

N_{sh} = total number of shafts required.

N_{fs} = number of shafts in first stage.

N_{st} = number of stages recommended.

2.55.6.4 Calculate minimum practical number of shafts needed. This step is necessary due to rounding in previous calculations.

$$N_{mp} = N_{fs} + (N_{st} - 1)\, N_{ss}$$

where

N_{mp} = minimum practical number of shafts needed.

N_{fs} = number of shafts needed in first stage.

N_{st} = number of stages recommended.

N_{ss} = number of shafts in each stage following the first stage.

2.55.6.5 For simplicity of design, it is assumed that shafts will be arranged in groups of eight. Each group will be called a bank and will consist of two end shafts and six intermediate shafts. Any shafts in excess of a multiple number of eight will fom a partial bank of from one to seven shafts as needed. Many other configurations are possible; however, varying the configuration should not affect earth work and concrete requirements significantly.

2.55.6.5.1 Calculate number of banks needed.

$$N_b = N_{mp}/8$$

If N_b is not an integer, use next larger integer.

where

N_b = number of full and partial banks.

N_{mp} = minimum practical number of shafts needed.

2.55.6.5.2 Calculate number of end and intermediate shafts needed.

$$N_{es} = 2\ N_b$$

$$N_{is} = N_{mp} - N_{es}$$

where

N_{es} = number of end shafts needed.

N_b = number of banks needed.

N_{is} = number of intermediate shafts needed.

N_{mp} = minimum practical number of shafts needed.

2.55.6.6 Calculate earthwork requirements.

$$V_{ew} = 130\ N_{es} + 142\ N_{is}$$

where

V_{ew} = volume of earthwork required, yd.^{3}.

N_{es} = number of end shafts needed.

N_{is} = number of intermediate shafts needed.

2.55.6.7 Calculate reinforced concrete requirements.

2.55.6.7.1 $V_{sc} = 23\ N_{es} + 20.7 N_{is}$

2.55.6.7.2 $V_{wc} = 11.5\ N_{es} + 8.6\ N_{is}$

where

V_{sc} = volume of slab concrete needed, yd.^{3}.

V_{wc} = volume of wall concrete needed, yd.^{3}.

N_{es} = number of end shafts needed.

N_{is} = number of intermediate shafts needed.

2.55.6.8 Calculate annual power consumption:

2.55.6.8.1 For a hydraulic loading between 0.5 and 2.5 gpd/ft^2.

$$KWH = [3^{(3.9 - 1.21 H_d)} + 7.12] \quad 6,534\ Q_{avg}$$

2.55.6.8.2 For a hydraulic loading between 2.5 and 4.5 gpd/ft.^{2}

$$KWH = (17.05 - 2.5H_d)\ 6,534\ Q_{avg}$$

where

KWH = annual power consumption, kwh/yr.

H_d = hydraulic loading, gpd/ft.^{2}.

Qavg = design flow rate, mgd.

2.55.6.9 Calculate annual operation and maintenance labor requirements.

2.55.6.9.1 For a plant with less than 30 shafts

$$L_{om} = (1.25 - 0.025\ N_{mp})\ 52\ N_{mp}$$

2.55.6.9.2 For a plant with 30 shafts or more.

$$L_{om} = 26\ N_{mp}$$

where

L_{om} = annual operation and maintenance labor requirements, person-hrs./yr.

N_{mp} = minimum practical number of shafts needed.

2.55.6.9.3 It is assumed that 70% of the man hour requirement is allocated for maintenance and the rest to operation. Thus,

Operation man hour requirement, OMH, man hr./yr.

$$OMH = 0.3 \times L_{om}$$

Maintenance man-hour requirement, MMH, man-hr./yr.

$$MMH = 0.7 \times L_{om}$$

2.55.6.10 Other Operation and Maintenance Material Costs. This item includes repair and replacement material cost and other minor costs, such as lubricating oil. It is expressed as a percent of installed costs of the RBC equipment.

OMMP = 2%

OMMP = Percent of the installed RBC equipment costs.

2.55.6.11 Other Construction Cost Items. From the above estimation, approximately 85% of the total construction costs have been accounted for. Other minor cost items such as piping, site work, control system, etc. would be the other 15 percent.

CF, correction factor would be $\frac{1}{.85} = 1.18$

2.55.7 Quantities Calculations Output Data.

2.55.7.1 Minimum practical number of shafts need, N_{mp}

2.55.7.2 Number of shafts in first stage, N_{fs}.

2.55.7.3 Earthwork required, V_{ew}, Cu. Yd.

2.55.7.4 R.C. wall quantity, V_{wc}, Cu. Yd.

2.55.7.5 R.C. slab quantity, V_{sc}, Cu. Yd.

2.55.7.6 Annual power consumption, KWH, kwhr/yr.

2.55.7.7 Operational man hour requirement, OMH man-hr./yr.

2.55.7.8 Maintenance man hour requirement, MMH man-hr./yr.

2.55.7.9 Other operation and maintenance material cost, OMMP, percent.

2.55.7.10 Correction factor, CF.

2.55.8 Unit Price Inputs Required.

2.55.8.1 Cost of earthwork, UPIEX, \$/cu yd.

2.55.8.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.55.8.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.55.8.4 Standard size RBC equipment cost, COSRBC, \$ (optional).

2.55.8.5 Marshall and Swift Equipment Cost Index, MSECI.

2.55.9 <u>Cost Calculations</u>.

2.55.9.1 Cost of earthwork, COSTE.

$$COSTE = V_{ew} \times UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of concrete wall, cu. yd.

UPIEX = unit price input of earthwork, \$/cu yd.

2.55.9.2 Cost of concrete wall in-place, COSTCW.

$$COSTCW = V_{CW} \times UPICW$$

where

COSTCW = cost of concrete wall in-place, \$.

V_{cw} = quantity of concrete wall, cu. yd.

UPICW = unit price input cost of concrete wall in-place. \$/cu. yd.

2.55.9.3 Cost of concrete slab in-place, COSTCS.

$$COSTCS = V_{cs} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of concrete slab, cu. yd.

UPICS = unit price input of R.C. slab in-place, \$/cu. yd.

2.55.9.4 Cost of Installed Equipment.

2.55.9.4.1 Purchase cost of RBC equipment. This includes the cost for the media, shaft and fiberglass cover. The first quarter, 1977 cost of a shaft with media surface area of 100,000 sq. ft. is approximately \$30,500. In this procedure, it is preferred that the cost of equipment is input from user. If no user input is given, the default value is given:

$$COSRBC = 30{,}500 \times \frac{MSECI}{491.6}$$

where

COSRBC = purchase cost of standard size media with surface area of 100,000 sq. ft, \$.

MSECI = current Marshall and Swift equipment cost index from input.

491.6 = Marshall and Swift cost index first quarter 1977.

2.55.9.4.2 Installation cost. It is estimated that a 15% markup should be added for installation of the RBC equipment. Thus, the installed equipment cost, IEC.

$$IEC = 1.15 \times COSRBC\ (N_{mp})$$

where

IEC = installed equipment cost, \$.

N_{mp} = minimum practical number of shafts needed.

2.55.9.5 Other Cost Items. This category includes cost of piping, walkways, electrical control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF to the sum of other costs.

2.55.9.5.1 Total bare construction costs, TBCC.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC) \times CF$$

where

TBCC = total bare construction costs, $.

CF = correction factor for minor cost items.

2.55.9.6 Operation and Maintenance Material Costs. Since this item of the operation and maintenance costs is expressed as a percentage of the installed equipment cost, it can be calculated by:

$$OMMC = \frac{OMMP}{100} \times IEC$$

where

OMMC = operation and maintenance material costs, $/yr.

OMMP = percentage of the total bare construction costs as operation and maintenance material cost, percent.

2.55.10 Cost Calculations Output Data.

2.55.10.1 Total bare construction costs of the RBC system cost, TBCC, $.

2.55.10.2 Operation and maintenance material costs, OMMC, $/yr.

2.55.11 Bibliography.

2.55.11.1 Antonie, R.L., Fixed Biological Surfaces - Wastewater Treatment, CRC Press, Inc. Cleveland, Ohio, 1976.

2.55.11.2 Antonie, R.L. "Response of the Bio-Disc Process to Fluctuating Wastewater Flows", Proce. 25th Cenf. of Purdue Ind. Waste, p. 427, 1970.

2.55.11.3 Autotrol, Co., Biosurf Design Manual, January, 1979.

2.57 SLUDGE DRYING BEDS

2.57.1 Background.

2.57.1.1 Sludge drying beds are a common method for dewatering digested sludge, especially in small plants. Drying beds are usually constructed using 4-9 inches of sand over 8-18 inches of graded gravel. The beds are usually divided into at least three sections for operational purposes. An underdrain system usually of vetrified clay pipe, spaced 9-20 ft apart, is used to remove water.

2.57.1.2 The design of sludge beds is influenced by many factors, such as weather conditions, sludge characteristics, land value, proximity of residences and use of sludge conditioning aids.

2.57.2 Input Data.

2.57.2.1 Sludge flow, gpd.

2.57.2.2 Solids in thickened sludge, %.

2.57.2.3 Solids desired, %.

2.57.3 Design Parameters.

2.57.3.1 Depth of sludge applied (8-12 in).

2.57.3.2 Days (T) in which drainage is the primary drying mechanism (1-8 days).

2.57.3.3 Solids after T days (15-25%).

2.57.3.4 Clearwater evaporation rate (available from the U. S. Weather Bureau).

2.57.3.5 Correction of evaporation rate for sludge (0.75).

2.57.3.6 Average rainfall in wet month (available from the U. S. Weather Bureau).

2.58.3.7 Fraction of rainfall absorbed by the sludge (0.57).

2.57.3.8 Number of sections desired.

2.57.4 Process Design Calculations.

2.57.4.1 Fill several columns with thickened/digested sludge to desired application depth. The bottom of the column should contain sand and gravel in similar depths to that expected in the bed. Check the bed daily until drainage from the bottom has essentially ceased. Record the number of days of drainage (t_d) and the percent solids in the sludge.

2.57.4.2 Calculate the required drying time.

$$T = \frac{30 \times H \times S_o}{aE - bR}\left(\frac{1}{S_1} - \frac{1}{S_2}\right) + t_d$$

where

T = total drying time, days.

H = depth to which sludge is applied, in.

S_o = initial solids, percent.

a = correction of evaporation rate for sludge, 0.75.

E = clearwater evaporation rate sludge, in/month.

b = fraction of water absorbed by sludge, 0.57.

R = rainfall during wet month, in/month.

S_1 = solids content after t_d days, percent.

S_2 = final solids content, percent.

t_d = time during which drainage is significant, days.

2.57.4.3 Calculate the surface area.

$$SA = \frac{Q_s \times T(12)}{H(7.48)}$$

where

SA = surface area required, ft^2.

Q_s = volumetric sludge flow, gpd.

2.57.4.4 Calculate the solids produced.

$$DTPY = Q_s S_o \left(\frac{8.34\ lb/gal}{100} \right) (365\ days/yr) \left(\frac{ton}{2000\ lb} \right)$$

where

DTPY = tons of dry solids per year.

2.57.4.5 Calculate weight of solids removed.

$$TPY = \frac{(DTPY)(100)}{S_2}$$

where

TPY = total tons per year removed.

S_2 = final solids content, %.

2.57.5 Process Design Output Data.

2.57.5.1 Area required, ft^2.

2.57.5.2 Depth of sludge application, in.

2.57.5.3 Number of sections.

2.57.5.4 Area of each section, ft^2.

2.57.5.5 Drying time in bed, days.

2.57.6 Quantities Calculations.

2.57.6.1 Uncovered drying beds with truck tracks for ease of sludge pickup (See Figure 2.57-1) will be utilized in this manual. A minimum of three beds will be provided for alternate sludge application, drying and cleanup. A typical width of 20 feet is utilized. This limitation is imposed by the constraint of manual sludge pickup and transfer to trucks. The length of bed is limited to less than 150 feet. This is due to the hydraulic constraint of the drainage pipes.

2.57.6.2 Selection of the number of beds. The selection of the number of drying beds is dependent on the total surface area required. The following rule will be followed in this selection.

$$N = 3 + \frac{(SA - 9,000)}{3000}$$

N must be an integer and should always be larger than 3.

where

N = number of beds.

SA = total drying bed surface area required, sq ft.

2.57.6.3 Dimensions of drying beds.

2.57.6.3.1 Surface area of each individual bed, SAN, sq ft.

$$SAN = \frac{SA}{N}$$

2.57.6.3.2 Length of each bed, LN, ft.

$$LN = \frac{SAN}{20}$$

where

20 = width of bed, ft.

2.57.6.4 Earthwork required for drying bed construction. It is assumed that the depth of cut would be 4 feet and the slope of cut would be 1:1. Thus the total volume of cut can be expressed by:

$$V_{ew} = [48\ N\ (LN) + 224\ N + 560\ N^2\ (LN)^2 + 4480\ N^2\ (LN)]\ \frac{4}{3}$$

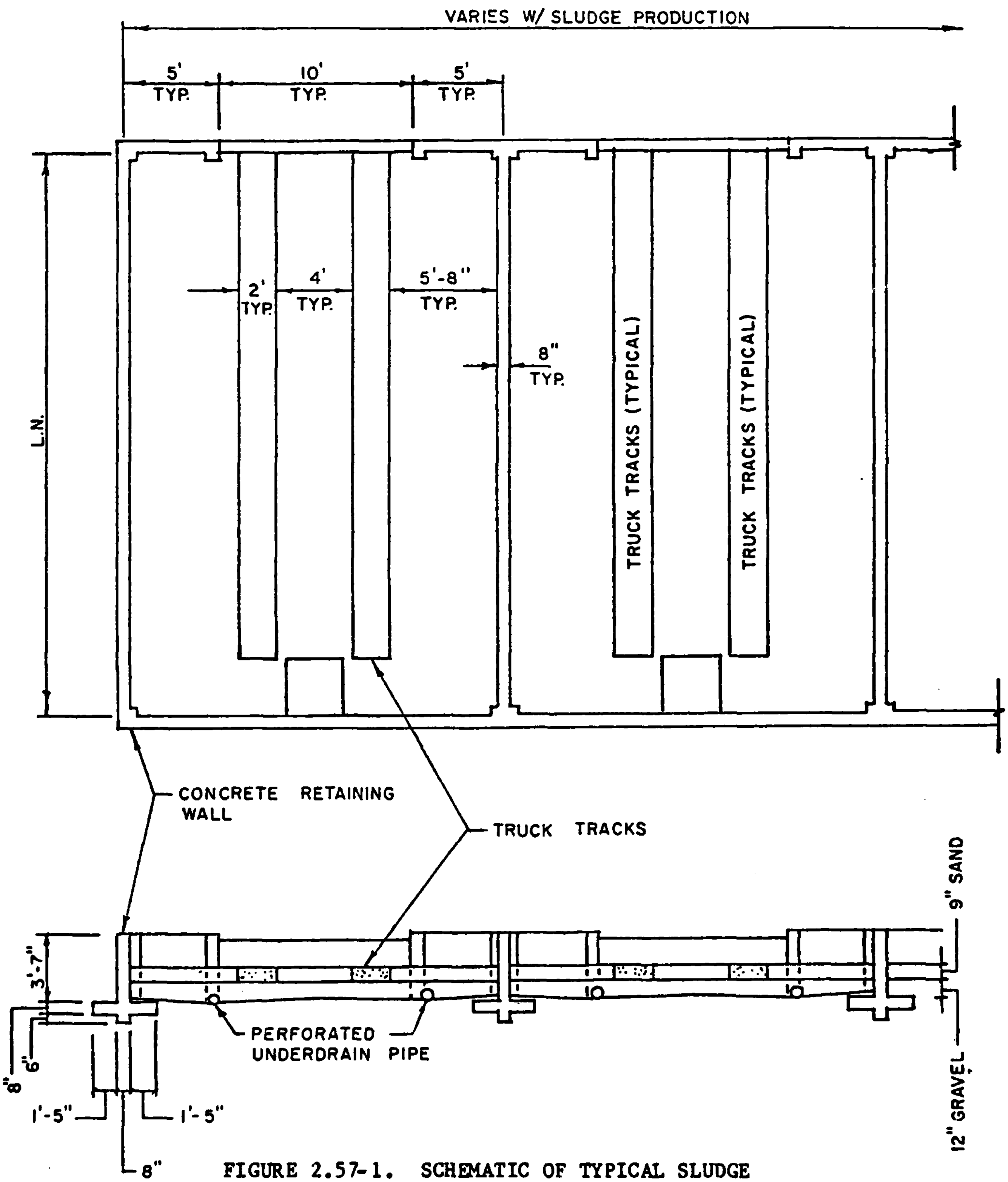

FIGURE 2.57-1. SCHEMATIC OF TYPICAL SLUDGE DRYING BED

where

V_{ew} = volume of earthwork in cu ft.

2.57.6.5 Quantity of reinforced concrete required.

2.57.6.5.1 This item includes the concrete dividing walls and the truck tracks.

2.57.6.5.2 The typical cross section of the dividing wall is shown in Figure 2.57-1. The quantity of wall would be:

$$V_{cw} = 5.06\ [40 \cdot N + (N)\ (LN) + (LN)]$$

where

V_{cw} = volume of reinforced concrete in-place, cu ft.

2.57.6.5.3 Volume of concrete required for the construction of truck tracks:

$$V_{cs} = 3\ (N)\ (LN)$$

where

V_{cs} = volume of R.C. in-place, cu ft.

2.57.6.6 Quantity of sand and gravel.

2.57.6.6.1 It is assumed that a 12-inch depth of gravel and a 9-inch depth of sand will be utilized in drying bed construction.

2.57.6.6.2 Volume of sand.

$$V_{ds} = 20\ (N)\ (LN)\ \frac{9}{12} = 15\ (N)\ (LN)$$

where

V_{ds} = volume of sand required, cu ft.

2.57.6.6.3 Volume of gravel.

$$V_{dg} = 20\ (N)\ (LN)$$

where

V_{dg} = volume of gravel, cu ft.

2.57.6.7 Perforated drain pipe required.

2.57.6.7.1 It is assumed that only the vitrified clay pipe will be used for this purpose.

2.57.6.7.2 Size of pipe to be used. Only 4-inch, 6-inch, and 8-inch diameter will be utilized. Selection of pipe size depends on length of bed. The following rule will be used:

Length of Bed, LN	Clay Pipe Diameter (DICLP) In
< 100'	4"
100 - 200	6"
> 200	8"

2.57.6.7.3 Total length of pipe, CPL.

$$CPL = 2\ (N)\ (LN)$$

2.57.6.8 Operation and maintenance manpower requirement.

2.57.6.8.1 Operation man-hours required, OMH.

When TPD < 0.09 (tons/day)

$$OMH = 360$$

When $0.09 \leq TPD \leq 0.8$ (tons/day)

$$OMH = 964\ (TPD)^{0.409}$$

When TPD > 0.8 (tons/day)

$$OMH = 1066.4\ (TPD)$$

where

TPD = sludge solids applied per day, tons/day.

OMH = operation man-hour requirement, man-hours/yr.

2.57.6.8.2 Maintenance man-hour requirement.

When TPD < 0.09 (tons/day)

$$MMH = 160$$

When $0.09 \leq TPD \leq 0.8$ (tons/day)

$$MMH = 432.8\ (TPD)^{0.409}$$

When TPD > 0.8 (tons/day)

$$MMH = 532.8\ (TPD)$$

where

MMH = maintenance man-hour requirement, man-hour/yr.

2.57.6.9 Operation and material costs. This item is principally related to replacement of sand removal in the bed cleaning process. It is expressed as a percent of total bare construction costs of the drying bed.

OMMP = [illegible].9%

where

OMMP = percent of the installed equipment costs for the operation and maintenance material costs, percent.

2.57.6.10 Other construction cost items. From the above calculations it can be seen that approximately 90 percent of the construction costs have been accounted for. Other items such as inlet piping, filtrate collection system, etc., would be 10 percent. Thus, the correction factor would be:

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correciton factor for minor construction costs.

2.57.7 <u>Quantities Calculations Output Data</u>.

2.57.7.1 Number of beds, N.

2.57.7.2 Surface area of each bed, SAN, sq ft.

2.57.7.3 Length of each bed, LN, ft.

2.57.7.4 Quantity of earthwork required, V_{ew}, cu ft.

2.57.7.5 Quantity of concrete wall, V_{cw}, cu ft.

2.57.7.6 Quantity of concrete slab, V_{cs}, cu ft.

2.57.7.7 Quantity of sand, V_{ds}, cu ft.

2.57.7.8 Quantity of gravel, V_{dg}, cu ft.

2.57.7.9 Perforated pipe size, DICLP, in.

2.57.7.10 Length of pipe, CPL, ft.

2.57.7.11 Operation manpower requirement, OMH, man-hours/yr.

2.57.7.12 Maintenance manpower requirement, MMH, man-hours/yr.

2.57.7.13 Operation and maintenance costs as a percent of capital costs, OMMP, percent.

2.57.7.14 Other construction cost items, CF.

2.57.8 <u>Unit Price Input Required</u>.

2.57.8.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.57.8.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

2.57.8.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

2.57.8.4 Current <u>Engineering News Record</u> Construction Cost Index, ENRCCI.

2.57.9 <u>Cost Calculations</u>.

2.57.9.1 Cost of earthwork, COSTE.

$$COSTE = \frac{V_{ew}}{27} \times UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.57.9.2 Cost of reinforced concrete wall in-place, COSTCW.

$$COSTCW = \frac{V_{cw}}{27} \times UPICW \times 0.7$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = total quantity of R.C. wall, cu ft.

UPICW = unit price input of R.C. wall in-place, \$/cu yd.

0.7 = due to the simple construction, the unit price is lowered to 70 percent.

2.57.9.3 Cost of R.C. slab in-place, COSTCS.

$$COSTCS = \frac{V_{cs}}{27} \times UPICS \times 0.6$$

where

COSTCS = cost of R.C. slab in-place, dollars.

V_{cs} = volume of R.C. slab, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

0.6 = due to the simple construction, the unit price is lowered to 60 percent.

2.57.9.4 Costs of sand and gravel media and drain pipe system.

2.57.9.4.1 Cost of perforated clay piping system. The installed piping cost for 1st quarter 1977 is summarized in the following table:

Pipe Size DICLP	Unit Cost of Installed Pipe COSTCP, \$/L.F.
4"	\$2.15/L.F.
6"	\$2.75/L.F.
8"	\$4.25/L.F.

Cost of drain pipe system, CDPS, dollars.

$$CDPS = COSTCP \cdot CPL$$

where

$CDPS$ = cost of drain pipe system, \$.

$COSTCP$ = unit cost of installed piping system, \$/L.F.

CPL = length of pipe, ft.

2.57.9.4.2 Cost of sand and gravel. The 1st quarter 1977 costs of in-place sand and gravel are:

$$COSAND = \$5.90/\text{cu yd}$$

$$COGRVL = \$4.30/\text{cu yd}$$

where

$COSAND$ = unit cost of sand in-place, \$/cu yd.

$COGRVL$ = unit cost of gravel in-place, \$/cu yd.

Thus cost of drying bed media, CODBM.

$$CODBM = (COSAND)\left(\frac{V_{ds}}{27}\right) + (COGRVL)\left(\frac{V_{dg}}{27}\right)$$

where

V_{ds} = volume of sand, cu ft.

V_{dg} = volume of gravel, cu ft.

3.57.9.4.3 For better estimates, the unit prices, COSAND, COGRVL, and COSTCP, should be obtained from local vendors and be treated as unit price input. Otherwise, the Engineering News Record (ENR) Construction Cost Index should be used to update these costs.

Thus, if unit prices are given from the input:

$$CODBMU = (CODBM + CDPS)$$

If no unit prices are given from the input:

$$CODBMU = (CODBM + CDPS)\frac{ENRCCI}{2470}$$

where

$CODBMU$ = updated costs of drying bed media and drainage system, \$.

ENRCCI = Engineering News Record Construction Cost Index.

2470 = ENRCCI of 1st quarter, 1977.

2.57.9.5 Total bare construction costs of drying bed, TBCC.

$$TBCC = CF\ (COSTE + COSTCW + COSTCS + CODBMU)$$

where

TBCC = total bare construction costs of sludge drying bed, \$.

CF = correction factors for minor construction items.

2.57.9.6 Operation and maintenance material costs, OMMC. O&M material costs are estimated as a percent of the total bare construction costs. Thus:

$$OMMC = \frac{OMMP}{100}\ (TBCC)$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = O&M material costs as percent of total construction cost, percent.

2.57.10 Cost Calculations Output Data.

2.57.10.1 Total bare construction cost for sludge drying bed, \$.

2.57.10.2 Operation and maintenance material costs, \$/yr.

2.57.11 Bibliography.

2.57.11.1 Keefer, C.E., Public Works, Vol 98, p. 7.

2.57.11.2 Metcalf and Eddy, Inc., "Water Pollution Abatement Technology: Capabilities and Costs, Public Owned Treatment Works", 1975, PB-250690-03, NTIS, Springfield, VA.

2.57.11.3 Metcalf and Eddy, Inc., Wastewater Engineering: Collection, Treatment, Disposal, 1972, McGraw Hill, New York.

2.57.11.4 Patterson and Banker, "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", Report No. 17090 DAN., Oct., 1971, USEPA, Washington, D.C.

2.57.11.5 WPCF Manual of Practice No. 8, Sewage Treatment Plant Design, 1959, Water Pollution Control Federation.

2.57.11.6 WPCF Manual of Practice No. 2, Utilization of Municipal Wastewater Sludge, 1971, Water Pollution Control Federation.

2.59 SLUDGE HAULING AND LANDFILLING

2.59.1 Background.

2.59.1.1 Landfilling can provide an acceptable and inexpensive method for ultimate sludge disposal, particularly for smaller facilities. The method may be of special importance if it can be integrated with solid waste disposal systems that have an operating sanitary landfill. Other methods of sludge treatment, such as drying beds or incineration, are considered to be methods of volume reduction that produce a residue requiring ultimate disposal.

2.59.1.2 Sludge hauling and landfilling may be approached in a manner similar to that for a typical solid waste disposal problem. Most solid waste disposal systems have at least four definable components: storage, collection, haul and disposal. In addition, sludge disposal systems usually require some form of pretreatment if associated costs are to be minimized.

2.59.1.3 Pretreatment of sludge is related to reducing the volume to a minimum before transporting. Typical unit processes used for volume reduction may include digestion, centrifugation, vacuum filtration and sludge drying beds. Costs associated with these processes are not considered to be part of sludge hauling or landfilling but are very important in the overall sludge handling train.

2.59.1.4 Storage costs are site-specific and depend largely upon the method selected in the sludge handling train. They may be simply the costs associated with the purchase of bins for storage of waste activated or primary sludge, a dump truck for storage of digested sludge solids that have been centrifuged or vacuum filtered, or the cost associated with sludge drying beds.

2.59.1.5 Collection costs are dependent upon a time-labor relationship to transfer the sludge from storage to the transporting vehicle, as a dump or tank truck. There may not be a collection cost associated with labor; however, a cost would be incurred to provide a vehicle during the loading period. Larger facilities may require that a driver be assigned to the vehicle during loading periods. Collection costs may be significant when it is necessary to shovel sludge from drying beds into trucks for transportation to the landfill. As indicated in the above paragraphs collection costs are site and system specific.

2.59.1.6 Transportation costs are associated with such parameters as truck cost, truck size, haul time, labor, and operating costs per unit time for items such as depreciation, fuel, insurance, maintenance, etc. Operating costs may be estimated from manufacturer's rating information and used in conjunction with estimates of sludge production from various wastewater treatment processes.

2.59.1.7 Disposal costs are related to the operation and management of the final disposal facility. This cost should be minimal if the facility will integrate ultimate sludge disposal with the disposal of refuse. When this is possible, the disposal costs may only include the costs of unloading and a landfill fee. On the other hand, if the landfill is to receive only waste sludge; costs may be very significant as other equipment for operation of the landfill will be required. The equipment used for landfill operation may include units for excavation, placing, covering, and compaction of fill.

2.59.1.8 The lowest possible moisture content attainable at a reasonable costs should be produced for economical sludge hauling and landfill operations. A reduction of moisture content will produce a savings in storage, initial equipment, operating, and labor costs.

2.59.2 Input Data.

2.59.2.1 Average wastewater flow, Q_{avg}, mgd.

2.59.2.2 Sludge flow, SF, gallons/million gallons.

2.59.2.3 Raw sludge suspended solids concentration, SS, percent.

2.59.2.4 Dewatered sludge solids concentration, CSS, percent.

2.59.2.5 Vehicle loading time, LT, hr.

2.59.2.6 Vehicle unloading time, ULT, hr.

2.59.2.7 Round trip haul time, HT, hr.

2.59.2.8 Solids capture in dewatering process, SCAP, percent.

2.59.2.9 Distance to disposal site, D, miles.

2.59.2.10 Work schedule, HPD, hrs/day.

2.59.3 Design Parameters.

2.59.3.1 Sludge volume per million gallons treated (Table 2.59-1).

2.59.3.2 Raw sludge solids concentration (Table 2.59-1), 1.5-15 percent.

2.59.3.3 Concentrated solids (Table 2.59-1), .6-60 percent.

2.59.3.4 Vehicle capacity, yd^3/truck.

2.59.3.5 Truck loading time, 0.5-2.0 hr.

2.59.3.6 Haul time, local conditions.

2.59.3.7 Daily work schedule, 6-8 hr.

2.59.3.8 Solids capture (Table 2.59-2), 70-99 percent.

2.59.4 Process Design Calculations.

2.59.4.1 Sludge volume hauled.

$$SV = \frac{(Q_{avg})(SF)(SS)}{(7.48)(27)(CSS)}$$

where

SV = sludge volume hauled, cu yd/day.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gallons/million gallons.

SS = raw sludge suspended solids concentration, percent.

CSS = dewatered sludge solids concentration, percent.

2.59.4.2 Capacity of trucks. The system which was selected to haul the sludge is a semi-trailer with a tractor. The trailer is equipped with a hydraulic ram for removal of the sludge. The following is given for selection of trailer size.

Q_{avg} (mgd)	Truck Capacity, CAP (cu yd)
0.5 to 12.5	19
12.5 to 25.0	22
25.0	30

2.59.4.3 Number of vehicles.

$$NTR = \frac{(SV)(LT + HT)}{(HPD)(CAP)}$$

where

NTR = number of trucks required.

LT = vehicle loading time.

HT = round trip hauling time.

HPD = work schedule, hrs/day.

CAP = vehicle capacity, cu yd/truck.

2.59.4.4 Tons of sludge hauled per day.

$$TSH = \frac{(Q_{avg})(SF)(SS)(SCAP)(8.34)}{100\ (CSS)(2000)}$$

where

TSH = tons of sludge hauled per day, tons/day.

SCAP = solids capture, percent.

2.59.4.5 Number of round trips per day per truck.

$$RTD = \frac{SV}{(NTR)(CAP)}$$

Round RTD up to next largest integer.

where

RTD = round trips per truck per day.

SV = sludge volume hauled, yd^3/day.

NTR = number of trucks required.

CAP = vehicle capacity, yd^3/truck.

2.59.5 Process Design Output Data.

2.59.5.1 Average wastewater flow, Q_{avg}, mgd.

2.59.5.2 Raw sludge suspended solids concentration, SS, percent.

2.59.5.3 Dewatered sludge solids concentration, CSS, percent.

2.59.5.4 Sludge volume hauled, SV, cu yd/day.

2.59.5.5 Vehicle capacity, CAP, cu yd.

2.59.5.6 Number of trucks required, NTR.

2.59.5.7 Tons of sludge hauled per day, TSH, tons/day.

2.59.5.8 Distance to disposal site, D, miles.

2.59.6 Quantities Calculations.

2.59.6.1 Sludge storage shed. Depending on the location of the landfill site, there will be periods, especially during the winter months, when landfilling operations cannot be continued. For this reason, a storage shed, sized to handle the anticipated volume of sludge produced during landfill shutdown periods, must be furnished.

2.59.6.1.1 Sludge storage shed floor area.

$$SSSA = \frac{(SV)(T)(27)}{h}$$

where

SSSA = sludge storage shed area, sq ft.

SV = sludge volume hauled, cu yd/day.

T = maximum anticipated landfill downtime, use 30 days, days.

h = anticipated sludge storage height, use 8 ft, ft.

2.59.6.1.2 Length and width of storage shed slab. The length of the shed is assumed to be twice the width.

$$L = 2W$$

$$SSSA = (L)(W) = 2W^2$$

$$W = \left(\frac{SSSA}{2}\right)^{0.5}$$

where

L = length of sludge storage shed slab, ft.

W = width of sludge storage shed slab, ft.

2.59.6.1.3 Volume of earthwork required for sludge storage shed. It is assumed that the top of the slab will be at the natural ground level. The side slopes of the excavation will be 1 to 1.

$$V_{ew} = [4W^2 + 15W + 25 + (4W^4 + 30W^3 + 50W^2)^{0.5}]\left[\frac{2.5}{3}\right]$$

where

V_{ew} = volume of earthwork, cu ft.

2.59.6.1.4 Volume of reinforced concrete required for slab.

$$V_{cs} = W(13.74 + 2W)$$

where

V_{cs} = volume of R.C. required for slab, cu ft.

2.59.6.1.5 Surface area of canopy roof for sludge storage shed.

$$SACR = SSSA$$

where

SACR = surface area of canopy roof, sq ft.

2.59.6.2 Operation manpower.

2.59.6.2.1 Calculate round-trip haul distance.

$$RTHD = 2D$$

where

D = distance to disposal site, miles.

RTHD = round trip haul distance, miles.

2.59.6.2.2 Calculate operation manpower.

If $0 < RTHD \leq 16.5$ miles, the operation manpower is calculated by:

$$OMH = (260)(TSH)(.0263)(RTHD)^{0.2708}$$

where

OMH = operation manpower required, man-hours/yr.

If $RTHD > 16.5$ miles, the operation manpower is calculated by:

$$OMH = (.0034)(260)(TSH)(RTHD)$$

2.59.6.3 Distance travelled per year per truck.

$$DTT = (RTHD)(RTD)(5)(50)$$

where

DTT = distance travelled per year per truck, miles/yr.

RTD = round trips per day per truck.

5 = days of operation per week.

50 = weeks of operation per year.

2.59.6.4 Operation and maintenance material and supply costs. These costs are given as a percent of the cost of the vehicle used for hauling. The costs include the cost for maintenance labor as well as the cost of repair and replacement parts.

2.59.6.4.1 For 19 cu yd vehicle:

$$OMMP = .0013 \ (DTT)$$

2.59.6.4.2 For 22 cu yd vehicle:

$$OMMP = .00106 \ (DTT)$$

2.59.6.4.3 For 30 cu yd vehicle:

$$OMMP = .00102 \ (DTT)$$

where

OMMP = O&M maintenance material and supply costs, as percent of vehicle cost, percent.

2.59.7 <u>Quantities Calculations Output Data</u>.

2.59.7.1 Sludge storage shed area, SSSA, sq ft.

2.59.7.2 Volume of earthwork, V_{ew}, cu ft.

2.59.7.3 Volume of R.C. slab required, V_{cs}, cu ft.

2.59.7.4 Surface area of canopy roof, SACR, sq ft.

2.59.7.5 Round-trip haul distance, RTHD, miles.

2.59.7.6 Operation and maintenance material and supply costs, as percent of vehicle cost, OMMP, percent.

2.59.7.7 Operation manpower required, OMH, man-hours, yr.

2.59.7.8 Distance travelled per year per truck, DTT, miles/yr.

2.59.8 <u>Unit Price Input Required</u>.

2.59.8.1 Cost of earthwork, COSTE, $/cu yd.

2.59.8.2 Cost of R.C. concrete slab in-place, COSTCS, \$/cu yd.

2.59.8.3 Cost of canopy-type roof, UPIR, \$/sq ft.

2.59.8.4 Cost of flat monthly or yearly charge for landfilling sludge, CMC or CYC, \$/mo or yr.

2.59.8.5 Cost per cubic yard to landfill sludge, CPCY, \$/cu yd.

2.59.8.6 Cost per ton of sludge to landfill, CPT, \$/ton.

2.59.8.7 Cost of standard size vehicle, COSTSSV, \$ (optional).

2.59.8.8 Marshall and Swift Equipment Cost Index, MSECI.

2.59.9 Cost Calculations.

2.59.9.1 Cost of earthwork.

$$COSTE = \left(\frac{V_{ew}}{27}\right) UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.59.9.2 Cost of concrete slab in-place.

$$COSTCS = \left(\frac{V_{cs}}{27}\right) (UPICS)$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab in-place required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.59.9.3 Cost of canopy-type roof.

$$COSTCR = (SACR)(UPIR)$$

where

COSTCR = cost of canopy-type roof, \$.

SACR = surface area of canopy-type roof, sq ft.

UPIR = unit price input for canopy-type roof, \$/sq ft.

2.59.9.4 Cost of sludge hauling vehicles.

2.59.9.4.1 Purchase cost of sludge hauling vehicles.

$$COSTSV = (NTR)(COSTSSV)(COSTRO)$$

where

COSTSV = cost of sludge hauling vehicle, \$.

NTR = number of vehicles.

COSTSSV = cost of standard size vehicle, 22 cu yd, \$.

COSTRO = ratio of cost of vehicle of desired capacity and the cost of the standard size vehicle.

2.59.9.4.2 COSTRO. Vehicles in three sizes are available. The cost ratio for each size is as shown:

Vehicle Size (cu yd)	COSTRO
19	.82
22	1.00
30	1.04

2.59.9.4.3 Cost of standard size vehicle (22 cu yd). The cost of a vehicle with a 22 cu yd capacity in the first quarter of 1977 is:

$$COSTSSV = \$51,700$$

For better cost estimation, COSTSSV should be obtained from the vehicle manufacturer and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSSV = (51,700)\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter 1977.

2.59.9.5 Cost of landfilling dewatered sludge. Generally in a private landfilling operation, the charges assessed to a municipality to dispose of dewatered sludge, are computed one of three ways: 1) Flat monthly or yearly charge, 2) Charge by the truck load (which can be related back to a charge per cu yd), 3) If the landfill has a set of scales, a charge per ton of sludge can be used.

2.59.9.5.1 Case I - Monthly or yearly disposal fee.

$$COSTLF = (12)(CMC) \text{ or } CYC$$

where

COSTLF = cost to landfill dewatered sludge, \$/yr.

CMC = constant monthly charge (local option - local condition), \$ per month.

CYC = constant yearly charge (local option - local condition), \$ per year.

2.59.9.5.2 Case II - Disposal fee based on yd^3 of sludge delivered to landfill site.

$$COSTLF = (SV)(CPCY)(365)$$

where

COSTLF = cost to landfill dewatered sludge, \$/yr.

SV = sludge volume hauled, yd^3/day.

CPCY = cost per cubic yard of sludge to landfill, $\$/yd^3$ (local conditions).

2.59.9.5.3 Case III - Disposal fee based on tons of sludge delivered to landfill site.

$$COSTLF = (TSH)(365)(CPT)$$

where

COSTLF = cost to landfill dewatered sludge, \$/yr.

TSH = tons of sludge hauled per day, tons/day.

CPT = cost per ton of sludge to landfill, \$/ton (local condition).

2.59.9.6 Operation and maintenance material and supply costs. In addition to the nomal material and supply costs, the cost of operating the landfill will be considered a part of this cost.

$$OMMC = (\frac{OMMP}{100})(COSTSV) + COSTLF$$

where

OMMC = operation and maintenance material and supply costs, \$/yr.

OMMP = operation and maintenance material and supply costs, as percent of purchase cost of sludge vehicles, percent.

2.59.9.7 Total bare construction cost.

$$TBCC = COSTE + COSTCS + COSTCR + COSTSV$$

where

TBCC = total bare construction cost, \$.

2.59.10 Cost Calculations Output Data.

2.59.10.1 Total bare construction cost, TBCC, \$.

2.59.10.2 O&M material and supply costs, OMMC, \$/yr.

2.59.11 Bibliography.

2.59.11.1 Helle, Steven C., "Estimating Costs of Wastewater Sludge Disposal", Public Works, March, 1977, Vol. 108, No. 3.

2.59.11.2 McMichael, Walter F., "Costs of Hauling and Land Spreading of Domestic Sewage Treatment Plant Sludge", EPA-670/2-74-010 National Environmental Research Center, February, 1974.

2.59.11.3 Metcalf and Eddy, Inc., Wastewater Engineering: Collection, Treatment, and Disposal, McGraw-Hill, New York, 1972.

2.59.11.4 Shea, Timothy G. and Stockton, John D., "Wastewater Sludge Utilization and Disposal Costs", Technical Report MCD-12, EPA-430/9-75-015, September, 1975.

2.59.11.5 USEPA Process Design Manual for Sludge Treatment and Disposal.

Table 2.59-1. Normal Quantities of Sludge Produced by Different Treatment Processes

Wastewater Treatment Process	Gallons Sludge/ mg Treated	Solids Percent	Sludge Specific Gravity
Primary sedimentation			
Undigested	2,950	5.0	1.02
Digested in separate tanks	1,450	6.0	1.03
Trickling filter	745	7.5	1.025
Chemical precipitation	5,120	7.5	1.03
Primary sedimentation and activated sludge			
Undigested	6,900	4.0	1.02
Digested in separate tanks	2,700	6.0	1.03
Activated sludge			
Waste sludge	19,400	1.5	1.005
Septic tanks, digested	900	10.0	1.04
Imhoff tanks, digested	500	15.0	1.04

Table 2.59-2. Process Efficiencies for Dewatering of Wastewater Sludge

Unit Process	Solids Capture, percent	Cake Solids, percent
Centrifugation		
Solid bowl	80-90	5-13
Disc-nozzle	80-97	5-7
Basket	70-90	9-10
Dissolved air flotation	95	4-6
Drying beds	85-99	8-25
Filter press	99	40-60
Gravity thickener	90-95	5-12
Vacuum filter	90+	28-35

2.61 THICKENING.

2.61.1 Background.

2.61.1.1 Thickening reduces the moisture content of a slurry. Thickening is used at most waste treatment plants, as an economic measure, to reduce the volume of sludge or for greater efficiency in subsequent processes. Thickening is normally accomplished by gravity or flotation thickeners; centrifuges have also been used as sludge thickeners.

2.61.1.2 In this section only gravity thickening and flotation thickening will be addressed.

2.61.2 General Description Flotation Thickening.

2.61.2.1 Flotation is a solid-liquid separation process. Separation is artifically induced by introducing fine gas bubbles (usually air) into the system. The gas-solid aggregate with an overall bulk density less than the density of the liquid; thus, these aggregates rise to the surface of the fluid. Once the solid particles have been floated to the surface, they can be collected by a skimming operation.

2.61.2.2 In wastewater treatment, flotation is used as a clarification process to remove suspended solids and as a thickening process to concentrate various types of sludges. However, high operating costs of the process generally limit its use to clarification of certain industrial wastes and for concentration of waste-activated sludge.

2.61.2.3 Air flotation systems may be classified as dispersed air flotation or dissolved air flotation. In dispersed air flotation, air bubbles are generated by introducing air through a revolving impeller or porous media. This type of flotation system is ineffective and finds very limited application in wastewater treatment. Dissolved air flotation may be subclassified as pressure flotation or vacuum flotation. Pressure flotation involves air being dissolved in the wastewater under elevated pressures and later released at atmospheric pressure. Vacuum flotation, however, consists of applying a vacuum to wastewater aerated at atmospheric pressure. Dissolved air-pressure flotation, considered herein is the most commonly used in waterwater treatment.

2.61.2.4 The principal components of a dissolved air-pressure flotation system as shown in Figure 2.61-1 are a pressurizing pump, air injection facilities, a retention tank, a back pressure regulating device, and a flotation unit. The primary variables for flotation design are pressure, recycle ratio, feed solid concentration, detention period, air-to-solids ratio, use of polymers and solids and hydraulic loadings. Optimum design parameters must be obtained from bench scale or pilot plant studies. Typical design parameters are listed in Table 2.61-1.

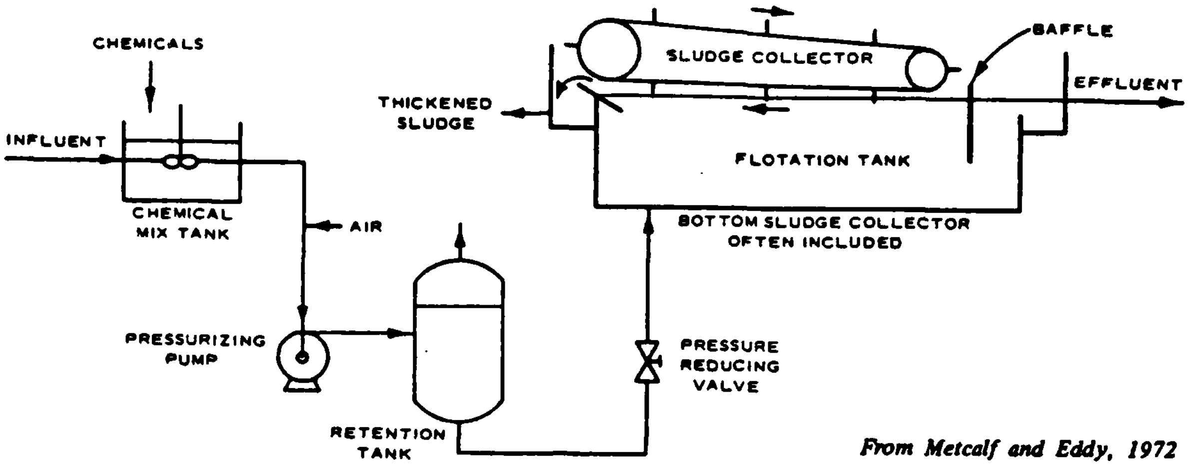

Figure 2.61-1. Schematic of Dissolved-Air Flotation Tank

2.61.3 General Description Gravity Thickening.

2.61.3.1 Gravity thickening is the most common process currently used for dewatering and for the concentration of sludge prior to digestion. Gravity thickening is essentially a sedimentation process similar to that which occurs in all settling tanks. The process is simple and is the least expensive of the available thickening processes.

2.61.3.2 Gravity thickening may be classified as plain settling and mechanical thickening. Plain settling usually results in the formation of scum at the surface and stratification of sludges near the bottom. Sludges from secondary clarifiers usually cannot be concentrated by plain settling. Gentle agitation is usually employed to stir the sludge, thereby opening channels for water escape and promoting densification. A common mechanical thickener consists of a circular tank equipped with a slowly revolving sludge collector. Primary and secondary sludges are usually mixed prior to thickening. A ratio of secondary sludge to primary sludge of 8 to 1 or greater is recommended to assure aerobic conditions in the thickener. Chlorine has been used to prevent sludge septicity and gasification which interfere with optimum solids concentration of organic materials. A chlorine residual of 0.5 to 1.0 mg/l in the thickening tank overhead prevents such problems. Organic polyelectrolytes (anionic, nonionic, and cationic) have been used successfully to increase the sludge settling rates, the overflow clarity, and the allowable tank loading.

2.61.3.3 Design of Thickeners. In the design of thickeners, concentration of the underflow and clarification of the overflow must be achieved. Mechanical thickeners (fig. 2.61-2) are designed on the basis of hydraulic surface loading and solid loading. These parameters are nomally obtained from laboratory batch settling tests. Procedures for conducting the tests and evaluating the design parameters are well documented in literaure. In the absence of laboratory data, Table 2.61-2 may be used as a guide for selecting solid loading rates. Typical surface loading rates of 600 to 800 gpd/ft^2 are recommended for most thickeners. Hydraulic loading rates of less than 400 gpd/ft^2 were reported to produce odor problems. Detention time of the thickener may range between 2 and 4 hours. Gravity thickening is the most common method currently used at wastewater treatment plants for concentrating sludges.

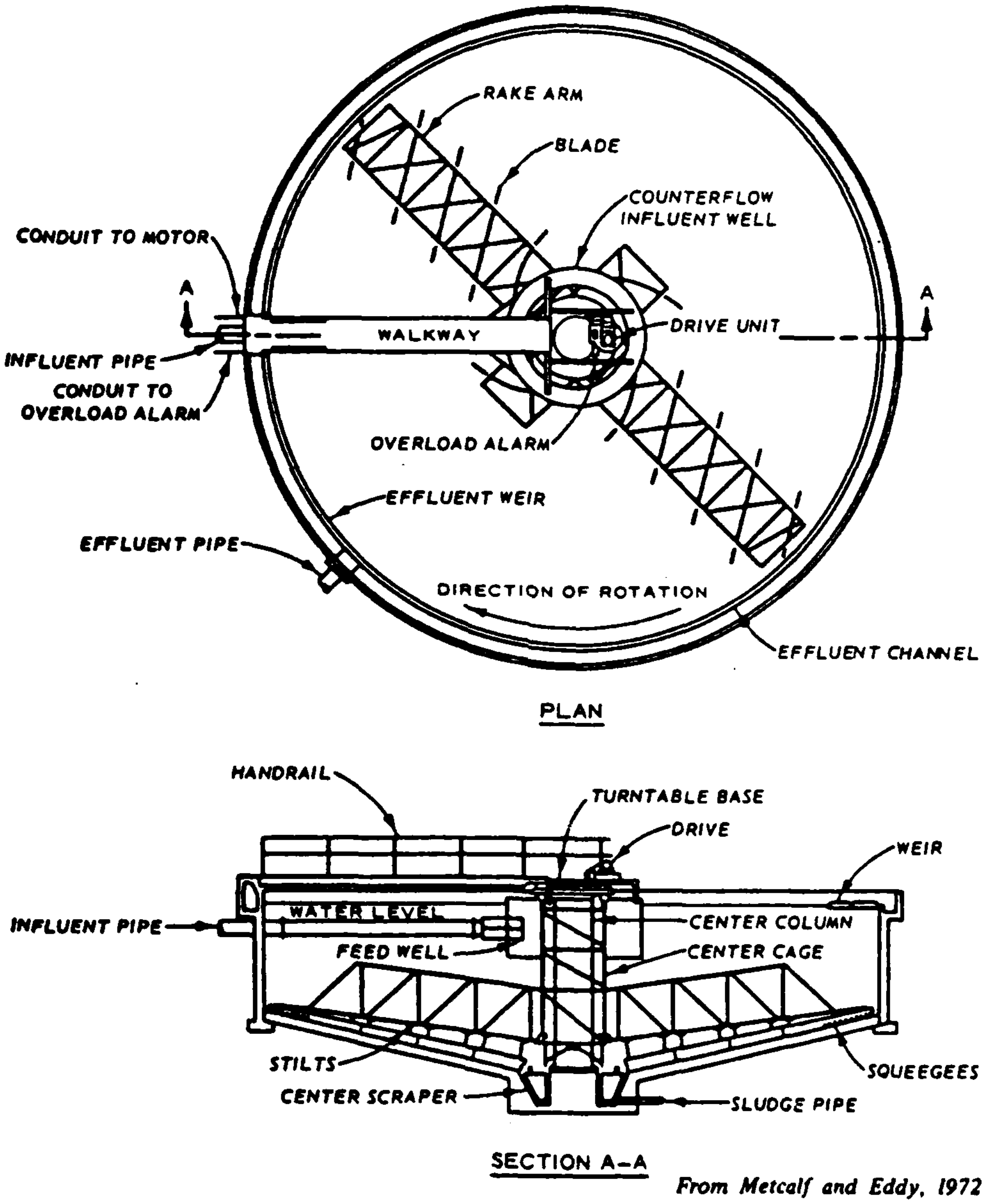

Figure 2.61-2. Schematic of a mechanical thickener.

2.61.4 Flotation Thickening.

2.61.4.1 Input Data.

2.61.4.1.1 Sludge flow, mgd.

2.61.4.1.2 Suspended solids concentration in the feed, mg/l.

2.61.4.1.2.1 Average concentration.

2.61.4.1.2.2 Variation in concentration.

2.61.4.1.3 Polymer dosage, lb/ton dry solids.

2.61.4.2 Design Parameters. From laboratory or pilot plant studies.

2.61.4.2.1 Air-to-solid ratio, A/S.

2.61.4.2.2 Air pressure, P, psig.

2.61.4.2.3 Detention time in flotation tank, DTFT, hr.

2.61.4.2.4 Solids loading, ML, lb/ft^2/day.

2.61.4.2.5 Hydraulic loading, HL, gpm/ft^2.

2.61.4.2.6 Detention time in pressure tank, DTPT, min.

2.61.4.2.7 Float concentration, C_F, percent.

2.61.4.3 Process Design Calculations.

2.61.4.3.1 Select air-to-solids ratio.

2.61.4.3.2 Assume air pressure (40 to 60 psig).

2.61.4.3.3 Calculate P in atmospheres = $\frac{\text{psig} + 14.7}{14.7}$

2.61.4.3.4 Calculate recycle flow.

$$R = \frac{(A/S)(Q)(Co)}{1.3\ S_a(0.5P-1)}$$

where

A/S = air-to-solid ratio.

S_a = air solubility at standard conditions, cc/l.

P = absolute pressure, atmospheres.

R = recycle flow, mgd.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

2.61.4.3.5 Calculate surface area required.

2.61.4.3.5.1 Select a solids loading rate and calculate surface area. If no pilot data is available, use the following mass loadings:

With polymer addition: 30 lb/sq ft/day

Without polymer addition: 10 lb/sq ft/day

$$SA = \frac{(Q)(C_o)(8.34)}{ML}$$

where

SA = surface area, ft^2.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

ML = solids loading rate, $lb/ft^2/day$.

2.61.4.3.5.2 Select a hydraulic loading rate and calculate the surface area.

$$SA = \frac{(Q+R)(10^6)}{(HL)(60)(24)}$$

where

SA = surface area, ft^2.

Q = feed flow, mgd.

R = recycle flow, mgd.

HL = hydraulic loading rate, gpm/ft^2.

2.61.4.3.5.3 Compare the surface areas calculated and use the larger of the two.

2.61.4.3.6 Select detention time in the flotation tank and calculate the volume.

$$VOLFT = (Q + R) \times \left(\frac{1}{7.48}\right)\left(\frac{1}{24}\right)(DTFT)(10^6)$$

where

VOLFT = volume of flotation tank, ft^3.

Q = total flow, mgd.

R = recycle flow, mgd.

DTFT = detention time in flotation tank, hr.

2.61.4.3.7 Select pressure tank detention time and calculate volume of pressure tank.

$$VOLPT = (R)\left(\frac{1}{7.48}\right)\left(\frac{1}{24}\right)\left(\frac{1}{60}\right)(DTPT)(10^6)$$

where

VOLPT = volume of pressure tank, ft^3.

R = recycle flow, mgd.

DTPT = detention time in pressure tank, min.

2.61.4.3.8 Calculate volume of sludge.

$$VS = \frac{(Q)(C_o)(\%\ removal)}{(C_F)(specific\ gravity)}$$

where

VS = volume of sludge, gpd.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

C_F = solids concentration in float, percent.

2.61.4.3.9 Calculate polymer usage (if applicable).

$$PU = \frac{(PD)(Q)(C_o)\ 8.34}{2000}$$

where

PU = polymer usage, lb/day.

PD = polymer dosage, lb/ton dry solids (if polymers are used and no dosage rates are given in input, use 10 lb/ton dry solids).

Q = sludge flow, mgd.

C_o = suspended solids concentration in the feed, mg/l.

2.61.4.4 Process Design Output Data.

2.61.4.4.1 Suspended solids concentration, C_o, mg/l.

2.61.4.4.2 Air-to-solid ratio, A/S.

2.61.4.4.3 Air pressure, P, psig.

2.61.4.4.4 Solids loading, ML, lb/ft^2/day.

2.61.4.4.5 Hydraulic loading, HL, gpm/ft^2.

2.61.4.4.6 Recycle flow, R, mgd.

2.61.4.4.7 Surface area, SA, ft^2.

2.61.4.4.8 Volume of pressure tank, VOLPT, ft^3.

2.61.4.4.9 Volume of flotation tank, VOLFT, ft^3.

2.61.4.4.10 Pressure tank detention time, DTPT, min.

2.61.4.4.11 Flotation tank detention time, DTFT, hr.

2.61.4.4.12 Polymer usage, PU, lb/day.

2.61.4.5 Quantities Calculations.

2.61.4.5.1 Select number and size of flotation units. The standard size units available commercially are 40, 50, 70, 100, 140, 200, 280, 350, 450, 570, 750, 960, and 1250 sq ft.

2.61.4.5.1.1 If SA is less than 1250 sq ft, then NU is one. Compare SA to the commercially available units and select the smallest unit that is larger than SA.

2.61.4.5.1.2 If SA is greater than 1250 sq ft, then NU must be two or greater. Try NU = 2 first, if SA/NU is greater than 1250, then NU = NU + 1. Repeat the procedure until SA/NU is less than 1250, then compare SA/NU to the commercially available units and select the smallest unit that is larger than SA/NU.

where

SA = calculated surface area, sq ft.

NU = number of units.

SAS = surface area of unit selected.

2.61.4.5.2 Calculate building area. In area where freezing weather may be expected, flotation units would normally be enclosed in buildings.

2.61.4.5.2.1 Calculate diameter of unit.

$$DIA = \left(\frac{4\ SAS}{\pi}\right)^{0.5}$$

where

DIA = diameter of unit selected, ft.

2.61.4.5.2.2 Calculate building area.

$$A_B = (DIA + 2)\ (DIA + 5)\ (NU)$$

where

A_B = area of building, sq ft.

2.61.4.5.3 Earthwork required for construction. The procedure to estimate the earthwork requirement is the same as that for circular clarifier.

$$V_{ew} = (1.15)\ NU[0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = earthwork required for construction, cu ft.

1.15 = 15 percent excess volume as safety factor.

NU = number of units.

2.61.4.5.4 Reinforced concrete quantities.

2.61.4.5.4.1 Calculate side water depth. The side water depth can be related to the diameter by the following equation:

$$SWD = 6.72 + 0.0476\ (DIA)$$

where

SWD = side water depth, ft.

2.61.4.5.4.2 Calculate the thickness of the slab. The thickness of the slab can be related to the side water depth by the following equation:

$$t_s = 7.9 + 0.25\ SWD$$

where

t_s = thickness of the slab, inches.

2.61.4.5.4.3 Calculate the wall thickness. The wall thickness can be related to the side water depth by the following:

$$t_w = 7 + (0.5)\ SWD$$

where

t_w = wall thickness, inches.

2.61.4.5.4.4 Calculate reinforced concrete slab quantity.

$$V_{cs} = 0.825\ (DIA + 4)^2\ \left(\frac{t_s}{12}\right)\ (NU)$$

where

V_{cs} = quantity of R.C. slab in-place, cu ft.

2.61.4.5.4.5 Calculate reinforced concrete wall quantity.

$$V_{cw} = (3.14)\ (SWD + 1.0)\ (DIA)\ \left(\frac{t_w}{12}\right)\ (NU)$$

where

V_{cw} = quantity of R.C. wall in-place, cu ft.

2.61.4.5.4.6 Quantity of concrete for splitter box.

$$V_{cb} = 100\ (NU)^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter box, cu ft.

NU = number of units.

2.61.4.5.4.7 Total quantity of R.C.

$$\text{wall:} \quad V_{cwt} = V_{cw} + V_{cb}$$

$$\text{slab:} \quad V_{cst} = V_{cs}$$

where

V_{cwt} = total quantity of R.C. wall in-place, cu ft.

V_{cst} = total quantity of R.C. slab in-place, cu ft.

2.61.4.5.5 Calculate dry solids produced.

$$DSTPD = \frac{(Q)\ (C_o)\ (8.34)}{2000}$$

where

DSTPD = dry solids produced, tpd.

Q = sludge flow, mgd.

C_o = suspended solids concentration in the feed, mg/l.

2.61.4.5.6 Calculate operational labor.

2.61.4.5.6.1 If DSTPD is between 0 and 2.3 tpd, the operational labor is calculated by:

$$OMH = 560\ (DSTPD)^{0.4973}$$

2.61.4.5.6.2 If DSTPD is greater than 2.3 tpd, the operational labor is calculated by:

$$OMH = 496\ (DSTPD)^{0.5092}$$

where

OMH = operation labor, man-hour/yr.

2.61.4.5.7 Calculate maintenance labor.

2.61.4.5.7.1 If DSTPD is between 0 and 3.0 tpd, the maintenance labor is calculated by:

$$MMH = 156.0\ (DSTPD)^{0.4176}$$

2.61.4.5.7.2 If DSTPD is greater than 3.0 tpd, the maintenance labor is calculated by:

$$MMH = 124.0\ (DSTPD)^{0.6429}$$

where

MMH = maintenance labor, man-hour/yr.

2.61.4.5.8 Calculate electrical energy requirements for operation.

$$KWH = 63{,}000\ (DSTPD)^{0.9422}$$

where

KWH = electrical energy requirement for operation, kwhr/yr.

2.61.4.5.9 Operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of total bare construction cost.

$$OMMP = 1\%$$

where

OMMP = percent of air flotation total bare construction cost as operation and maintenance materials costs, percent.

2.61.4.5.10 Other construction cost items.

2.61.4.5.10.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

2.61.4.5.10.2 Other minor cost items such as piping, electrical wiring and conduit, concrete slab for pumps and pressure tanks, etc., would be 15 percent.

2.61.4.5.10.3 The correction factor would be

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for construction costs.

2.61.4.6 Quantities Calculations Output Data.

2.61.4.6.1 Surface area of unit selected, SAS, sq ft.

2.61.4.6.2 Number of units, NU.

2.61.4.6.3 Area of building, A_B, sq ft.

2.61.4.6.4 Earthwork required, V_{ew}, cu ft.

2.61.4.6.5 Total quantity of R.C. Wall in-place, V_{cwt}, cu ft.

2.61.4.6.6 Total quantity of R.C. slab in-place, V_{cst}, cu ft.

2.61.4.6.7 Operational labor, OMH, man-hour/yr.

2.61.4.6.8 Maintenance labor, MMH, man-hour/yr.

2.61.4.6.9 Electrical energy requirement for operation, KWH, kwhr/yr.

2.61.4.6.10 Operation and maintenance material costs as percent of air flotation total bare construction cost, percent.

2.61.4.6.11 Correction factor for construction costs, CF.

2.61.4.7 Unit Price Input Required.

2.61.4.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

2.61.4.7.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

2.61.4.7.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

2.61.4.7.4 Cost of standard size flotation equipment, COSTFS, \$, (optional).

2.61.4.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.61.4.8 Cost Calculations.

2.61.4.8.1 Cost of building.

$$COSTB = A_B \times UPIBC\ (.75)$$

where

COSTB = cost of building, \$.

A_B = building area, sq ft.

UPIBC = unit price input for building cost, \$/ft^2.

.75 = correction factor since slab is already accounted for in concrete costs.

2.61.4.8.2 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27}\ UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.61.4.8.3 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cwt}}{27}\ UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cwt} = total quantity of R.C. wall in-place, cu ft.

UPICW = unit price input for R.C. wall in-place, $/cu yd.

2.61.4.8.5 Purchase cost of flotation equipment. The costs given include the basic mechanism to be mounted in the concrete tank, air pressurization tank, pressurization pump, pressure release valve, air injection system, and electrical panel.

$$COSTF = COSTFS \times \frac{COSTRO}{100}$$

where

COSTF = purchase cost of flotation equipment of SAS surface area, $.

COSTFS = cost of standard size air flotation unit of 350 sq ft, $.

COSTRO = cost of unit of SAS sq ft expressed as percent of cost of standard size unit.

2.61.4.8.6 Calculate COSTRO.

2.61.4.8.6.1 If SAS is less than 240 sq ft, COSTRO is calculated by:

$$COSTRO = 0.3\ (SAS) + 25$$

2.61.4.8.6.2 If SAS is between 240 sq ft and 480 sq ft, COSTRO is calculated by:

$$COSTRO = 0.092\ (SAS) + 75$$

2.61.4.8.6.3 If SAS is greater than 480 sq ft, COSTRO is calculated by:

$$COSTRO = 0.161\ (SAS) + 43$$

2.61.4.8.7 Cost of standard size unit. The cost of a dissolved air flotation unit with 350 sq ft of surface area for the first quarter of 1977 is:

$$COSTFS = \$44{,}200$$

For better cost estimation, COSTFS should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTFS = \$44,200 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Cost Index.

491.6 = MSECI first quarter 1977.

2.61.4.8.8 Equipment Installation Costs. These costs would include mounting of a flotation mechanism in the flotation tank, setting pumps and tanks, interconnecting piping, electrical installation, etc. These costs are estimated as 75 percent of the purchase cost of the equipment.

$$EIC = .75\ COSTF$$

where

EIC = equipment installation costs, dollars.

2.61.4.8.9 Installed equipment cost.

$$IEC = (COSTF + EIC)\ NU$$

where

IEC = installed equipment cost, \$.

NU = number of units of area SAS sq ft.

2.61.4.8.10 Total bare construction costs.

$$TBCC = (COSTB + COSTE + COSTCW + COSTCS + IEC)\ CF$$

where

TBCC = total bare construction costs, \$.

CF = construction cost correction factor,

2.61.4.8.11 Operation and maintenance material costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance costs, \$/yr.

2.61.4.9 <u>Cost Calculations Output Data</u>.

2.61.4.9.1 Total bare construction cost for dissolved air flotation unit, TBCC, \$.

2.61.4.9.2 Operation and maintenance material costs, OMMC, \$/yr.

Table 2.61-1 Air Flotation Parameters

Parameter	Typical Value: Thickening	Typical Value: Clarification
Air pressure, psig	40 to 70	40 to 70
Effluent recycle, %	130 to 150	30 to 120
Detention time, hr	3	0.25 to 0.5
Air-to-solids ratio (lb air/lb solids)	(0.005 to 0.06)	
Solid loading, $lb/ft^2/day$		
Activated sludge (mixed liquor)	5 to 15	
Activated sludge (settled)	10 to 20	
50% primary +50% activated	20 to 40	
Primary only	to 50	
Hydraulic loading, gpm/ft^2	0.2 to 4	1 to 4
Detention time, min (pressurizing tank)	1 to 3	1 to 3

2.61.5 Gravity Thickening

2.61.5.1 Input Data.

2.61.5.1.1 Sludge flow (Q), gpd.

2.61.5.1.1.1 Average daily flow (Q_{avg}), gpd.

2.61.5.1.1.2 Maximum flow (Q_{max}) and minimum flow (Q_{min}), gpd.

2.61.5.1.2 Solids concentration (C_o), percent = (mg/l) x 10^{-4}.

Note: 1 percent solids = 0.624 lb/ft^3.

2.61.5.2 Design Parameters.

2.61.5.2.1 Desired underflow concentration (C_u), lb/ft^3 = (mg/l x 0.624 x 10^{-4}).

2.61.5.2.2 Mass loading (ML), lb/ft^2/day (from settling test).

2.61.5.2.3 Hydraulic loading (HL), gpd/ft^2 (400 to 800 gpd/ft^2).

2.61.5.2.4 Detention time (t), hr (2 to 6 hr).

2.61.5.2.5 Number of tanks (N).

2.61.5.3 Process Design Calculations.

2.61.5.3.1 Calculate unit area, using data from settling test.

$$UA = \frac{\frac{1}{C_i} - \frac{1}{C_u}}{U_i} \times \frac{1}{0.624}$$

where

UA = unit area, ft^2/lb/day.

C_i = solids concentration at settling velocity U_i, percent.

= (mg/l x (10^{-4}) = $\frac{C_o H_o}{H_i}$ (settling test, Fig. 2.61-3).

C_o = initial solids concentration, percent = (mg/1) x (10^{-4}).

H_o = initial height, ft.

H_i = intercept of tangent to the settling curve, ft (Fig. 2.61-3).

C_u = underflow concentration, percent.

U_i = settling velocity at the interface, ft/day (settling velocity, Fig. 2.61-3).

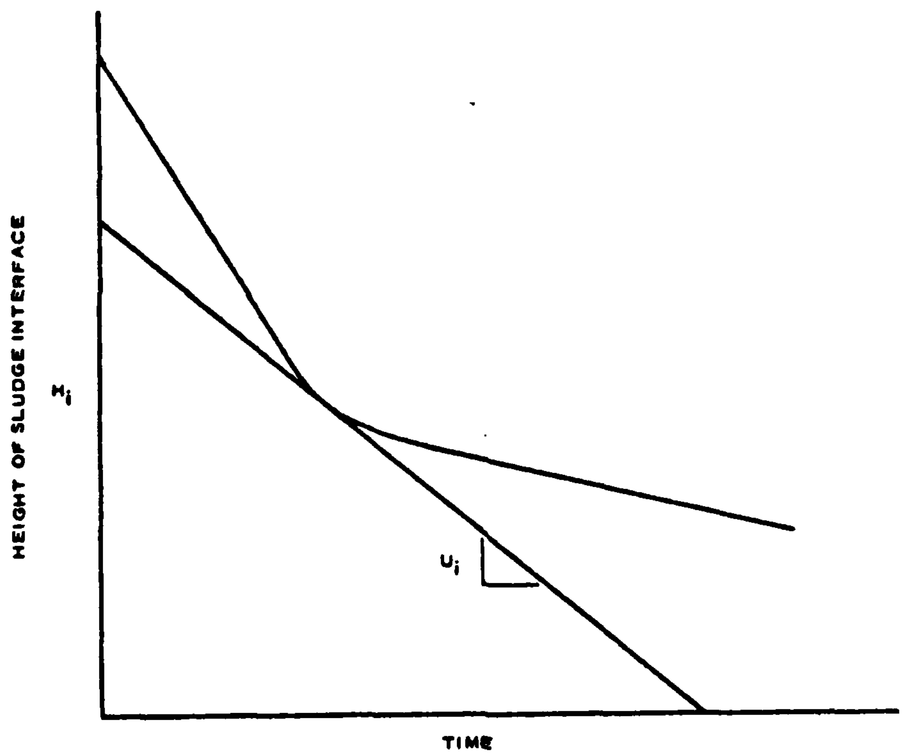

Figure 2.61-3. Typical settling curve from laboratory test.

2.61.5.3.2 Calculate mass loading.

$$ML = \frac{1}{UA}$$

where

ML = mass loading, lb/ft^2/day.

UA = unit area, ft^2/lb/day.

2.61.5.3.3 If settling data are not available, select mass loading from Table 2.61-2.

2.61.5.3.4 Calculate total surface area.

$$TSA = \frac{(Q_{avg})\ (C_o)\ (0.624)}{(ML)\ (7.48)}$$

where

TSA = total surface area, ft^2.

Q_{avg} = average daily flow, gpd.

C_o = initial solids concentration, percent.

ML = mass loading, lb/ft^2/day.

2.61.5.3.5 Check hydraulic loading.

$$HL = \frac{Q_{min}}{SA} \quad (\geq 400 \text{ gpd/ft}^2)$$

$$HL = \frac{Q_{max}}{SA} \quad (\leq 800 \text{ gpd/ft}^2)$$

where

HL = hydraulic loading, gpd/ft^2.

Q_{min} = minimum flow, gpd.

Q_{max} = maximum flow, gpd.

SA = surface area, ft^2.

2.61.5.3.6 Select number of tanks and calculate surface area per tank.

$$SAPT = \frac{TSA}{N}$$

where

$SAPT$ = surface area per tank, ft^2.

TSA = total surface area, ft^2.

N = number of tanks.

2.61.5.3.7 Select a detention time (2 to 6 hr) and calculate tank volume.

$$V = (Q_{avg})\ (t)\ \left(\frac{1}{24}\right)\ \left(\frac{1}{7.48}\right)$$

where

V = volume, ft^3.

Q_{avg} = average daily flow, gpd.

t = detention time, hr.

2.61.5.3.8 Calculate depth.

$$D = \frac{(V)}{(SAPT)}$$

where

D = depth, ft.

V = volume, ft^3.

SAPT = surface area per tank, ft^2.

2.61.5.3.9 Calculate volume of thickened sludge.

$$VTS = \frac{(Q)\ (C_o)\ (0.9)}{(C_u)\ (\text{Specific Gravity})}$$

where

VTS = volume of thickened sludge, gpd.

Q = sludge flow, gpd.

C_o = initial solid concentration, percent.

C_u = desired underflow concentration, percent.

2.61.5.4 Process Design Output Data.

2.61.5.4.1 Average sludge flow, mgd.

2.61.5.4.2 Initial concentration, percent.

2.61.5.4.3 Thickened concentration, percent.

2.61.5.4.4 Mass loading, $lb/ft^2/day$.

2.61.5.4.5 Hydraulic loading, $gal/day/ft^2$.

2.61.5.4.6 Detention time, hr.

2.61.5.4.7 Number of units.

2.61.5.4.8 Depth, ft.

2.61.5.4.9 Volume, ft^3.

2.61.5.4.10 Surface area per tank, ft^2.

2.61.5.4.11 Volume of thickened sludge, gpd.

2.61.5.5 <u>Quantities Calculations</u>.

2.61.5.5.1 Calculate diameter of each tank.

$$DIA = \left(\frac{4\ SAPT}{3.1416}\right)^{0.5}$$

If DIA is not an integer, use the next larger integer.

where

DIA = diameter of the unit, ft.

2.61.5.5.2 Calculate earthwork required for construction. The procedure to estimate the earthwork requirement is the same as that for the circular clarifier.

$$V_{ew} = (1.15)\ N\ [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = volume of earthwork required for construction, cu ft.

1.15 = addition of 15 percent for safety factor.

2.61.5.5.3 Calculate reinforced concrete quantities.

2.61.5.5.3.1 Reinforced concrete slab quantity for tanks.

$$V_{cst} = (N)\ (0.825)\ (DIA + 4)^2\ \frac{t_s}{12}$$

where

V_{cst} = quantity of R.C. slab in-place, cu ft.

t_s = thickness of the slab, inches, can be obtained by:

$$t_s = 7.9 + 0.25\ SWD$$

where

SWD = sidewater depth, ft.

2.61.5.5.3.2 Reinforced concrete wall quantity for tanks.

$$V_{cst} = (N)\ (3.14)\ (SWD + 1.5)\ (DIA)\ \frac{t_w}{12}$$

where

V_{cwt} = quantity of R.C. wall in-place for tank, cu ft.

t_w = wall thickness, inches, can be calculated by:

$$t_w = 7 + (0.5)\ (SWD)$$

2.61.5.5.3.3 Reinforced concrete for splitter boxes.

$$V_{cb} = (100)\ N^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter boxes, cu ft.

2.61.5.5.3.4 Total quantity of reinforced concrete in-place.

Wall: $V_{cw} = V_{cwt} + V_{cb}$

Slab: $V_{cs} = V_{cst}$

where

V_{cw} = total quantity of R.C. wall in-place, cu ft.

V_{cs} = total quantity of R.C. slab in-place, cu ft.

2.61.5.5.4 Calculate dry solids produced.

$$DSTPD = \frac{Q_{avg}\ (C_o)\ 8.34}{100\ (2000)}$$

where

DSTPD = tons of dry solids produced per day, tpd.

2.61.5.5.5 Calculate the maintenance manpower requirement.

2.61.5.5.5.1 If $0 < DSTPD \leq 2.7$ tpd, the maintenance manpower is calculated by:

$$MMH = 141.36 \ (DSTPD)^{0.566}$$

where

MMH = maintenance manpower requirements, man-hours/yr.

2.61.5.5.5.2 If $2.7 < DSTPD \leq 13$ tpd, the maintenance manpower is calculated by:

$$MMH = 164.8 \ (DSTPD)^{0.4093}$$

2.61.5.5.5.3 If $DSTPD > 13$ tpd, the maintenance manpower is calculated by:

$$MMH = 91.04 \ (DSTPD)^{0.6415}$$

2.61.5.5.6 Calculate the operation manpower reuqirement.

2.61.5.5.6.1 If $0 < DSTPD \leq 2.7$ tpd, the operation manpower is calculated by:

$$OMH = 152.0 \ (DSTPD)^{0.7066}$$

where

OMH = operation manpower requirement, man-hours/yr.

2.61.5.5.6.2 If $2.7 < DSTPD \leq 13$ tpd, the operation manpower is calculated by:

$$OMH = 184.16 \ (DSTPD)^{0.5046}$$

2.61.5.5.6.3 If $DSTPD > 13$ tpd, the operation manpower is calculated by:

$$OMH = 93.12 \ (DSTPD)^{0.7704}$$

2.61.5.5.7 Calculate the energy requirement for operation.

2.61.5.5.7.1 If $0 < DSTPD \leq 50$ tpd, the energy requirement for operation is calculated by:

$$KWH = 4500 \ (DSTPD)^{0.301}$$

where

KWH = energy requirement for operation, kwhr/yr.

2.61.5.5.7.2 If DSTPD 50 tpd, the energy requirement for operation is calculated by:

$$KWH = 1463.6\ (DSTPD)^{0.5881}$$

2.61.5.5.8 Other operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of the total bare construction cost of the sludge thickening system.

$$OMMP = 1\%$$

where

OMMP = O&M material costs as percent of total bare construction cost for sludge thickening system, percent.

2.61.5.5.9 Other construction cost items.

2.61.5.5.9.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

2.61.5.5.9.2 Other minor cost items such as piping, site cleaning, control panel, etc., would be 15 percent of the total bare construction cost.

2.61.5.5.9.3 The correction factor would be:

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor cost items.

2.61.5.6 <u>Quantities Calculations Output Data</u>.

2.61.5.6.1 Diameter of unit, DIA, ft.

2.61.5.6.2 Quantity of earthwork required, V_{ew}, cu ft.

2.61.5.6.3 Total quantity of R.C. wall in-place required, V_{cw}, cu ft.

2.61.5.6.4 Total quantity of R.C. slab in-place required, V_{cs}, cu ft.

2.61.5.6.5 Maintenance manpower required, MMH, man-hours/yr.

2.61.5.6.6 Operation manpower required, OMH, man-hours/yr.

2.61.5.6.7 Energy requirement for operation, KWH, kwhr/yr.

2.61.5.6.8 O&M material costs as percent of total bare construction cost of sludge thickening system, OMMP, percent.

2.61.5.6.9 Correction factor for other minor cost items, CF.

2.61.5.7 Unit Price Inputs Required.

2.61.5.7.1 Cost of earthwork, UPIEX, dollars/cu yd.

2.61.5.7.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.61.5.7.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.61.5.7.4 Standard size thickener mechanism (90-ft diameter) cost, COSTTS, \$, (optional).

2.61.5.7.5 Marshall and Swift Equipment Cost Index, MSECI.

2.61.5.7.6 Equipment installation labor rate, LABRI, \$/man-hour.

2.61.5.7.7 Crane rental rate, UPICR, \$/hour.

2.61.5.8 Cost Calculations.

2.61.5.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} \cdot UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.61.5.8.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} \cdot UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = quantity of R.C. required for walls, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

2.61.5.8.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \cdot UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = quantity of R.C. required for slab, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.61.5.8.4 Cost of installed equipment.

2.61.5.8.4.1 Purchase cost of thickener equipment. The purchase cost of the thickener mechanism can be obtained from the following equation:

$$COSTTM = COSTTS \times \frac{COSTRO}{100}$$

where

COSTTM = purchase cost of thickener mechanism with diameter of DIA ft, \$.

COSTTS = purchase cost of standard size thickener mechanism with diameter of 90 ft, \$.

COSTRO = cost of mechanism with diameter of DIA ft, as percent of cost of standard size mechanism, percent.

2.61.5.8.4.2 Calculate COSTRO.

$$COSTRO = 2.16\ (DIA)^{0.8515}$$

2.61.5.8.4.3 Cost of standard size mechanism. The cost of the mechanism for a 90-ft diameter thickener for the first quarter of 1977 is

$$COSTTS = \$82,500$$

For better cost estimation COSTTS should be obtained from equipment vendor and treated as a unit price input. If COSTES is not treated as a unit price input, the cost will be automatically updated by using the Marshall and Swift Equipment Cost Index.

$$COSTTS = 82,500 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift Cost Index for first quarter 1977.

2.61.5.8.4.4 Installation man-hours for thickener mechanism. The man-hour requirement for field erection of thickener mechanism can be estimated by:

$$IMH = 2.04\ (DIA)$$

where

IMH = installation man-hour requirement, man-hours.

2.61.5.8.4.5 Crane requirement for installation, CH.

$$CH = (0.1)\ (IMH)$$

where

CH = crane time requirement for installation, hr.

2.61.5.8.4.6 Other minor costs associated with the installed equipment. This category includes the cost for electrical controls, influent pipe, effluent weirs, scum baffles, special materials, painting, etc., and can be added as percent of purchase equipment cost.

$$PMINC = 15\%$$

where

PMINC = percentage of purchase cost of equipment as minor costs, percent.

2.61.5.8.4.7 Installed equipment costs.

$$IEC = (N)\ [(COSTIM)\ (1 + \frac{PMINC}{100}) + (IMH)\ (LABRI) + (CH)\ (UPICR)]$$

where

IEC = installed equipment costs, $.

LABRI = labor rate, $/man-hour.

UPICR = crane rental rate, $/hr.

2.61.5.8.5 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC)\ (CF)$$

where

TBCC = total bare construction cost, $.

CF = correction factor for minor cost items.

2.61.5.8.6 Operation and maintenance material costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, dollars/yr.

OMMP = percentage of total bare construction cost as operation and maintenance material costs, percent.

2.61.5.9 Cost Calculations Output Data.

2.61.5.9.1 Total bare construction cost of the sludge thickener, TBCC, \$.

2.61.5.9.2 Operation and maintenance material costs, OMMC, \$/yr.

Table 2.61-2. Concentrations of Unthickened and Thickened Sludges and Solids Loadings for Mechanical Thickeners

Type of Sludge	Sludge, Solid, percent Unthickened	Thickened	Solids Loading for Mechanical Thickeners $lb/ft^2/day$
Separate sludges			
Primary	2.5 to 5.5	8.0 to 10.0	20 to 30
Trickling filter	4.0 to 7.0	7.0 to 9.0	8 to 10
Modified aeration	2.0 to 4.0	4.3 to 7.9	7 to 18
Activated	0.5 to 1.2	2.5 to 3.3	4 to 18
Combined sludges			
Primary and trickling filter	3.0 to 6.0	7.0 to 9.0	12 to 20
Primary and modified aeration	3.0 to 4.0	8.3 to 11.6	12 to 20
Primary and activated	2.6 to 4.8	4.6 to 9.0	8 to 16

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2.63 TRICKLING FILTERS.

2.63.1 Background.

2.63.1.1 Trickling filters were one of the earliest forms of wastewater treatment used in the United States. They are classified as low- or high-rate filters, depending on the hydraulic loading (1-4 mgad for low, 10-40 mgad for high). In the trickling filtration process wastewater is distributed uniformly over the filter media by a flow distributor. The majority of such units use a reaction drive rotary distributor. A large portion of the wastewater rapidly passes through the filter; the remainder slowly trickles over the slime layer formed on the filter surface. BOD removal is achieved by biosorption and coagulation from the rapidly moving portion of the flow and by progressive removal of soluble constituents from the more slowly moving portion of the flow.

2.63.1.2 The quantity of biological slime produced is controlled by the available food; the growth will increase as the organic load increases until a maximum effective thickness is reached. The maximum growth is controlled by physical factors including hydraulic dosage rate, type of media, type of organic matter, amount of essential nutrients present, oxygen transfer, and nature of the particular biological growth.

2.63.1.3 The use of recently developed synthetic media has increased the popularity as well as the capability of trickling filters in domestic wastewater treatment. The granite stone medium is rarely used in modern sewage treatment systems due to its higher capital costs (compared with synthetic media) and other limitations such as growth of filter flies, odor, and ponding problems. Thus only the synthetic medium filter tower will be designed here.

2.63.2 Input Data.

2.63.2.1 Wastewater flow.

2.63.2.1.1 Average daily flow, mgd.

2.63.2.1.2 Peak hourly flow, mgd.

2.63.2.2 Influent BOD, mg/l.

2.63.2.3 Desired effluent BOD, mg/l.

2.63.2.4 Temperature, °C.

2.63.2.5 Recirculation ratio.

2.63.3 Design Parameters.

2.63.3.1 Reaction rate constant, k (0.0015-0.003) (from laboratory).

2.63.3.2 Specific surface area of the media, ft^2/ft^3, from manufacturer = A_p.

2.63.3.3 Media factor = n (from laboratory).

2.63.3.4 Hydraulic loading, gpm/ft^2 = Q_o (from laboratory).

2.63.3.5 Sludge production factor = PF (0.42-0.65) lb solids/lb BOD_5.

2.63.4 <u>Process Design Calculations</u>.

2.63.4.1 Eckenfelder's design procedure will be utilized.

$$\frac{S}{S_o} = \text{Exp}\left[\frac{-K\ A_p \cdot D}{Q_o^n}\right]$$

where

S = desired effluent BOD_5, mg/l, from input data.

S_o = influent BOD_5, mg/l, from input data.

K = treatability constant.

A_p = specific surface area of the media, ft^2/ft^3.

D = depth of the filter tower, ft.

Q_o = hydraulic loading, gpm/ft^2.

n = media factor.

2.63.4.1.1 Treatability Constant, K. K is a parameter dependent on the wastewater characteristics and temperature, but independent of the type of media used. The values of K extracted from the literature are presented in the following table:

BOD TREATABILITY FACTORS OF SETTLED SEWAGE IN
TRICKLING FILTER WITH VARIOUS MEDIA

Type of Media	$A_p (ft^2/ft^3)$	K
1½" Raschig rings	35	0.0023
2½" slag	20	0.00145
1" - 3" gravel	19	0.00311
Surfpac	25	0.0020
Surfpac	25	0.0020
Surfpac	25	0.0018
Surfpac	28	0.00282
	Average	0.00220

Thus K can be expressed as:

$$K = (1.035)^{T-20} \cdot (0.0022)$$

where

T = waste temperature, $^{\circ}C$.

1.035 = temperature correction factor.

2.63.4.1.2 Specific Surface Area, A_p.

Filter packings have been composed of 1 to 3 inches of rock with specific surface area of 9 to 20 (ft^2/ft^3) and the new plastic media of 20 to 35 or more (ft^2/ft^3) specific surface area. As recommended by Eckenfelder, a maximum A_p of about 30 ft^2/ft^3 is recommended for the treatment of carbonaceous wastes to avoid filter plugging and ponding.

Note: It is to be emphasized that the treatability factor or rate constant used by some media manufacturers is different from the treatability constant K used in this section. Their rate constant is equivalent to $(K \cdot A_p)$.

2.63.4.1.3 Media Factor, n. The media factor, n, is an empirical factor devised to relate the film fluid flow conditions to various packing methods. It is shown that n does not appear related to Q_o, the filter hydraulic loading. The media factor is related to the specific surface area, A_p, in the following way.

$$n = 0.91 - 6.45/A_p$$

It is suggested by Eckenfelder and plastic media manufacturers that a media factor of 0.5 should be used for the surfpac and/or vinyl core media.

2.63.4.2 When recirculation is employed, the following equation should be used to consider dilution of the filter influent:

$$\frac{S}{S_o} = \frac{\text{Exp}\ [-K\ A_p\ D/Q_1{}^n]}{(1 + R) - R\ \text{Exp}\ [-K\ A_p\ D/Q_1{}^n]}$$

where

Q_1 = total hydraulic loading, gpm/ft^2.

$Q_1 = Q_o\ (1 + R)$

R = the recirculation ratio.

There are two purposes for recycling the tower effluent back to the filter tower. One is that the plastic media requires an hydraulic loading of at least 0.5 gpm/ft^2 for complete wetting of the surface. The other reason is to enhance BOD removal when high strength waste is treated. It has been shown that when the influent BOD_5 concentration exceeds 400-500 mg/l, oxygen limiting can happen in the tower. Recirculation helps to lower the influent BOD as well as increase aeration in the tower.

The following equation is given to relate the recommended recirculation rate, R, to the influent BOD concentration.

$$R = 0.004\ S_o - 0.6$$

R must always be larger than or equal to zero.

2.63.4.3 Sizing a trickling filter system.

2.63.4.3.1 Select a hydraulic loading, usually use 0.75 gpm per ft^2. If the hydraulic loading Q_1 is smaller than 0.5 gpm/ft^2, set Q_1 = 0.5 gpm/ft^2 and readjust $R = \frac{0.5}{Q_o} - 1$.

2.63.4.3.2 Calculate the required depth, D, by using:

$$D = - \frac{\ln\left(\frac{S + S^{\cdot} R}{S_o + S^{\cdot} R}\right) \cdot Q_1^{\,n}}{K \cdot A_p}$$

The height or depth of a plastic media trickling filter tower can be up to 27 feet without external structural support. Thus, it is to our advantage to limit the tower height below 30 ft to avoid expensive structure.

2.63.4.3.2.1 If D is larger than 60 ft, it is recommended that other process should be tried.

2.63.4.3.2.2 If D is larger than 40 ft but smaller than 60 ft, a 2 stage trickling filter system would be used. Thus

$$Nsg = 2$$

where

Nsg = number of stages

2.63.4.3.2.3 If D is larger than 27 ft but smaller than 40 ft, a single stage system would be used.

$$Nsg = 1$$

and, a smaller hydraulic loading (such as 0.5 gpm per sq ft) should be used and the sizing procedure should be repeated until D is smaller than 27 ft.

2.63.4.3.2.4 If D is smaller than 27 ft, the sizing procedure is completed.

$$Nsg = 1$$

2.63.4.3.2.5 If D is smaller than 8 ft, use 8 ft as depth. It is recommended by some media manufacturers that a minimum D of 8-10 ft should be provided to insure good contact between wastewater and the biofilm.

2.63.4.3.3 Calculate surface area of filter.

$$SA = \frac{Q_{avg} \times 10^6}{Q_o (1440)}$$

where

SA = surface area, ft^2.

Q_{avg} = influent waste flow, mgd.

Q_o = hydraulic loading, gpm/ft^2.

2.63.4.3.4 Calculate the filter volume.

$$V = (SA) \cdot D$$

where

V = media volume, cu ft.

2.63.4.3.5 The depth of each tower.

$$D_f = D/Nsg$$

where

D_f = depth of each tower, ft.

2.63.4.4 Filter feed pumping station design.

2.63.4.4.1 Pumping capacity: The largest of the three flow values would be used to design the pump capacity.

2.63.4.4.1.1 To take care of peak loading

$$Q_{pump} = Q_{pK}$$

2.63.4.4.1.2 To take care of recirculation

$$Q_{pump} = (1 + R) \quad Qavg$$

2.63.4.4.1.3 To take care of complete wetting

$$Q_{pump} = (SA) \cdot (0.5) \times \frac{1440}{10^6}$$

where

Q_{pump} = firm pumping capacity of the filter feed pumping station, mgd.

2.63.4.4.2 Pumping head: The pump systems should have the pumping head of

$$H_{pump} = 1.5 (D_f)$$

where

H_{pump} = pumping head, ft.

2.63.4.4.3 Number of pump stations: For a two stage system double pumping is necessary. The number of filter feed pumping station would be:

$$N_{ps} = N_{sg}$$

where

N_{ps} = number of pump stations.

2.63.4.5 Sludge Production. The sludge production is dependent on the loading rate of the trickling filter. However, no information is available from the literature concerning this relationship. Based on field measurements, the solids production rate is in the range of 0.42 to 0.65 lbs solids per lbs of BOD_5 applied. A typical value of 0.45 lbs per lbs of BOD_5 for a typical domestic waste is suggested. Thus, the sludge production rate would be:

$$SP = (8.34) (Q_{avg}) (S_o) \cdot (SPR)$$

where

SP = sludge solids produced, lbs/day.

SPR = solids production rate.

2.63.4.6 Effluent Quality.

2.63.4.6.1 Suspended Solids.

The effluent suspended solids concentration is specified by the user.

2.63.4.6.2 BOD_5.

$$BODE = S_e + (0.84)\ (f')\ (SSE)$$

S_e is specified by the user.

where

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

f' = degradable fraction of MLVSS.

SSE = effluent suspended solids concentration, mg/l.

2.63.4.6.3 COD.

$$CODE = 1.5\ BODE$$
$$CODES = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODES = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

2.63.4.6.4 Nitrogen.

$$TKNE = (0.7)\ TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3 + (0.3)\ (TKN)$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

NO2 = influent NO_2 concentration, mg/l.

2.63.4.6.5 Phosphorus.

$$PO4E = (0.7)\ (PO4)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

2.63.4.6.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, lmg/l.

2.63.4.6.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

2.63.5 <u>Process Design Output Data.</u>

2.63.5.1 Removal efficiency, percent.

2.63.5.2 Volume of filter media, ft^3, V.

2.63.5.3 Surface area, sq ft. SA

2.63.5.4 Total tower depth, D, ft.

2.63.5.5 Sludge production, lbs/day, SP.

2.63.5.6 Number of stages, N_{sg}.

2.63.5.7 Depth of individual filter, D_f, ft.

2.63.5.8 Recirculation ratio, R.

2.63.5.9 Tower filter pump capacity, Q_{pump}, mgd.

2.63.5.10 Number of pump stations, N_{ps}.

2.63.6 Quantities Calculations.

2.63.6.1 Select number of filters. Field experience indicates that a single filter will be used when the total media volume is less than approximately 50,000 cu ft. Above this value two towers are generally utilized. The surface area of the filter tower is limited by the available sizes of the distribution arms. The distribution arms are generally in the range of 20 to 200 feet in diameter. Thus the maximum volume of a single trickling filter tower is limited by the diameter of the distributor arms and the depth of the media. Using the following values as the maximum

Diameter of tower = 150 ft

The maximum surface area of the filter tower would be:

$$SA_{max} = (150)^2 \times 1/4 \times \quad = 17{,}700 \text{ sq ft}$$

The number of towers per stage, NTS, would be determined by the following rule:

NTS	Surface area of Single Stage (sq ft)
1	2,000
2	2,000 - 17,700
3	17,700 - 35,400
4	35,400 - 53,100
6	53,100 - 88,500
8	88,500 - 123,900
10	123,900 - 159,300
12	159,300 - 194,700
14	194,700 - 247,800
15	247,800 - 265,500
16	265,500 - 283,200
18	283,200 - 300,900
20	300,900 - Above

Total number of towers = $N = N_{TS} \cdot Nsg$

where

N = total number of towers.

N_{TS} = number of tower per stage.

Nsg = number of stages.

2.63.6.2 Calculate the volume of the filter tower.

The volume to be handled by one filter, V_N, would be

$$V_N = \frac{V}{N}$$

where

V_N = volume of each individual filter tower, cu ft.

2.63.6.3 Calculate the diameter of the filter tower.

$$DIA = 1.128 \left(\frac{V_N}{D_f}\right)^{0.5}$$

where

DIA = diameter of the filter tower, ft.

D_f = *depth of the tower, ft.*

2.63.6.4 Trickling Filter Construction.

2.63.6.4.1 A typical section of a plastic media trickling filter is shown in Figure 2.63-1. It can generally be divided into four components: the medium and the outside wall, the distributor arms, the medium support system and the underdrain.

2.63.6.4.2 The plastic medium usually is supplied and installed by the manufacturers and cost of the installed system is estimated as dollars/cu ft. Polyester fiberglass, lightweight steel, and precast double-tee constructions have been utilized as the medium containment structure. In this design, it is assumed a 6-inch reinforced concrete wall will be utilized. The distributor arms include the center column and rotary distributors and their support. They are available from several manufacturers with size ranging from 20-feet to 200-feet diameter.

2.63.6.4.3 The medium support system consists of precast beams and concrete support posts as shown in Figure 2.63-2.

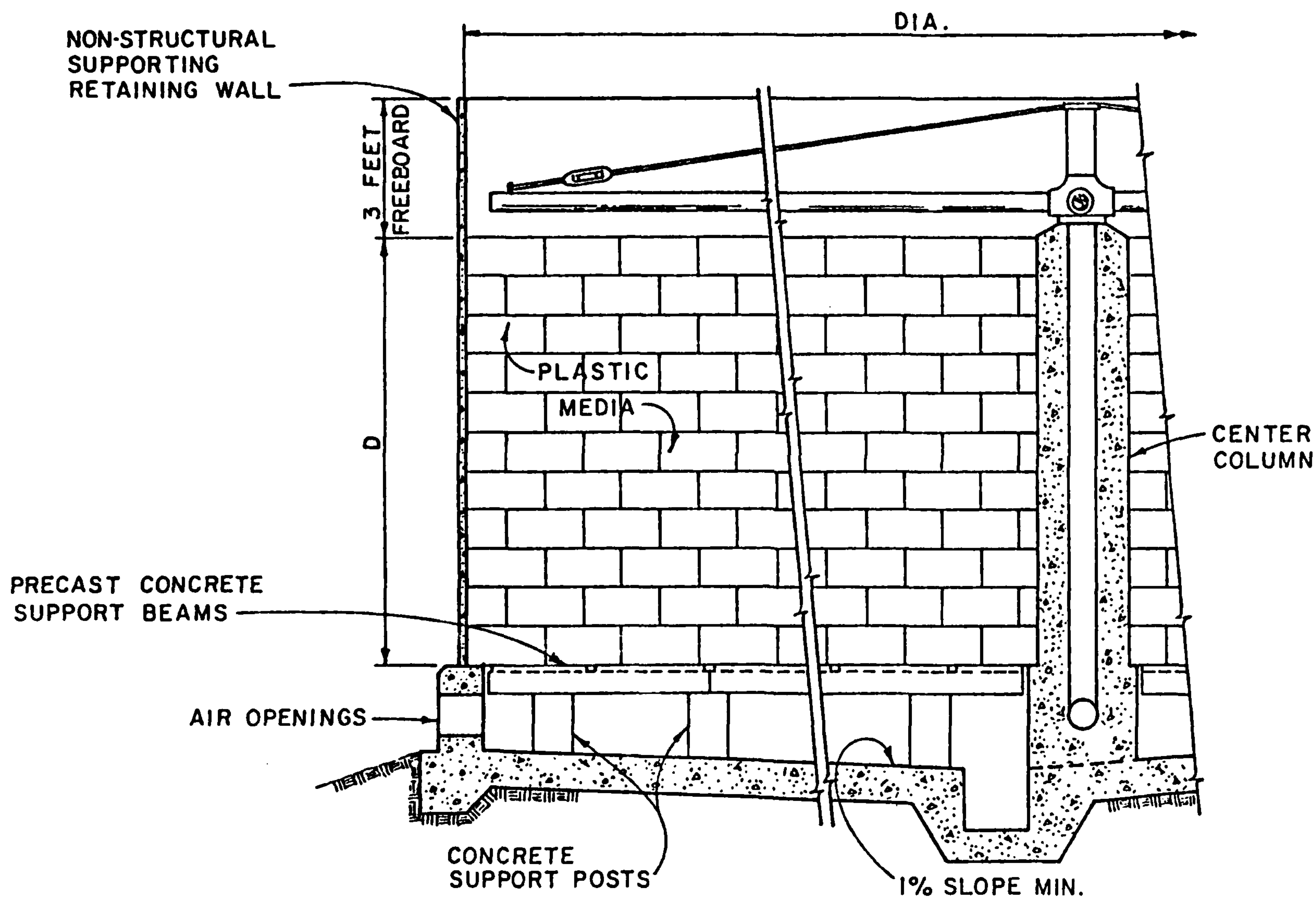

FIGURE 2.63-1. TYPICAL SECTION OF A PLASTIC MEDIA
TRICKLING FILTER TOWER

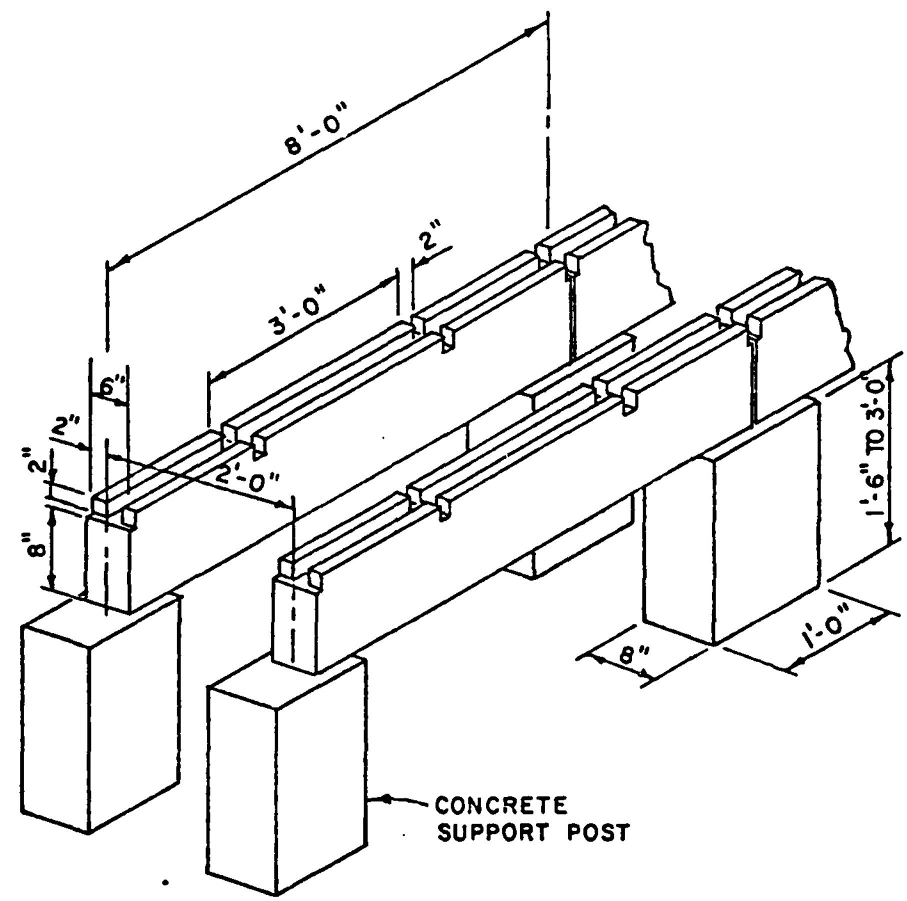

FIGURE 2.63-2. TYPICAL PRECAST CONCRETE BEAM SUPPORT SYSTEM FOR TRICKLING FILTER

2.63.6.4.4 The underdrain system includes the drainage floor and channel, sidewall with air openings, and center column for distributor support.

2.63.6.5 External Wall Construction. A reinforced wall with 6-inch thickness will be assumed for the wall. Thus, the reinforced concrete wall quantity would be:

$$V_{cwe} = 1.57\ (D_f + 3')\ (DIA)$$

where

V_{cwe} = quantity of RC wall, cu ft.

D_f = depth of filter media, ft.

DIA = diameter of the filter tower, ft.

2.63.6.6 The Media Support System. The supporting system consists of concrete supporting posts and precast concrete beams. The number of posts depends on the size of the filter and can be approximated by the following equations:

2.63.6.6.1 When DIA $\leq$ 40 ft

$$NCP = 0.00108\ (DIA)^{3.08}$$

2.63.6.6.2 When DIA > 40 ft

$$NCP = 0.0739\ (DIA)^{1.935}$$

where

NCP = total number of posts.

2.63.6.6.3 With the dimensions shown in Figure 2.63-2 and using an average depth of two feet, the volume of concrete required for the posts would be:

$$V_{cwp} = 1.333\ (NCP)$$

where

V_{cwp} = volume of R.C. wall for the supporting post, cu ft.

2.63.6.6.4 The precast concrete beam quantity, expressed as total length, can be related to the size of the filter as follows:

2.63.6.6.4.1 When DIA $\leq$ 40 ft

$$LCB = (0.0119)\ (N)\ (DIA)^{2.963}$$

2.63.6.6.4.2 When DIA > 40 ft

$$LCB = (0.364)\ (N)\ (DIA)^{2.035}$$

where

LCB = total length of the precast beams, ft.

2.63.6.7 The Underdrain System.

2.63.6.7.1 Floor concrete volume, assuming a thickness of 8"

$$V_{csf} = 0.524\ (DIA)^2$$

where

V_{csf} = volume of R.C. slab, cu ft.

2.63.6.7.2 Drainage Channel. The quantity of concrete for the drainage channel can be approximated by:

2.63.6.7.2.1 When DIA < 70 ft

$$V_{cwd} = (10)\ (DIA)$$

2.63.6.7.2.2 When DIA $\geq$ 70 ft

$$V_{cwd} = (17)\ (DIA)$$

where

V_{cwd} = volume of R.C. wall, cu ft.

2.63.6.7.3 The center column for the distributor arms. The quantity of concrete for the column can be estimated by:

2.63.6.7.3.1 When DIA < 70 ft

$$V_{cwc} = 4\ (D_f + 5')$$

2.63.6.7.3.2 When DIA $\geq$ 70 ft

$$V_{cwc} = 16\ (D_f + 5')$$

where

V_{cwc} = volume of concrete for the center column, cu ft.

2.63.6.7.4 The outside ring wall quantity.

$$V_{cwr} = 12.6\ (DIA)$$

where

V_{cwr} = volume of concrete for outside ring wall, cu ft.

2.63.6.8 Earthwork. The volume of earthwork can be estimated by:

$$V_{ewn} = (1.15) \times [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ewn} = earthwork required for the construction of a single tower, cu ft.

1.15 = safety factor for conservative design.

2.63.6.9 Total reinforced concrete wall quantity, V_{cw}.

$$V_{cw} = N\ [V_{cwe} + V_{cwp} + V_{cwd} + V_{cwc} + V_{cwr}]$$

where

V_{cw} = total quantity of R.C. wall in place, cu ft.

N = number of trickling filter towers.

2.63.6.10 Total reinforced concrete slab in place, V_{cs}.

$$V_{cs} = N \cdot V_{csf}$$

where

V_{cs} = total quantity of R.C. slab in place, cu ft.

2.63.6.11 Total earthwork required, V_{ew}.

$$V_{ew} = N \cdot V_{ewn}$$

where

V_{en} = total earthwork required, cu ft.

2.63.6.12 Electrical energy required for operation. The electric energy required for operation depends on the flow to be pumped and the total dynamic head. It can be estimated by the following equation.

$$KWH = 21,000\ (Q_{avg})^{0.961}$$

where

KWH = electric energy required, kwhr/yr.

Q_{avg} = average daily flow, mgd.

2.63.6.13 Operation and maintenance manpower requirement.

2.63.6.13.1 Operation man-hours required, OMH.

2.63.6.13.1.1 When $Q_{avg} \leq 1.0$ mgd.

$$OMH = 128\ (Q_{avg})^{0.301}$$

2.63.6.13.1.2 When $1.0 < Q_{avg} \leq 10$ mgd

$$OMH = 128\ (Q_{avg})^{0.6088}$$

2.63.6.13.1.3 When $Q_{avg} > 10$ mgd

$$OMH = 68\ (Q_{avg})^{0.8861}$$

where

OMH = operational manpower requirement, man-hour/yr.

Q_{avg} = average daily flow, mgd.

2.63.6.13.2 Maintenance man-hour requirement, MMH.

2.63.6.13.2.1 When $Q_{avg} \leq 1.0$ mgd

$$MMH = 112\ (Q_{avg})^{0.2430}$$

2.63.6.13.2.2 When $1.0 < Q_{avg} \leq 10$ mgd

$$MMH = 112\ (Q_{avg})^{0.6021}$$

2.63.6.13.2.3 When $Q_{avg} > 10$ mgd

$$MMH = 80\ (Q_{avg})^{0.7066}$$

where

MMH = maintenance man-hours required, man-hour/yr.

2.63.6.14 Other operation and maintenance material costs. This item includes repair and replacement material costs. It is expressed as a percent of total installed cost of equipment which includes the plastic media and distributor arms.

$$OMMP = 1\%$$

OMMP = percent of the installed equipment costs for the operation and maintenance material costs, %.

2.63.6.15 Other minor construction items. Items such as piping, walkways around the towers, and site cleaning would be approximately 20 percent of the total construction costs.

CF, the correction factor for this minor cost would be:

$$CF = \frac{1}{0.8} = 1.25$$

2.63.7 <u>Quantities Calculations Output Data.</u>

2.63.7.1 Total volume of media, V_d, cu ft.

2.63.7.2 Number of filter towers, N.

2.63.7.3 Total R.C. wall in-place, V_{cw}, cu ft.

2.63.7.4 Total R.C. slab in-place, V_{cs}, cu ft.

2.63.7.5 Total earthwork, V_{ew}, cu ft.

2.63.7.6 Total length of precast concrete media support beam, LCB, ft.

2.63.7.7 Operational manpower requirement, OMH, MH/yr.

2.63.7.8 Maintenance manpower requirement, MMH, MH/yr.

2.63.7.9 Electric energy requirement, KWH, kwhr/yr.

2.63.7.10 Other operation and maintenance material costs, OMMP, percent.

2.63.7.11 Correction factor for other capital costs, CF.

2.63.8 <u>Unit Price Input Required</u>.

2.63.8.1 Cost of earthwork, UPIEX, \$/cu yd.

2.63.8.2 Cost of R.C. wall in-place, UPICW, \$/cu yd.

2.63.8.3 Cost of R.C. slab in-place, UPICS, \$/cu yd.

2.63.8.4 Unit price of filter media, UPFM, \$/cu ft.

2.63.8.5 Unit price for a 50-foot diameter distributor system, CODAS, \$ (optional).

2.63.9 <u>Cost Calculations</u>.

2.63.9.1 Cost of earthwork, COSTE, \$.

$$\text{COSTE} = \frac{V_{ew}}{27} \times \text{UPIEX}$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

2.63.9.2 Cost of reinforced concrete wall in-place, COSTCW, \$.

$$\text{COSTCW} = \frac{V_{cw}}{27} \times \text{UPICW}$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = total quantity of R.C. wall cu ft.

UPICW = unit price input of R.C. wall in-place, \$/cu yd.

2.63.9.3 Cost of R.C. slab in-place, COSTCS, \$.

$$COSTCS = \frac{V_{cs}}{27} \times UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab, cu yd.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

2.63.9.4 Cost of the filter media, COSTFM, \$.

$$COSTFM = V_d \cdot UPFM$$

where

COSTFM = cost of installed filter media, \$.

V_d = volume of plastic medium, cu ft.

UPFM = unit price of plastic media installed, \$/cu ft.

The first quarter 1977 price for UPFM is \$2.50/cu ft. For a better estimate this unit price should be obtained from equipment vendor.

2.63.9.5 Cost of distributor arms.

2.63.9.5.1 Purchase cost of distributor arms. The purchase cost of distributor arms can be obtained by using the following equation:

$$CODA = CODAS \cdot CRIODA$$

where

CODA = purchase cost of distributor arm with diameter of DIA ft, \$.

CODAS = purchase cost of a standard sized distributor arm with diameter of 50 feet.

The 1st quarter 1977 price for CODAS is \$39,000. However, for a better estimate, CODAS should be obtained from equipment vendors and treated as a unit price input. Otherwise, the following equation will be utilized for cost escalation purposes:

$$CODAS = 39,000 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift equipment cost index for input.

491.6 = MSECI value, 1st quarter 1977.

CROIDA = ratio of cost of distributor arm with diameter of DIA feet to that of the standard size arm.

and CROIDA can be estimated by:

$$CROIDA = 0.367 + 0.01265\ (DIA)$$

2.63.9.5.2 Installation costs. An additional 32 percent of the purchase cost would be added for the installation costs.

2.63.9.5.3 Total installed costs for the distributor arms, ICODA.

$$ICODA = (N)\ (CODA)\ (1.32)$$

where

ICODA = total installed cost for the distributor arm system, \$.

2.63.9.6 Cost of the precast concrete media support beams:

2.63.9.6.1 The unit cost of the precast concrete beams can be approximated by using five times the unit cost of reinforced concrete wall in-place. The quantity of precast beam is V_{pcb}.

$$V_{pcb} = \left(\frac{6}{12}\right) \left(\frac{8}{12}\right) (LCB)$$

where

LCB = total length of beams, ft.

$\frac{6}{12}$ = width of the beam, ft.

$\frac{8}{12}$ = length of beam, ft.

2.63.9.6.2 Cost of precast concrete beam, CPCB, \$.

$$CPCB = \frac{5}{27} (V_{pcb})\ (UPICW)$$

where

CPCB = cost of precast concrete beams, \$.

UPICW = unit price input of R.C. wall in-place, \$/cu yd.

2.63.9.8 Total bare construction cost of the trickling filter system.

$$TBCC = CF\ (COSTE + COSTCW + COSTCS + COSTFM + ICODA + CPCB)$$

where

TBCC = total bare construction cost, \$.

CF = correction factor for minor construction cost.

2.63.9.8 Operation and maintenance material cost, OMMC. Since O&M material cost is estimated as a percent of the installed equipment cost, it is thus:

$$OMMC = \frac{OMMP}{100}\ (COSTFM + ICODA)$$

where

OMMC = operation and maintenance material cost, \$/yr.

OMMP = operation and maintenance material cost as percent of installed equipment cost, %.

2.63.10 <u>Cost Calculations Output Data.</u>

2.63.10.1 Total bare construction cost of the trickling filter system, TBCC, \$.

2.63.10.2 Operation and maintenance material cost, OMMC, \$/yr.

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2.65 VACUUM FILTRATION

2.65.1 Background.

2.65.1.1 Vacuum filtration is one of the most widely used methods for mechanical dewatering of wastewater sludges. The process is carried out using a slowly rotating drum, the outside of which is covered by a filter medium. A portion (about 20-40 percent) of the drum is submerged in sludge in the vat below the drum. Vacuum (10-26 in. of mercury) is applied to the submerged portion of the trough. As a result, water is drawn into the drum and a thin mat of sludge is formed on the filter medium. As the filter rotates, the vacuum is continued, and further moisture reduction occurs. In addition, the deposited cake is further dried by air which rushes through the cake into the drum. Before the filter cake reaches the sludge vat again, it passes over a roller and is broken off onto a conveyor for ultimate disposal. The time the drum spends submerged in the slurry is called the "filter time"; the time the cake spends on the drum above the vat is called the "drying time".

2.65.1.2 Vacuum filtration facilities are generally sold as a package by various filter manufacturers. In addition to the filter itself, the package normally includes vacuum pumps, sludge feed pumps, filtrate pumps, sludge conditioning tanks, chemical feed pumps, and belt conveyors that transport dewatered filter cake. Filter medium made of cloth (cotton, wood, nylon, dacron, or other synthetic material), coil springs, or a wire-mesh stainless steel fabric are available in various weaves of different porosities.

2.65.1.3 Vacuum filter performance is measured by filtration rate and dryness of the filter cake. Several factors affecting the performance of a vacuum filter include:

2.65.1.3.1 Vacuum -- As the vacuum increases, the filtration rate and the dryness of the cake also increase; this process is limited, obviously, by capacity of the drum. An ideal filter design would incorporate two independent vacuum systems: one operating while the cake is being formed, and other after it comes out of submergence and is being dried. A vacuum of at least 20 in of mercury is desirable.

2.65.1.3.2 Feed Solid Concentration -- In general, the sludge filtration rates increase directly in proportion to the increase in feed sludge solids concentration; a smaller filtrate volume has to be removed per pound of filter cake formed. The practical limit for optimum operation for sewage sludges is 4-8 percent.

2.65.1.3.3 Drum Speed and Submergence -- The drum speed influences the cycle time and consequently influences filter yield and filter cake moisture. A decrease in filter cycle time should increase the yield. However, lower filter cake moisture will be obtained by increasing the filter cycle, thereby extending the drying cycle. A submergence level of 20-40 percent has been used. It is usually more economical to run the filter at lower submergence since it will increase the ratio of dewatering time to cake formation time, and will still allow a short cycle for greater filtration rates.

2.65.1.3.4 Chemical Conditioning of Sludge -- Chemical conditioning of sludge is usually a necessary step prior to sludge vacuum filtration. Chemical conditioning agglomerates solids and causes a release of water, thereby making the sludge easier to filter. A wide variety of chemicals have been evaluated for conditioning sludges prior to vacuum filtration. In general, lime and ferric chloride are the most commonly used conditioners. Recently, some organic polyelectrolytes have become popular as sludge conditioners. The amount and type of conditioning chemicals required depend on the physical and chemical characteristics of the sludge. Tables 2.65-1 and 2.65-2 summarize chemical doses reported from the operating records of various treatment facilities.

2.65.1.4 Vacuum filter systems are designed from data describing quantities of sludge to be filtered, sludge characteristics, filtration rates, cake moisture, and filter operation cycles. The data could be generated from laboratory or pilot investigations of the sludge. The Buchner funnel test and the filter leaf test are commonly used in laboratory testing programs for estimating the filterability of sludges. The Buchner funnel test evaluates the optimum chemical requirements and sludge filtration characteristics measured in terms of specific resistance. The filter leaf test determines the effect of different fabrics, fabric forms, and drying times on filter yield. Table 2.65-3 summarizes specific resistance; Table 2.65-4 presents filter yields recommended for various sewage sludges.

2.65.1.5 Filter yield, or production rate, is the basic factor used in determining the size of vacuum filter installations. A conservative design rate of 3.5 $lb/ft^2/hr$ has been widely used. However, assuming the yield to be equal to the solids concentration of the sludge to be filtered is more accurate. Generally, the yield may vary from 2 to 10 $lb/ft^2/hr$. The low values represent filtration of fresh and digested activated sludge; the high values are typical for raw primary or primary plus trickling filter humus sludge filtration.

2.65.1.6 Vacuum filtration is being widely used for dewatering sewage sludges in small treatment facilities because of its flexibility, small space requirement, and the excellent characteristics of the cake. Thus, vacuum filtration is a viable alternative for sludge dewatering in treatment facilities serving recreation areas.

2.65.2 <u>Input Data</u>.

2.65.2.1 Volume of sludge to be dewatered, gpd.

2.65.2.2 Initial moisture content of sludge, percent.

2.65.3 <u>Design Parameters</u>.

2.65.3.1 Final moisture content of sludge, percent.

2.65.3.2 Specific resistance, sec^2/g (Buchner funnel test).

2.65.3.3 Applied vacuum, psi.

2.65.3.4 Fraction of cycle time for cake formation (formation time/cycle time), depends on degree of submergence.

2.65.3.5 Cycle time, min (usually 1.5 to 5 min).

2.65.3.6 Filtrate viscosity, centipoises.

2.65.3.7 Chemical dose, percent of dry weight in solids fed to filter.

2.65.3.8 Operation per week, days.

2.65.3.9 Operation per day, hr.

2.65.3.10 Loading rate, $lb/ft^2/hr$.

2.65.3.11 Number of units.

2.65.4 <u>Process Design Calculations</u>.

2.65.4.1 Calculate filter loading rate.

Table 2.65-1. Average Chemical Doses for Vacuum Filtration

Type of Sludge	Chemical Dose Rate, percent Ferric Chloride	Chemical Dose Rate, percent Lime	Yield $lb/ft^2/hr$	Cake Moisture percent
Raw primary	2.1	8.8	6.9	69.0
Digested primary	3.8	12.1	7.2	73.0
Elutriated digested primary	3.4	0.0	7.5	69.0
Raw primary plus filter humus	2.6	11.0	7.1	75.0
Raw primary plus activated	2.6	10.1	4.5	77.5
Raw activated	7.5	0.0	0.0	84.0
Digested primary plus filter humus	5.3	15.0	4.6	77.5
Digested primary plus activated	5.6	18.6	4.0	78.5
Elutriated digested primary plus activated:				
Average without lime	8.4	0.0	3.8	79.0
Average with lime	2.5	6.2	3.8	76.2

Table 2.65-2. Polyelectrolyte Doses (Usual Ranges) for Vacuum Filtration

Type of Sludge	Dose Rate Percent	Yield $lb/ft^2/hr$	Cake Moisture Percent
Raw primary or raw primary plus filter humus	0.2 to 1.2	6 to 20	63 to 72
Digested primary	0.2 to 1.5	4 to 15	66 to 74
Digested primary plus activated	0.5 to 2.0	4 to 8	68 to 76

Table 2.65-3. Specific Resistance of Some Industrial and Municipal Sludges

Type of Sludge	Specific Resistance $sec^2/g \times 10^7$ at 500 g/cm^2	Coefficient of Compressibility
Neutralization of sulfuric acid with lime slurry	1 to 2	
Neutralization of sulfuric acid with dolomitic lime slurry	3	0.77
Processing of aluminum	3	0.44
Paper industry	6	0.0
Neutralization of fatty acids with sodium carbonate	7	0.0
Froth flotation of coal	80	1.6
Malt whisky distillery	200	1.3
Mixed chrome and vegetable tannery	300	0.0
Biological treatment of chemical wastes	300	0.0
Activated (domestic)	2880	0.81
Raw conditioned (domestic)	3.1	1.00
Digested conditioned (domestic)	10.5	0.0
Digested and activated (conditioned)	14.6	1.10
Raw (domestic)	470	0.54

Table 2.65-4. Expected Performance of Vacuum Filters
Handling Properly Conditioned Sludge

Type of Sludge	Yield, $lb/ft^2/hr$
Fresh solids	
Primary	4 to 12
Primary plus trickling filter	4 to 8
Primary plus activated	4 to 5
Activated (alone)	2.5 to 3.5
Digested solids (with or without elutriation)	
Primary	4 to 8
Primary plus trickling filter	4 to 5
Primary plus activated	4 to 5

$$LR = 35.7 \left(\frac{XCP}{RT} \right)^{½}$$

$$R = r(10^{-7}) = sec^2/g$$

$$C = \frac{1}{\frac{C_i}{100 - C_i} - \frac{C_f}{100 - C_f}}$$

where

LR = filter loading rate, $lb/ft^2/hr$.

X = form time/cycle time (0.1 to 0.6).

C = weight of dry solids in cake, g/ml.

P = applied vacuum, psi (5 to 15 psi).

R = filtrate viscosity, centipoises.

r = specific resistance (Buchner funnel test or Table 2.31-3, sec^2/g.

t = cycle time (1.5 to 5 min), time of revolution of drum.

C_i = initial moisture content of sludge, percent.

C_f = final moisture content of sludge, percent, (31%).

2.65.4.2 If data are not available, select loading rate from Table 2.31-4 or use 3.5 $lb/ft^2/hr$.

2.65.4.3 Calculate required total filter area.

$$TFA = \frac{V(100 - C_i)(8.34)(7)24(100+C_d)}{LR(100)(HPD)(DPW)}$$

where

TFA = total filter area, ft^2.

V = sludge volume, gal/day.

C_i = initial moisture content of sludge, percent.

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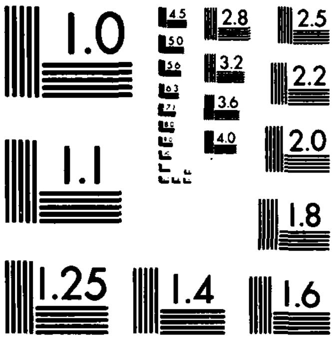

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

LR = filter loading rate, $lb/ft^2/hr$.

HPD = operati^n per day, hr.

DPW = days/week.

2.65.4.4 Calculate amount of chemicals required.

$$CR = \frac{C_d V}{(HPD)(DPW)(100)} (7)(8.34) \frac{(100 - C_i)}{100}$$

where

CR = amount of chemicals required, lb/hr.

C_d = chemical dosage, percent of dry weight of solids fed to filter.

V = sludge volume, gal/day.

HPD = operation per day, hr.

DPW = days/week.

C_i = initial moisture content of sludge, percent.

2.65.4.5 Select number of filters and calculate area per filter.

$$APF = \frac{TFA}{NF}$$

where

APF = area per filter, ft^2.

TFA = total filter area, ft^2.

NF = number of filters.

2.65.4.6 Effluent Sludge Characteristics.

2.65.4.6.1 Sludge Volume.

$$SF = \frac{V(100 - Ci)\ (100 + C_d)\ (.9)}{100 - C_f}$$

where

SF = sludge volume after filtration, gpd.

V = sludge volume before filtration, gpd.

Ci = initial moisture content of sludge, %.

C_f = final moisture content of sludge, %.

C_d = chemical dosage, percent of dry weight of solids fed to filter, %.

2.65.4.6.2 Volatiles in sludge.

$$PVOLAT = \frac{(SF)\ (C_f)\ (VOLAT)}{(V)\ (100 - Ci)\ (100 + C_d)\ (.9)}$$

where

$PVOLAT$ = percent volatile in effluent sludge, %.

SF = sludge volume after filtration, gpd.

C_f = final moisture content of sludge, %.

$VOLAT$ = percent volatile in influent sludge, %.

V = sludge volume before filtration, gpd.

Ci = initial moisture content of sludge, %.

C_d = chemical dosage, percent of dry weight of solids fed to filter, %.

2.65.5 Process Design Output Data.

2.65.5.1 Loading rate, lb/ft^2/hr.

2.65.5.2 Total filter area, ft^2.

2.65.5.3 Chemical requirements, lb/hr.

2.65.5.4 Initial moisture content, percent.

2.65.5.5 Final moisture content, percent.

2.65.5.6 Cycle time, min.

2.65.5.7 Formation time, min.

2.65.5.8 Chemical dose, percent.

2.65.5.9 Applied vacuum, psi.

2.65.5.10 Operation per day, hr.

2.65.5.11 Operation per week, days.

2.65.6 Quantities Calculations.

2.65.6.1 Filter Selection.

2.65.6.1.1 Units must be one of the following sizes (FA_A) as these are the sizes commercially available: 60, 85, 100, 125, 150, 200, 250, 300, 360, 430, 500, 575, 675, 750 sq ft.

where

FA_A = filter area of commercially available units, sq ft.

2.65.6.1.2 If the total filter area is less than 750 sq ft, only one filter will be used as it is assumed that plants of this size will run the filter one shift per day and will have adequate sludge storage should the filter be out of service for any period of time. The total filter area (TFA) should be compared to the commercially available units (FA_A) and the smallest available unit which is larger than TFA should be selected.

2.65.6.1.3 If the total filter area (TFA) is greater than 750 sq ft, then a minimum of two units will be used. For maintenance purposes all units will be the same size.

$$A_F = \frac{TFA}{N_F}$$

where

A_F = calculated filter area for a single filter, sq ft.

N_F = the number of filters.

The selection of the correct filter size must be done by trial and error. Since the minimum number of units is two, the first trial will be:

Try $N_F = 2$: If A_F is greater than 750 sq ft, try $N_F = N_F + 1$ and repeat until A_F is less than 750 sq ft. When an A_F less than 750 sq ft is obtained, compare A_F to FA_A and select the smallest FA_A that is greater than A_F. If $FA_A \times N_F$ is larger than TFA by more than 10%, go to $N_F = N_F + 1$ and repeat the procedure. When $FA_A \times N_F$ is larger than TFA by less than 10%, then $FA_A = FA_S$.

where

FA_S = filter area of the commercial unit selected, sq ft.

2.65.6.2 Calculate building size for housing filters. The area of the building was determined from the size of the equipment to be installed such as vacuum filter, sludge pumps, vacuum pumps, filtrate tanks, chemical pumps, belt conveyors, etc.

$$A_B = [190 + 1.2\ (FA_S)]\ N_F$$

where

A_B = area of building, sq ft.

FA_S = filter area of the commercial unit selected, sq ft.

N_F = number of filters.

2.65.6.3 Calculate dry solids produced.

$$DS = \frac{V\ (100 - C_1)\ (8.34)\ (1 + {}^{C}d/100)}{100\ (2000)}$$

where

DS = dry solids produced, tpd.

V = volume of sludge, gpd.

C_1 = initial moisture content of sludge, percent.

2.65.6.4 Calculate operational labor.

2.65.6.4.1 If the dry solids produced (DS) is betwen .01 tpd and .09 tpd.

$$OMH = 520 \text{ manhours/yr}$$

2.65.6.4.2 If the dry solids produced (DS) is between .09 tpd and 9.0 tpd,

$$OMH = 1760 \ (DS)^{0.504}$$

2.65.6.4.3 If the dry solids produced (DS) is between 9.0 tpd and 300 tpd,

$$OMH = 1200 \ (DS)^{0.734}$$

where

OMH = operating labor, man-hours/yr.

DS = dry solids produced, tpd.

2.65.6.5 Calculate maintenance labor.

2.65.6.5.1 If the dry solids produced (DS) is between .01 tpd and .09 tpd,

$$MMH = 64 \text{ man-hours/yr}$$

2.65.6.5.2 If the dry solids produced (DS) is between .09 tpd and 9.0 tpd,

$$MMH = 240 \ (DS)^{0.548}$$

2.65.6.5.3 If the dry solids produced (DS) is between 9.0 tpd and 300 tpd,

$$MMH = 136 \ (DS)^{0.808}$$

where

MMH = maintenance labor, man-hours/yr.

DS = dry solids produced, tpd.

2.65.6.6 Calculate energy requirement for operation.

$$KWH = 28{,}000 \ (DS)^{0.933}$$

where

KWH = energy requirement, kwhr/yr.

DS = dry solids produced, tpd.

2.65.6.7 Calculate O&M material and supply cost.

2.65.6.7.1 Material and supply costs include such items as lubrication oil, paint, and replacement material, etc. These costs are estimated as 5% of the installed equipment cost.

2.65.7 <u>Quantities Calculations Output Data</u>.

2.65.7.1 Filter area of the commercial unit selected, FA_S, sq ft.

2.65.7.2 Number of filters, N_F.

2.65.7.3 Area of building, A_B, sq ft.

2.65.7.4 Dry solids produced, DS, tpd.

2.65.7.5 Operating labor, OMH, man-hours/yr.

2.65.7.6 Maintenance labor, MMH, man-hours/yr.

2.65.7.7 Energy requirement, KWH, kwhr/yr.

2.65.7.8 Chemical requirement, CR, lb/hr.

2.65.8 <u>Unit Price Input Required</u>.

2.65.8.1 Standard size vacuum filter cost, COSTSF, $, (optional).

2.65.8.2 Equipment installation labor rate, LABRI, $/MH.

2.65.8.3 Building construction costs, UPIBC, $/sq ft.

2.65.8.4 Current Marshall and Swift equipment cost index, MSECI.

2.65.9 <u>Cost Calculations</u>.

2.65.9.1 Calculate cost of installed equipment.

2.65.9.1.1 Purchase cost of vacuum filter includes the cost of the vacuum filter, vacuum pump, filtrate pump, filtrate tank, sludge pump, chemical pumps, conditioning tank, conveyor belt, electric motors, and electrical control panel.

$$COSTF = \frac{COSTSF \cdot COSTR \cdot N_F}{100}$$

where

COSTF = purchase cost of vacuum filter and accessories \$.

COSTSF = purchase cost of standard size filter, 300 sq ft, \$.

COSTR = cost of filter of area FA_S as percent of the cost of the standard size filter, percent.

N_F = number of filters.

2.65.9.1.2 Calculate COSTR. The percent of the cost of the standard size unit is calculated by:

$$COSTR = 52 + .16\ (FA_S)$$

where

COSTR = cost of filter of area FA_S as percent of the cost of the standard size filter, %.

FA_S = filter area of the commercial unit selected, sq ft.

2.65.9.1.3 Purchase cost of standard size filter. The 300 sq ft vacuum filter was selected as the standard size unit as it is midrange of the sizes available. The costs of a 300 sq ft vacuum filter and accessories for the first quarter of 1977 are:

$$COSTSF = \$150,000$$

For better estimation, COSTSF should be obtained from the equipment vendor and treated as a unit price input. If COSTSF is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSF = \$150,000 \cdot \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index 1st quarter 1977.

2.65.9.1.4 Calculate filter installation labor. If the filter selected (FA_S) is smaller than 400 sq ft, then:

$$IL = [544 + 0.32\ (FA_S)]\ N_F$$

If the filter selected (FA_S) is larger than 400 sq ft, then:

$$IL = [476 + 0.48\ (FA_S)]\ N_F$$

where

IL = filter installation labor, man-hours.

FA_S = filter area of the commercial unit selected, sq ft.

N_F = number of filters.

2.65.9.1.5 Calculate filter installation cost.

$$ICOST = (IL)\ (LABRI)$$

where

ICOST = filter installation cost, \$.

IL = filter installation labor, man-hours.

LABRI = installation labor rate, \$/man-hour.

2.65.9.1.6 Calculate other equipment installation costs. This includes costs for installation of vacuum pump, filtrate pump, filtrate tank, sludge tank, sludge pump, chemical pumps, conveyor belt, electrical panel, and piping. These costs are estimated as 60% of the purchase costs of the vacuum filter and accessories.

$$OICOST = COSTF\ (.6)$$

where

OICOST = other equipment installation costs, \$.

COSTF = purchase cost of vacuum filter and accessories, \$.

2.65.9.1.7 Calculate total installed equipment costs.

$$IEC = COSTF + ICOST + OICOST$$

where

IEC = total installed equipment costs, \$.

COSTF = purchase cost of vacuum filter and accessories \$.

ICOST = filter installation labor, \$.

OICOST = other equipment installation costs, \$.

2.65.9.2 Calculate cost of filter building.

$$COSTB = A_B \cdot UPIBC$$

where

COSTB = cost of filter building, \$.

A_B = area of building, sq ft.

UPIBC = unit price input for building construction costs, \$/sq ft.

2.65.9.3 Calculate total bare construction costs.

$$TBCC = IEC + COSTB$$

where

TBCC = total bare construction costs, \$.

IEC = total installed equipment costs, \$.

COSTB = cost of filter building, \$.

2.65.9.4 Calculate operation and maintenance material and supply costs.

$$OMMC = (IEC)\ (.05)$$

where

OMMC = operation and maintenance material and supply costs, $.

IEC = total installed equipment costs.

2.65.10 Cost Calculations Output Data.

2.65.10.1 Total bare construction costs, TBCC, $.

2.65.10.2 Operation and maintenance material and supply costs, OMMC, $/yr.

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2.67 WET OXIDATION

2.67.1 Background.

2.67.1.1 Wet oxidation, also known as wet air oxidation, wet incineration, and wet combustion, is based on the principle that any substance capable of burning can be oxidized in the presence of liquid water at temperatures between 250°F and 700°F. In general, any degree of oxidation desired can be accomplished by providing the proper temperature, pressure, reaction time, and sufficient compressed air or oxygen.

2.67.1.2 The process has been patented and is commercially known as the Zimpro process. Th process is capable of operating satisfactorily at sludge solids concentrations as low as 1%, much less than that required for conventional combustion processes.

2.67.1.3 The descriptions that follow are intended to be used to evaluate the wet air oxidation process as a disposal technique. It is not intended to give or lend the rights established by the patent held on the process described herein.

2.67.1.4 The general flow diagram of the wet air oxidation system is illustrated in Figure 2.67-1. The raw sludge is ground, reducing particle sizes to a maximum of ¼ inch, and mixed with a quantity of compressed air or oxygen. When the process is operated continuously, the air-waste mixture is pumped through a series of heat exchangers, thereby being brought to the initial reaction temperature, and then into the pressurized reactor. When the process is operated as a batch system, the heat exchangers are eliminated as a source of heating the reactor influent.

2.67.1.5 The oxidation taking place in the reactor causes an increase in temperature. The oxidized effluent is cooled in the heat exchangers. Gases are separated from the liquid, which is carrying the residual oxidized material and released through a pressure-reducing valve into an odor-controlling, catalytic oxidation unit. These gases, when economical, may be expanded in a turbine as opposed to the pressure released to the atmosphere. Liquid and solids residue pass through a separate pressure-reducing valve and the residue removed by conventional separation equipment. The solids are inert and may be disposed of accordingly. Wet air oxidation process wastewater must be returned to the beginning of the wastewater treatment facilities. The process can be designed to be thermally autogenous. When additional heat is needed, as in the cases of batch operation or process start-up, steam is injected into the reactor. This may also be necessary when low levels of oxidation are being used in process operation or if the sludge being oxidized has a low energy value.

2.67.1.6 There are certain disadvantages to using this system. After a period of from 30 to 60 days the heat exchangers must be isolated from the system and cleaned. Operationally, the need to return the oxidized liquors, which are very high in organics, phosphorus, and nitrogen, through the wastewater treatment system represents a considerable load and must be taken into consideration in over-all treatment plant design. The high pressure-high temperature system also introduces some safety problems which will play a role in the design of the plant facilities.

2.67.1.7 Oxygen must be present in stoichiometric proportions to prevent impedement of combustion during the wet air oxidation process. Oxygen in excess of the required stoichiometric quantity will not accelerate the process. The chemical oxygen demand of the waste has been found to be a very convenient parameter of the oxygen required in the combustion process. Another parameter utilized and verified through experiments is the steam-to-air ratio.

2.67.2 Input Data.

2.67.2.1 Average wastewater flow, mgd.

2.67.2.2 Sludge volume, gallons/million gallons.

2.67.2.3 Raw sludge solids concentration, %.

2.67.2.4 Sludge COD (mg/l).

2.67.2.5 Operating temperature, °F.

2.67.2.6 Operating pressure, psia. From steam tables, psia.

2.67.2.7 Specific volume of saturate steam, from steam tables, ft^3/lb.

2.67.2.8 Effluent COD, mg/l. To be recycled to head of separate biological treatment facilities.

2.67.3 Design Parameters.

2.67.3.1 Sludge volume per million gallons treated.

2.67.3.2 Raw sludge solids concentration, %, 1.5-15%.

2.67.3.3 Sludge COD. (20-40 mg/l) Average 30 mg/l.

2.67.3.4 Operating temperature (250-700°F). May use 450°F as an average.

2.67.3.5 Specific volume of saturated steam, from steam tables. Use 1.446 ft^3/lb at 450°F.

2.67.3.6 COD reduction. Approximately 60-90%. Typically 75%.

2.67.3.7 Sludge retention time, select from Figure 2.67-2 for specified conditions. Use 1 hour.

2.67.4 Process Design Calculations.

2.67.4.1 Calculate the sludge volume per day.

$$SV = (Q_{avg})(SF)$$

where

SV = sludge volume, gallons/day.

Q_{avg} = average wastewater flow, mgd.

SF = sludge flow, gallons/million gallons.

2.67.4.2 Calculate the air requirements.

$$A = \frac{COD}{27.8}$$

where

A = air requirements, lb dry air/gallon of sludge.

COD = chemical oxygen demand of sludge, mg/l.

2.67.4.3 Calculate the steam-to-air ratio, assuming the gallon of sludge produces steam.

$$\frac{S}{A_t} = \frac{8.34}{0.8A}$$

where

$\frac{S}{A_t}$ = steam-to-air ratio, lb of steam/lb dry air.

0.8 = a factor to insure some water will be retained in the liquid phase.

2.67.4.4 Calculate the reactor pressure.

$$P_t = \frac{53.3(T + 460)}{144 \frac{S}{A_t} V_s + P_s}$$

where

P_t = reactor pressure, psia.

T = operating temperature, °F.

V_s = specific volume of steam, ft^3/lb.

P_s = saturated steam pressure, psia.

2.67.4.5 Calculate the reactor volume.

$$V_r = \frac{8.34(SV)(V_s)(RT)}{24}$$

where

V_r = reactor volume, ft^3.

RT = sludge retention time, hours.

2.67.4.6 Calculate the heating value of the influent waste.

$$H_w = 50(COD_r)$$

where

H_w = heating value of influent waste, btu/gallon.

COD_r = chemical oxygen demand removed, mg/l.

2.67.4.7 Calculate additional heat required.

$$H_g = 1500 - H_w - H_e$$

where

H_g = additional heat required, Btu/gallon.

H_e = heat supplied by the heat exchangers. Use 250 btu/gallon.

1500 = Btu/gallon required for an autogenous reactor.

2.67.4.8 Supernatant Return.

2.67.4.8.1 Quantity.

2.67.4.8.1.1 Activated Sludge and Oxidation Ditch.

$$QSUP = (Q_{avg})\ (0.02)$$

where

QSUP = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.67.4.8.1.2 Trickling Filter and Rotating Biological Contactor.

$$QSUP = (Q_{avg})\ (0.007)$$

where

QSUP = quantity of supernatant returned, mgd.

Q_{avg} = average daily wastewater flow, mgd.

2.67.4.8.2 Supernatant Quality.

TSS = 100

BOD = 3000

COD = 9500

TKN = 800

NH3 = 700

PH = 7.1

where

TSS = total suspended solids concentration, mg/l.

BOD = BOD_5 concentration, mg/l.

COD = COD concentration, mg/l.

TKN = total Kjeldahl nitrogen concentration, mg/l.

NH3 = ammonia nitrogen concentration, mg/l.

PH = pH.

2.67.5 Process Design Output Data.

2.67.5.1 Sludge volume, gallons, day.

2.67.5.2 Air requirements, pounds of dry air per gallon of sludge.

2.67.5.3 Reactor pressure, psia.

2.67.5.4 Reactor temperature, °F.

2.67.5.5 Reactor volume, ft^3.

2.67.5.6 Additional heat required, Btu/gallon.

2.67.6 Quantities Calculations. Not Used.

2.67.7 Quantities Calculations Output Data. Not Used.

2.67.8 Unit Price Input Required. Not Used.

2.67.9 Cost Calculations.

2.67.9.1 Unit price costing is not available for this treatment process, therefore parametric costing will be used.

2.67.9.2 Calculate operation and maintenance costs.

$$Z = 3.4263 - 0.50377(X) + 0.022872(X)^2$$

$$O\&M = \frac{e^Z(X)}{100}$$

where

O&M = operation and maintenance cost, \$/yr.

X = sludge feed rate, 1000 gpd.

2.67.9.3 Calculate total bare construction cost.

$$Y = 4.3685 - 0.39095(X) + 0.013492(X)^2$$

$$TBCC = e^Y$$

where

TBCC = total bare construction cost, \$.

X = sludge feed rate, 1000 gpd.

2.67.10 Cost Calculations Output Data.

2.67.10.1 Operation and maintenance costs, O&M, \$/yr.

2.67.10.2 Total bare construction cost, TBCC, \$.

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2.67.11.12 U.S. Environmental Protection Agency, Process Design Manual for Sludge Treatment and Disposal, EPA 625/1-74-006, Oct., 1974.

2.67.11.13 Zimmermann, F.J., "New Waste Disposal Process" Chemical Engineering, 25 Aug. 1958.

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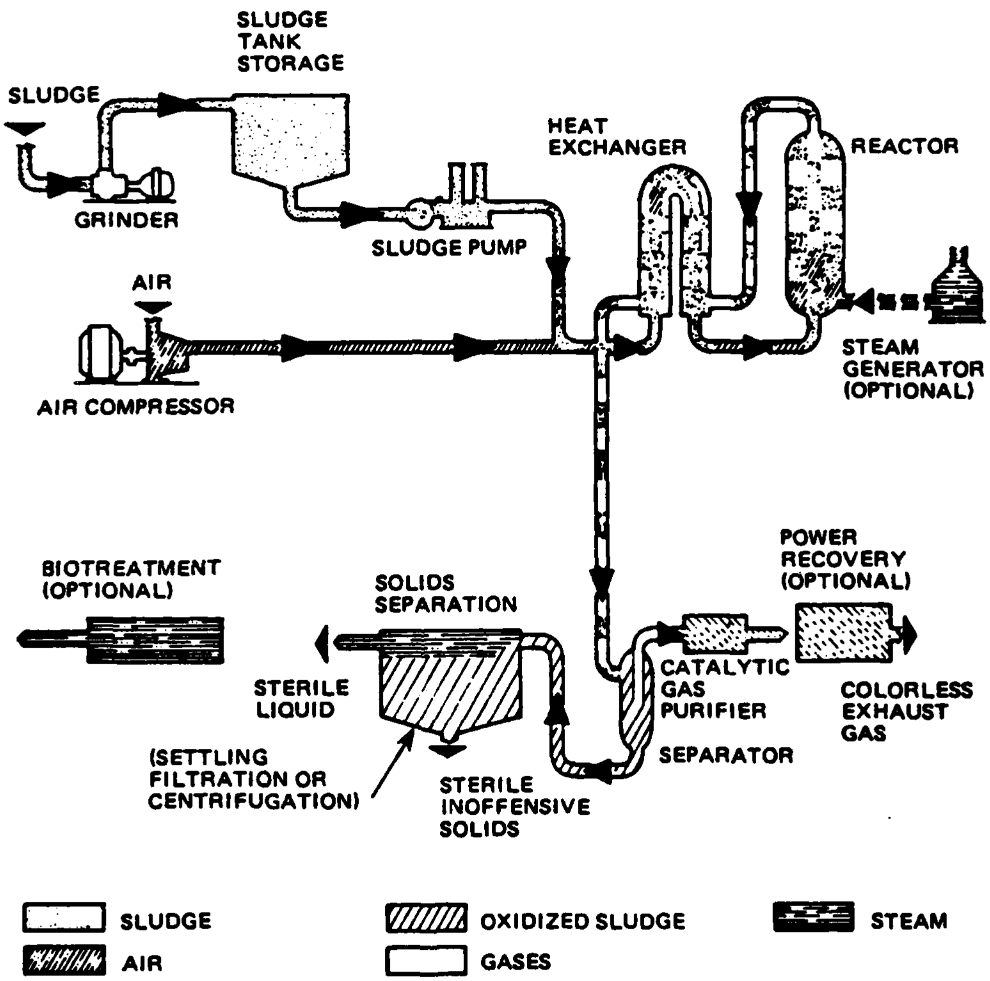

FIGURE 2.67-1. WET AIR OXIDATION SYSTEM

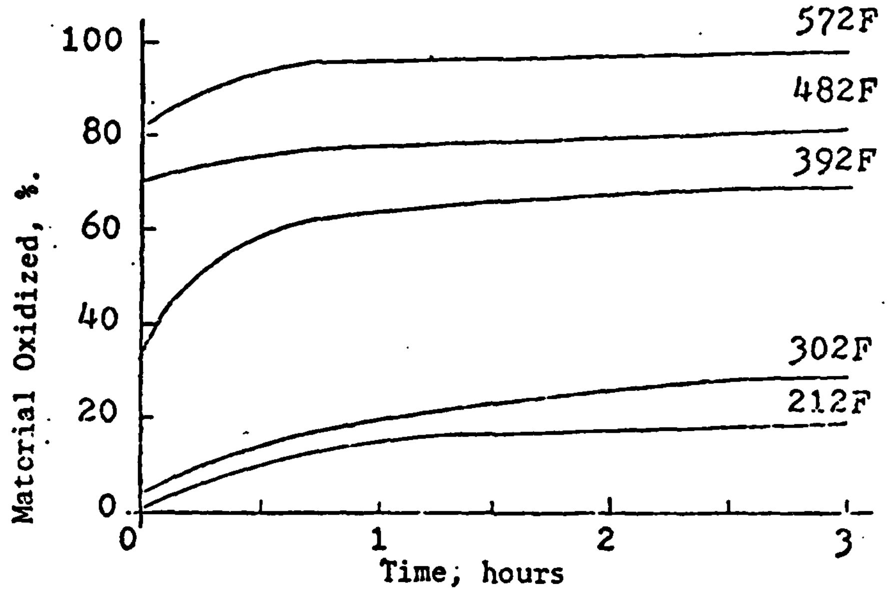

FIGURE 2.67-2. MATERIAL OXIDIZED VERSUS TIME AT VARIOUS TEMPERATURES.

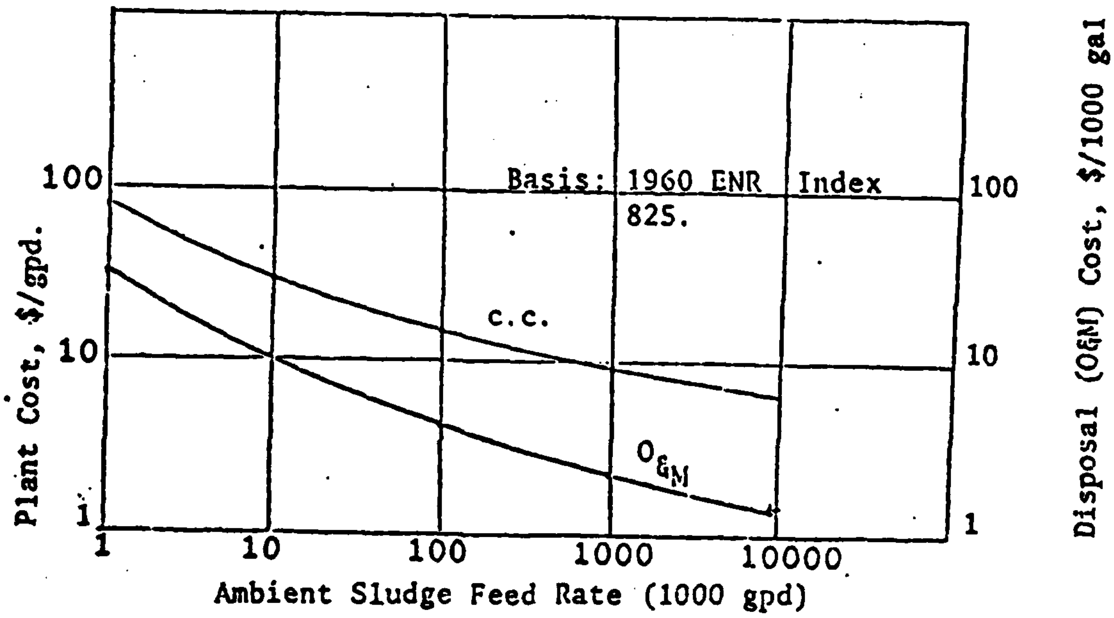

FIGURE 2.67-3. COST DATA FOR WET AIR OXIDATION

3.1 ACTIVATED SLUDGE

3.1.1 Background. There are a number of variations of the activated sludge process. However in the range of flows which are being addressed (0-.5 mgd) the costs for the various systems are very nearly the same. Also in this flow range activated sludge plants are generally package units with everything except the foundations and raw pumping stations furnished as a single unit. As a general rule the extended air activated sludge process is selected for small plants, because it is easier to operate than other modifications of the activated sludge process and most small plants can not obtain adequately skilled operators. The following calculations for sizing and costing will be based on the extended air activated sludge process.

3.1.2 Input Data.

3.1.2.1 Wastewater flow.

3.1.2.1.1 Average daily flow, Q_{avg}, mgd.

3.1.2.1.2 Peak flow, Qp, mgd.

3.1.2.2 Influent wastewater characteristics.

3.1.2.2.1 COD, mg/l.

3.1.2.2.3 Total suspended solids, TSS, mg/l.

3.1.2.2.4 Volatile suspended solids, VSS, mg/l.

3.1.2.2.5 Nonbiodegradable volatile suspended solids, SSi, mg/l.

3.1.2.2.6 pH.

3.1.2.2.7 Acidity and/or alkalinity, mg/l.

3.1.2.2.8 Nitrogen (kjeldahl or NH_3), mg/l.

3.1.2.2.9 Phosphorus (total and soluble), mg/l.

3.1.2.2.10 Oils and greases, mg/l.

3.1.2.2.11 Heavy metals, mg/l.

3.1.2.2.12 Toxic or special characteristics, mg/l.

3.1.2.2.13 Temperature, ^{o}F or ^{o}C.

3.1.2.3 Effluent quality requirements.

3.1.2.3.1 BOD_5, Fe, mg/l.

3.1.2.3.2 Suspended solids, SS, mg/l.

3.1.2.3.3 Total kjeldahl nitrogen, TKN, mg/l.

3.1.2.3.4 Phosphorus, P, mg/l.

3.1.3 Process Design Calculations.

3.1.3.1 McKinney's equations will be used for the design calculations. Assume the following design parameters.

3.1.3.1.1 Metabolism constant, K_m, use 15/hr at 20°C.

3.1.3.1.2 Synthesis factor, K_s, use 10.4/hr at 20°C.

3.1.3.1.3 Endogenous respiration factor, K_e, use 0.02/hr at 20°C.

3.1.3.1.4 Temperature correction coefficient, θ, use 1.0 to 1.03.

3.1.3.1.5 Hydraulic detention time, t, use 18 to 36 hours.

3.1.3.1.6 Solids retention time, t_s, use 20 to 30 days.

3.1.3.2 Adjust constants for temperature. Must adjust the metabolism constant, synthesis factor, and endogenous respiration factor from 20°C to operating temperature.

$$K_T = K_{20} \, (\theta)^{(T-20)}$$

where

K_T = rate constant at operating temperature.

K_{20} = rate constant at 20°C.

θ = temperature correction coefficient.

T = operating temperature, °C.

3.1.3.3 Determine aeration tank volume.

$$V = Q_{avg}\ (t/24)$$

where

V = volume of aeration tank, million gal.

Q_{avg} = average daily wastewater flow, mgd.

t = hydraulic detention time, hrs.

3.1.3.4 Calculate effluent soluble BOD_5.

$$Fe = \frac{Fi}{1 + K_m t}$$

If Fe 10 mg/l, assume a longer detention time (t) and recalculate Fe until Fe 10 mg/l.

where

Fe = soluble effluent BOD_5, mg/l.

Fi = influent BOD_5, mg/l.

K_m = metabolism constant at operating temperature, 1/hr.

t = hydraulic detention time, hrs.

3.1.3.5 Calculate MLSS concentration and check it to be sure it is between 3000 and 6000 mg/l.

3.1.3.5.1 Calculate (Ma) living active mass.

$$Ma = \frac{Ks\ Fe}{Ke + \frac{1}{24t_s}}$$

where

Ma = living active mass, mg/l.

K_s = synthesis factor, 1/hr.

K_e = endogenous respiration factor, 1/hr.

Fe = soluble effluent BOD_5, mg/l.

t_s = solids retention time, days, use 20 to 30 days.

3.1.3.5.2 Calculate (Me) endogenous mass.

$$M_e = 0.2\ K_e\ M_a\ t_s\ (24)$$

where

M_e = endogenous mass, mg/1.

K_e = endogenous respiration factor, 1/hr.

M_a = living active mass, mg/1.

t_s = solids retention time, days.

3.1.3.5.3 Calculate (M_i) inert nonbiodegradable organic mass.

$$M_i = SSi \times \frac{24\ t_s}{t}$$

where

M_i = inert nonbiodegradable organic mass, mg/1.

SSi = inert nonbiodegradable organic solids in influent, mg/1.

t_s = solids retention time, days.

t = hydraulic detention time, hrs.

3.1.3.5.4 Calculate (M_{ii}) inert inorganic suspended solids, mg/1.

$$M_{ii} = SS_{ii} \times \frac{24\ t_s}{t} + .1(M_a + M_e)$$

where

M_{ii} = inert inorganic suspended solids, mg/1.

SS_{ii} = inert inorganic suspended solids in the influent, mg/1.

t_s = solids retention time, days.

t = hydraulic detention time, hrs.

M_a = living active mass, mg/1.

M_e = endogenous mass, mg/1.

3.1.3.5.5 Calculate (MLSS) mixed liquor suspended solids.

$$MLSS = M_a + M_e + M_i + M_{ii}$$

where

$MLSS$ = mixed liquor suspended solids, mg/l.

M_a = living active mass, mg/l.

M_e = endogenous mass, mg/l.

M_i = inert nonbiodegradable organic mass, mg/l.

M_{ii} = inert inorganic suspended solids, mg/l.

3.1.3.5.6 Check organic loading (F/M).

$$F/M = \frac{24\ Fi}{(MLSS)(t)}$$

where

F/M = food to microorganism ratio.

F_i = influent BOD_5, mg/l.

MLSS = mixed liquor suspended solids, mg/l.

t = hydraulic detention time, hr.

If $F/M < 0.05$ reduce t and recalculate MLSS.

If $F/M \geq 0.15$ increase t and recalculate MLSS.

3.1.3.6 Calculate sludge production.

$$M_T = \frac{(MLSS)\ (V)\ (8.34)}{t_s}$$

where

M_T = sludge produced, lb/day.

MLSS = mixed liquor suspended solids, mg/l.

V = volume of aeration tank, million gal.

t_s = solids retention time, days.

3.1.3.7 Calculate pounds of solids produced per pounds of BOD removed.

$$PSPBOD = \frac{M_t}{(Q_{avg})\ (F_i - F_e)\ (8.34)}$$

where

where

PSPBOD = pounds of solids produced per pounds of BOD removed.

M_T = sludge produced, lb/day.

Q_{avg} = average daily flow, mgd.

F_i = influent BOD_5, mg/l.

F_e = soluble effluent BOD_5, mg/l.

8.34 = conversion factor.

3.1.3.8 Calculate sludge recycle ratio.

$$RR = \frac{MLSS}{M_u - MLSS}$$

where

RR = sludge recycle ratio (.75-1.50).

MLSS = mixed liquor suspended solids, mg/l.

M_u = solids concentration in return sludge (10,000 to 12,000 mg/l) mg/l.

3.1.3.9 Effluent Characteristics.

3.1.3.9.1 Calculate total effluent BOD_5.

$$BODE = F_e + (0.84)\ (SS_{eff})\ \left(\frac{M_a}{MLSS}\right)\ (0.76)$$

where

BODE = total effluent BOD_5, mg/l.

F_e = effluent soluble BOD_5, mg/l.

SS_{eff} = suspended solids in effluent, mg/l.

M_a = living active mass, mg/l.

MLSS = mixed liquor suspended solids, mg/l.

3.1.3.9.2 COD.

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ F_e$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent COD soluble concentration, mg/l.

F_e = effluent BOD_5 soluble concentration, mg/l.

3.1.3.9.3 Nitrogen.

The effluent Kjedahl nitrogen concentration is specified by the user.

NH3E = TKNE
NO3E = NO3
NO2E = 0.0

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

3.1.3.9.4 Oil and Grease.

OAGE = 0.0

where

OAGE = effluent oil and grease concentration, mg/l.

3.1.3.9.5 Settleable Solids.

SETSO = 0.0

where

SETSO = effluent settleable solids concentration, mg/l.

3.1.4 Process Design Output Data.

3.1.4.1 Volume of aeration tank, V, million gal.

3.1.4.2 Hydraulic detention time, t, hrs.

3.1.4.3 Effluent soluble BOD_5, F_e, mg/l.

3.1.4.4 Mixed liquor suspended solids, MLSS, mg/l.

3.1.4.5 Food to microorganism ratio, F/M.

3.1.4.6 Sludge production, M_T, lb/day.

3.1.4.7 Sludge recycle ratio, RR.

3.1.4.8 Total effluent BODE, BOD_5, mg/l.

3.1.5 Quantities Calculations.

3.1.5.1 Calculate size of package plant.

3.1.5.1.1 Determine size of package plant required from flow.

$$DIA = 126.4\ (Q_{avg})^{0.4943}$$

where

DIA = diameter of package plant, ft.

Q_{avg} = average daily flow, mgd.

3.1.5.1.2 Determine size of package plant required from aeration tank volume.

$$DIA = 114.4\ (V)^{0.4957}$$

where

DIA = diameter of package plant, ft.

V = aeration tank volume million gal.

3.1.5.1.3 Select the larger of the two diameters as the size of the package plant.

3.1.5.2 Determine quantity of concrete slab.

Assume:
The slab thickness is 9 inches.
The slab diameter is 3 ft greater than the unit diameter.

$$V_{cs} = (0.589)\ (DIA + 3)^2$$

where

V_{cs} = volume of R.C. slab required, cu ft.

DIA = diameter of package plant, ft.

3.1.5.3 Determine operation manhours required.

3.1.5.3.1 If $Q_{avg} \leq 0.1$ mgd

OMH = 1200

3.1.5.3.2 If $Q_{avg} > 0.1$ mgd

$$OMH = 1683 \ (Q_{avg})^{0.1469}$$

where

OMH = operation manhours required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.1.5.4 Determine maintenance manhours required.

3.1.5.4.1 If $Q_{avg} \leq 0.1$

$MMH = 640$

3.1.5.4.2 If $Q_{avg} > 0.1$

$$MMH = 1143 \ (Q_{avg})^{0.2519}$$

where

MMH = maintenance manhour requirements, MH/yr.

Q_{avg} = average daily flow, mgd.

3.1.5.5 Determine energy requirement.

$$KWH = 75{,}000 \ (Q_{avg})$$

where

KWH = energy required, Kwhr/yr.

Q_{avg} = average daily flow, mgd.

3.1.5.6 Operation and maintenance material and supply costs.

$$OMMP = 1.74 \ (Q_{avg})^{-0.2497}$$

where

$OMMP$ = O&M material and supply costs as percent installed package plant cost.

Q_{avg} = average daily flow, mgd.

3.1.5.7 Other construction cost items. Most of the cost associated with a package activated sludge plant have been accounted for. Some minor items such as painting, electrical, minor piping, etc., have not been identified and these costs would represent approximately 10% of the total cost.

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correction factor for other construction cost.

3.1.6 <u>Quantities Calculations Output Data</u>.

3.1.6.1 Diameter of package plant, DIA, ft.

3.1.6.2 Volume of R.C. slab required, V_{cs}, cu ft.

3.1.6.3 Operation manhours required, OMH, MH/yr.

3.1.6.4 Maintenance manhours required, MMH, MH/yr.

3.1.6.5 Energy required, KWH, Kwhr/yr.

3.1.6.6 O&M material and supply costs as percent installed package plant cost, OMMP, %.

3.1.6.7 Correction factor for other construction costs, CF.

3.1.7 <u>Unit Price Input Required</u>.

3.1.7.1 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

3.1.7.2 Cost of standard size package plant (100,000 gpd), COSTAS, $.

3.1.7.3 Current Marshall and Swift Equipment Cost Index, MSECI.

3.1.8 <u>Cost Calculations</u>.

3.1.8.1 Cost of concrete.

$$COSTCS = \frac{V_{cs}}{27} \; UPICS$$

where

COSTCS = cost of R.C. slab in-place, $.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab, \$/cu yd.

3.1.8.2 Cost of package plant installed.

$$COSTPP = (COSTAS) \frac{COSTRO}{100}$$

where

COSTPP = installed cost of package plant of diameter DIA, \$.

COSTAS = cost of standard size package plant (100,000 gpd), \$.

COSTRO = cost of package plant of diameter, DIA as percent of cost of standard unit.

3.1.8.3 Determine COSTRO.

$$COSTRO = 6.28\ (DIA)^{0.8316}$$

where

COSTRO = cost of package plant of diameter, DIA, as percent of cost of standard unit.

DIA = diameter of package plant, ft.

3.1.8.4 Determine COSTAS. COSTAS is the cost of the standard package extended aeration activated sludge plant. The standard unit is a package plant capable of treating 100,000 gpd of municipal sewage of average composition and strength. This would include all equipment except raw wastewater pumps. It does not include the slab or site preparation. These type units are nomally furnished to a general contractor who prepares the site and furnishs the slab.

$$COSTAS = \$36,500$$

For better cost estimation COSTAS should be obtained from an equipment vendor and treated as a unit price input. If this is not done the equipment cost will be adjusted for inflation using the Marshall and Swift Equipment Cost Index.

$$COSTAS = \$36,500\ \frac{MSECI}{491.6}$$

where

COSTAS = cost of standard size package plant (100,000 gpd), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for first quarter, 1977.

3.1.8.5 Determine total bare construction cost.

$$TBCC = (COSTCS + COSTPP)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTCS = cost of R.C. slab in-place, \$.

COSTPP = installed cost of package plant of diameter, DIA, \$.

CF = correction factor for other construction costs.

3.1.8.6 O&M material and supply costs.

$$OMMC = (COSTPP)\ \left(\frac{OMMP}{100}\right)$$

where

OMMC = O&M material and supply costs, \$/yr.

COSTPP = cost of package plant of diameter, DIA, \$.

OMMP = O&M material and supply costs as percent installed package plant cost, %.

3.1.9 Cost Calculations Output Data.

3.1.9.1 Total bare construction cost, TBCC, \$.

3.1.9.2 O&M material and supply cost, OMMC, \$/yr.

3.1.10 Bibliography.

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3.1.10.4 Metcalf and Eddy, Inc., Wastewater Engineering: Collection, Treatment and Disposal, McGraw Hill, New York, 1972.

3.1.10.5 Metcalf and Eddy, Inc., "Water Pollution Abatement Technology: Capabilities and Cost, Public Owned Treatment Works", 1975, PB-250690-03, NTIS, Springfield, Va.

3.1.10.6 Patterson and Bunker, "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", WPCR Series 17090 DAN 10/71, USEPA.

3.3 BAR SCREENS

3.3.1 Background. Bar screens are located at the head of treatment plants to remove large objects which may damage or clog pumps, valves, pipelines and other equipment within the treatment plant. Bar screens consist of vertical or inclined bars spaced at equal intervals across the channel where the wastewater flows. Bar screens may be manually or mechanically cleaned. Only the mechanically cleaned screens will be considered here, as most small systems do not usually have labor available for cleaning the screens.

The design of bar screens is based mainly on average and peak flows. The main consideration is pressure loss through the bar screens during peak flow periods when the largest quantity of debris would be expected.

3.3.2 Input Data.

3.3.2.1 Wastewater flow.

3.3.2.1.1 Average daily flow, Q_{avg}, mgd.

3.3.2.1.2 Maximum daily flow, Q_{max}, mgd.

3.3.2.1.3 Peak wet weather flow, Q_p, mgd.

3.3.3 Design Parameters.

3.3.3.1 Velocity through the bar screen, V_b, fps. (Use 2 fps for Q_{max} and 3 fps for Q_p).

3.3.3.2 Approach velocity, V, fps.

3.3.3.3 Maximum head loss through screen, h_e, ft (.5 ft).

3.3.3.4 Bar spacing, B_s, in (1 in).

3.3.3.5 Width of bar, B_w, in (5/16 in).

3.3.3.6 Bar shape factor, , (1.67) See Table 3.3-1.

3.3.3.7 Slope of bars, Θ, degrees. (10°) See Table 3.3-2.

3.3.4 Process Design Calculations.

3.3.4.1 Calculate area of bar screen required.

3.3.4.1.1 Based on maximum daily flow.

$$A_N = \frac{(Q_{max})\ (1.5470)}{2}$$

where

A_N = net flow area of screen, ft^2.

Q_{max} = maximum daily flow, mgd.

1.547 = conversion from mgd to cfs.

2 = maximum velocity through screen, fps.

3.3.4.1.2 Based on peak wet weather flow.

$$A_N = \frac{(Q_p)\ (1.547)}{3}$$

where

A_N = net flow area of screen, ft^2.

Q_p = peak wet weather flow.

Use the larger of the two net area.

3.3.4.2 Calculate total area of screen.

$$A_T = \frac{(A_N)\ (B_s + B_w)}{B_s}$$

where

A_T = total screen area, ft^2.

A_N = net screen area, ft^2.

B_s = bar spacing, inches.

B_w = width of bar, inches.

3.3.4.3 Calculate width of flow channel. The smallest mechanically cleaned bar screen available is 2 ft wide. In the flow range we are concerned with a 2 ft wide screen should handle all the flows.

$$W_c = 2 \text{ ft}$$

where

W_c = width of channel, ft.

3.3.4.4. Calculate the depth of water in channel.

$$D = \frac{A_T}{2}$$

where

D = water depth in the channel, ft.

A_T = total area of screen, ft.

2 = width of channel, ft.

3.3.4.5 Calculate the depth of channel. The channel shall have 2 ft freeboard above the water in the channel.

$$D_c = D + 2$$

where

D_c = depth of channel, ft.

D = water depth in the channel, ft.

3.3.4.6 Calculate head loss through screen.

$$h_e = \beta\left(\frac{B_w}{B_s}\right)^{1.33}\left(\frac{1.547\ Q_{avg}}{2D}\right)^2 \frac{\sin^2 \theta}{2g}$$

where

h_e = head loss through screen, ft.

β = bar shape factor

B_w = width of bar, inches.

B_s = bar spacing, inches.

Q_{avg} = average daily flow, mgd.

D = water depth in channel, ft.

θ = slope of bars, degrees.

3.3.5 Process Design Output Data.

3.3.5.1 Wastewater flow.

3.3.5.1.1 Average daily flow, Q_{avg}, mgd.

3.3.5.1.2 Maximum daily flow, Q_{max}, mgd.

3.3.5.1.3 Peak wet weather Q_p, mgd.

3.3.5.2 Width of channel, W_c, ft.

3.3.5.3 Depth of channel, D_c, ft.

3.3.5.4 Head loss through screen, ft.

3.3.5.5 Width of bars, B_w, inches.

3.3.5.6 Bar spacing, B_s, inches.

3.3.5.7 Bar shape factor, .

3.3.6 <u>Quantities Calculations</u>.

3.3.6.1 Determine length of channel. A uniform velocity distribution across the width of the channel is desirable for screening operation. To assure reasonably uniform velocity distribution the channel should extend a distance of five times the width of the channel upstream of the screen and two times the width downstream.

$$L_c = 7 W_c$$

where

L_c = length of channel, ft.

W_c = width of channel, ft.

3.3.6.2 Calculate volume of earthwork required. Assume as with other structures, it is 3 ft in ground. Assume 2 ft overexcavation.

$$V_{ew} = (W_c + 4)\ (L_c + 4)\ (D_c + 1)$$

where

V_{ew} = volume of earthwork required, cu ft.

W_c = width of channel, ft.

L_c = length of channel, ft.

D_c = depth of channel, ft.

3.3.6.3 Calculate volume of concrete required. Assume concrete is 9" thick.

3.3.6.3.1 Volume of reinforced concrete wall.

$$V_{cw} = [(L_c + 1.5)\ (2) + 2\ W_c]\ (D_c)\ (.75) + (W_c)\ (6)\ (.75)$$

where

V_{cw} = volume of R.C. wall required, cu ft.

L_c = length of channel, ft.

W_c = width of channel, ft.

D_c = depth of channel, ft.

3.3.6.3.2 Volume of reinforced concrete slab.

$$V_{cs} = (W_c + 1.5)\ (L_c + 1.5)$$

where

V_{cs} = volume of R.C. slab required, cu ft.

W_c = width of channel, ft.

L_c = length of channel, ft.

3.3.6.4 Calculate operation and maintenance manpower required.

3.3.6.4.1 Operation manpower required.

$$OMH = 46\ (Q_{avg})^{1.354}$$

where

OMH = operation manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.3.6.4.2 Maintenance manpower required.

$$MMH = 16.7\ (Q_{avg})^{0.372}$$

where

MMH = maintenance manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.3.6.5 Calculate electrical energy requirements.

$$\text{Log (KWH)} = 3.0803 + 0.1833\ (\text{Log}\ Q_{avg}) - 0.0467\ (\text{Log}\ Q_{avg})^2 + 0.0428\ (\text{Log}\ Q_{avg})^3$$

where

KWH = electrical energy required, kwhr/yr.

Q_{avg} = average daily flow, mgd.

3.3.6.6 O&M material and supply costs.

$$OMMP = 2\%$$

where

OMMP = O&M material costs as percent of total bare construction cost, %.

3.3.6.7 Other construction costs. The above calculations account for approximately 90% of the construction cost. The other 10% consists of miscellaneous electrical, painting, etc.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs.

3.3.7 <u>Quantities Calculations Output Data</u>.

3.3.7.1 Length of channel, L_c, ft.

3.3.7.2 Volume of earthwork required, V_{ew}, cu ft.

3.3.7.3 Volume of R.C. wall required, V_{cw}, cu ft.

3.3.7.4 Volume of R.C. slab required, V_{cs}, cu ft.

3.3.7.5 Operation manpower required, OMH, MH/yr.

3.3.7.6 Maintenance manpower required, MMH, MH/yr.

3.3.7.7 Electrical energy required, KWH, kwhr/yr.

3.3.7.8 O&M material and supply costs as percent total bare construction cost, OMMP, %.

3.3.7.9 Correction factor for other construction costs, CF.

3.3.8 <u>Unit Price Inputs Required</u>.

3.3.8.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.3.8.2 Unit Price input for R.C. wall in-place, UPICW, $/cu yd.

3.3.8.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

3.3.8.4 Purchase cost of bar screen, COSTBS, \$, (Optional).

3.3.8.5 Marshall and Swift Equipment Cost Index, MSECI.

3.3.9 Cost Calculations.

3.3.9.1 Calculate cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} (UPIEX)$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.3.9.2 Calculate cost of reinforced concrete.

3.3.9.2.1 Calculate cost of R.C. wall.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.3.9.2.2 Calculate cost of R.C. slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab, cu ft.

UPICS = unit price input for R.C. slab, \$/cu yd.

3.3.9.3 Calculate installed cost of bar screens. For flows of 500,000 gal and less a 2 ft wide bar screen is adequate. The purchase cost of a 2 ft wide mechanically cleaned bar screen in the first quarter of 1977 is \$10,350. For better cost estimation the cost of the bar screen should be obtained from a vendor and treated as a unit price input. If this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTBS = \$10,350 \frac{MSECI}{491.6}$$

where

COSTBS = purchase cost of bar screen, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index 1st quarter, 1977.

3.3.9.3.1 Installed cost of bar screen. The installation cost is about 20% of the equipment cost.

$$ICS = (1.2)\ (COSTBS)$$

where

ICS = installed cost of bar screen, \$.

COSTBS = purchase cost of bar screen, \$.

3.3.9.4 Calculate total bare construction cost.

$$TBCC = (ICS + COSTE + COSTCW + COSTCS)\ (CF)$$

where

TBCC = total bare construction cost, \$.

ICS = installed cost of bar screen, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

CF = correction factor for other construction costs.

3.3.9.5 Calculate O&M material and supply costs.

$$OMMC = TBCC \frac{OMMP}{100}$$

where

OMMC = O&M material and supply cost, \$/yr.

TBCC = total bare construction cost, \$.

OMMP = O&M material and supply costs as percent of total bare construction cost, %.

3.3.10 Cost Calculations Output Data.

3.3.10.1 Total bare construction cost, TBCC, \$.

3.3.10.2 O&M material and supply cost, OMMC, \$.

TABLE 3.3-1

BAR SHAPE FACTORS

Shape Factor	Description of Bar
2.42	Sharp-edged rectangular bars
1.83	Rectangular bars with semicircular upstream faces
1.79	Circular Bars
1.67	Rectangular bars with semicircular upstream and downstream faces

TABLE 3.3-2

GENERAL CHARACTERISTICS OF BAR SCREENS

Item	Dimensions
Bar, sizes	
Width, inches	1/4 to 5/8
Depth, inches	1 to 3
Spacing, inches	5/8 to 3
Slope from Vertical, deg	0 to 30
Approach velocity, fps	2 to 3
Allowable head loss, inches	6

3.3.11 Bibliography.

3.3.11.1 American Society of Civil Engineers and the Water Pollution Control Federation, "Sewage Treatment Plant Design", Manual of Practice No. 8, 1959, 1961, 1967, 1968, Water Pollution Control Federation, Washington, D.C.

3.3.11.2 Equipment Manufacturers' Catalogs.

3.3.11.3 Middlebrooks, E. Joe and Charlotte H. Middlebrooks, "Energy Requirements for Small Flow Wastewater Treatment Systems", Office of Chief of Engineers, Washington, D.C., April, 1979.

3.3.11.4 Metcalf and Eddy, Inc., Wastewater Engineering; Collection, Treatment, and Disposal, McGraw-Hill, New York, 1972.

3.5 CHLORINATION

3.5.1 Background. Chlorination is one of several processes used in the disinfection of wastewater. Some of the other methods of disinfection are use of other halogens, ozone, and ultraviolet radiation. While these other methods are sometimes used, chlorination is used almost exclusively in small plants.

Several factors influence the effectiveness of disinfection by chlorine. The chlorine dosage rate, contact time, presence of organic matter, pH, and temperature all have an affect upon the disinfection with chlorine.

3.5.2 Input Data.

3.5.2.1 Wastewater flow.

3.5.2.1.1 Average flow, Q_{avg}, mgd.

3.5.2.1.2 Peak flow, Q_p, mgd.

3.5.3 Design Parameters.

3.5.3.1 Contact time at peak flow, CT, min. (minimum of 15 min.).

3.5.3.2 Length to width ratio, RLW (40 to 1 to approach plug flow).

3.5.3.3 Number of tanks, N (usually 1 for this flow range).

3.5.3.4 Chlorine dosage, CD, mg/l (about 8 mg/l for average sewage).

3.5.4 Process Design Calculations.

3.5.4.1 Calculate contact tank volume. Select a contact time (CT) at peak flow (15 minutes minimum).

$$VCT = \frac{Q_p(CT) \times 10^6}{1440}$$

where

VCT = volume of contact tank, gal.

Q_p = peak flow, mgd.

CT = contact time at peak flow, min.

3.5.4.2 Calculate surface area of contact tank. Select a side water depth (SWD).

$$SA = \frac{(VCT)}{(7.48)(SWD)}$$

where

SA = surface area of contact tank, sq ft.

VCT = volume of contact tank, gal.

SWD = side water depth, ft. 8 ft.

3.5.4.3 Calculate tank dimensions. Select a length to width channel ratio (min. 40 to 1).

3.5.4.3.1 Calculate channel width.

$$CW = \left(\frac{SA}{RLW}\right)^{0.5}$$

where

CW = channel width, ft (CW is always 1 ft).

SA = surface area of contact tank, sq ft.

RLW = length to width channel ratio.

3.5.4.3.2. Calculate channel length.

$$CL = \frac{SA}{CW}$$

where

CL = channel length, ft.

SA = surface area of tank, sq ft.

CW = channel width, ft.

3.5.4.3.3 Calculate contact tank length and width. Assume tank will have 4 baffles with minimum distance between baffles of 1 ft. Also assume the baffles will be 6" thick concrete.

$$CTL = \frac{CL}{5} + CW$$

$$CTW = (CW + .5)\ 5$$

where

CTL = contact tank length, ft.

CL = channel length, ft.

CW = channel width, ft.

CTW = contact tank width, ft.

3.5.4.4 Calculate chlorine requirement. Select a chlorine dosage (See Table 3.5-1).

$$CR = (Q_{avg})\ (CD)\ (8.34)$$

where

CR = chlorine requirement, lb/day.

Q_{avg} = average flow, mgd.

CD = chlorine dosage, mg/l.

3.5.5 Process Design Output Data.

3.5.5.1 Average flow, Q_{avg}, mgd.

3.5.5.2 Peak flow, Q_p, mgd.

3.5.5.3 Contact time, CT, min.

3.5.5.4 Volume of contact tank, VCT, gal.

3.5.5.5 Chlorine requirement, CR, lb/day.

3.5.5.6 Contact tank length, CTL, ft.

3.5.5.7 Contact tank width, CTW, ft.

3.5.5.8 Side water depth, SWD, ft.

3.5.6 Quantities Calculations.

3.5.6.1 Calculate earthwork required. Assume tank is 4 ft in ground and sides of excavation are at 1 to 1 side slope.

$$V_{ew} = 4(CTL)(CTW) + 16\ CTW + 16\ CTL + 128$$

where

V_{ew} = volume of earthwork, cu ft.

CTL = contact tank length, ft.

CTW = contact tank width, ft.

3.5.6.2 Calculate quantity of concrete slab required.

$$V_{cs} = (CTL + 1)\ (CTW + 1)\ (.5)$$

where

V_{cs} = volume of R.C. slab required, cu ft.

CTL = contact tank length, ft.

CTW = contact tank width, ft.

3.5.6.3 Calculate quantity of concrete wall required.

$$V_{cw} = (6\ CTL + 2\ CTW)\ (.5)\ (SWD + 2)$$

where

V_{cw} = volume of R.C. wall required, cu ft.

CTL = contact tank length, ft.

CTW = contact tank width, ft.

SWD = side water depth, ft.

3.5.6.4 Calculate size and number of chlorine cylinders.

3.5.6.4.1 If chlorine requirement (CR) is less than 50 lb/day use 150 lb cylinders. Assume that a minimum of 30 days supply will be kept on hand. The minimum number of cylinders regardless of chlorine requirement, will be two 150 lb cylinders.

$$N_c = \frac{(CR)\ 30}{150}$$

If N_c is not an integer, use next largest integer.

where

N_c = number of cylinders required.

CR = chlorine requirement, lb/day.

3.5.6.4.2 If the chlorine requirement is greater than 50 lb/day, use 1-ton cylinders. The minimum number of cylinders is 2.

$$N_c = \frac{CR\ (30)}{2000}$$

where

N_c = number of cylinders.

CR = chlorine requirement, lb/day.

3.5.6.5 Calculate chlorination building area. The chlorination building size is determined by the anticipated equipment to be used for the various chlorine requirements. For flows less than 0.5 mgd only one chlorinater and one evaporator would be required. Therefore, the same size building would be used between flows of 0 to .5 mgd.

$$BC_c = 220 \text{ sq ft}$$

where

BA_c = chlorination building area, sq ft.

3.5.6.6 Calculate chlorine storage area.

3.5.6.6.1 If CR $\leq$ 50 lb/day then $A_s = 16 \times N_c$.

3.5.6.6.2 If CR $>$ 50 lb/day then $A_s = 140 \times N_c$.

where

A_s = area of chlorine storage building, sq ft.

N_c = number of chlorine cylinders required.

3.5.6.7 Calculate operational labor.

$$OMH = 40.48\ (CR)^{0.5316}$$

where

OMH = operational labor, MH/yr.

CR = chlorine requirement, lb/day.

3.5.6.8 Calculate maintenance labor.

3.5.6.8.1 If the chlorine requirement is between 0 and 55 lb/day,

$$MMH = 15.82\ (CR)^{0.3141}$$

3.5.6.8.2 If the chlorine requirement is greater than 55 lb/day,

$$MMH = 7.83\ (CR)^{0.4914}$$

where

MMH = maintenance labor, MH/yr.

CR = chlorine requirement, lb/day.

3.5.6.9 Calculate energy requirement for operation. In this low flow range the energy requirement is constant.

$$KWH = 118{,}000$$

where

KWH = energy requirement, kwhr/yr.

3.5.6.10 Calculate operation and maintenance material and supply costs.

$$OMMP = 6.255\ (CR)^{-0.0797}$$

where

$OMMP$ = O&M material and supply cost as percent of construction cost.

CR = chlorine requirement, lb/day.

3.5.6.11 Other construction cost items for chlorine contact chamber. From the calculations, approximately 85% of the cost of the chlorine contact chamber have been accounted for. Other minor costs such as influent and effluent piping and weirs, handrails, painting, etc. would account for the other 15%.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for construction of chlorine contact chamber.

3.5.7 Quantities Calculations Output Data

3.5.7.1 Chlorine requirement, CR, lb/day.

3.5.7.2 Chlorination building area, BA_c, sq ft.

3.5.7.3 Area of chlorine storage building, A_s, sq ft.

3.5.7.4 Volume of earthwork, V_{ew}, cu ft.

3.5.7.5 Volume of R.C. slab required, V_{cs}, cu ft.

3.5.7.6 Volume of R.C. wall required, V_{cw}, cu ft.

3.5.7.7 Correction factor for construction cost of chlorine contact chamber, CF.

3.5.7.8 Operation labor, OMH, MH/yr.

3.5.7.9 Maintenance labor, MMH, MH/yr.

3.5.7.10 Energy requirement for operation, KWH, kwhr/yr.

3.5.7.11 O&M material and supply costs, OMMP, %.

3.5.8 <u>Unit Price Input Required</u>.

3.5.8.1 Standard size chlorinator costs, COSTCLE, \$ (optional).

3.5.8.2 Unit price input building cost, UPIBC, \$/sq ft.

3.5.8.3 Unit price input for excavation, UPIEX, \$/cu yd.

3.5.8.4 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

3.5.8.5 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

3.5.8.6 Marshall and Swift Equipment Cost Index.

3.5.9 <u>Cost Calculations</u>.

3.5.9.1 Calculate cost of earthwork.

$$\mathrm{COSTE} = \frac{V_{ew}}{27}\,\mathrm{UPIEX}$$

where

COSTE = cost of excavation, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for excavation, \$/cu yd.

3.5.9.2 Cost of R.C. walls in-place.

$$\mathrm{COSTCW} = \frac{V_{cw}}{27}\,\mathrm{UPICW}$$

where

$COSTCW$ = cost of R.C. walls in-place, \$.

V_{cw} = volume of R.C. wall required, cu ft.

$UPICW$ = unit price input for R.C. wall, \$/cu yd.

3.5.9.3 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} \, UPICS$$

where

$COSTCS$ = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

$UPICS$ = unit price input for concrete slab, \$/cu yd.

3.5.9.4 Calculate cost for chlorine contact chamber.

$$COSTCC = (COSTE + COSTCS + COSTCW)\ (CF)$$

where

$COSTCC$ = cost of chlorine contact chamber, \$.

$COSTE$ = cost of excavation, \$.

$COSTCS$ = cost of R.C. slab in-place, \$.

$COSTCW$ = cost of R.C. wall in-place, \$.

CF = correction factor for construction cost of chlorine contact chamber.

3.5.9.5 Calculate cost of chlorination building.

$$COSTCB = BA_c \times UPIBC$$

where

$COSTCB$ = cost of chlorination building, \$.

BA_c = chlorination building area, sq ft.

$UPIBC$ = unit price input building cost, \$/sq ft.

3.5.9.6 Calculate cost of chlorine storage building.

$$COSTS = A_s \times (.5)\ (UPIBC)$$

where

COSTS = cost of chlorine storage building, \$.

A_s = area of chlorine storage building, sq ft.

UPIBC = unit price input building cost, \$/sq ft.

0.5 = factor to convert UPIBC to unit cost for storage building.

3.5.9.7 Calculate installed chlorination equipment cost.

3.5.9.7.1 The equipment purchase cost includes all equipment required such as chlorinator, evaporator, scales, leak detector, flow recorder, and booster pump. In this flow range residual analyzers are not required as chlorine is controlled based on flow.

3.5.9.7.2 If CR is between 0 lb/day and 50 lb/day.

$$COSTCE = 4.33 \times COSTCLE$$

3.5.9.7.3 If CR is greater than 50 lb/day.

$$COSTCE = 5.83 \times COSTCLE$$

where

COSTCE = purchase cost of chlorination equipment, \$.

COSTCLE = purchase cost of standard size chlorinator (2,000 lb/day).

3.5.9.7.4 Determine cost of standard size chlorinator (COSTCLE).

The approximate cost of a 2,000 lb/day chlorinator for the first quarter of 1977 is:

$$COSTCLE = \$2,700$$

For a better cost estimation, COSTCLE should be obtained from equipment vendor and treated as unit price input. If COSTCLE is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTCLE = \$2{,}700 \frac{MSECI}{491.6}$$

where

COSTCLE = cost of standard size chlorinator, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index first quarter 1977.

3.5.9.7.5 Calculate installation cost of chlorination equipment.

$$ICOST = (0.3)\ (COSTCE)$$

where

ICOST = installation cost of chlorination equipment, \$.

COSTCE = purchase cost of chlorination equipment, \$.

3.5.9.7.6 Calculate installed equipment cost.

$$IEC = COSTCE + ICOST$$

where

IEC = installed equipment costs, \$.

COSTCE = purchase cost of chlorination equipment, \$.

ICOST = installation cost of chlorination equipment, \$.

3.5.9.8 Calculate total bare construction cost.

$$TBCC = IEC + COSTCB + COSTS + COSTCC$$

where

TBCC = total bare construction cost, \$.

IEC = installed equipment cost, \$.

COSTCB = cost of chlorination building, \$.

COSTS = cost of chlorine storage building, \$.

COSTCC = cost of chlorine contact chamber, \$.

3.5.9.9 Calculate operation and maintenance material and supply cost.

$$OMMC = (TBCC) \frac{OMMP}{100}$$

where

OMMC = O&M material and supply cost, $.

TBCC = total bare construction cost, $.

OMMP = O&M material and supply costs as percent of construction cost, %.

3.5.10 Cost Calculations Output Data.

3.5.10.1 Total bare construction cost, TBCC, $.

3.5.10.2 O&M material and supply cost, OMMC, $.

TABLE 3.5-1

CHLORINE DOSAGE

Wastewater Source	Dosage Range (mg/l)
Untreated wastewater	6 to 25
Primary sedimentation	5 to 20
Chemical precipitation plant	2 to 6
Trickling filter plant	3 to 15
Activated sludge plant	2 to 8
Multimedia filter after activated sludge plant	1 to 5

3.5.11 Bibliography.

3.5.1.1 Keefer, C.E., Public Works, vol. 98, p.7.

3.5.11.2 Paterson and Banker, "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", EPA Report 17090 DAN, 10/71.

3.5.11.3 Metcalf and Eddy, Inc., "Water Pollution Abatement Technology: Capabilities and Cost, Public Owned Treatment Works", P.B.-250 690-01, Mar. 1976, NTIS, Springfield, Va.

3.5.11.4 White, George Clifford, Handbook of Chlorination, Van Nostrand Reinhold Company, New York, New York, 1972.

3.7 LAGOONS

3.7.1 Background. Lagoons have been extensively used for municipal and industrial waste where sufficient land area is available. Some of the reasons for the popularity of lagoons are operational stability with fluctuating loads, require relatively unskilled operators, low O&M costs, and low construction costs.

Six different types of lagoons will be addressed; aerated aerobic lagoons, aerated facultative lagoons, anaerobic lagoons, facultative lagoons, oxidation lagoons, and sludge lagoons.

3.7.2 General Description Aerated Aerobic Lagoons. The contents of an aerobic aerated lagoon must be completely mixed so that the incoming solids and the biological solids produced in the lagoon do not settle. Effluent quality is a function of detention time and will normally have a BOD ranging from one-third to one-half of the influent value. This BOD is due to the endogenous respiration of the biological solids escaping in the effluent. Before the effluent is discharged, the solids may be removed by settling.

3.7.3 General Description Aerated Facultative Lagoon.

3.7.3.1 The contents of this type of lagoon are not completely mixed. Thus, portions of the incoming solids and the biologically produced solids settle out and undergo anaerobic decomposition. As a result, the effluent from the facultative lagoon would contain higher soluble BOD concentration than from the aerobic one.

3.7.3.2 Algal growth is possible, due to the non-complete mixing. The contribution of effluent suspended solids can be very high, dependent on the season, temperature and mixing intensity in the lagoon.

3.7.4 General Description Anaerobic Lagoon. As the name suggests these lagoons are anaerobic throughout their depth except for a very shallow upper layer. These lagoons are constructed deep in order to insure anaerobic conditions and to conserve heat. Typically they are from 8 to 20 feet deep. Reductions of 70% of the influent BOD_5 are common with anaerobic lagoons and under ideal condition reductions of 85% are possible.

3.7.5 General Description Facultative Lagoon. In a facultative lagoon three zones exist; aerobic, facultative and anaerobic. The aerobic zone is near the surface and is like the aerobic or oxidation pond with aerobic bacteria and algae existing in a symbiotic relationship. The anaerobic zone is at the bottom of the lagoon where accumulated solids are decomposed by anaerobic bacteria. The facultative zone is an intermediate zone between the surface and bottom of the lagoon which is partly aerobic and partly anaerobic. Decomposition of the waste in this zone is accomplished by facultative bacteria. Normal depths for these lagoons is 3 to 8 feet.

3.7.6 General Description Oxidation Lagoon. The aerobic or oxidation lagoon is one in which aerobic bacteria and algae coexist in an aerobic environment. The oxygen required for reduction of the organic waste by the aerobic bacteria is supplied by algae production of oxygen through photosynthesis and atmospheric re-aertion. The pond depths are very shallow, usually not greater than 4 feet, because of the dependency of algae photosynthesis upon sunlight. Soluble BOD_5 removal is high, but this is misleading because the high concentration of algae in the effluent.

3.7.7 General Description Sludge Lagoons. Lagoons have been used extensively in small systems for the dewatering of sludge. Drying lagoons are similar to sandbeds in that they are both designed for the dewatered sludge to be removed periodically and the lagoon refilled. However they differ from sandbeds in that they use earthern levees and are built on the natural ground, therefore they are inherently cheaper to build.

Several factors must be considered in designing drying lagoons. The major factors include climate, subsoil permeability, lagoon depth, solids loading rates, and sludge characteristics.

3.7.8 Aerated Aerobic Lagoon.

3.7.8.1 Input Data.

3.7.8.1.1 Waste flow.

3.7.8.1.1.1 Average daily flow, mgd.

3.7.8.1.1.2 Peak (hourly) flow, mgd.

3.7.8.1.2 Wastewater characteristics.

3.7.8.1.2.1 BOD influent, mg/l.

3.7.8.1.2.2 Influent suspended solids, mg/l.

3.7.8.1.2.3 Influent volatile suspended solids, mg/l.

3.7.8.1.2.4 Nitrogen, mg/l as N.

3.7.8.1.2.5 Phosphorus, mg/l as P.

3.7.8.1.2.6 Non-biodegradable fraction of VSS.

3.7.8.1.3 Desired degree of treatment.

3.7.8.1.4 Temperature, °C (summer and winter).

3.7.8.2 <u>Design Parameters</u>.

3.7.8.2.1 Reaction rate constant, 0.0007-0.002 l/mg-hr.

3.7.8.2.2 MLSS, 200-500 mg/l.

3.7.8.2.3 MLVSS, 140-350 mg/l.

3.7.8.2.4 Fraction of BOD synthesized, 0.73.

3.7.8.2.5 Fraction of BOD oxidized for energy, 0.52.

3.7.8.2.6 Endogenous respiration rate per day, (b = 0.075/day, b' = 0.15/day).

3.7.8.2.7 Temperature coefficient, 1.035.

3.7.8.2.8 Hydraulic detention time, 2-4 days.

3.7.8.2.9 Depth, 6-12 ft.

3.7.8.3 <u>Process Design Calculations</u>.

3.7.8.3.1 Calculate volume of lagoon.

3.7.8.3.1.1 Adjust reaction rate constant for winter and summer temperatures.

$$K_T = K_{20}\,\theta^{(T-20)}$$

where

K_T = adjusted reaction rate constant, l/mg hr.

K_{20} = reaction rate constant at 20° C, l/mg hr.

θ = temperature coefficient, (1.035).

T = temperature, $^\circ$C.

3.7.8.3.1.2 Calculate detention time.

$$t = \frac{1}{24aK_TS_e - b}$$

where

t = detention time, days.

a = fraction of BOD synthesized.

K_T = adjusted reaction rate constant, l/mg hr.

S_e = effluent soluble BOD, mg/l.

b = endogenous respiration rate, 1/day.

Calculate the detention time based on summer temperature and winter temperature and desired treatment, then select the larger detention time.

3.7.8.3.1.3 Calculate lagoon volume.

$$V = Q_{avg}\ t$$

where

V = volume of lagoon, million gallons.

Q_{avg} = average daily flow, mgd.

t = detention time, days.

3.7.8.3.2 Calculate mixed liquor volatile suspended solids.

$$X_V = \frac{X_o + a\ (S_o - S_e)}{1 + bt}$$

where

X_V = mixed liquor volatile suspended solids, mg/l.

X_o = influent suspended solids, mg/l.

a = fraction of BOD synthesized.

S_o = influent BOD, mg/l.

S_e = effluent soluble BOD, mg/l.

b = endogenous respiration rate, 1/day.

t = detention time, days.

3.7.8.3.3 Calculate oxygen requirement.

$$O_2 = (a')\ (S_o - S_e)\ (Q_{avg})\ (8.34) + (b')\ (X_V)\ (V)\ (8.34)$$

where

O_2 = oxygen required, lb/day.

a' = fraction of BOD oxidized for energy (0.52).

S_o = influent BOD, mg/l.

S_e = effluent soluble BOD, mg/l.

Q_{avg} = average daily flow, mgd.

b' = endogenous respiration rate, 1/day (0.15/day).

X_V = mixed liquor volatile suspended solids, mg/l.

V = volume of lagoon, million gallons.

3.7.8.3.4 Design mechanical aeration system.

3.7.8.3.4.1 Assume the following design parameters.

3.7.8.3.4.1.1 Standard transfer efficiency, STE, lb/hp-hr $\approx$ 5.

3.7.8.3.4.1.2 O_2 transfer in waste/O_2 transfer in water, 0.9.

3.7.8.3.4.1.3 O_2 saturation in waste/O_2 saturation in water, 0.9.

3.7.8.3.4.1.4 Correction factor for pressure, P, $\approx$ 1.0.

3.7.8.3.4.1.5 O_2 saturation at selected summer temperature, $(C_s)_T$, mg/l.

3.7.8.3.4.1.6 Minimum dissolved oxygen to be maintained in the basin, 2.0 mg/l.

3.7.8.3.4.2 Adjust standard transfer efficiency to operating efficiency.

$$OTE = STE \frac{[(C_s)_T\ (\beta)\ (P) - C_L]}{9.17} \alpha\ (1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature, mg/l.

β = O_2 saturation in waste/O_2 saturation in water.

P = correction factor for pressure.

C_L = minimum dissolved oxygen to be maintained in basin, mg/l.

α = O_2 transfer in waste/O_2 transfer in water.

T = temperature, ^oC.

3.7.8.3.4.3 Calculate horsepower for aerators.

$$HP = \frac{(O_2)\ (1000)}{(OTE)\ (24)\ (V)\ (10^6)}$$

where

HP = aerator horsepower required per 1000 gallons, hp/1000 gallons.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of lagoon, million gal.

3.7.8.3.4.4 Check the calculated horsepower versus minimum required for complete mixing.

If HP 0.06 hp/1000 gallons set hp = 0.06 hp/1000 gal.

3.7.8.3.4.5 Calculate total horsepower required.

$$THP = \frac{(HP)\ (V)\ (10^6)}{1000}$$

where

THP = total aerator horsepower required, hp.

HP = aerator horsepower required per 1000 gal., hp/1000 gal.

3.7.8.3.5 Calculate nutrient requirements.

BOD: Nitrogen: Phosphorus = 100:5:1

3.7.8.3.6 Effluent Characteristics.

3.7.8.3.6.1 Calculate total BOD of the effluent.

$$BODE = S_e + 0.3\ X_V$$

where

BODE = total BOD of effluent, mg/l.

S_e = effluent soluble BOD_5, mg/l.

X_V = mixed liquor volatile suspended solids, mg/l.

3.7.8.3.6.2 Suspended Solids.

$$SSE = \frac{X_v}{.8}$$

where

SSE = effluent suspended solids concentration, mg/l.

X_v = mixed liquor volatile suspended solids, mg/l.

3.7.8.3.6.3 COD.

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration.

CODSE = effluent COD soluble concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.8.3.6.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = .25\ TKN$$
$$NO2E = 0.0$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia nitrogen concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

3.7.8.3.6.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

3.7.8.4 Process Design Output Data

3.7.8.4.1 Average daily flow, Q_{avg}, mgd.

3.7.8.4.2 Peak flow, Q_p, mgd.

3.7.8.4.3 Detention time, t, days.

3.7.8.4.4 Effluent soluble BOD, S_e, mg/l.

3.7.8.4.5 Volume of lagoon, V, million gal.

3.7.8.4.6 Oxygen required, O_2 lb/day.

3.7.8.4.7 Total aerator horsepower required, THP, hp.

3.7.8.4.8 Total BOD of effluent, BOD_{eff}, mg/l.

3.7.8.5 Quantities Calculations.

3.7.8.5.1 Calculate number of basins required, the Number of basins may be designated, if not, it will be selected from the following, based on flow.

Flow Range (Q_{avg})	Number of Basins (N)
Less than 0.5 mgd	1
From 0.5 to 2.0 mgd	2
From 2.0 to 5.0 mgd	3
Greater than 5.0 mgd	4

where

W_{avg} = average daily flow, mgd.

N = number of basins.

3.7.8.5.2 Select size and number of aerators.

3.7.8.5.2.1 Calculate aerator horsepower per basin.

$$HP_b = \frac{THP}{N}$$

where

HP_b = aerator horsepower required per basin, hp.

THP = total aerator horsepower required, hp.

N = number of basins.

3.7.8.5.2.2 Determine number of aerators per basin. The number of aerators per basin must be one of the following 2, 3, 4, 6, 8. Also the aerators must be one of the following sizes, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100 or 150. The selection process will be trial and error.

Assume number of aerator per basin (K) is 2. If $\frac{HP_b}{K} > 150$, go to next trials K = 3, 4, 6, 8 until $\frac{HP_b}{K} \leq 150$, then compare $\frac{HP_b}{K}$ with values for individual aerators (HP_a) given above. Select the smallest value of HP_a that is greater than $\frac{HP_b}{K}$. Compare $HP_a \times K$ with HP_b. If $HP_a \times K$ is larger than HP_b by 5% or more, go to next trial using the next larger K, until $HP_a \times K$ is within 5% of HP_b.

3.7.8.5.3 Calculate basin depth. The basin water depth (Dw) is controlled by the aerator sizes used. This is true because each size aerator has a maximum depth at which it can be used and still achieve mixing without the use of a draft tube.

3.7.8.5.3.1 If $HP_a < 100$ HP

$$DW = 4.82\ (HP_a)^{0.2467}$$

3.7.8.5.3.2 If $100 \leq HP_a \leq 150$

$$DW = 15 \text{ ft}$$

where

DW = basin water depth, ft.

HP_a = horsepower of individual aerators, hp.

3.7.8.5.4 Calculate basin dimensions.

3.7.8.5.4.1 Calculate length to width ratio.

$$\text{If } K \leq 4,\ r = K$$

$$\text{If } K > 4,\ r = k/2$$

where

K = number of aerators per basin.

r = basin length to width ratio.

3.7.8.5.4.2 Calculate the length of the basin at water level.

$$L_w = \frac{r\ 4/r\ A_a - 6/r\ (3DW)^2 + 2\ (3DW)^{2\ 0.5} + 3DW\ (1+r)}{2}$$

$$A_a = \frac{V \times 10^6}{(N)\ (DW)\ (7.48)}$$

where

L_w = length of basin at water level, ft.

r = length to width ratio.

s = side slope = 3 to 1 for all basins.

A_a = average lagoon surface area, ft^2.

DW = basin water depth, ft.

3.7.8.5.4.3 Calculate basin width at water level.

$$W_w = \frac{Lw}{r}$$

where

W_w = width of basin at water level, ft.

L_w = length of basin at water level, ft.

r = length to width ratio.

3.7.8.5.5 Calculate volume of earthwork required. The following assumptions were made concerning basin construction.

Basins will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
A 2 ft. freeboard will be used on all lagoons.
Common levee construction will be used where practical.

3.7.8.5.5.1 The volume of earthwork must be determined by trial and error. Assume a depth of cut of 1 ft.

3.7.8.5.5.1.1 Calculate the length and width at original ground level.

$$L_c = L_w - 6\ (DW - DC)$$

$$W_c = \frac{L_c}{r}$$

where

L_c = length of lagoon at original ground level, ft.

W_c = width of lagoon at original ground level, ft.

r = length to width ratio.

DW = basin water depth, ft.

DC = depth of cut, ft.

3.7.8.5.5.1.2 Calculate the volume of cut.

$$V_c = (1.3)(N)(DC)\left[W_cL_c - 3(DC)(W_c) - 3(DC)(L_c) + 12(DC)^2\right]$$

where

V_c = volume of cut, cu ft.

N = number of lagoons.

DC = depth of cut, ft.

W_c = width of lagoon at original ground level, ft.

L_c = length of lagoon at original ground level, ft.

3.7.8.5.5.1.3 Calculate length and width at top of levee.

$$L_T = L_W + 12$$

$$W_T = \frac{L_T}{r}$$

where

L_T = length of lagoon at top of levee, ft.

W_T = width of lagoon at top of levee, ft.

L_W = length of lagoon of water level, ft.

r = length to width ratio.

3.7.8.5.5.1.4 Calculate the depth of fill.

$$DF = DW + 2 - DC$$

where

DF = depth of fill, ft.

DW = basin water depth, ft.

DC = depth of cut, ft.

3.7.8.5.5.1.5 Calculate number of levees.

$$N_L = N + 1$$

$$N_w = 2\,N$$

where

N_L = number of levees of length, L_T.

N_w = number of levees of width, W_T.

N = number of lagoons.

3.7.8.5.5.1.6 Calculate volume of fill.

$$V_F = [10DF + 3(DF)^2]\,(L_T)\,(N_L) + (W_T)\,(N_w)$$

where

V_F = volume of fill, cu ft.

DF = depths of fill, ft.

L_T = length of lagoon at the top of the levee, ft.

W_T = width of the lagoon at the top of the levee, ft.

N_L = number of levees of length, L_T.

N_w = number of levees of width, W_T.

3.7.8.5.5.1.7 Compare V_c and V_f.

If $V_c < V_F$ then assume $DC > 1$ ft and recalculate V_c and V_F.

If $V_c > V_F$ then assume $DC < 1$ ft and recalculate V_c and V_F.

Repeat this procedure until $V_c = V_F$. This is the volume of earthwork required.

$$V_c = V_F = VLEW$$

where

DC = depth of cut, ft.

V_c = volume of cut, cu ft.

V_F = volume of fill, cu ft.

VLEW = volume of earthwork required, cu ft.

3.7.8.5.6 Calculate reinforced concrete requirement.

3.7.8.5.6.1 Influent structure. The influent structure would be a flow splitter box. The size of the structure would be determined by the number of basins, since it would have to accommodate weirs for each basin. The following quantities will be used.

$$N = 1, \text{ or } 2 \quad V_{cwi} = 81 \text{ cu ft}$$
$$33 \text{ cu ft}$$

$$N = 3 \quad V_{cwi} = 97 \text{ cu ft}$$
$$V_{csi} = 43 \text{ cu ft}$$

$$N = 4 \quad V_{cwi} = 135 \text{ cu ft}$$
$$V_{csi} = 66 \text{ cu ft}$$

where

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

N = number of basins.

3.7.8.5.6.2 Effluent structure. Each basin would have a separate effluent structure. The size of the structure would be approximately the same regardless of flow.

$$V_{cwe} = (81)\ (N)$$

$$V_{cse} = (32)\ (N)$$

where

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

N = number of basins.

3.7.8.5.6.3 Total reinforced concrete required.

$$V_{cw} = V_{cwi} + V_{cwe}$$

$$V_{cs} = V_{csi} + V_{cse}$$

where

V_{cw} = total volume of R.C. wall required, cu ft.

V_{cs} = total volume of R.C. slab required, cu ft.

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

3.7.8.5.7 Calculate volume of concrete required for embankment protection. In large lagoons and in aerated lagoons the earthern levees require protection from the wave action of the water. For this purpose concrete is placed around the interior of the levee at the water line. The concrete would extend from the top of the levee to about 1.5 ft below the water surface. Using a 3 to 1 side slope, the width of the slab would be 11 ft. The slab would be 8 inches thick.

$$V_{cep} = (2L_w + 2W_w)\ (11)\ (0.67)\ (N)$$

where

V_{cep} = volume of concrete for embankment protection, cu ft.

L_w = length of levee at water level, ft.

W_w = width of levee at water level, ft.

N = number of basins.

3.7.8.5.8 Calculate lagoon surface area.

$$A_s = \frac{(N)\ (Lw)\ (Ww)}{43560}$$

where

A_s = lagoon water surface area, acres.

N = number of basins.

Lw = length of levee at water surface, ft.

Ww = width of levee at water surface, ft.

3.7.8.5.9 Calculate operation manpower required.

3.7.8.5.9.1 If $A_S \leq 4$ acres, the operation manpower is calculated by:

$$OMH = 700\ (A_S)^{0.2571}$$

where

OMH = operation manpower required, MH/yr.

A_s = lagoon water surface area, acres.

3.7.8.5.9.2 If $A_S > 4$ acres, the operation manpower is calculated by:

$$OMH = 539.3\ (A_S)^{0.4453}$$

3.7.8.5.10 Calculate maintenance manpower required.

3.7.8.5.10.1 If $A_S \leq 4$ acres, the maintenance manpower is calculated by:

$$MMH = 130.0\ (A_S)^{0.2925}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

3.7.8.5.10.2 If $A_S > 4$ acres, the maintenance manpower is calculated by:

$$MMH = 105\ (A_S)^{0.4466}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

3.7.8.5.11 Calculate electrical energy required for operation. Assuming that all the aerators operate 90% of the time and the efficiency of the electric motors is 0.877.

3.7.8.5.11.1 Calculate total installed horsepower.

$$HP_T = (HP_a)\ (K)\ (N)$$

where

HP_a = horsepower of individual aerators, hp.

HP_T = total installed horsepower, hp.

K = number of aerators per basin.

N = number of basins.

3.7.8.5.11.2 Calculate electric energy required.

$$KWH = (HP_T)\ (365)\ (24)\ (.9)\ (.85)\ (.877)$$

where

KWH = electrical energy required, kwhr/yr.

HP_T = total installed horsepower, hp.

3.7.8.5.12 Calculate quantity for lagoon liner. In some areas the soils are such that lagoons must be lined to prevent loss of water. User should designated whether liner is required.

3.7.8.5.12.1 Calculate area of the bottom of the lagoon.

$$SA_B = (L_w - 6DW)\ (W_w - 6DW)$$

where

SA_B = area of the bottom of lagoon, ft^2.

L_w = length of basin at water level, ft.

W_w = width of basin at water level, ft.

DW = basin water depth, ft.

3.7.8.5.12.2 Calculate area of lagoon embankment.

$$SA_E = 2(L_T + W_T)\ \left(\frac{DW+2}{.3159} + 5\right)$$

where

SA_E = area of lagoon embankment, ft^2.

L_T = length of basin at top of levee, ft.

W_T = width of basin at top of levee, ft.

DW = basin water depth, ft.

3.7.8.5.12.3 Calculate total surface area required for liner.

$$SA_L = N\ (SA_B + SA_E)$$

where

SA_L = total area of lagoon liner, ft^2.

N = number of basins.

SA_B = area of bottom of lagoon, ft^2.

SA_E = area of lagoon embankment, ft^2.

3.7.8.5.13 Other operation and maintenance material and supply costs. This item includes such items as lubrication oil, paint, repair and replacement parts. These costs are estimated as a percent of the installed equipment cost.

$$OMMP = 4.93\ (HP_T)^{-0.1827}$$

where

OMMP = O&M maintenance and supply costs as percent of the installed equipment cost, %.

3.7.8.5.14 Other minor construction cost items. From the above calculations approximately 90% of the construction costs have been accounted for. Other minor items such as piping, grass seeding, etc., would be 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs items.

3.7.8.6 Quantities Calculations Output Data.

3.7.8.6.1 Number of basins, N.

3.7.8.6.2 Basin water depth, DW, ft.

3.7.8.6.3 Number of aerators per basin, K.

3.7.8.6.4 Basin length to width ratio, r.

3.7.8.6.5 Horsepower of individual aerators, HP_a, hp.

3.7.8.6.6 Length of basin at water level, L_w, ft.

3.7.8.6.7 Width of basin at water level, W_w, ft.

3.7.8.6.8 Volume of earthwork required, VLEW, cu ft.

3.7.8.6.9 Total volume of R.C. wall required, V_{cw}, cu ft.

3.7.8.6.10 Total volume of R.C. slab required, V_{cs}, cu ft.

3.7.8.6.11 Volume of concrete for embankment protection, V_{cep}, cu ft.

3.7.8.6.12 Operation manpower required, OMH, MH/yr.

3.7.8.6.13 Maintenance manpower required, MMH, MH/yr.

3.7.8.6.14 Electrical energy required, KWH, Kwhr/yr.

3.7.8.6.15 Total area of lagoon liner, SA_L, ft^2.

3.7.8.6.16 O&M maintenance and supply costs as percent of the installed equipment cost, OMMP, %.

3.7.8.6.17 Correction factor for other minor construction costs, CF.

3.7.8.7 <u>Unit Price Input Required</u>.

3.7.8.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.7.8.7.2 Unit price input for R.C. wall in-place, UPICW, $/cu yd.

3.7.8.7.3 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

3.7.8.7.4 Cost of standard size aerator (50 hp), COSTA, $, (optional).

3.7.8.7.5 Current Marshall and Swift Equipment Cost Index, MSECI.

3.7.8.7.6 Installation labor rate, LABRI, \$/MH.

3.7.8.7.7 Unit price input for lagoon liner, UPILL, \$/ft^2.

3.7.8.8 Cost Calculations.

3.7.8.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.7.8.8.2 Cost of reinforced concrete wall.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, cu yd.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.7.8.8.3 Cost of reinforced concrete slab.

$$COSTCS = \frac{V_{cs}}{27} UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

3.7.8.8.4 Cost of concrete embankment protection.

$$COSTEP = \frac{V_{cep}}{27} (0.5)(UPICS)$$

where

COSTEP = cost of concrete embankment protection in-place, \$.

V_{cep} = volume of concrete for embankment protection, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

0.5 = factor to adjust R.C. concrete unit price to nonreinforced.

3.7.8.8.5 Calculate purchase cost of aerators.

$$COSTA = \frac{(COSTSA)\ (COSTR)\ (K)\ (N)}{100}$$

where

COSTA = purchase cost of aerators, \$.

COSTSA = cost of standard size aerator (50 hp), \$.

COSTR = cost of aerator of horsepower HP_a as a percent of the cost of the standard size aerators, %.

K = number of aerators per basin.

N = number of basins.

3.7.8.8.5.1 Calculate COSTR.

If HP_a 25 hp; COSTR is calculated by:

$$COSTR = 20.7\ (HP_a)^{0.2686}$$

If HP_a 25 hp; COSTR is calculated by:

$$COSTR = 4.12\ (HP_a)^{0.7878}$$

3.7.8.8.5.2 Purchase cost of standard size aerator. The standard size aerator is a 50 hp, high-speed floating aerator. The cost of the 50 hp aerator in the first quarter of 1977 is:

$$COSTSA = \$13{,}960$$

For better cost estimation, COSTSA should be obtained from the equipment vendor and treated as a unit price input. However, if COSTA is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTA = \$13{,}960 \frac{MSECI}{491.6}$$

where

COSTA = cost of standard size aerator (50 hp), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

3.7.8.8.6 Calculate total installed equipment cost.

3.7.8.8.6.1 Calculate aerator installation labor.

$$IMH = 0.633\ (HP_a) + 40$$

where

IMH = aerator installation labor, MH.

HP_a = horsepower of individual aerators, hp.

3.7.8.8.6.2 Calculate aerator installation cost.

$$AIC = (IMH)\ (K)\ (N)\ (LABRI)$$

where

AIC = aerator installation cost, \$.

IMH = aerator installation labor, MH.

K = number of aerators per basin.

N = number of basins.

LABRI = installation labor rate, \$/MH.

3.7.8.8.6.3 Calculate installed cost for electrical/mechanical.

$$EMC = 0.589\ (HP_a)^{-0.1465}\ (COSTA)$$

where

EMC = installed cost for electrical/mechanical, \$.

HP_a = horsepower of individual aerators, hp.

COSTA = purchase cost of aerators, \$.

3.7.8.8.6.4 Calculate total installed equipment cost.

$$IEC = COSTA + AIC + EMC$$

where

IEC = total installed equipment cost, \$.

AIC = aerator installation cost, \$.

EMC = installed cost of electrical/mechanical, \$.

3.7.8.8.7 Calculate cost of lagoon liner.

$$COSTLL = (SA_L)\ (UPILL)$$

COSTLL = installed cost of lagoon liner, \$.

SA_L = total area of lagoon liner, ft^2.

UPILL = unit price input for lagoon liner, $\$/ft^2$.

3.7.8.8.8 Calculate total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + COSTEP + COSTLL + IEC)\ (CF)$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

COSTEP = cost of concrete embankment protection in-place, \$.

COSTLL = installed cost of lagoon liner, \$.

IEC = total installed equipment cost, \$.

3.7.8.8.9 Calculate O&M maintenance and supply cost.

$$OMMC = \frac{OMMP}{100} IEC$$

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as percent of installed equipment cost, %.

IEC = total installed equipment cost, $.

3.7.8.9 <u>Cost Calculations Output Data</u>.

3.7.8.9.1 Total bare construction cost, TBCC, $.

3.7.8.9.2 O&M material and supply costs, $/yr.

3.7.9 **Aerated Facultative Lagoon.**

3.7.9.1 **Input Data.**

3.7.9.1.1 Wastewater flow.

3.7.9.1.1.1 Average daily flow, mgd.

3.7.9.1.1.2 Peak (hourly) flow, mgd.

3.7.9.1.2 Wastewater characteristics.

3.7.9.1.2.1 BOD influent, mg/l.

3.7.9.1.2.2 Influent suspended solids, mg/l.

3.7.9.1.2.3 Influent volatile suspended solids, mg/l.

3.7.9.1.2.4 Nitrogen, mg/l as N.

3.7.9.1.2.5 Phosphorus, mg/l as P.

3.7.9.1.2.6 Non-biodegradable fraction of VSS.

3.7.9.1.3 Desired degree of treatment.

3.7.9.1.4 Temperature, ^{o}C (summer and winter).

3.7.9.2 **Design Parameters.**

3.7.9.2.1 Reaction rate constant/day (0.5-1.0, avg 0.75).

3.7.9.2.2 Temperature correction coefficient 1.075.

3.7.9.2.3 Fraction BOD removed for respiration (0.9-1.4).

3.7.9.2.4 BOD feedback from bottom or sediment (summer = 20 percent; winter = 5 percent).

3.7.9.2.5 MLVSS, mg/l, (50-150) average 100.

3.7.9.3 **Process Design Calculations.**

3.7.9.3.1 Select the rate constant, K. Adjust K for summer and winter temperatures.

$$K_T = K_{20}\theta^{(T-20)}$$

where

K_T = rate constant for desired temperature, ^{o}C.

K_{20} = rate constant at 20°C.

θ = temperature correction coefficient.

T = temperature, °C.

3.7.9.3.2 For summer and winter efficiencies, calculate detention times to meet winter efficiency.

$$\frac{S_e}{S_o} = \frac{1}{1 + Kt} \ (1.05)$$

where

S_e = effluent soluble BOD_5, mg/l.

S_o = influent BOD_5, mg/l.

K = reaction rate constant.

t = detention time, days.

and summer efficiency:

$$\frac{S_e}{S_o} = \frac{1}{1 + Kt} \quad (1.2)$$

Select the larger of the detention times.

3.7.9.3.3 Calculate volume.

$$V = Q_{avg}t$$

where

V = volume, million gal.

Q_{avg} = average daily flow, mgd.

t = detention time, days.

3.7.9.3.4 Determine oxygen requirements, assume a'.

$$O_2 = a'S_rQ(8.34)\ (1.2)$$

where

O_2 = oxygen required, lb/day.

a' = fraction of BOD oxidized for energy.

S_r = BOD removed, mg/l.

Q = flow, mgd.

3.7.9.3.5 Design mechanical aeration system and check horsepower supply to allow solids to settle: horsepower required to allow solids to settle (0.01-0.02 hp/1000 gal).

3.7.9.3.5.1 Assume the following design parameters.

3.7.9.3.5.1.1 Standard transfer efficiency, lb/hp-hr (O_2 dissolved oxygen, 20°C, and tap water).

3.7.9.3.5.1.2 O_2 transfer in waste/O_2 transfer in water $\approx$ 0.9.

3.7.9.3.5.1.3 O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

3.7.9.3.5.1.4 Correction factor for pressure $\approx$ 1.0.

3.7.9.3.5.2 Select summer operating temperature (25-30°C) and determine (from standard tables) O_2 saturation.

3.7.9.3.5.3 Adjust standard transfer efficiency to operating conditions.

$$OTE = STE \frac{(C_s)_T (\beta)(p) - C_L}{9.17} (\alpha)(1.02)^{T-20}$$

where

OTE = operating transfer efficiency, lb O_2/hp-hr.

STE = standard transfer efficiency, lb O_2/hp-hr.

$(C_s)_T$ = O_2 saturation at selected summer temperature, mg/l.

β = O_2 saturation in waste/O_2 saturation in water $\approx$ 0.9.

p = correction factor for pressure $\approx$ 1.0.

C_L = minimum dissolved oxygen to be maintained in the basin $\approx$ 2.0 mg/l.

$= O_2$ transfer in waste/O_2 transfer in water 0.9.

T = temperature, ^oC.

3.7.9.3.5.4 Calculate horsepower requirement.

$$HP = \frac{O_2}{OTE \frac{lb\ O_2}{hp\text{-}hr} (24)(V)(10^6)} \times 1000$$

where

HP = horsepower required per 1000 gal.

O_2 = oxygen required, lb/day.

OTE = operating transfer efficiency, lb O_2/hp-hr.

V = volume of the basin, million gal.

3.7.9.3.5.5 Calculate total horsepower requirement.

$$THP = \frac{(HP)(V)(10^6)}{1000}$$

where

THP = total horsepower required, hp.

V = volume of lagoon, million gal.

HP = horsepower required per 1000 gal, hp/1000 gal.

3.7.9.3.6 Effluent Characteristics.

3.7.9.3.6.1 Effluent suspended solids concentration. The suspended solids from an aerated facultative lagoon can be divided into three categories; inert solids, bacteria cells and algal cells. The growth of algae in sewage lagoons is a very complex process. Geographic location, temperature, season, organic loading, nutrient concentration and light penetration are the factors that govern the ecological system in a sewage lagoon. However, White and Rich have presented the following which relates the algal concentration with mixing level in aerated lagoon systems.

$$SS_{AL} = \frac{0.407}{HP} - 10.7$$

where

SS_{AL} = algal concentration of suspended solids, mg/l.

HP = horsepower required/1000 gal.

The suspended solids due to inert material and bacteria cells is more or less independent of the mixing level. It is usually in the range of 25 to 30 mg/l and approximately 85% volatile.

$$SS_{oi} = 25 \text{ mg/l}$$

Thus, the total suspended solids concentration would be

$$(SS)_{eff} = SS_{AL} + SS_{oi}$$

where

$(SS)_{eff}$ = total suspended solids in effluent, mg/l.

SS_{AL} = algal concentration of suspended solids, mg/l.

SS_{oi} = suspended solids due to inert material and bacteria cells, mg/l.

3.7.9.3.6.2 Calculate BOD in the final effluent.

$$BODE = Se + 0.3\ (0.85 \times SS_{oi}) + 0.12\ (SS_{AL})$$

where

BODE = BOD in effluent in mg/l.

Se = soluble effluent BOD, mg/l.

0.12 = BOD concentration contributed by algae in the effluent.

3.7.9.3.6.3 COD.

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.9.3.6.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

3.7.9.3.6.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

3.7.9.3.6.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

3.7.9.3.7 Detemine nutrient requirements.

BOD:Nitrogen:Phosphorus = 100:5:1

3.7.9.4 <u>Process Design Output Data.</u>

3.7.9.4.1 Average daily flow, Q_{avg}, mgd.

3.7.9.4.2 Peak flow, Q_p, mgd.

3.7.9.4.3 Detention time, t, days.

3.7.9.4.4 Effluent soluble BOD, S_e, mg/l.

3.7.9.4.5 Volume of lagoon, V, million gal.

3.7.9.4.6 Oxygen required, O_2, lb/day.

3.7.9.4.7 Total aerator horsepower required, THP, hp.

3.7.9.4.8 Total BOD of effluent, BOD_{eff}, mg/l.

3.7.9.5 Quantities Calculations.

3.7.9.5.1 Calculate number of basins required. The number of basins may be designated, if not, it will be selected from the following, based on flow.

Flow Range (Q_{avg})	Number of Basins (N)
Less than 0.5 mgd	1
From 0.5 to 2.0 mgd	2
From 2.0 to 5.0 mgd	3
Greater than 5.0 mgd	4

where

Q_{avg} = average daily flow, mgd.

N = number of basins.

3.7.9.5.2 Select size and number of aerators.

3.7.9.5.2.1 Calculate aerator horsepower per basin.

$$HP_b = \frac{THP}{N}$$

where

HP_b = aerator horsepower required per basin, hp.

THP = total aerator horsepower required, hp.

N = number of basins.

3.7.9.5.2.2 Determine number of aerators per basin. The number of aerators per basin must be one of the following 2, 3, 4, 6, 8. Also the aerators must be one of the following sizes, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100, or 150. The selection process will be trial and error.

Assume number of aerator per basin (K) is 2. If $\frac{HP_b}{K}$ 150, go to next trials K=3, 4, 6, 8 until $\frac{HP_b}{K}$ 150, then compare $\frac{HP_b}{K}$ with values for individual aerators (HP_a) given above. Select the smallest value of HP_a that is greater than $\frac{HP_b}{K}$. Compare HP_a x K with HP_b. If HP_a x K is larger than HP_b by 5% or more, go to next trial using the next larger K, until HP_a x K is within 5% of HP_b.

3.7.9.5.3 Calculate basin depth. The basin water depth (Dw) is controlled by the aerator sizes used. This is true because each size aerator has a maximum depth at which it can be used and still achieve mixing without the use of a draft tube.

3.7.9.5.3.1 If HP_a 100 HP

$$DW = 4.82\ (HP_a)^{0.2467}$$

3.7.9.5.3.2 If 100 HP_a 150

$$DW = 15\ \text{ft}$$

where

DW = basin water depth, ft.

HP_a = horsepower of individual aerators, hp.

3.7.9.5.4 Calculate basin dimensions.

3.7.9.5.4.1 Calculate length to width ratio.

If K 4, r = K

If K 4, r = K/2

where

K = number of aerators per basin.

r = basin length to width ratio.

3.7.9.5.4.2 Calculate the length of the basin at water level.

$$L_w = \frac{r\ 4/r\ A_a - 6/r\ (3DW)^2 + 2\ (3DW)^2\ \ {}^{0.5} + 3DW\ (1+r)}{2}$$

$$A_a = \frac{V \times 10^6}{(N)\ (DW)\ (7.48)}$$

where

L_w = length of basin at water level, ft.

r = length to width ratio.

s = side slope = 3 to 1 for all basins.

A_a = average lagoon surface area, ft^2.

DW = basin water depth, ft.

3.7.9.5.4.3 Calculate basin width at water level.

$$W_w = \frac{LW}{r}$$

where

W_w = width of basin at water level, ft.

L_w = length of basin at water level, ft.

r = length to width ratio.

3.7.9.5.5 Calculate volume of earthwork required. The following assumptions were made concerning basin construction.

Basins will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
A 2 ft freeboard will be used on all lagoons.
Common levee construction will be used where practical.

3.7.9.5.5.1 The volume of earthwork must be determined by trial and error. Assume a depth of cut of 1 ft.

3.7.9.5.5.1.1 Calculate the length and width at original ground level.

$$L_c = L_w - 6\ (DW - DC)$$

$$W_c = \frac{L_c}{r}$$

where

L_c = length of lagoon at original ground level, ft.

W_c = width of lagoon at original ground level, ft.

r = length to width ratio.

DW = basin water depth, ft.

DC = depth of cut, ft.

3.7.9.5.5.1.2 Calculate the volume of cut.

$$V_c = (1.3)(N)(DC)\ W_cL_c - 3(DC)(W_c) - 3(DC)(L_c) + 12(DC)^2$$

where

V_c = volume of cut, cu ft.

N = number of lagoons.

DC = depth of cut, ft.

W_c = width of lagoon at original ground level, ft.

L_c = length of lagoon at original ground level, ft.

3.7.9.5.5.1.3 Calculate length and width at top of levee.

$$L_T = L_w + 12$$

$$W_T = \frac{L_T}{r}$$

where

L_T = length of lagoon at top of levee, ft.

W_T = width of lagoon at top of levee, ft.

L_w = length of lagoon at water level, ft.

r = length to width ratio.

3.7.9.5.5.1.4 Calculate the depth of fill.

$$DF = DW + 2 - DC$$

where

DF = depth of fill, ft.

DW = basin water depth, ft.

DC = depth of cut, ft.

3.7.9.5.5.1.5 Calculate number of levees.

$$N_L = N + 1$$

$$N_w = 2\ N$$

where

N_L = number of levees of length, L_T.

N_W = number of levees of width, W_T.

N = number of lagoons.

3.7.9.5.5.1.6 Calculate volume of fill.

$$V_F = [10DF + 3(DF)^2] (L_T) (N_L) + (W_T) (N_W)$$

where

V_F = volume of fill, cu ft.

DF = depths of fill, ft.

L_T = length of lagoon at the top of the levee, ft.

W_T = width of the lagoon at the top of the levee, ft.

N_L = number of levees of length, L_T.

N_W = number of levees of width, W_T.

3.7.9.5.5.1.7 Compare V_c and V_f.

If $V_c < V_F$ then assume DC > 1 ft and recalculate V_c and V_F.

If $V_c > V_F$ then assume DC < 1 ft and recalculate V_c and V_F.

Repeat this procedure until $V_c = V_F$. This is the volume of earthwork required.

$$V_c = V_F = VLEW$$

where

DC = depth of cut. ft.

V_c = volume of cut, cu ft.

V_F = volume of fill, cu ft.

VLEW = volume of earthwork required, cu ft.

3.7.9.5.6 Calculate reinforced concrete requirement.

3.7.9.5.6.1 Influent structure. The influent structure would be a flow splitter box. The size of the structure would be determined by the number of basins, since it would have to accommodate weirs for each basin. The following quantities will be used.

$$N = 1, \text{ or } 2 \quad V_{cwi} = \begin{matrix} 81 \text{ cu ft.} \\ 33 \text{ cu ft.} \end{matrix}$$

$$N = 3 \qquad V_{cwi} = 97 \text{ cu ft.}$$
$$V_{csi} = 43 \text{ cu ft.}$$
$$N = 4 \qquad V_{cwi} = 135 \text{ cu ft.}$$
$$V_{csi} = 66 \text{ cu ft.}$$

where

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

N = number of basins.

3.7.9.5.6.2 Effluent Structure. Each basin would have a separate effluent structure. The size of the structure would be approximately the same regardless of flow.

$$V_{cwe} = (81)\ (N)$$

$$V_{cse} = (32)\ (N)$$

where

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

N = number of basins.

3.7.9.5.6.3 Total reinforced concrete required.

$$V_{cw} = V_{cwi} + V_{cwe}$$

$$V_{cs} = V_{csi} + V_{cse}$$

where

V_{cw} = total volume of R.C. wall required, cu ft.

V_{cs} = total volume of R.C. slab required, cu ft.

V_{cwi} = volume of R.C. wall required for influent structure, cu ft.

V_{cwe} = volume of R.C. wall required for effluent structure, cu ft.

V_{csi} = volume of R.C. slab required for influent structure, cu ft.

V_{cse} = volume of R.C. slab required for effluent structure, cu ft.

3.7.9.5.7. Calculate volume of concrete required for embankment protection. In large lagoons and in aerated lagoons the earthern levees require protection from the wave action of the water. For this purpose concrete is placed around the interior of the levee at the water line. The concrete would extend from the top of the levee to about 1.5 ft. below the water surface. Using a 3 to 1 side slope, the width of the slab would be 11 ft. The slab would be 8 inches thick.

$$V_{cep} = (2L_w + 2W_w)(11)(0.67)(N)$$

where

V_{cep} = volume of concrete for embankment protection, cu ft.

L_w = length of levee at water level, ft.

W_w = width of levee at water level, ft.

N = number of basins.

3.7.9.5.8 Calculate lagoon surface area.

$$A_s = \frac{(N)(Lw)(Ww)}{43560}$$

where

A_s = lagoon water surface area, acres.

N = number of basins.

Lw = length of levee at water surface, ft.

Ww = width of levee at water surface, ft.

3.7.9.5.9 Calculate operation manpower required.

3.7.9.5.9.1 If $A_S \leq 4$ acres, the operation manpower is calculated by:

$$OMH = 700\ (A_S)^{0.2571}$$

where

OMH = operation manpower required, MH/yr.

A_S = lagoon water surface area, acres.

2.4.9.5.9.2 If $A_S > 4$ acres, the operation manpower is calculated by:

$$OMH = 539.3 \ (A_S)^{0.4453}$$

3.7.9.5.10 Calculate maintenance manpower required.

3.7.9.5.10.1 If $A_S \leq 4$ acres, the maintenance manpower is calculated by:

$$MMH = 130.0 \ (A_S)^{0.2925}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

3.7.9.5.10.2 If $A_S > 4$ acres, the maintenance manpower is calculated by:

$$MMH = 105 \ (A_S)^{0.4466}$$

where

MMH = maintenance manpower required, MH/yr.

A_S = lagoon water surface area, acres.

3.7.9.5.11 Calculate electrical energy required for operation. Assuming that all the aerators operate 90% of the time and the efficiency of the electric motors is 0.877, the power required would be.

3.7.9.5.5.11.1 Calculate total installed horsepower.

$$HP_T = (HP_a) \ (K) \ (N)$$

where

HP_a = horsepower of individual aerators, hp.

HP_T = total installed horsepower, hp.

K = number of aerators per basin.

N = number of basins.

3.7.9.5.11.2 Calculate electric energy required.

$$KWH = (HP_T)\ (365)\ (24)\ (.9)\ (.85)\ (.877)$$

where

KWH = electrical energy required, kwhr/yr.

HP_T = total installed horsepower, hp.

3.7.9.5.12 Calculate quantity for lagoon liner. In some areas the soils are such that lagoons must be lined to prevent loss of water. User should designate whether liner is required.

3.7.9.5.12.1 Calculate area of the bottom of the lagoon.

$$SA_B = (L_w - 6Dw)\ (W_w - 6Dw)$$

where

SA_B = area of the bottom of lagoon, ft^2.

L_w = length of basin at water level, ft.

W_w = width of basin at water level, ft.

DW = basin water depth, ft.

3.7.9.5.12.2 Calculate area of lagoon embankment.

$$SA_E = 2(L_T + W_T)\ \left(\frac{DW+2}{.3159} + 5\right)$$

where

SA_E = area of lagoon embankment, ft^2.

L_T = length of basin at top of levee, ft.

W_T = width of basin at top of levee, ft.

DW = basin water depth, ft.

3.7.9.5.12.3 Calculate total surface area required for liner.

$$SA_L = N\ (SA_B + SA_E)$$

where

SA_L = total area of lagoon liner, ft^2.

N = number of basins.

SA_B = area of bottom of lagoon, ft^2.

SA_E = area of lagoon embankment, ft^2.

3.7.9.5.13 Other operation and maintenance material and supply costs. This item includes such items as lubrication oil, paint, repair and replacement parts. These costs are estimated as a percent of the installed equipment cost.

$$OMMP = 4.93\ (HP_T)^{-0.1827}$$

where

OMMP = O&M maintenance and supply costs as percent of the installed equipment cost, %.

3.7.9.5.14 Other minor construction cost items. From the above calculations approximately 90% of the construction costs have been accounted for. Other minor items such as piping, grass seeding, etc., would be 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs items.

3.7.9.6 <u>Quantities Calculations Output Data</u>

3.7.9.6.1 Number of basins, N.

3.7.9.6.2 Basin water depth, DW, ft.

3.7.9.6.3 Number of aerators per basin, K.

3.7.9.6.4 Basin length to width ratio, r.

3.7.9.6.5 Horsepower of individual aerators, HP_a, hp.

3.7.9.6.6. Length of basin at water level, L_w, ft.

3.7.9.6.7 Width of basin at water level, W_w, ft.

3.7.9.6.8 volume of earthwork required, VLEW, cu ft.

3.7.9.6.9 Total volume of R.C. wall required, V_{cw}, cu ft.

3.7.9.6.10 Total volume of R.C. slab required, V_{cs}, cu ft.

3.7.9.6.11 Volume of concrete for embankment protection, V_{cep}, cu ft.

3.7.9.6.12 Operation manpower required, OMH, MH/yr.

3.7.9.6.13 Maintenance manpower required, MMH, MH/yr.

3.7.9.6.14 Electrical energy required, KWH, Kwhr/yr.

3.7.9.6.15 Total area of lagoon liner, SA_L, $ft.^2$.

3.7.9.6.16 O&M maintenance and supply costs as percent of the installed equipment cost, OMMP, %.

3.7.9.6.17 Correction factor for other minor construction costs, CF.

3.7.9.7 Unit Price Input Required.

3.7.9.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.7.9.7.2 Unit price input for R.C. slab in-place, UPICW, $/cu yd.

3.7.9.7.3 Unit price input for R.C. slab in-place UPICS, $/cu yd.

3.7.9.7.4 Cost of standard size aerator (50 hp), COSTA, $, (optional).

3.7.9.7.5 Current Marshall and Swift Equipment Cost Index, MSECI.

3.7.9.7.6 Installation labor rate, LABRI, $/MH.

3.7.9.7.7 Unit price input for lagoon liner, UPILL, $\$/ft^2$.

3.7.9.8 Cost Calculations.

3.7.9.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, $.

VLEW = volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.7.9.8.2 Cost of reinforced concrete wall.

$$COSTCW = \frac{V_{cw}}{27} \; UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, cu yd.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.7.9.8.3 Cost of reinforced concrete slab.

$$COSTCS = \frac{V_{cs}}{27} \; UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

3.7.9.8.4 Cost of concrete embankment protection.

$$COSTEP = \frac{V_{cep}}{27} \; (0.5)(UPICS)$$

where

COSTEP = cost of concrete embankment protection in-place, \$.

V_{cep} = volume of concrete for embankment protection, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

0.5 = factor to adjust R.C. concrete unit price to non reinforced.

3.7.9.8.5 Calculate purchase cost of aerators.

$$COSTA = \frac{(COSTSA)\ (COSTR)\ (K)\ (N)}{100}$$

where

COSTA = purchase cost of aerators, \$.

COSTSA = cost of standard size aerator (50 hp), \$.

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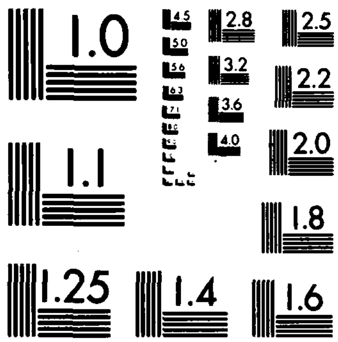

MICROCOPY RESOLUTION TEST CHART
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COSTR = cost of aerator of horsepower HP_a as a percent of the cost of the standard size aerators, %.

K = number of aerators per basin.

N = number of basins.

3.7.9.8.5.1 Calculate COSTR.

If HP_a 25 hp; COSTR is calculated by:

$$COSTR = 20.7\ (HP_a)^{0.2686}$$

If HP_a 25 hp; COSTR is calculated by:

$$COSTR = 4.12\ (HP_a)^{0.7878}$$

3.7.9.8.5.2 Purchase cost of standard size aerator. The standard size aerator is a 50 hp, high-speed floating aerator. The cost of the 50 hp aerator in the first quarter of 1977 is:

$$COSTSA = \$13,960$$

For better cost estimation, COSTSA should be obtained from the equipment vendor and treated as a unit price input. However, if COSTA is not treated as a unit price input, the cost will be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTA = \$13,960\ \frac{MSECI}{491.6}$$

where

COSTA = cost of standard size aerator (50 hp), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

3.7.9.8.6 Calculate total installed equipment cost.

3.7.9.8.6.1 Calculate aerator installation labor.

$$IMH = 0.633\ (HP_a) + 40$$

where

IMH = aerator installation labor, MH.

HP_a = horsepower of individual aerators, hp.

3.7.9.8.6.2 Calculate aerator installation cost.

$$AIC = (IMH)\ (K)\ (N)\ (LABRI)$$

where

AIC = aerator installation cost, $.

IMH = aerator installation labor, MH.

K = number of aerators per basin.

N = number of basins.

LABRI = installation labor rate, $/MH.

3.7.9.8.6.3 Calculate installed cost for electrical/mechanical.

$$EMC = 0.589\ (HP_a)^{-0.1465}\ (COSTA)$$

where

EMC = installed cost for electrical/mechanical, $.

HP_a = horsepower of individual aerators, hp.

COSTA = purchase cost of aerators, $.

3.7.9.8.6.4 Calculate total installed equipment cost.

$$IEC = COSTA + AIC + EMC$$

where

IEC = total installed equipment cost, $.

AIC = aerator installation cost, $.

EMC = installed cost of electrical/mechanical, $.

3.7.9.8.7 Calculate cost of lagoon liner.

$$COSTLL = (SA_L)\ (UPILL)$$

COSTLL = installed cost of lagoon liner, $.

SA_L = total area of lagoon liner, ft^2.

UPILL = unit price input for lagoon liner, $\$/ft^2$.

3.7.9.8.8 Calculate total bare construction cost.

TBCC = (COSTE + COSTCW + COSTCS + COSTEP + COSTLL + IEC) (CF)

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

COSTEP = cost of concrete embankment protection in-place, \$.

COSTLL = installed cost of lagoon liner, \$.

IEC = total installed equipment cost, \$.

3.7.9.8.9 Calculate O&M maintenance and supply cost.

$$OMMC = \frac{OMMP}{100} IEC$$

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as percent of installed equipment cost, %.

IEC = total installed equipment cost, \$.

3.7.9.9 Cost Calculations Output Data.

3.7.9.9.1 Total bare construction cost, TBCC, \$.

3.7.9.9.2 O&M material and supply costs, \$/yr.

3.7.10 Anaerobic Lagoons

3.7.10.1 Input Data.

3.7.10.1.1 Wastewater flow.

3.7.10.1.1.1 Average daily flow, mgd.

3.7.10.1.1.2 Peak hourly flow, mgd.

3.7.10.1.2 Wastewater strength, BOD_5, mg/1.

3.7.10.1.3 Other characteristics.

3.7.10.1.3.1 pH.

3.7.10.1.3.2 Temperature (maximum and minimum).

3.7.10.2 Design Parameters (See Table 3.7-1).

3.7.10.3 Process Design Calculations.

3.7.10.3.1 Calculate BOD_5 in the waste.

$$BOD = (Q_{avg}) (BODI) (8.34)$$

where

BOD = quantity of BOD_5 in waste, lb/day.

Q_{avg} = average daily flow, mgd.

BODI = concentration of BOD_5 in influent, mg/1.

8.34 = conversion factor.

3.7.10.3.2 Determine lagoon surface area.

- Based on type of lagoon and climate select a loading rate (LBOD).

$$SA = \frac{BOD}{LBOD}$$

where

SA = lagoon surface area, acres.

BOD = quantity of BOD_5 in waste, lb/day.

LBOD = lagoon loading rate, lb/day acre.

3.7.10.3.3 Determine volume of lagoon.

- Based on type of lagoon select an operating depth.

$$V = (SA)\ (D)\ (0.32585)$$

where

V = volume of lagoon, million gal.

SA = lagoon surface area, acres.

D = lagoon operating depth, ft.

0.32585 = conversion factor, acre ft to million gallons.

3.7.10.3.4 Determine detention time.

$$DT = \frac{V}{Q_{avg}}$$

where

DT = detention time, days.

V = volume of lagoon, million gal.

Q_{avg} = average daily flow, mgd.

Check detention time against minimum detention time in Table 3.7-1. If DT is less than minimum increase the surface area (SA) until the minimum detention time is obtained.

3.7.10.3.5 Effluent Characteristics.

3.7.10.3.5.1 Determine effluent BOD_5 concentration. The mechanisms in lagoons are complex and cannot be accurately predicted, therefore, effluent concentrations will be determined based on percent reduction from actual experience. The average soluble BOD_5 reduction for lagoons is 65%.

$$BODE = (1 - .65)\ (BODI)$$
$$S_e = 0.75\ BODE$$

where

BODE = concentration of BOD_5 in effluent, mg/l.

BODI = concentration of BOD_5 in influent, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.10.3.5.2 Suspended Solids.

$$SSE = 100$$

where

SSE = effluent suspended solids concentration, mg/l.

3.7.10.3.5.3 COD.

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.10.3.5.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

3.7.10.3.5.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4 = influent phosphorus concentration, mg/l.

PO4E = effluent phosphorus concentration, mg/l.

3.7.10.3.5.6 Oil and Grease.

$$OAGE = 1.5\ OAG$$

where

OAG = influent oil and grease concentration, mg/l.

OAGE = effluent oil and grease concentration, mg/l.

3.7.10.3.5.7 pH.

$$PH = 6.8$$

where

PH = effluent pH.

3.7.10.4 Process Design Output Data.

3.7.10.4.1 Lagoon loading rate, LBOD, lb/day acre.

3.7.10.4.2 Lagoon surface area, SA, acres.

3.7.10.4.3 Lagoon operating depth, D, ft.

3.7.10.4.4 Volume of lagoon, V, million gal.

3.7.10.4.5 Concentration of BOD_5 in effluent, BODE, mg/l.

3.7.10.5 Quantities Calculations.

3.7.10.5.1 Detemine quantity of earthwork.

The following assumptions are made concerning the construction of the lagoons.

A minimum of 2 cells will always be used.
Anaerobic lagoon cells will not be greater than 2 acres in surface area.
Lagoon cells will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
Lagoon cells will be square.

3.7.10.5.1.1 Detemine the number and size of lagoon cells.

TABLE 3.7-1
DESIGN PARAMETERS FOR STABILIZATION PONDS

Parameter	Type of Pond			
	Aerobic[a]	Facultative	Facultative	Anaerobic
Flow regime	Intermittently mixed	--	Mixed surface layer	--
Pond size, acres	10 multiples	2 to 10 multiples	2 to 10 multiples	0.5 to 2.0 multiples
Operation[b]	Series or parallel	Series or parallel	Series or parallel	Series
Detention time, days[b]	10 to 40	7 to 30	7 to 20	20 to 50
Depth, ft	3 to 4	3 to 6	3 to 8	8 to 15
pH	6.5 to 10.5	6.5 to 9.0	6.5 to 8.5	6.8 to 7.2
Temperature range, °C	0 to 40	0 to 50	0 to 50	6 to 50
Optimum temperature, °C	20	20	20	30
BOD_5 loading, lb/acre/day[c]	60 to 120[d]	15 to 50	30 to 100	200 to 500
BOD_5 conversion	60 to 70	60 to 70	60 to 70	50 to 70
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue
Algal concentration, mg/l	80 to 200	40 to 160	10 to 40	--
Effluent suspended solids, mg/l[e]	140 to 340	160 to 400	110 to 340	80 to 160

(a) Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produced
(b) Depends on climatic conditions.
(c) Typical values (much higher values have been applied at various loadings). Loading values are often specified by state control agencies.
(d) Some states limit this figure to 50 or less.
(e) Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/l and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/l.

3.7.10.5.1.1.1 For Anaerobic lagoons.

$$\text{If SA} \quad 4, \text{ NLC} = 2$$

$$\text{If SA} \quad 4, \text{ NLC} = \frac{SA}{2}$$

NLC must be an even number.

$$CSA = \frac{SA}{NLC}$$

where

NLC = number of lagoon cells.

SA = lagoon surface area, acres.

CSA = lagoon cell surface area, acres.

3.7.10.5.1.2 Determine lagoon cell dimensions.

$$L = 208.7\ (CSA)^{0.5} + 12$$

where

L = length of one side of lagoon cell, ft.

CSA = lagoon cell surface area, acres.

208.7 = conversion factor acres to sq ft.

12 = additional length required for 2 ft freeboard.

3.7.10.5.1.3 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = D + 2$$

$$VF = [3\ (DF)^2 + 10\ DF]\ \left[\frac{5NLC}{2} + 2\right]\ (L)$$

$$VC = (1.3)\ (NLC)\ (DC)\ [L^2 - 6(DF)(L) + 12\ (DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC $<$ VF then assume DC $>$ 1 ft and recalculate VC and VF.
If VC $>$ VF then assume DC $<$ 1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

The smallest pipe to be used will be 4 inches. If DIA < 4 inches set DIA = 4 inches. DIA must be one of the following 4, 6, 8, 10, 12, 14, etc. Always use next higher diameter.

where

DIA = pipe diameter, inches.

Q_{max} = peak hourly flow, mgd.

9.72 = combined conversion factors.

3.7.10.5.3.2 Determine length of pipe.

$$LDIA = (6D + 10)\ NLC$$

where

LDIA = length of pipe of diameter DIA, ft.

D = lagoon operating depth, ft.

NLC = number of lagoon cells.

3.7.10.5.4 Valve for lagoons. Each lagoon cell will be capable of being isolated by the use of valves. There will be one valve for each lagoon cell and the valves will be the same size as the pipe feeding the cell. The valves will be butterfly valves.

$$NBV = NLC + 1$$

$$DBV = DIA$$

where

NBV = number of valves.

NLC = number of lagoon cells.

DBV = diameter of valves, inches.

DIA = pipe diameter, inches.

3.7.10.5.5 Effluent structure. The effluent structure for all flows in this range is assumed to be a concrete structure 4 feet by 4 feet with 6" thick walls. The depth will be the same as the total depth of the lagoon.

$$V_{cw} = (8)(D+2)$$

$$V_{cs} = 8$$

where

V_{cw} = volume of concrete wall, ft^3.

D = lagoon operating depth.

V_{cs} = volume of concrete slab, ft^3.

3.7.10.5.6 Calculate operation and maintenance manpower.

$$\text{If } Q_{avg} \leq 0.1 \quad \text{OMMH} = 160$$

$$\text{If } Q_{avg} > 0.1 \quad \text{OMMH} = 313.8\ (Q_{avg})^{0.2925}$$

where

Q_{avg} = average daily flow, mgd.

OMMH = operation and maintenance manhours, MH/yr.

3.7.10.5.7 Other miscellaneous construction costs. The item already calculated represents approximately 90% of the construction cost. The other 10% consists of items such as seeding, miscellaneous concrete pads, walkways, etc.

$$CF = \frac{1}{.9} = 1.11$$

where

CF = correction factor for miscellaneous construction.

3.7.10.6 Quantities Calculations Output Data.

3.7.10.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

3.7.10.6.2 Area of lagoon liner, ALL, ft^2.

3.7.10.6.3 Pipe diameter, DIA, inches.

3.7.10.6.4 Length of pipe of diameter DIA, LDIA, ft.

3.7.10.6.5 Number of valves, NBV.

3.7.10.6.6 Diameter of valves, DBV, inches.

3.7.10.6.7 Volume of concrete wall, V_{cw}, ft^3.

3.7.10.6.8 Volume of concrete slab, V_{cs}, ft^3.

3.7.10.6.9 Operation and maintenance manpower, OMMH, MH/yr.

3.7.10.6.10 Correction factor for miscellaneous construction, CF.

3.7.10.7 <u>Unit Price Input Required</u>.

3.7.10.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

3.7.10.7.2 Unit price input for concrete wall, UPICW, \$/cu yd.

3.7.10.7.3 Unit price input for concrete slab, UPICS, \$/cu yd.

3.7.10.7.4 Cost of standard size pipe (12" Ø), COSP, \$/ft.

3.7.10.7.5 Cost of standard size valve (12" butterfly), COSTSV, \$.

3.7.10.8 <u>Cost Calculations</u>.

3.7.10.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

27 = conversion from ft^3 to cu yd.

3.7.10.8.2 Calculate cost of piping.

3.7.10.8.2.1 Installed cost of pipe.

$$ICP = \frac{COSTP}{100} (COSP)\ (LDIA)$$

where

ICP = installed cost of pipe, \$.

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12"Ø), \$/ft.

LDIA = length of pipe of diameter DIA, ft.

3.7.10.8.2.2 Determine COSTP.

$$COSTP = 6.842\ (DIA)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

DIA = pipe diameter, inches.

3.7.10.8.2.3 Determine COSP. COSP is the cost per foot of 12" Ø welded steel pipe. This cost is $13.50 per foot in 4th quarter, 1977.

3.7.10.8.3 Calculate cost of concrete.

3.7.10.8.3.1 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})}{27} (UPICW)$$

where

COSTCW = cost of concrete wall, $.

V_{cw} = volume of concrete wall, ft^3.

UPICW = unit price input for concrete wall, $/cu yd.

27 = conversion factor ft^3 to cu yd.

3.7.10.8.3.2 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})}{27} UPICS$$

where

COSTCS = cost of concrete slab, $.

V_{cs} = volume of concrete slab, ft^3.

UPICS = unit price input for concrete slab, $/cu yd.

27 = conversion factor from ft^3 to cu yd.

3.7.10.8.4 Calculate cost of valves.

3.7.10.8.4.1 Installed cost of valves.

$$IBV = \frac{(COSTBV)\ (COSTSV)\ (NBV)}{100}$$

where

IBV = installed cost of valves, $.

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

COSTSV = Cost of standard size valve, \$.

NBV = number of valves.

3.7.10.8.4.2 Determine COSTBV.

$$COSTBV = 3.99\ (DBV)^{1.395}$$

where

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

DBV = diameter of the valve, inches.

3.7.10.8.4.2 Determine COSTSV. COSTSV is the cost of a 12" ⌀ butterfly valve suitable for water service. This cost if \$1004 for 4th quarter, 1977.

3.7.10.8.5 Calculate total bare construction cost.

$$TBCC = (COSTE + ICP + COSTCW + COSTCS + IBV)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

ICP = installed cost of pipe, \$.

COSTCW = cost of concrete wall, \$.

COSTCS = cost of concrete slab, \$.

IBV = installed cost of valves, \$.

CF = correction factor for miscellaneous construction.

3.7.10.9 <u>Cost Calculations Output Data</u>.

3.7.10.9.1 Total bare construction cost, TBCC, \$.

3.7.11 Facultative Lagoon

3.7.11.1 Input Data.

3.7.11.1.1 Wastewater flow.

3.7.11.1.1.1 Average daily flow, mgd.

3.7.11.1.1.2 Peak hourly flow, mgd.

3.7.11.1.2 Wastewater strength, BOD_5, mg/l.

3.7.11.1.3 Other characteristics.

3.7.11.1.3.1 pH.

3.7.11.1.3.2 Temperature (maximum and minimum).

3.7.11.2 Design Parameters (See Table 3.7-2).

3.7.11.3 Process Design Calculations.

3.7.11.3.1 Calculate BOD_5 in the waste.

$$BOD = (Q_{avg}) (BODI) (8.34)$$

where

BOD = quantity of BOD_5 in waste, lb/day.

Q_{avg} = average daily flow, mgd.

BODI = concentration of BOD_5 in influent, mg/l.

8.34 = conversion factor.

3.7.11.3.2 Determine lagoon surface area.

- Based on type of lagoon and climate select a loading rate (LBOD).

$$SA = \frac{BOD}{LBOD}$$

where

SA = lagoon surface area, acres.

BOD = quantity of BOD_5 in waste, lb/day.

LBOD = lagoon loading rate, lb/day acre.

3.7.11.3.3 Determine volume of lagoon.

- Based on type of lagoon select an operating depth.

$$V = (SA)\ (D)\ (0.32585)$$

where

V = volume of lagoon, million gal.

SA = lagoon surface area, acres.

D = lagoon operating depth, ft.

0.32585 = conversion factor, acre ft to million gallons.

3.7.11.3.4 Determine detention time.

$$DT = \frac{V}{Q_{avg}}$$

where

DT = detention time, days.

V = volume of lagoon, million gal.

Q_{avg} = average daily flow, mgd.

Check detention time against minimum detention times in Table 3.7-2. If DT is less than minimum increase the surface area (SA) until the minimum detention time is obtained.

3.7.11.3.5 Effluent Characteristics.

3.7.11.3.5.1 Determine effluent BOD_5 concentration. The mechanisms in lagoons are complex and can not be accurately predicted, therefore effluent concentrations will be determined based on percent reduction from actual experience. The average soluble BOD_5 reduction for lagoons is 65%.

$$BODE = (1 - .65)\ (BODI)$$
$$S_e = 0.75\ BODE$$

where

BODE = concentration of BOD_5 in effluent, mg/l.

BODI = concentration of BOD_5 in influent, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.11.3.5.2 Suspended Solids.

$$SSE = 100$$

where

SSE = effluent suspended solids concentration, mg/l.

3.7.11.3.5.3 COD.

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.11.3.5.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

3.7.11.3.5.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4 = influent phosphorus concentration, mg/l.

PO4E = effluent phosphorus concentration, mg/l.

3.7.11.3.5.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAG = influent oil and grease concentration, mg/l.

OAGE = effluent oil and grease concentration, mg/l.

3.7.11.3.5.7 pH.

$$PH = 6.8$$

where

PH = effluent pH.

3.7.11.4 Process Design Output Data.

3.7.11.4.1 Lagoon loading rate, LBOD, lb/day acre.

3.7.11.4.2 Lagoon surface area, SA, acres.

3.7.11.4.3 Lagoon operating depth, D, ft.

3.7.11.4.4 Volume of lagoon, V, million gal.

3.7.11.4.5 Concentration of BOD_5 in effluent, BODE, mg/l.

3.7.11.5 Quantities Calculations.

3.7.11.5.1 Determine quantity of earthwork.

The following assumptions are made concerning the construction of the lagoons.

A minimum of 2 cells will always be used.
Facultative lagoon cells will not be greater than 10 acres in surface area.
Lagoon cells will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
Lagoon cells will be square.

3.7.11.5.1.1 Determine the number and size of lagoon cells.

3.7.11.5.1.1.1 For facultative lagoons.

$$\text{If } SA \leq 20 \quad NLC = 2$$

$$\text{If } SA > 20, \; NLC = \frac{SA}{10}$$

$$CSA = \frac{SA}{NLC}$$

where

NLC = number of lagoon cells.

SA = lagoon surface area, acres.

CSA = lagoon cell surface area, acres.

3.7.11.5.1.2 Determine lagoon cell dimensions.

$$L = 208.7\ (CSA)^{0.5} + 12$$

where

L = length of one side of lagoon cell, ft.

CSA = lagoon cell surface area, acres.

208.7 = conversion factor acres to sq ft.

12 = additional length required for 2 ft freeboard.

3.7.11.5.1.3 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = D + 2$$

$$VF = [3\ (DF)^2 + 10\ DF]\ [\ \frac{5NLC}{2} + 2\ ]\ (L)$$

$$VC = (1.3)\ (NLC)\ (DC) \quad L^2 - 6(DF)(L) + 12\ (DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC VF then assume DC 1 ft and recalculate VC and VF.
IF VC VF then assume DC 1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

TABLE 3.7-2
DESIGN PARAMETERS FOR STABILIZATION PONDS

Parameter	Type of Pond: Aerobic[(a)]	Facultative	Facultative	Anaerobic
Flow regime	Intermittently mixed	--	Mixed surface layer	--
Pond size, acres	10 multiples	2 to 10 multiples	2 to 10 multiples	0.5 to 2.0 multiple
Operation[(b)]	Series or parallel	Series or parallel	Series or parallel	Series
Detention time, days[(b)]	10 to 40	7 to 30	7 to 20	20 to 50
Depth, ft	3 to 4	3 to 6	3 to 8	8 to 15
pH	6.5 to 10.5	6.5 to 9.0	6.5 to 8.5	6.8 to 7.2
Temperature range, °C	0 to 40	0 to 50	0 to 50	6 to 50
Optimum temperature, °C	20	20	20	30
BOD_5 loading, lb/acre/day[(c)]	60 to 120[(d)]	15 to 50	30 to 100	200 to 500
BOD_5 conversion	60 to 70	60 to 70	60 to 70	50 to 70
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacteria cell tissue
Algal concentration, mg/l	80 to 200	40 to 160	10 to 40	--
Effluent suspended solids, mg/l[(e)]	140 to 340	160 to 400	110 to 340	80 to 160

(a) Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produ
(b) Depends on climatic conditions.
(c) Typical values (much higher values have been applied at various loadings). Loading values are often specified by state control agencies.
(d) Some states limit this figure to 50 or less.
(e) Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/l and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/l.

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = lagoon operating depth, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

$VLEW$ = volume of earthwork required for lagoon construction, ft^3.

3.7.11.5.2 Detemine requirement for lagoon liner. In some areas due to soil conditions the lagoons must be lined with an impervious material to prevent percolation of the wastewater into the natural ground.

3.7.11.5.2.1 Calculate area to be lined.

$$ALL = NLC \quad (4)\ (3D)(L-3D-6) + (L-6D-12)^2$$

where

ALL = area of lagoon liner, ft^2.

NLC = number of lagoon cells.

D = lagoon operating depth, ft.

L = length of one side of lagoon cell, ft.

3.7.11.5.3 Piping for lagoon cells.

Assume:

Pipes are flowing full.
Velocity is 3 fps.
Pipes going through the levee extend 10 ft past toe of the levee.

3.7.11.5.3.1 Detemine pipe size.

$$DIA = 9.72\ (Q_{max})^{0.5}$$

The smallest pipe to be used will be 4 inches. If DIA < 4 inches set DIA = 4 inches. DIA must be one of the following 4, 6, 8, 10, 12, 14, etc. Always use next higher diameter.

where

DIA = pipe diameter, inches.

Q_{max} = peak hourly flow, mgd.

9.72 = combined conversion factors.

3.7.11.5.3.2 Determine length of pipe.

$$LDIA = (6D + 10)\ NLC$$

where

LDIA = length of pipe of diameter DIA, ft.

D = lagoon operating depth, ft.

NLC = number of lagoon cells.

3.7.11.5.4 Valve for lagoons. Each lagoon cell will be capable of being isolated by the use of valves. There will be one valve for each lagoon cell and the valves will be the same size as the pipe feeding the cell. The valves will be butterfly valves.

$$NBV = NLC + 1$$

$$DBV = DIA$$

where

NBV = number of valves.

NLC = number of lagoon cells.

DBV = diameter of valves, inches.

DIA = pipe diameter, inches.

3.7.11.5.5 Effluent structure. The effluent structure for all flows in this range is assumed to be a concrete structure 4 feet by 4 feet with 6" thick walls. The depth will be the same as the total depth of the lagoon.

$$V_{cw} = (8)(D+2)$$

$$V_{cs} = 8$$

where

V_{cw} = volume of concrete wall, ft^3.

D = lagoon operating depth.

V_{cs} = volume of concrete slab, ft^3.

3.7.11.5.6 Calculate operation and maintenance manpower.

$$\text{If } Q_{avg} \leq 0.1 \quad OMMH = 160$$

$$\text{If } Q_{avg} > 0.1 \quad OMMH = 313.8\ (Q_{avg})^{0.2925}$$

where

Q_{avg} = average daily flow, mgd.

OMMH = operation and maintenance manhours, MH/yr.

3.7.11.5.7 Other miscellaneous construction costs. The item already calculated represents approximately 90% of the construction cost. The other 10% consists of items such as seeding, miscellaneous concrete pads, walkways, etc.

$$CF = \frac{1}{.9} = 1.11$$

where

CF = correction factor for miscellaneous construction.

3.7.11.6 Quantities Calculations Output Data.

3.7.11.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

3.7.11.6.2 Area of lagoon liner, ALL, ft^2.

3.7.11.6.3 Pipe diameter, DIA, inches.

3.7.11.6.4 Length of pipe of diameter DIA, LDIA, ft.

3.7.11.6.5 Number of valves, NBV.

3.7.11.6.6 Diameter of valves, DBV, inches.

3.7.11.6.7 Volume of concrete wall, V_{cw}, ft^3.

3.7.11.6.8 Volume of concrete slab, V_{cs}, ft^3.

3.7.11.6.9 Operation and maintenance manpower, OMMH, MH/yr.

3.7.11.6.10 Correction factor for miscellaneous construction, CF.

3.7.11.7 <u>Unit Price Input Required</u>.

3.7.11.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

3.7.11.7.2 Unit price input for concrete wall, UPICW, \$/cu yd.

3.7.11.7.3 Unit price input for concrete slab, UPICS, \$/cu yd.

3.7.11.7.4 Cost of standard size pipe (12" Ø), COSP, \$/ft.

3.7.11.7.5 Cost of standard size valve (12" butterfly), COSTSV, \$.

3.7.11.8 <u>Cost Calculations</u>.

3.7.11.8.1 Calculate cost of earthwork.

$$\text{COSTE} = \frac{\text{VLEW}}{27}\,\text{UPIEX}$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

27 = conversion from ft^3 to cu yd.

3.7.11.8.2 Calculate cost of piping.

3.7.11.8.2.1 Installed cost of pipe.

$$\text{ICP} = \frac{\text{COSTP}}{100}\,(\text{COSP})\,(\text{LDIA})$$

where

ICP = installed cost of pipe, \$.

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12"Ø), \$/ft.

LDIA = length of pipe of diameter DIA, ft.

3.7.11.8.2.2 Determine COSTP.

$$\text{COSTP} = 6.842\,(\text{DIA})^{1.2255}$$

where

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

DIA = pipe diameter, inches.

3.7.11.8.2.3 Determine COSP. COSP is the cost per foot of 12" ⌀ welded steel pipe. This cost is $13.50 per foot in 4th quarter, 1977.

3.7.11.8.3 Calculate cost of concrete.

3.7.11.8.3.1 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})}{27}(UPICW)$$

where

COSTCW = cost of concrete wall, $.

V_{cw} = volume of concrete wall, ft^3.

UPICW = unit price input for concrete wall, $/cu yd.

27 = conversion factor ft^3 to cu yd.

3.7.11.8.3.2 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})}{27} UPICS$$

where

COSTCS = cost of concrete slab, $.

V_{cs} = volume of concrete slab, ft^3.

UPICS = unit price input for concrete slab, $/cu yd.

27 = conversion factor from ft^3 to cu yd.

3.7.11.8.4 Calculate cost of valves.

3.7.11.8.4.1 Installed cost of valves.

$$IBV = \frac{(COSTBV)\ (COSTSV)\ (NBV)}{100}$$

where

IBV = installed cost of valves, $.

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

COSTSV = Cost of standard size valve, \$.

NBV = number of valves.

3.7.11.8.4.2 Determine COSTBV.

$$COSTBV = 3.99\ (DBV)^{1.395}$$

where

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

DBV = diameter of the valve, inches.

3.7.11.8.4.3 Determine COSTSV. COSTSV is the cost of a 12" ϕ butterfly valve suitable for water service. This cost if \$1004 for 4th quarter, 1977.

3.7.11.8.5 Calculate total bare construction cost.

$$TBCC = (COSTE + ICP + COSTCW + COSTCS + IBV)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

ICP = installed cost of pipe, \$.

COSTCW = cost of concrete wall, \$.

COSTCS = cost of concrete slab, \$.

IBV = installed cost of valves, \$.

CF = correction factor for miscellaneous construction.

3.7.11.9 Cost Calculations Output Data.

3.7.11.9.1 Total bare construction cost, TBCC, \$.

3.7.12 Oxidation Lagoon.

3.7.12.1 Input Data.

3.7.12.1.1 Wastewater flow.

3.7.12.1.1.1 Average daily flow, mgd.

3.7.12.1.1.2 Peak hourly flow, mgd.

3.7.12.1.2 Wastewater strength, BOD_5, mg/l.

3.7.12.1.3 Other characteristics.

3.7.12.1.3.1 pH.

3.7.12.1.3.2 Temperature (maximum and minimum).

3.7.12.2 Design Parameters (See Table 3.7-3).

3.7.12.3 Process Design Calculations.

3.7.12.3.1 Calculate BOD_5 in the waste.

$$BOD = (Q_{avg}) (BODI) (8.34)$$

where

BOD = quantity of BOD_5 in waste, lb/day.

Q_{avg} = average daily flow, mgd.

BODI = concentration of BOD_5 in influent, mg/l.

8.34 = conversion factor.

3.7.12.3.2 Determine lagoon surface area.

- Based on type of lagoon and climate select a loading rate (LBOD).

$$SA = \frac{BOD}{LBOD}$$

where

SA = lagoon surface area, acres.

BOD = quantity of BOD_5 in waste, lb/day.

LBOD = lagoon loading rate, lb/day acre.

3.7.12.3.3 Determine volume of lagoon.

- Based on type of lagoon select an operating depth.

$$V = (SA)\ (D)\ (0.32585)$$

where

V = volume of lagoon, million gal.

SA = lagoon surface area, acres.

D = lagoon operating depth, ft.

0.32585 = conversion factor, acre ft to million gallons.

3.7.12.3.4 Determine detention time.

$$DT = \frac{V}{Q_{avg}}$$

where

DT = detention time, days.

V = volume of lagoon, million gal.

Q_{avg} = average daily flow, mgd.

Check detention time against minimum detention times in Table 3.7-3. If DT is less than minimum increase the surface area (SA) until the minimum detention time is obtained.

3.7.12.3.5 Effluent Characteristic.

3.7.12.3.5.1 Determine effluent BOD_5 concentration. The mechanisms in lagoons are complex and can not be accurately predicted, therefore effluent concentrations will be determined based on percent reduction from actual experience. The average soluble BOD_5 reduction for lagoons is 65%.

$$BODE = (1 - .65)\ (BODI)$$
$$S_e = 0.75\ BODE$$

where

BODE = concentration of BOD_5 in effluent, mg/l.

BODI = concentration of BOD_5 in influent, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.12.3.5.2 Suspended Solids.

$$SSE = 100$$

where

SSE = effluent suspended solids concentration, mg/l.

3.7.12.3.5.3 COD.

$$CODE = 1.5\ BODE$$
$$CODSE = 1.5\ S_e$$

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.7.12.3.5.4 Nitrogen.

$$TKNE = TKN$$
$$NH3E = TKNE$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

NO3 = influent NO3 concentration, mg/l.

NO3E = effluent NO3 concentration, mg/l.

NO2 = influent NO2 concentration, mg/l.

NO2E = effluent NO2 concentration, mg/l.

3.7.12.3.5.5 Phosphorus.

$$PO4E = 0.7\ PO4$$

where

PO4 = influent phosphorus concentration, mg/l.

PO4E = effluent phosphorus concentration, mg/l.

3.7.12.3.5.6 Oil and Grease.

$$OAGE = 0.15\ OAG$$

where

OAG = influent oil and grease concentration, mg/l.

OAGE = effluent oil and grease concentration, mg/l.

3.7.12.3.5.7 pH.

$$PH = 6.8$$

where

PH = effluent pH.

TABLE 3.7-3
DESIGN PARAMETERS FOR STABILIZATION PONDS

Parameter	Type of Pond: Aerobic[a]	Facultative	Facultative	Anaerobic
Flow regime	Intermittently mixed	--	Mixed surface layer	--
Pond size, acres	10 multiples	2 to 10 multiples	2 to 10 multiples	0.5 to 2.0 multiples
Operation[b]	Series or parallel	Series or parallel	Series or parallel	Series
Detention time, days[b]	10 to 40	7 to 30	7 to 20	20 to 50
Depth, ft	3 to 4	3 to 6	3 to 8	8 to 15
pH	6.5 to 10.5	6.5 to 9.0	6.5 to 8.5	6.8 to 7.2
Temperature range, °C	0 to 40	0 to 50	0 to 50	6 to 50
Optimum temperature, °C	20	20	20	30
BOD_5 loading, lb/acre/day[c]	60 to 120[d]	15 to 50	30 to 100	200 to 500
BOD_5 conversion	60 to 70	60 to 70	60 to 70	50 to 70
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue
Algal concentration, mg/l	80 to 200	40 to 160	10 to 40	--
Effluent suspended solids, mg/l[e]	140 to 340	160 to 400	110 to 340	80 to 160

(a) Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produced

(b) Depends on climatic conditions.

(c) Typical values (much higher values have been applied at various loadings). Loading values are often specified by state control agencies.

(d) Some states limit this figure to 50 or less.

(e) Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/l and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/l.

3.7.12.4 Process Design Output Data.

3.7.12.4.1 Lagoon loading rate, LBOD, lb/day acre.

3.7.12.4.2 Lagoon surface area, SA, acres.

3.7.12.4.3 Lagoon operating depth, D, ft.

3.7.12.4.4 Volume of lagoon, V, million gal.

3.7.12.4.5 Concentration of BOD_5 in effluent, BODE, mg/l.

3.7.12.5 Quantities Calculations.

3.7.12.5.1 Determine quantity of earthwork.

The following assumptions are made concerning the construction of the lagoons.

A minimum of 2 cells will always be used.
Oxidation lagoon cells will not be greater than 10 acres in surface area.
Lagoon cells will be constructed using equal cut and fill.
Levee side slopes will be 3 to 1.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
Lagoon cells will be square.

3.7.12.5.1.1 Determine the number and size of lagoon cells.

3.7.12.5.1.1.1 For oxidation lagoons.

$$\text{If } SA \leq 20 \quad NLC = 2$$

$$\text{If } SA > 20, \; NLC = \frac{SA}{10}$$

$$CSA = \frac{SA}{NLC}$$

where

NLC = number of lagoon cells.

SA = lagoon surface area, acres.

CSA = lagoon cell surface area, acres.

3.7.12.5.1.2 Determine lagoon cell dimensions.

$$L = 208.7\ (CSA)^{0.5} + 12$$

where

L = length of one side of lagoon cell, ft.

CSA = lagoon cell surface area, acres.

208.7 = conversion factor acres to sq ft.

12 = additional length required for 2 ft freeboard.

3.7.12.5.1.3 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = D + 2$$

$$VF = [\ 3\ (DF)^2 + 10\ DF\ ]\ [\frac{5NLC}{2} + 2] \quad (L)$$

$$VC = (1.3)\ (NLC)\ (DC)\ [\ L^2 - 6(DF)(L) + 12\ (DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC < VF then assume DC > 1 ft and recalculate VC and VF.
IF VC > VF then assume DC < 1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = lagoon operating depth, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

3.7.12.5.2 Determine requirement for lagoon liner. In some areas due to soil conditions the lagoons must be lined with an impervious material to prevent percolation of the wastewater into the natural ground.

3.7.12.5.2.1 Calculate area to be lined.

$$ALL = NLC\ [\ (4)\ (3D)(L-3D-6) + (L-6D-12)^2\ ]$$

where

ALL = area of lagoon liner, ft^2.

NLC = number of lagoon cells.

D = lagoon operating depth, ft.

L = length of one side of lagoon cell, ft.

3.7.12.5.3 Piping for lagoon cells.

Assume:

Pipes are flowing full.
Velocity is 3 fps.
Pipes going through the levee extend 10 ft past toe of the levee.

3.7.12.5.3.1 Determine pipe size.

$$DIA = 9.72\ (Q_{max})^{0.5}$$

The smallest pipe to be used will be 4 inches. If DIA < 4 inches set DIA = 4 inches. DIA must be one of the following 4, 6, 8, 10, 12, 14, etc. Always use next higher diameter.

where

DIA = pipe diameter, inches.

Q_{max} = peak hourly flow, mgd.

9.72 = combined conversion factors.

3.7.12.5.3.2 Determine length of pipe.

$$LDIA = (6D + 10)\ NLC$$

where

$LDIA$ = length of pipe of diameter DIA, ft.

D = lagoon operating depth, ft.

NLC = number of lagoon cells.

3.7.12.5.4 Valve for lagoons. Each lagoon cell will be capable of being isolated by the use of valves. There will be one valve for each lagoon cell and the valves will be the same size as the pipe feeding the cell. The valves will be butterfly valves.

$$NBV = NLC + 1$$

$$DBV = DIA$$

where

NBV = number of valves.

NLC = number of lagoon cells.

DBV = diameter of valves, inches.

DIA = pipe diameter, inches.

3.7.12.5.5 Effluent structure. The effluent structure for all flows in this range is assumed to be a concrete structure 4 feet by 4 feet with 6" thick walls. The depth will be the same as the total depth of the lagoon.

$$V_{cw} = (8)(D+2)$$

$$V_{cs} = 8$$

where

V_{cw} = volume of concrete wall, ft^3.

D = lagoon operating depth.

V_{cs} = volume of concrete slab, ft^3.

3.7.12.5.6 Calculate operation and maintenance manpower.

$$\text{If } Q_{avg} \leq 0.1 \quad OMMH = 160$$

$$\text{If } Q_{avg} > 0.1 \quad OMMH = 313.8\ (Q_{avg})^{0.2925}$$

where

Q_{avg} = average daily flow, mgd.

OMMH = operation and maintenance manhours, MH/yr.

3.7.12.5.7 Other miscellaneous construction costs. The item already calculated represents approximately 90% of the construction cost. The other 10% consists of items such as seeding, miscellaneous concrete pads, walkways, etc.

$$CF = \frac{1}{.9} = 1.11$$

where

CF = correction factor for miscellaneous construction.

3.7.12.6 <u>Quantities Calculations Output Data</u>.

3.7.12.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

3.7.12.6.2 Area of lagoon liner, ALL, ft^2.

3.7.12.6.3 Pipe diameter, DIA, inches.

3.7.12.6.4 Length of pipe of diameter DIA, LDIA, ft.

3.7.12.6.5 Number of valves, NBV.

3.7.12.6.6 Diameter of valves, DBV, inches.

3.7.12.6.7 Volume of concrete wall, V_{cw}, ft^3.

3.7.12.6.8 Volume of concrete slab, V_{cs}, ft^3.

3.7.12.6.9 Operation and maintenance manpower, OMMH, MH/yr.

3.7.12.6.10 Correction factor for miscellaneous construction, CF.

3.7.12.7 <u>Unit Price Input Required</u>.

3.7.12.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

3.7.12.7.2 Unit price input for concrete wall, UPICW, \$/cu yd.

3.7.12.7.3 Unit price input for concrete slab, UPICS, \$/cu yd.

3.7.12.7.4 Cost of standard size pipe (12" Ø), COSTSP, \$/ft.

3.7.12.7.5 Cost of standard size valve (12" butterfly), COSTSV, $.

3.7.12.8 Cost Calculations.

3.7.12.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VLEW}{27} UPIEX$$

where

COSTE = cost of earthwork, $.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, $/cu yd.

27 = conversion from ft^3 to cu yd.

3.7.12.8.2 Calculate cost of piping.

3.7.12.8.2.1 Installed cost of pipe.

$$ICP = \frac{COSTP}{100} (COSP) (LDIA)$$

where

ICP = installed cost of pipe, $.

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12"ϕ), $/ft.

LDIA = length of pipe of diameter DIA, ft.

3.7.12.8.2.2 Determine COSTP.

$$COSTP = 6.842 (DIA)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIA as percent of cost of standard size pipe, %.

DIA = pipe diameter, inches.

3.7.12.8.2.3 Determine COSP. COSP is the cost per foot of 12" ø welded steel pipe. This cost is \$13.50 per foot in 4th quarter, 1977.

3.7.12.8.3 Calculate cost of concrete.

3.7.12.8.3.1 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})}{27} (UPICW)$$

where

COSTCW = cost of concrete wall, \$.

V_{cw} = volume of concrete wall, ft^3.

UPICW = unit price input for concrete wall, \$/cu yd.

27 = conversion factor ft^3 to cu yd.

3.7.12.8.3.2 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})}{27} UPICS$$

where

COSTCS = cost of concrete slab, \$.

V_{cs} = volume of concrete slab, ft^3.

UPICS = unit price input for concrete slab, \$/cu yd.

27 = conversion factor from ft^3 to cu yd.

3.7.12.8.4 Calculate cost of valves.

3.7.12.8.4.1 Installed cost of valves.

$$IBV = \frac{(COSTBV)\ (COSTSV)\ (NBV)}{100}$$

where

IBV = installed cost of valves, \$.

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

COSTSV = Cost of standard size valve, \$.

NBV = number of valves.

3.7.12.8.4.2 Determine COSTBV.

$$COSTBV = 3.99\ (DBV)^{1.395}$$

where

COSTBV = cost of valve of diameter DBV as percent of cost of standard size valve, %.

DBV = diameter of the valve, inches.

3.7.12.8.4.2 Determine COSTSV. COSTSV is the cost of a 12" ϕ butterfly valve suitable for water service. This cost if $1004 for 4th quarter, 1977.

3.7.12.8.5 Calculate total bare construction cost.

$$TBCC = (COSTE + ICP + COSTCW + COSTCS + IBV)\ CF$$

where

TBCC = total bare construction cost, $.

COSTE = cost of earthwork, $.

ICP = installed cost of pipe, $.

COSTCW = cost of concrete wall, $.

COSTCS = cost of concrete slab, $.

IBV = installed cost of valves, $.

CF = correction factor for miscellaneous construction.

3.7.12.9 <u>Cost Calculations Output Data</u>.

3.7.12.9.1 Total bare construction cost, TBCC, $.

3.7.13 Sludge Lagoon.

3.7.13.1 Input Data.

3.7.13.1.1 Sludge flow, Q_s, gpd.

3.7.13.1.2 Initial solids content in sludge, S_o, %.

3.7.13.1.3 Solids loading rate, LRS, lb/yr cu ft (2.2 to 2.4 lb/yr cu ft.)

3.7.13.2 Design Parameters.

3.7.13.2.1 Solids loading rate, lb/yr cu ft.

3.7.13.2.2 Sludge characteristics.

3.7.13.2.3 Soil permeability.

3.7.13.3 Process Design Calculations.

3.7.13.3.1 Calculate dry solids produced.

$$DSP = \frac{(Q_s)(S_o)(8.34)(365)}{100}$$

where

DSP = dry solids produced, lb/yr.

S_o = initial solids content in sludge, %.

8.34 = conversion from gal to lb, lb/gal.

365 = conversion, days/yr.

3.7.13.3.2 Calculate volume of lagoons.

$$LV = \frac{DSP}{LRS}$$

where

LV = lagoon volume, cu ft.

DSP = dry solids produced, lb/yr.

LRS = solids loading rate, lb/yr cu ft.

3.7.13.3.3 Calculate lagoon surface area.

$$TLSA = \frac{LV}{D}$$

where

TLSA = total lagoon surface area, sq ft.

LV = lagoon volume, cu ft.

D = sludge depth in lagoon, ft.

3.7.13.3.4 Calculate number of lagoons. There should always be a minimum of 2 lagoons so that one is drying while the other is being filled. In this flow range no more than 2 lagoons should be required.

$$NL = 2$$

where

NL = number of lagoons.

3.7.13.3.5 Final Sludge Volume.

$$Q_f = \frac{(Q_s)\ (So)}{30}$$

where

Q_f = final sludge volume, gpd.

Q_s = initial sludge volume, gpd.

So = initial solids content, %.

30 = final solids content, %.

3.7.13.4 <u>Process Design Output Data.</u>

3.7.13.4.1 Dry solids produced, DSP, lb/yr.

3.7.13.4.2 Sludge flow, Q_s, gpd.

3.7.13.4.3 Initial solid content in sludge, S_o, %.

3.7.13.4.4 Solids loading rate, LRS, lb/yr cu ft.

3.7.13.4.5 Sludge depth in lagoon, D, ft.

3.7.13.4.6 Lagoon volume, LV, cu ft.

3.7.13.4.7 Total lagoon surface area, TLSA, sq ft.

3.7.13.4.8 Number of lagoons, NL.

3.7.13.5 Quantities Calculations.

3.7.13.5.1 Assumptions. The following assumptions are made concerning the construction of the lagoons.

3.7.13.5.1.1 The levees will have a 3 to 1 side slope.

3.7.13.5.1.2 The levees will have a 10 ft wide flat top for access.

3.7.13.5.1.3 The lagoons will be constructed with equal cut and fill.

3.7.13.5.1.4 There will be a minimum of 2 lagoons for operational purposes.

3.7.13.5.1.5 The sludge depth in the lagoons will be a maximum of 2 ft with 2 ft of freeboard.

3.7.13.5.1.6 Common levee construction will be used.

3.7.13.5.1.7 Lagoons will be square.

3.7.13.5.2 Calculate surface area per lagoon.

$$SAL = \frac{TLSA}{2}$$

where

SAL = surface area per lagoon, sq ft.

TLSA = total lagoon surface area, sq ft.

2 = number of lagoons.

3.7.13.5.3 Calculate dimensions of lagoon.

$$L = (SAL)^{0.5} + 12$$

where

L = length of one side of lagoon at top of levee, ft.

SAL = surface area per lagoon, sq ft.

12 = additional length for 2 ft freeboard.

3.7.13.5.4 Calculate volume of earthwork required. The volume of earthwork must be detemined by trial and error using the following equations:

$$DF + DC = D + 2$$

$$VF = [3(DF)^2 + 10\ DF]\quad [7L]$$

$$VC = (2.6)(DC)\ [L^2 - 6(DF)(L) + 12(DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 foot. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

IF VC $<$ VF then assume DC$>$1 ft and recalculate VC and VF.
IF VC $>$ VF then assume DC$<$1 ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the lagoons.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

D = sludge depth in lagoon, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

L = length of one side of lagoon at the to of levee, ft.

VLEW = volume of earthwork required, ft^3.

3.7.13.5.5 Calculate concrete required for overflow-decant structure. It is assumed that the same structure will be used for all flows. The structure will be 4' x 4' with 6" thick walls. The height of the structure will be 3 ft above the depth of sludge in the lagoon.

$$V_{cw} = 8 + (8)\ (D + 3)\ \ 2$$

where

V_{cw} = volume of concrete wall required, ft^3.

D = sludge depth in lagoon, ft.

2 = number of lagoons.

3.7.13.5.6 Calculate operation manpower required.

3.7.13.5.6.1 If DSP $\leq$73,000 lb/yr.

$$OMH = 46$$

3.7.13.5.6.2 If DSP $>$ 73,000 lb/yr.

$$OMH = 5.81\ (DSP)^{0.1847}$$

where

DSP = dry solids produced, lb/yr.

OMH = operation man-hour requirement, MH/yr.

3.7.13.5.7 Calculate maintenance manpower required.

3.7.13.5.7.1 If DSP 73,000 lb/yr.

$$MMH = 24$$

3.7.13.5.7.2 If DSP 73,000 lb/yr.

$$MMH = 1.47\ (DSP)^{0.2491}$$

where

DSP = dry solids produced, lb/yr.

MMH = maintenance man-hours required, MH/yr.

3.7.13.5.8 Other construction cost items. The previous calculations account for approximately 80% of the cost of the drying lagoons. The other 20% includes influent piping, slide gates for decanting, grassing slopes, etc.

$$CF = \frac{1}{0.8} = 1.25$$

where

CF = correction factor for other construction cost items.

3.7.13.6 Quantities Calculations Output Data.

3.7.13.6.1 Volume of earthwork required, VLEW, ft^3.

3.7.13.6.2 Volume of concrete wall required, V_{cw}, ft^3.

3.7.13.6.3 Operation man-hour requirement, OMH, MH/yr.

3.7.13.6.4 Maintenance man-hour requirement, MMH, MH/yr.

3.7.13.6.5 Correction factor for other construction cost items, CF.

3.7.13.7 Unit Price Input Required.

3.7.13.7.1 Unit price input for excavation, UPIEX, \$/cu yd.

3.7.13.7.2 Unit price input for R.C. wall in-place, UPICW, \$/cu yd.

3.7.13.8 Cost Calculations.

3.7.13.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VLEW}{27} (UPIEX)$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required, ft^3.

UPIEX = unit price input for excavation, \$/cu yd.

3.7.13.8.2 Calculate cost of concrete

$$COSTCW = \frac{V_{cw}}{27} (UPICW)$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, ft^3.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.7.13.8.3 Calculate total care construction cost.

$$TBCC = (COSTE + COSTCW) CF$$

where

TBCC = total care construction cost, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

CF = correction factor for other construction cost.

3.7.13.9 Cost Calculations Output Data.

3.7.13.9.1 Total bare construction cost, TBCC, \$.

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3.9 LAGOON UPGRADING

3.9.1 Background. Many small facilities have utilized lagoons of one type or another for waste treatment because of their ease of operation and low construction cost. For a long period of time very little reliable data on the effectiveness of lagoons was available. As more data is amassed and with the advent of stricter effluent standards in many areas, it has become clear that many lagoon systems may require upgrading. Four potential methods of upgrading lagoon effluents will be addressed; coagulation and settling, dissolved air flotation, gravity filters, and intermittant sand filtration.

3.9.2 General Description Coagulation and Settling.

3.9.2.1 Chemical coagulation involves the aggregation of small particles into large, more readily settleable conglomerates. Chemical coagulation is used in wastewater treatment to remove colloidal and suspended matter from raw wastes, remove phosphorus, remove algae from lagoon effluents, and enhance sludge dewaterability.

3.9.2.2 Wastewater can be coagulated using any of the coagulants common in water treatment. The coagulants have been limited to three for purpose of this program. They are alum, ferric chloride, and lime.

3.9.2.3 The essential processes involved in chemical coagulation are rapid mixing, flocculation, and sedimentation. These processes can be carried out in separate basins or in a flocculator-clarifier system. Because of its popularity and lower capital costs, only the flocculator-clarifier system will be considered.

3.9.3 General Description Dissolved Air Flotation.

3.9.3.1 Dissolved air flotation is a solid-liquid separation process. Separation is accomplished by introducing fine gas bubbles into the system. The gas bubbles become attached to the solid particles, forming an aggregate which has a bulk density less than water causing the aggregate to float to the surface. The aggregate can then be removed from the surface by skimming. This process has been evaluated for use in upgrading lagoons in small systems. The main component to be removed is algae. While flotation has been shown to work well when properly operated with the addition of coagulants, its' use in small systems is hampered in that it requires a relatively high degree of operator skill, which generally will not be available at this level.

3.9.4 General Description Gravity Filters.

3.9.4.1 Gravity filters are used in upgrading the effluents from secondary treatment plants and stabilization ponds by removing suspended solids. In the flow range addressed here the filters are nomally package, multimedia filtration units. These units come complete except for concrete slab and connecting piping, which would be provided by the general contractor.

3.9.4.2 Tertiary filters are generally at the end of the treatment system and involve substantial head loss. For this reason pumping is usually required in front of the filters. Design and costing of the pumping facilities is not included in this section, this should be done in the section entitled, "Intermediate Pumping".

3.9.5 General Description Intermittent Sand Filtration.

3.9.5.1 The intermittent sand filter has been used in wastewater treatment since the late nineteenth century. However, it was used to treat raw or primary settled sewage. Little work has been done in using the intermittent sand filter as an effluent polishing process until recent years. Recent studies show that it gives good results when used as a polishing process for secondary treatment processes. For purposes of this program the intermittent sand filter will be used only in conjunction with suspended solids removal from lagoon effluent.

3.9.6 Coagulation and Settling.

3.9.6.1 Input Data.

3.9.6.1.1 Wastewater flow.

3.9.6.1.1.1 Average daily flow, Q_{avg}, mgd.

3.9.6.1.1.2 Peak flow, Q_p, mgd.

3.9.6.1.2 Wastewater characteristics.

3.9.6.1.2.1 BOD, total and soluble, mg/l.

3.9.6.1.2.2 COD, total and soluble, mg/l.

3.9.6.1.2.3 Phosphorus, mg/l.

3.9.6.1.2.4 Suspended solids, mg/l.

3.9.6.1.2.5 pH.

3.9.6.1.2.6 Alkalinity, mg/l.

3.9.6.2 Design Parameters.

3.9.6.2.1 Coagulant used.

3.9.6.2.1.1 Alum.

3.9.6.2.1.2 Ferric chloride.

3.9.6.2.1.3 Lime.

3.9.6.2.2 Coagulant dosage, mg/l.

3.9.6.2.3 Desired quality of treated effluent, mg/l.

3.9.6.2.4 Surface loading rates for clarifier, gpd/ft^2.

3.9.6.3 Process Design Calculations.

3.9.6.3.1 Calculate coagulant requirements. Select the type of coagulant and the dosage rate required.

$$CR = (CD)\ (Q_{avg})\ (8.34)$$

where

CR = coagulant requirements, lb/day.

CD = coagulant dosage, mg/l.

Q_{avg} = average daily flow, mgd.

3.9.6.3.2 Sludge from suspended solids removal. This is the same regardless of the coagulant used.

$$X_s = (8.34)\ (Q_{avg})\ (SS_{inf} - SS_{eff})$$

where

X_s = sludge produced due to suspended solids removed, lb/day.

SS_{inf} = influent suspended solids, mg/l.

SS_{eff} = effluent suspended solids, mg/l.

Q_{avg} = average daily flow, mgd.

3.9.6.3.3 Sludge production when alum is used as coagulant.

3.9.6.3.3.1 Sludge from phosphorus removal.

$$X_p = (8.34)\ (4)\ (Q_{avg})\ (P_{inf} - P_{eff})$$

where

X_p = sludge from phosphorus removal, lb/day.

P_{inf} = influent phosphorus concentration, mg/l.

P_{eff} = effluent phosphorus concentration, mg/l.

Q_{avg} = average daily flow, mgd.

3.9.6.3.3.2 Alum required for phosphorus precipitation.

$$AP = (P_{inf} - P_{eff})\ 9.6$$

where

AP = alum required for phosphorus precipitation, mg/l.

P_{inf} = influent phosphorus concentration, mg/l.

P_{eff} = effluent phosphorus concentration, mg/l.

3.9.6.3.3.3 Sludge form hydroxide formation.

$$X_{OH} = (8.34)\ (0.263)\ (Q_{avg})\ (CD - AP)$$

where

X_{OH} = sludge from hydroxide formation, lb/day.

Q_{avg} = average wastewater flow, mgd.

CD = coagulant dosage, mg/l.

AP = alum required for phosphorus precipitation, mg/l.

3.9.6.3.3.4 Total sludge production when alum is the coagulant.

$$X_{AL} = X_s + X_p + X_{OH}$$

where

X_{AL} = total sludge production with alum, lb/day.

X_s = sludge produced due to suspended solids removed, lb/day.

X_p = sludge produced from phosphorus removal, lb/day.

X_{OH} = sludge fom hydroxide formation, lb/day.

3.9.6.3.4 Sludge production when ferric chloride is used as coagulant.

3.9.6.3.4.1 Sludge from phosphorus removal.

$$X_p = (8.34)\ (4.9)\ (Q_{avg})\ (P_{inf} - P_{eff})$$

where

X_p = sludge from phosphorus removal, lb/day.

Q_{avg} = average daily flow, mgd.

P_{inf} = influent phosphorus concentration, mg/l.

P_{eff} = effluent phosphorus concentration, mg/l.

3.9.6.3.4.2 Ferric chloride requirement for phosphorus removal.

$$FP = (P_{inf} - P_{eff})\ (5.23)$$

where

FP = ferric chloride requirement for phosphorus removal, mg/l.

P_{inf} = influent phosphorus concentration, mg/l.

P_{eff} = effluent phosphorus concentration, mg/l.

3.9.6.3.4.3 Sludge from hydroxide formation.

$$X_{OH} = (8.34)\ (0.65)\ (Q_{avg})\ (CD - FP)$$

where

X_{OH} = sludge from hydroxide formation.

Q_{avg} = average daily flow, mgd.

CD = coagulant dosage, mg/l.

FP = ferric chloride required for phosphorus removal, mg/l.

3.9.6.3.4.4 Total sludge production when ferric chloride is used as coagulant.

$$X_{Fe} = X_s + X_p + X_{OH}$$

where

X_{Fe} = total sludge production with ferric chloride, lb/day.

X_s = sludge produced due to suspended solids removal, lb/day.

X_p = sludge produced from phosphorus removal, lb/day.

X_{OH} = sludge from hydroxide formation.

3.9.6.3.5 Sludge production when lime is used as a coagulant.

3.9.6.3.5.1 Chemical sludge production. The chemical reactions involved in the lime precipitation process are quite complex. Factors such as raw waste, hardness, alkalinity, calcium or magnesium ratio, pH, and others can significantly affect the sludge quantity. For preliminary estimation purposes, the following equation will be used:

$$X_{Ca} = (8.34)\ (1.5)\ (Q_{avg})\ (CD)$$

where

X_{Ca} = chemical sludge production due to lime precipitation, lb/day.

Q_{avg} = average daily flow, mgd.

CD = coagulant dosage, mg/l.

3.9.6.3.5.2 Total sludge production when lime is used as coagulant.

$$X_{CaT} = X_s + X_{Ca}$$

where

X_{CaT} = total sludge production with lime, lb/day.

X_s = sludge produced due to suspended solids removal, lb/day.

X_{Ca} = chemical sludge production due to lime precipitation, lb/day.

3.9.6.3.6 Selection of flocculator-clarifier. This type of system integrates the rapid mixing tank, flocculator, and clarifier into one unit. It is sometimes referred to as an upflow clarifier or a sludge blanket clarifier.

3.9.6.3.6.1 The size of an upflow clarifier is based on overflow rate. The design overflow rate varies with the type of coagulant used.

3.9.6.3.6.1.1 Lime coagulation.

$$Q_{of} = 630 \text{ gpd/ft}^2$$

3.9.6.3.6.1.2 Alum coagulation.

$$Q_{of} = 360 \text{ gpd/ft}^2$$

3.9.6.3.6.1.3 Iron salt caogulation.

$$Q_{of} = 500 \text{ gpd/ft}^2$$

where

Q_{of} = design overflow rate, gpm/ft^2.

3.9.6.3.6.2 Unit selection. Assume in this flow range only one unit will be used.

$$DIA = 1.13 {\frac{Q_{avg} \times 10^6}{Q_{of}}}^{0.5}$$

DIA must be an integer.

If DIA $\geq$200 ft. assume 2 units and recalculate DIA.

where

DIA = diameter of flocculator-clarifier, ft.

Q_{avg} = average daily flow, mgd.

Q_{of} = design overflow rate, gpd/ft^2.

3.9.6.3.6.3 Determine the depth of clarifier. The side water depth is a function of the diameter of the unit.

$$SWD = 10.67 + (0.067) (DIA)$$

where

SWD = side water depth of flocculator-clarifier, ft.

DIA = diameter of flocculator-clarifier, ft.

3.9.6.3 Effluent Characteristics.

3.9.6.3.1 BOD_5.

$$BODE = (BOD5)\ (1.0 - \frac{BODR}{100})$$

If BODE < BOD5S then BODE = BOD5S

where

BODE = effluent BOD_5, mg/l.

BODR = BOD_5 removal rate, %.

BOD5 = influent BOD_5, mg/l.

BOD5S = effluent soluble BOD_5, mg/l.

3.9.6.3.2 COD.

$$CODE = (COD)\ (1 - \frac{CODR}{100})$$

If CODE < CODS then CODE = CODS

where

CODE = effluent COD concentration, mg/l.

COD = influent COD concentration, mg/l.

CODR = COD removal rate, %.

CODS = effluent soluble COD concentration, mg/l.

3.9.6.3.3 Phosphorus.

$$PO4E = PO4\ (1.0 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

3.9.6.3.4 Suspended Solids.

$$SSE = SSI\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SSI = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

3.9.6.3.5 Nitrogen.

$$TKNE = (TKN)\ (1 - \frac{TKNR}{100})$$

If TKNE < NH3 then TKN = NH3

where

TKNE = effluent TKN concentration, mg/l.

TKN = influent TKN concentration, mg/l.

TKNR = TKN removal rate, %.

NH3 = effluent ammonia concentration.

3.9.6.3.5 Oil and Grease.

$$OAG = 0.0$$

where

OAG = effluent oil and grease concentration, mg/l.

3.9.6.3.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = settleable solids concentration, mg/l.

3.9.6.4 Process Design Output Data

3.9.6.4.1 Coagulant used.

3.9.6.4.2 Coagulant dosage, CD, mg/l.

3.9.6.4.3 Coagulant requirement, CR, lb/day.

3.9.6.4.4 Sludge production, X, lb/day.

3.9.6.4.5 Design overflow rate, Q_{of}, gpd/ft^2.

3.9.6.4.6 Diameter of flocculator-clarifier, DIA, ft.

3.9.6.4.7 Side water depth of flocculator-clarifier, SWD, ft.

3.9.6.4.8 Number of units, N.

3.9.6.5 <u>Quantities Calculations</u>.

3.9.6.5.1 Determine quantity of earthwork required.

$$V_{ew} = (1.15)\ (N)\ [0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = volume of earthwork required, cu ft.

DIA = diameter of flocculator-clarifier, ft.

N = number of units.

3.9.6.5.2 Calculate reinforced concrete required.

3.9.6.5.2.1 Calculate R.C. slab required.

$$V_{cs} = (N)\ (0.825)\ (DIA + 4)^2\ \left(\frac{t_s}{12}\right)$$

$$T_s = 7.9 + 0.25\ SWD$$

where

V_{cs} = volume of R.C. slab required, cu ft.

DIA = diameter of flocculator-clarifier, ft.

t_s = thickness of slab, inches.

SWD = side water depth of flocculator-clarifier, ft.

N = number of units.

3.9.6.5.2.2 Calculate R.C. wall required.

$$V_{cw} = (3.14)\ (N)\ (SWD + 1.5)\ (DIA)\ \frac{t_w}{12}$$

$$t_w = 7 + (0.5)\ (SWD)$$

where

V_{cw} = volume of R.C. wall required, cu ft.

SWD = side water depth of flocculator-clarifier, ft.

DIA = diameter of flocculator-clarifier, ft.

t_w = wall thickness, inches.

N = number of units.

3.9.6.5.3 Determine electrical energy requirement.

3.9.6.5.3.1 Calculate mechanism horsepower. The mechanism horsepower is a function of the diameter of the unit.

$$\text{If } DIA \leq 20 \text{ ft.}$$
$$MHP = 1.0$$

$$\text{If } 20 < DIA \leq 90$$

$$MHP = 0.0053\ (DIA)^{1.75}$$

$$\text{If } DIA > 90$$

$$MHP = 7.62\ (DIA)^{0.1351}$$

where

MHP = mechanism horsepower, hp.

DIA = diameter of flocculator-clarifier, ft.

3.9.6.5.3.2 Calculate electric energy required for operation.

$$KWH = (6701)\ (N)\ (MHP)$$

where

KWH = electric energy requirement, kw hr/yr.

MHP = mechanism horsepower, hp.

N = number of units.

3.9.6.5.4 Operation and maintenance manpower requirement.

3.9.6.5.4.1 Operation manpower requirement.

$$\text{If } Q_{avg} \leq 0.1 \text{ mgd}$$

$$OMH = 304$$

$$\text{If } Q_{avg} > 0.1 \text{ mgd}$$

$$OMH = 1093.8\ (Q_{avg})^{0.5561}$$

where

OMH = operation manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.9.6.5.4.2 Maintenance manpower required.

If $Q_{avg} \leq 0.1$ mgd

$$MMH = 128$$

If $Q_{avg} > 0.1$ mgd

$$MMH = 474.8 \ (Q_{avg})^{0.5693}$$

3.9.6.5.5 Operation and maintenance material and supply cost. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of the total bare construction cost of the coagulation system.

$$OMMP = 1.0\%$$

where

$OMMP$ = O&M material and supply costs as percent of the total bare construction cost, %.

3.9.6.5.6 Calculate chemical quantity used.

3.9.6.5.6.1 When lime is used as coagulant.

$$LIME = (CR)\ (365)$$

where

$LIME$ = quantity of lime required per year, lb/yr.

CR = coagulant requirement, lb/day.

3.9.6.5.6.2 When alum is used as coagulant.

$$ALUM = (CR)\ (365)$$

where

$ALUM$ = quantity of ALUM required per year, lb/yr.

CR = coagulant requirement, lb/day.

3.9.6.5.6.3 When ferric chloride is used as coagulant.

$$IRON = (CR)\ (365)$$

where

IRON = quantity of ferric chloride required per year, lb/yr.

CR = coagulant requirement, lb/day.

3.9.6.5.7 Other construction cost items. The calculations account for approximately 85% of the construction cost of the system. The remaining 15% covers other minor costs, such as piping, control panel, painting, etc.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor.

3.9.6.6 Quantities Calculations Output Data.

3.9.6.6.1 Volume of earthwork required, V_{ew}, cu ft.

3.9.6.6.2 Volume of R.C. slab required, V_{cs}, cu ft.

3.9.6.6.3 Volume of R.C. wall required, V_{cw}, cu ft.

3.9.6.6.4 Electric energy requirement, KWH, kw hr/yr.

3.9.6.6.5 Operation manpower required, OMH, MH/yr.

3.9.6.6.6 Maintenance manpower required, MMH, MH/yr.

3.9.6.6.7 Quantity of lime required per year, LIME, lb/yr.

3.9.6.6.8 Quantity of alum required per year, ALUM, lb/yr.

3.9.6.6.9 Quantity of ferric chloride required per year, IRON, lb/yr.

3.9.6.6.10 O&M material and supply costs as percent of the total bare construction cost, OMMP, %.

3.9.6.6.11 Correction factor, CF.

3.9.6.7 Unit Price Inputs Required.

3.9.6.7.1 Cost of earthwork, UPIEX, $/cu yd.

3.9.6.7.2 Cost of R.C. wall in-place, UPICW, $/cu yd.

3.9.6.7.3 Cost of R.C. slab in-place, UPICS, $/cu yd.

3.9.6.7.4 Standard size flocculator-clarifier mechanism (60-feet diameter) cost, COSTCL, $ (optional).

3.9.6.7.5 Marshall and Swift Equipment Cost Index, MSECI.

3.9.6.7.6 Equipment installation labor rate, \$/MH.

3.9.6.7.7 Crane rental rate, UPICR, \$/hr.

3.9.6.8 Cost Calculations.

3.9.6.8.1 Calculate cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} \text{ (UPIEX)}$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.9.6.8.2 Calculate cost of concrete wall.

$$COSTCW = \frac{V_{cw}}{27} \text{ (UPICW)}$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. Wall required, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.9.6.8.3 Calculate cost of concrete slab.

$$COSTCS = \frac{V_{cs}}{27} \text{ (UPICS)}$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = volume of R.C. slab required, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

3.9.6.8.4 Cost of installed equipment.

3.9.6.8.4.1 Purchase cost of clarifier mechanism. The purchase cost of mechanisms can be obtained by using the following equation:

$$COSTCM = COSTCL \times COSTRO$$

where

COSTCM = purchase cost of mechanism with diameter DIA feet, \$.

COSTCL = purchase cost of standard size mechanism with diameter of 60 ft.

COSTRO = ratio of cost of mechanism with diameter of DIA, feet and the cost of standard size clarifier.

3.9.6.8.4.2 COSTRO. The cost ratio can be expressed as:

$$COSTRO = (0.0164) \times DIA$$

3.9.6.8.4.3 Cost of standard size mecahnism. The cost of mechanism for a 60-foot diameter upflow clarifier for the first quarter of 1977 is:

$$COSTCL = \$110,000$$

For better estimate, COSTCL should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift equipment cost index.

$$COSTCL = 110,000 \times \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index from input.

491.6 = Marshall and Swift cost index 1st quarter of 1977.

3.9.6.8.4.4 Equipment installation man-hour requirement. The man-hour requirement for field erection of clarifier mechanism can be estimated as:

$$IMH = 200 + (25) \times (DIA)$$

where

IMH = installation man-hour requirement, man-hours.

3.9.6.8.4.5 Crane requirement for installation, hr.

$$CH = (0.1)\ (IMH)$$

CH = crane time requirement for installation, hr.

3.9.6.8.4.6 Other minor costs associated with the installed equipment. This category includes the costs for electric wiring, piping, painting, etc., and can be added as percent of purchased equipment cost.

$$PMINC = 5\%$$

where

PMINC = percentage of purchasing costs of equipment as minor costs, %.

3.9.6.8.4.7 Installed equipment costs, IEC.

$$IEC = [COSTCM\ (1 + \frac{PMINC}{100}) + IMH \times LABRI + CH \times UPICR] \times N$$

where

IEC = installed equipment costs, \$.

LABRI = labor rate, \$/MH.

UPICR = crane rental rate, \$/yr.

3.9.6.8.5 Other cost items. This category includes costs of piping, walkways, electrical control instruments, site work, etc. Costs can be adjusted by multiplying the correction factor CF by the sum of other costs.

3.9.6.8.6 Total bare construction costs, TBCC, \$.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC) \times CF$$

where

TBCC = total bare construction costs, \$.

CF = correction factor for minor cost items.

3.9.6.8.7 Operation and maintenance material costs. Since this item of the operation and maintenance costs is expressed as a percentage of the total bare construction costs, it can be calculated by:

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, \$/yr.

OMMP = percentage of the total bare construction costs as operation and maintenance material cost, %.

3.9.6.9 Cost Calculations Output Data.

3.9.6.9.1 Total bare construction costs of the chemical coagulation system TBCC, $.

3.9.6.9.2 O&M material and supply cost, OMMC, $.

3.9.7 **Dissolved Air Flotation.**

3.9.7.1 **Input Data.**

3.9.7.1.1 Wastewater flow, mgd.

3.9.7.1.2 Suspended solids concentration in the feed, mg/l.

3.9.7.1.2.1 Average concentration.

3.9.7.1.2.2 Variation in concentration.

3.9.7.1.3 Chemical dosage, mg/l.

3.9.7.2 **Design Parameters.** From laboratory or pilot plant studies.

3.9.7.2.1 Air-to-solid ratio, A/S.

3.9.7.2.2 Air pressure, P, psig.

3.9.7.2.3 Detention time in flotation tank, DTFT, hr.

3.9.7.2.4 Hydraulic loading, HL, gpm/ft^2.

3.9.7.2.5 Detention time in pressure tank, DTPT, min.

3.9.7.2.6 Float concentration, C_F, percent.

3.9.7.3 **Process Design Calculations.**

3.9.7.3.1 Select air-to-solid ratio. A/S between .01 to .03 use .02 if none specified.

3.9.7.3.2 Assume pressure (40 to 60 psig).

3.9.7.3.3 Calculate P in atmosphere $= \frac{\text{psig} + 14.7}{14.7}$

3.9.7.3.4 Calculate recycle flow.

$$\frac{A}{S} = \frac{1.3S_a(0.5P - 1)R}{QC_o}$$

where

A/S = air-to-solid ratio.

S_a = air solubility at standard conditions, cc/l.

P = absolute pressure, atmospheres.

R = recycle flow, mgd.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

3.9.7.3.5 Select a hydraulic loading rate and calculate surface area.

$$SA = \frac{(Q + R)(10^6)}{(HL)(60)(24)}$$

where

SA = surface area, ft^2.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

R = recycle flow, mgd.

HL = hydraulic loading rate, gpm ft^2. HL is between 1 and 4 gpm/ft^2, if no HL selected use 2.0 gpm/ft^2.

3.9.7.3.6 Select detention time in the flotation tank and calculate the volume.

$$VOLFT = (Q + R) \times \left(\frac{1}{7.48}\right)\left(\frac{1}{24}\right) (DTFT) (10^6)$$

where

VOLFT = volume of flotation tank, ft^3.

Q = feed flow, mgd.

R = recycle flow, mgd.

DTFT = detention time in flotation tank, hr.

3.9.7.3.7 Select pressure tank detention time and calculate volume of pressure tank.

$$VOLPT = (R)\left(\frac{1}{7.48}\right)\left(\frac{1}{24}\right)\left(\frac{1}{60}\right) (DTPT) (10^6)$$

where

VOLPT = volume of pressure tank, ft^3.

R = recycle flow, mgd.

DTPT = detention time in pressure tank, min.

3.9.7.3.8 Calculate volume of sludge.

$$VS = \frac{(Q)(C_o)(\%\ removal)}{(C_F)(specific\ gravity)}$$

where

VS = volume of sludge, gpd.

Q = feed flow, mgd.

C_o = influent suspended solids concentration, mg/l.

C_F = solids concentration in float, percent.

3.9.7.3.9 Calculate chemical usage (if applicable).

$$CU + (CD)\ (Q)\ 8.34$$

where

CU = chemical usage, lb/day.

CD = chemical dosage, mg/l.

Q = feed flow, mgd.

3.9.7.3.10 Effluent Characteristics.

3.9.7.3.10.1 Suspended Solids.

$$SSE = Co\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

Co = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

3.9.7.3.10.2 BOD_5.

$$BODE = BOD\ (1 - \frac{BODR}{100})$$

If BODE < BODSE set BODE = BODSE

where

BODE = effluent BOD_5 concentration, mg/l.

BODSE = effluent soluble BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, mg/l.

BOD = influent BOD_5 concentration, mg/l.

3.9.7.3.10.3 COD.

$$CODE = COD\ (1 - \frac{CODR}{100})$$

If CODE < CODSE set CODE = CODSE

where

CODE = effluent COD concentration, mg/l.

COD = influent COD concentration, mg/l.

CODR = COD removal rate, %.

CODSE = effluent soluble COD concentration, mg/l.

3.9.7.3.10.4 Nitrogen.

$$TKNE = TKN\ (1 - \frac{TKNR}{100})$$

$$NH3E = NH3$$
$$NO3E = NO3$$
$$NO2E = NO2$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNR = Kjedahl nitrogen removal rate, %.

NH3E = effluent ammonia concentration, mg/l.

NH3 = influent ammonia concentration, mg/l.

NO3E = effluent NO_3 concentration, mg/l.

NO3 = influent NO_3 concentration, mg/l.

NO2E = effluent NO_2 concentration, mg/l.

NO2 = influent NO_2 concentration, mg/l.

3.9.7.3.10.5 Phosphorus.

$$PO4E = PO4 \left(1 - \frac{PO4R}{100}\right)$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, mg/l.

3.9.7.3.10.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

3.9.7.3.10.7 Oil and Grease.

$$OAGE = (0.05)\ (OAG)$$

where

OAGE = effluent oil and grease concentration, mg/l.

OAG = influent oil and grease concentration, mg/l.

3.9.7.4 Process Design Output Data.

3.9.7.4.1 Suspended solids concentration, C_o, mg/l.

3.9.7.4.2 Air-to-solid ratio, A/S.

3.9.7.4.3 Air pressure, P, psig.

3.9.7.4.4 Hydraulic loading, HL, gpm/ft^2.

3.9.7.4.5 Recycle flow, R, mgd.

3.9.7.4.6 Surface area, SA, ft^2.

3.9.7.4.7 Volume of pressure tank, VOLPT, ft^3.

3.9.7.4.8 Volume of flotation tank, VOLFT, ft^3.

3.9.7.4.9 Pressure tank detention time, DTPT, min.

3.9.7.4.10 Flotation tank detention time, DTFT, hr.

3.9.7.4.11 Chemical usage, CU, lb/day.

3.9.7.5 <u>Quantities Calculations</u>.

3.9.7.5.1 Flotation units are usually sized based on surface area. The smaller units from 40 sq ft to 100 sq ft are usually supplied in one piece already assembled. Large units up to 1250 sq ft are supplied with all components to be installed in a concrete tank supplied by the contractor. This equipment includes the basic mechanism, air saturation tank, pressurization pump, air compressor, pressure release valve, rotameter, air regulator, and throttling valve. Both circular and rectangular flotation units are available. For purposes of simplification we have chosen to use the circular units. However, it should be noted that circular and rectangular units of the same surface area are comparable in cost.

3.9.7.5.2 Select number and size of flotation units. The standard size units available commercially are 40, 50, 70, 100, 140, 200, 280, 350, 450, 570, 750, 960, and 1250 sq ft.

3.9.7.5.2.1 If SA is less than 1250 sq ft, then NU is one. Compare SA to the commercially available units and select the smallest unit that is larger than SA.

3.9.7.5.2.2 If SA is greater than 1250 sq ft, then NU must be two or greater. Try NU = 2 first, if SA/NU is greater than 1250, then NU = NU + 1. Repeat the procedure until SA/NU is less than 1250, then compare SA/NU to the commercially available units and select the smallest unit that is larger than SA/NU.

where

SA = calculate surface area, sq ft.

NU = number of units.

SAS = surface area of unit selected.

3.9.7.5.3 Calculate building area. In area where freezing weather may be expected, flotation units would normally be enclosed in buildings.

3.9.7.5.3.1 Calculate diameter of unit.

$$DIA = (1.13)\ (SAS)^{0.5}$$

where

DIA = diameter of unit selected, ft.

3.9.7.5.3.2 Calculate building area.

$$A_B = (DIA + 2)\ (DIA + 5)\ (NU)$$

where

A_B = area of building, sq ft.

3.9.7.5.4 Earthwork required for construction. The procedure to estimate the earthwork requirement is the same as that for circular clarifier.

$$V_{ew} = (1.15)\ NU[0.035\ (DIA)^3 + 4.88\ (DIA)^2 + 77\ (DIA) + 350]$$

where

V_{ew} = earthwork required for construction, cu ft.

1.15 = 15 percent excess volume as safety factor.

NU = number of units.

3.9.7.5.5 Reinforced concrete quantities.

3.9.7.5.5.1 Calculate side water depth. The side water depth can be related to the diameter by the following equation:

$$SWD = 6.72 + 0.0476\ (DIA)$$

where

SWD = side water depth, ft.

3.9.7.5.5.2 Calculate the thickness of the slab. The thickness of the slab can be related to the side water depth by the following equation:

$$t_s = 7.9 + 0.25\ SWD$$

where

t_s = thickness of the slab, inches.

3.9.7.5.5.3 Calculate the wall thickness. The wall thickness can be related to the side water depth by the following:

$$t_w = 7 + (0.5)\ SWD$$

where

t_w = wall thickness, inches.

3.9.7.5.5.4 Calculate reinforced concrete slab quantity.

$$V_{cs} = 0.825\ (DIA + 4)^2\ \left(\frac{t_s}{12}\right)\ (NU)$$

where

V_{cs} = quantity of R.C. slab in-place, cu ft.

3.9.7.5.5.5 Calculate reinforced concrete wall quantity.

$$V_{cw} = (3.14)\ (SWD + 1.0)\ (DIA)\ (\frac{t_w}{12})\ (NU)$$

where

V_{cw} = quantity of R.C. wall in-place, cu ft.

3.9.7.5.5.6 Quantity of concrete for splitter box.

$$V_{cb} = 100\ (NU)^{1.13}$$

where

V_{cb} = quantity of R.C. for splitter box, cu ft.

NU = number of units.

3.9.7.5.5.7 Total quantity of R.C.

$$\text{wall: } V_{cwt} = V_{cw} + V_{cb}$$

$$\text{slab: } V_{cst} = V_{cs}$$

where

V_{cwt} = total quantity of R.C. wall in-place, cu ft.

V_{cst} = total quantity of R.C. slab in-place, cu ft.

3.9.7.5.6 Calculate dry solids produced.

$$DSTPD = \frac{(Q)\ (C_o)\ (8.34)}{2000}$$

where

DSTPD = dry solids produced, tpd.

Q = feed flow, mgd.

C_o = suspended solids concentration in the feed, mg/l.

3.9.7.5.7 Calculate operational labor.

3.9.7.5.7.1 If DSTPD is between 0 and 0.1 tpd, the operational labor is calculated by:

$$OMH = 240$$

3.9.7.5.7.2 If DSTPD is greater than 0.1 tpd, the operational labor is calculated by:

$$OMH = 558.4\ (DSTPD)^{0.3667}$$

where

OMH = operation labor, man-hour/yr.

3.9.7.5.8 Calculate maintenance labor.

3.9.7.5.8.1 If DSTPD is between 0 and 0.1 tpd, the maintenance labor is calculated by:

$$MMH = 64.0$$

3.9.7.5.8.2 If DSTPD is greater than 0.1 tpd, the maintenance labor is calculated by:

$$MMH = 160.0\ (DSTPD)^{0.3979}$$

where

MMH = maintenance labor, man-hour/yr.

3.9.7.5.9 Calculate electrical energy requirements for operation.

3.9.7.5.9.1 If DSTPD is between 0 and 0.35 tpd, the energy requirement is:

$$KWH = 23{,}500$$

3.9.7.5.9.2 If DSTPD is greater than 0.35 tpd, the energy is calculated by:

$$KWH = 63{,}000\ (DSTPD)^{0.9422}$$

where

KWH = electrical energy requirement for operation, kwhr/yr.

3.9.7.5.10 Operation and maintenance material costs. This item includes repair and replacement material costs and other minor costs. It is expressed as a percent of total bare construction cost.

$$OMMP = 1\%$$

where

OMMP = percent of air flotation total bare construction cost as operation and maintenance materials cost, percent.

3.9.7.5.11 Other construction cost items.

3.9.7.5.11.1 From the above estimation, approximately 85 percent of the construction costs have been accounted for.

3.9.7.5.11.2 Other minor cost items such as piping, electrical wiring and conduit, concrete slab for pumps and pressure tanks, etc., would be 15 percent.

3.9.7.5.11.3 The correction factor would be:

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for construction costs.

3.9.7.6 Quantities Calculations Output Data.

3.9.7.6.1 Surface area of unit selected, SAS, sq ft.

3.9.7.6.2 Number of units, NU.

3.9.7.6.3 Area of building, A_B, sq ft.

3.9.7.6.4 Earthwork required, V_{ew}, cu ft.

3.9.7.6.5 Total quantity of R.C. wall in-place, V_{cwt}, cu ft.

3.9.7.6.6 Total quantity of R.C. slab in-place, V_{cst}, cu ft.

3.9.7.6.7 Operational labor, OMH, man-hour/yr.

3.9.7.6.8 Maintenance labor, MMH, man-hour/yr.

3.9.7.6.9 Electrical energy requirement for operation, KWH, kwhr/yr.

3.9.7.6.10 Operation and maintenance material costs as percent of air flotation total bare construction cost, percent.

3.9.7.6.11 Correction factor for construction costs, CF.

3.9.7.7 Unit Price Input Required.

3.9.7.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.9.7.7.2 Unit price input for R.C. wall in-place, UPICW, $/cu.

3.9.7.7.3 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

3.9.7.7.4 Cost of standard size flotation equipment, COSTFS, $, (optional).

3.9.7.7.5 Marshall and Swift Equipment Cost Index, MSECI.

3.9.7.8 Cost Calculations.

3.9.7.8.1 Cost of building.

$$COSTB = A_B \times UPIBC\ (.75)$$

where

COSTB = cost of building, \$.

A_B = building area, sq ft.

UPIBC = unit price input for building cost, \$/sq ft.

.75 = correction factor since slab is already accounted for in concrete costs.

3.9.7.8.2 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27}\ UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = quantity of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.9.7.8.3 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cst}}{27}\ UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cwt} = total quantity of R.C. wall in-place, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.9.7.8.4 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cst}}{27}\ UPICS$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cst} = total quantity of R.C. slab in-place, cu ft.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

3.9.7.8.5 Purchase cost of flotation equipment. The costs given include the basic mechanism to be mounted in the concrete tank, air pressurization tank, pressurization pump, press[illegible] release valve, air injection system, and electrical panel.

$$COSTF = COSTFS \times \frac{COSTRO}{100}$$

where

COSTF = purchase cost of flotation equipment of SAS surface area, \$.

COSTFS = cost of standard size air flotation unit of 350 sq ft.

COSTRO = cost of unit of SAS sq ft expressed as percent of cost of standard size unit.

3.9.7.8.6 Calculate COSTRO.

3.9.7.8.6.1 If SAS is less than 240 sq ft, COSTRO is calculated by:

$$COSTRO = 0.3\ (SAS) + 25$$

3.9.7.8.6.2 If SAS is between 240 sq ft and 480 sq ft, COSTRO is calculated by:

$$COSTRO = 0.092\ (SAS) + 75$$

3.9.7.8.6.3 If SAS is greater than 480 sq ft, COSTRO is calculated by:

$$COSTRO = 0.161\ (SAS) + 43$$

3.9.7.8.7 Cost of standard size unit. The cost of a dissolved air flotation unit with 350 sq ft of surface area for the first quarter of 1977 is:

$$COSTFS = \$44{,}200$$

For better cost estimation, COSTFS should be obtained from equipment vendor and treated as a unit price input. Otherwise, for future escalation the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTFS = \$44{,}200\ \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Cost Index from input.

491.6 = Marshall and Swift Cost Index first quarter 1977.

3.9.7.8.8 Equipment installation costs. These costs would include mounting of flotation mechanism in the flotation tank, settling pumps, and tanks, interconnecting piping, electrical installation, etc. These costs are estimated as 75 percent of the purchase cost of equipment.

$$EIC = .75\ COSTF$$

where

EIC = equipment installation costs, \$.

3.9.7.8.9 Installed equipment cost.

$$IEC = (COSTF + EIC)\ NU$$

where

IEC = installed equipment cost, \$.

NU = number of units of area SAS sq ft.

3.9.7.8.10 Total bare construction costs.

$$TBCC = (COSTB + COSTE + COSTCW + COSTCS + IEC)\ CF$$

where

TBCC = total bare construction costs, \$.

CF = construction cost correction factor.

3.9.7.8.11 Operation and maintenance material costs.

$$OMMC = TBCC \times \frac{OMMP}{100}$$

where

OMMC = operation and maintenance material costs, dollars.

3.9.7.9 Cost Calculations Output Data.

3.9.7.9.1 Total bare construction cost for dissolved air flotation unit, TBCC, \$.

3.9.7.9.2 Operation and maintenance material costs, OMMC, \$/yr.

3.9.8 **Gravity Filter.**

3.9.8.1 **Input Data.**

3.9.8.1.1 Wastewater characteristics.

3.9.8.1.1.1 Average daily flow, Q_{avg}, mgd.

3.9.8.1.1.2 Peak flow, Q_p, mgd.

3.9.8.1.2 Influent suspended solids concentration SS, mg/l.

3.9.8.1.3 Hydraulic loading rate, LR, gpm/ft^2.

3.9.8.2 **Design Parameters.**

3.9.8.2.1 Hydraulic loading rate, gpm/ft^2.

3.9.8.2.2 Backwash rate, gpm/ft^2.

3.9.8.3 **Process Design Calculations.**

3.9.8.3.1 Calculate required filter surface area.

3.9.8.3.1.1 If no hydraulic loading rate is specified use 3.5 gpm/ft^2.

$$SA = \frac{(Q_{avg})\ (10^6)}{(1440)\ (LR)}$$

where

SA = filter surface area required, ft^2.

LR = hydraulic loading rate, gpm/ft^2.

3.9.8.3.1.2 Package filter units usually have four (4) filter cells for operational reasons. The Ten State Standards specify that the filtration rate should not exceed 5 gpm/ft^2 during maximum flow with one filter cell out of service. Therefore, the filter surface area should be calculated based on this requirement and checked against the area calculated based on average flow.

$$SAP = \frac{(Q_p)\ (10^6)\ (NC)}{(5)\ (1440)\ (NC-1)}$$

where

SAP = filter surface area based on peak flow, ft^2.

Q_p = Peak flow, mgd.

NC = number of filter cells (use 4 if not specified).

3.9.8.3.1.3 Compare SA and SAP and use the larger of the values as the design filter surface area, SAD.

3.9.8.3.2 Backwash Requirements.

3.9.8.3.2.1 Ten State Standards require that provision be made for a minimum backwash rate of 20 gpm/ft^2 and a minimum backwash period of 10 minutes. Usually, filtered water is used for backwash purposes. It is assumed in this model that the filter unit will be followed by chlorine contact tanks which would supply ample filtered water for backwash purposes, therefore no additional storage will be provided.

3.9.8.3.2.2 Ten State Standards also state that the rate of return of waste filter backwash water to treatment units should not exceed 15 percent of the design flow rate. Surge tanks should be provided with a minimum capacity of twice the backwash water volume required.

3.9.8.3.2.3 Calculate volume of backwash surge tank.

$$V_{bws} = \frac{(SAD)\ (20)\ (10)\ (2)}{(NC)\ (7.48)}$$

where

V_{bws} = volume of backwash surge tank, ft^3.

SAD = design filter surface area, ft^2.

NC = number of cells

20 = backwash flow rate, gpm/ft^2.

10 = backwash period, min.

2 = number of backwash volumes.

7.48 = conversion factor, gal/ft^3.

3.9.8.3.3 Effluent Characteristics.

3.9.8.3.3.1 Suspended Solids.

$$SSE = (0.4)\ (SS)$$

where

SSE = effluent suspended solids concentration, mg/l.

SS = influent suspended solids concentration, mg/l.

3.9.8.3.3.2 BOD_5.

$$BODE = BODS$$

where

BODE = effluent BOD_5 concentration, mg/l.

BODS = influent soluble BOD_5 concentration, mg/l.

3.9.8.3.3.3 COD.

$$CODE = CODS$$

where

CODE = effluent COD concentration, mg/l.

CODS = influent soluble COD concentration, mg/l.

3.9.8.3.3.4 Nitrogen.

$$TKNE = NH3 - NH3E$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3 = influent ammonia concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

3.9.8.3.3.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

3.9.8.3.3.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

3.9.8.4 <u>Process Design Output Data</u>.

3.9.8.4.1 Design filter surface area, SAD, ft^2.

3.9.8.4.2 Number of filter cells, NC.

3.9.8.4.3 Volume of backwash surge tank, ft^3.

3.9.8.5 Quantities Calculations.

3.9.8.5.1 Calculate reinforced concrete slab for package filter. The concrete slab is assumed to be 1 ft thick.

$$V_{cs} = 1.434\ (SAD) + 32.8$$

where

V_{cs} = volume of R.C. slab for filter unit, ft^3.

SAD = design filter surface area.

3.9.8.5.2 Backwash surge tank design. The backwash water would be drained from the bottom of the filter unit into the surge tank. It is assumed that the surge tank would be a concrete structure built below grade.

Assumptions:

Water depth in surge tank is 7 ft.
Freeboard of 2 ft.
Length to width ratio of 2.
Concrete is 1 ft thick.
Excavation side slope is 1 to 1.
Slab extends 1-1/2 ft past wall of tank.

3.9.8.5.2.1 Calculate dimensions of surge tank.

$$W = \left(\frac{V_{bws}}{14}\right)^{0.5}$$

$$L = 2W$$

where

V_{bws} = Volume of backwash surge tank, ft^3.

W = width of backwash surge tank, ft.

L = length of backwash surge tank, ft.

3.9.8.5.2.2 Calculate earthwork required.

$$V_{ew} = 10/3\ (A_1 + A_2 + \sqrt{A_1 \times A_2})$$

$$A_1 = (L + 5)\ (W + 5)$$

$$A_2 = (L + 25)\ (W + 25)$$

where

V_{ew} = volume of earthwork required, ft^3.

A_1 = area at bottom of excavation.

A_2 = area of ground level.

3.9.8.5.2.3 Volume of concrete required. Consider all concrete as wall concrete because of forming required.

$$V_{cw} = 2W^2 + 75W + 25$$

where

V_{cw} = volume of R.C. wall required, ft^3.

W = width of backwash surge tank, ft.

3.9.8.5.3 Electric energy required for operation. The major energy user in filtration is pumping of the main stream of wastewater. Other energy requirements are power for backwashing, surface sprays, and air blowers. The energy for pumping of the main wastewater stream is included in the section entitled "Intermediate Pumping". The following equations gives energy requirements for filter backwash, surface spray and air blowers.

$$KWH = (8213)\ (Q_{avg})^{0.972}$$

where

KWH = electrical energy required, kw hr/yr.

Q_{avg} = average daily flow, mgd.

3.9.8.5.4 Operation and maintenance manpower requirement.

3.9.8.5.4.1 Maintenance manpower required,

$$\text{If } Q_{avg} \leq .1 \text{ mgd}$$

$$MMH = 112$$

$$\text{If } Q_{avg} > .1 \text{ mgd}$$

$$MMH = 432\ (Q_{avg})^{0.585}$$

where

MMH = maintenance manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.9.8.5.4.2 Operation manpower required.

$$\text{If } Q_{avg} \leq .1 \text{ mgd}$$

$$OMH = 176$$

$$\text{If } Q_{avg} > .1 \text{ mgd}$$

$$OMH = 643.2 \, (Q_{avg})^{0.572}$$

where

OMH = operation manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.9.8.5.5 Operation and maintenance material and supply costs.

$$OMMP = 5\%$$

where

$OMMP$ = O&M material and supply costs as percent of installed package filtration equipment.

3.9.8.5.6 Replacement Schedule.

3.9.8.5.6.1 Equipment. The service life of the equipment is approximately 20 yrs.

$$RSE = 20$$

where

RSE = replacement schedule for equipment, yrs.

3.9.8.5.7 Other construction cost items. The preceding calculations account for approximately 80 percent of the construction cost. The remaining 20 percent would include costs for items such as backwash auxiliary supply pumps, piping, housing, etc.

$$CF = \frac{1}{0.8} = 1.25$$

where

CF = correction factor for other construction costs.

3.9.8.6 Quantities Calculations Output Data.

3.9.8.6.1 Design filter surface area, SAD, ft^2.

3.9.8.6.2 Volume of R.C. slab for filter unit, V_{cs}, ft^3.

3.9.8.6.3 Volume of earthwork required, V_{ew}, ft^3.

3.9.8.6.4 Volume of R.C. wall required, V_{cw}, ft^3.

3.9.8.6.5 Electrical energy requirement, KWH, kw hr/yr.

3.9.8.6.6 Maintenance manpower required, MMH, MH/yr.

3.9.8.6.7 Operation manpower required, OMH, MH/yr.

3.9.8.6.8 O&M material and supply costs as percent of installed package filtration equipment, OMMP, %.

3.9.8.6.9 Correction factor for other construction costs, CF.

3.9.8.7 Unit Price Input Required.

3.9.8.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.9.8.7.2 Unit price input for R.C. wall in-place, UPICW, $/cu yd.

3.9.8.7.3 Unit price input for R.C. slab in-place, UPICS, $/cu yd.

3.9.8.7.4 Cost of standard size filter, COSTSF, $ (optional).

3.9.8.7.5 Marshall and Swift Equipment Cost Index, MSECI.

3.9.8.8 Cost Calculations.

3.9.8.8.1 Calculate cost for earthwork.

$$COSTE = \frac{(V_{ew})}{27} (UPIEX)$$

where

COSTE = cost of earthwork, $.

V_{ew} = volume of earthwork required, ft^3.

UPIEX = unit price input for earthwork, $/cu yd.

3.9.8.8.2 Calculate cost of reinforced concrete.

3.9.8.8.2.1 Cost of R.C. slab in-place.

$$COSTCS = \frac{V_{cs}}{27} (UPICS)$$

where

COSTCS = cost of R.C. slab in-place, $.

V_{cs} = volume of R.C. slab required for filter unit, ft^3.

UPICS = unit price input for R.C. slab in-place, $/cu yd.

3.9.8.8.2.2 Cost of R.C. wall in-place.

$$COSTCW = \frac{V_{cw}}{27} UPICW$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = volume of R.C. wall required, ft^3.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.9.8.8.2.3 Total cost of reinforced concrete.

$$COSTC = COSTCW + COSTCS$$

where

COSTC = total cost of reinforced concrete, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

3.9.8.8.3 Calculate installed cost package multimedia filter unit.

3.9.8.8.3.1 Calculate COSTF.

$$COSTF = \frac{COSTRO}{100} (COSTSF)$$

where

COSTF = cost of package filter of size SAD, \$.

COSTRO = cost of package filter of size SAD as percent of cost of standard size filter (120 sq ft), %.

COSTSF = cost of standard size filter (120 sq ft), \$.

3.9.8.8.3.2 Calculate COSTRO.

$$\text{If } SAD \leq 175 \text{ ft}^2$$

$$COSTRO = 65\ (SAD)^{0.0898}$$

$$\text{If } SAD > 175 \text{ ft}^2$$

$$COSTRO = 14.5\ (SAD)^{0.381}$$

where

COSTRO = cost of package filter of size SAD as percent of cost of standard size filter (120 sq ft), %.

SAD = design filter surface area, ft^2.

3.9.8.8.3.3 Determine COSTSF. COSTSF is the cost of a 120 sq ft package multimedia filter unit. These units normally are complete including blowers for air wash, backwash pumps, underdrain system, backwash troughs, filter media, control panel, valving and piping connecting cells, and steel tanks. The general contractor is usually required to furnish the concrete slab, backwash surge tank, and pumping of main waste stream. The cost of this unit for the first quarter of 1977 is:

$$COSTSF = \$73,760$$

For better cost estimation COSTSF should be obtained from a vendor and treated as a unit price input. If this is not done the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTSF = \$73,760\ \frac{MSECI}{491.6}$$

where

MSECI = the current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index for 1st quarter of 1977.

3.9.8.8.3.4 Calculate installed filter cost.

$$IFC = (1.5)\ (COSTF)$$

where

IFC = installed filter cost, \$.

COSTF = cost of package filter of size SAD, \$.

3.9.8.8.4 Calculate total bare construction cost.

$$TBCC = (COSTE + COSTC + IFC)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

COSTC = total cost of reinforced concrete, \$.

IFC = installed filter cost, \$.

CF = correction factor for other construction cost items.

3.9.8.8.5 Calculate O&M material and supply costs.

$$OMMC = (IFC)\ \frac{OMMP}{100}$$

where

OMMC = O&M material and supply costs, \$/yr.

OMMP = O&M material and supply costs as percent of installed filter cost, %.

IFC = install filter cost, \$.

3.9.8.9 Cost Calculations Output Data.

3.9.8.9.1 Total bare construction cost, TBCC, \$.

3.9.8.9.2 O&M material and supply costs, OMMC, \$.

3.9.9 Intemittant Sand Filtration.

3.9.9.1 Input Data.

3.9.9.1.1 Average daily flow.

3.9.9.1.2 Wastewater characteristics.

3.9.9.1.2.1 BOD_5 (total and soluble).

3.9.9.1.2.2 Suspended solids.

3.9.9.1.3 Hydraulic loading rate. (200,000-800,000 gpad) default 400,000.

3.9.9.1.4 Filter bed depth. (24 to 36 inches) Default 30 inches.

3.9.9.2 Design Parameters.

3.9.9.2.1 Hydraulic loading rate, gpad.

3.9.9.2.2 Filter bed depth, inches.

3.9.9.3 Process Design Calculations.

3.9.9.3.1 The following assumptions are made.

3.9.9.3.1.1 In this flow range three filter beds will always be used.

3.9.9.3.1.2 The filter bed levees will be earthen.

3.9.9.3.1.3 The underdrain system will be perforated PVC pipe.

3.9.9.3.2 Calculate filter area required.

$$FA = \frac{Q_{avg} (10^6)}{HLR}$$

where

FA = filter area, acres.

Q_{avg} = average daily flow, mgd.

HLR = hydraulic loading rate, gpad.

3.9.9.3.3 Calculate area per filter bed.

$$AFB = \frac{(FA)\ (43560)}{3}$$

where

AFB = area per filter bed, ft^2.

FA = filter area, acres.

43560 = conversion acres to ft^2.

3 = number of filter beds.

3.9.9.3.4 Estimate effluent quality.

3.9.9.3.4.1 BOD_5 concentration. The effluent BOD_5 concentration does vary slightly with hydraulic loading rate, size of sand, and depth of sand. However, the effluent can be estimated by the following average removal efficiency.

$$BODE = (1.0-0.7)\ (BOD)$$

If BODE < BODS, set BODE = BODS

where

BODE = effluent BOD_5 concentration, mg/l.

0.7 = average removal.

BOD = influent BOD_5 concentration, mg/l.

BODS = influent soluble BOD_5 concentration, mg/l.

3.9.9.3.4.2 Suspended solids concentration. The effluent suspended solids concentration from a well designed and operated filter will be approximately 40% of the influent suspended solids concentration.

$$SSE = (0.4)\ (SS)$$

where

SSE = effluent suspended solids concentration, mg/l.

SS = influent suspended solids concentration, mg/l.

3.9.9.3.4.3 COD.

$$CODE = (0.3)\ (COD)$$

If CODE < CODS, set CODE = CODS

where

CODE = effluent COD concentration, mg/l.

COD = influent COD concentration, mg/l.

CODS = influent soluble COD concentration, mg/l.

3.9.9.3.4.4 Nitrogen.

$$TKNE = NH3 = NH3E$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

NH3 = influent ammonia concentration, mg/l.

NH3E = effluent ammonia concentration, mg/l.

3.9.9.3.4.5 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/l.

3.9.9.3.4.6 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/l.

3.9.9.4 <u>Process Design Output Data</u>.

3.9.9.4.1 Average daily flow, Q_{avg}, mgd.

3.9.9.4.2 Hydraulic loading rate, HLR, gpad.

3.9.9.4.3 Filter bed depth, FBD, inches.

3.9.9.4.4 Area per filter bed, AFB, ft^2.

3.9.9.4.5 Effluent BOD_5 concentration, BOD_e, mg/l.

3.9.9.4.6 Effluent suspended solids concentration, SS_e, mg/l.

3.9.9.5 Quantities Calculations.

3.9.9.5.1 The following assumptions are made concerning the construction of the filter beds.

3.9.9.5.1.1 The beds will be square.

3.9.9.5.1.2 The levees will have 3 to 1 side slopes.

3.9.9.5.1.3 The top of the levees will be 10 ft wide for maintenance and access.

3.9.9.5.1.4 The levees will extend 2 ft above the top of the sand media.

3.9.9.5.1.5 The filter beds will be constructed using equal cut and fill.

3.9.9.5.1.6 The underdrain pipes will be 4" ϕ perforated PVC pipe.

3.9.9.5.1.7 Underdrains will be spaced 10 ft apart.

3.9.9.5.1.8 Gravel bedding 1 ft deep will be placed around and over the underdrain pipes to prevent sand from entering the underdrain system.

3.9.9.5.2 Calculate filter bed dimensions.

3.9.9.5.2.1 Calculate the length of filter bed at the top of sand.

$$L_s = (AFB)^{0.5}$$

where

L_s = length of filter bed at the top of sand, ft.

AFB = area per filter bed, ft^2.

3.9.9.5.2.2 Calculate length at top of levee.

$$L_T = L_s + 12$$

where

L_T = length of filter bed at the top of levee, ft.

L_s = length of filter bed at the top of sand, ft.

12 = additional length due to 3 to 1 side slope.

3.9.9.5.3 Calculate volume of earthwork required. The volume of earthwork required must be calculated by trial and error. Assume a depth of cut (DC) of 1 ft.

3.9.9.5.3.1 Calculate the length of filter bed at original ground level.

$$L_c = L_s - 6\left(\frac{FBD}{12} + 1 - DC\right)$$

where

L_c = length of filter bed at original ground level, ft.

L_s = length of filter bed at top of sand, ft.

FBD = filter bed depth, inches.

1 = one foot depth for underdrain system.

DC = depth of cut, ft.

3.9.9.5.3.2 Calculate the volume of cut.

$$V_c = (1.3)\ (3)\ (DC)\ [(L_c)^2 - 6(DC)\ (L_c) + 12(DC)^2]$$

where

V_c = volume of cut, cu ft.

3 = number of filter beds.

DC = depth of cut, ft.

L_c = length of filter bed at original ground level, ft.

3.9.9.5.3.3 Calculate the depth of fill.

$$DF = \frac{FBD}{12} + 3 - DC$$

where

DF = depth of fill, ft.

FBD = filter bed depth, inches.

DC = depth of cut, ft.

3 = additional depth for underdrain and freeboard, ft.

3.9.9.5.3.4 Calculate volume of fill.

$$V_F = [10DF + 3(DF)^2] (L_T) (10)$$

where

V_F = volume of fill, cu ft.

DF = depth of fill, ft.

L_T = length of filter bed at the top of levee, ft.

10 = number of levees of length, L_T.

3.9.9.5.3.5 Compare V_c and V_F.

If $V_c < V_F$ then assume DC > 1 ft and recalculate V_c and V_F.

If $V_c > V_F$ then assume DC < 1 ft and recalculate V_c and V_F.

Repeat this procedure until $V_c = V_F$. This is the volume of earthwork required.

$$V_c = V_F = VEW$$

where

V_c = volume of cut, cu ft.

V_F = volume of fill, cu ft.

VEW = volume of earthwork required, cu ft.

3.9.9.5.4 Calculate volume of filter sand required.

$$VS = (3) \left(\frac{FBD}{12}\right) [(L_s)^2 - 6 \left(\frac{FBD}{12}\right) (L_s) + 12 \left(\frac{FBD}{12}\right)^2]$$

where

VS = volume of sand required, cu ft.

3 = number of filter beds.

FBD = filter bed depth, inches.

L_s = length of filter bed at top of sand, ft.

3.9.9.5.5 Calculate volume of gravel media required.

3.9.9.5.5.1 Calculate the length of filter bed at top of gravel media.

$$L_G = L_s - 6\ (\frac{FBD}{12})$$

where

L_G = length of bed at the top of gravel media, ft.

L_s = length of bed at the top of sand, ft.

FBD = filter bed depth, inches.

3.9.9.5.5.2 Calculate the volume of gravel media required.

$$V_{GL} = [(L_G)^2 - 6\ L_G + 12]\ 3$$

where

V_{GL} = volume of gravel media required, cu ft.

L_G = length of bed at the top of gravel media, ft.

3 = number of filter beds.

3.9.9.5.6 Calculate quantity of drain pipe required. It is assumed that 4 inch diameter perforated PVC pipe will be used.

$$LDP = \frac{L_c - 6(DC+1)}{10} + 1 \quad (L_T)\ (3)$$

where

LDP = length of drain pipe, ft.

L_c = length of filter bed at the original ground level, ft.

DC = depth of cut, ft.

L_T = length of filter bed at the top of the levee, ft.

3 = number of beds.

10 = spacing of drain pipe, ft.

3.9.9.5.7 Calculate size and quantity of influent pipe. The flow for dosing the sand filters should not exceed 1 cubic foot per second for every 5000 square feet of bed area.

3.9.9.5.7.1 Calculate flowrate.

$$Q_d = \frac{(AFB)\ (7.48)\ (60)}{5000}$$

where

Q_d = flowrate to beds, gpm.

AFB = area per filter bed, sq ft.

3.9.9.5.7.2 Calculate pipe size. Assume a 5 ft/sec velocity.

$$D = 0.286\ (Q_d)^{0.5}$$

D must be one of the following 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

where

D = influent pipe size, inches.

Q_d = flow rate to beds, gpm.

3.9.9.5.7.3 Calculate length of pipe required.

$$LP = 3[L_T + 10 + (3)\ (DF)]$$

where

LP = length of influent pipe required, ft.

3 = number of filter beds.

L_T = length of filter bed at the top of levee, ft.

DF = depth of fill, ft.

3.9.9.5.8 Calculate operation and maintenance manpower.

3.9.9.5.8.1 Operation manpower.

$$OMH = 1128.7\ (Q_{avg})^{1.016}$$

where

OMH = operation manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.9.5.8.2 Maintenance manpower.

$$MMH = 1128.7\ (Q_{avg})^{1.016}$$

where

MMH = maintenance manpower required, MH/yr.

Q_{avg} = average daily flow, mgd.

3.9.9.5.9 Operation and maintenance material and supply cost. The O&M material and supply required for the intemittant sand filter is loss of sand filter media from cleaning.

$$OMMP = 15\%$$

where

$OMMP$ = O&M material and supply costs as percent of sand media cost.

3.9.9.5.10 Other construction cost items. The preceding calculations account for approximately 90% of the construction cost. The remaining 10% would include items such as miscellaneous concrete structures, drain collection pipe, valves, etc.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction cost.

3.9.9.6 <u>Quantities Calculation Output Data.</u>

3.9.9.6.1 Length of filter bed at the top of levee, L_T, ft.

3.9.9.6.2 Volume of earthwork required, VEW, cu ft.

3.9.9.6.3 Volume of sand media required, VS, cu ft.

3.9.9.6.4 Volume of gravel media required, V_{GL}, cu ft.

3.9.9.6.5 Length of drain pipe required, LDP, ft.

3.9.9.6.6 Influent pipe size, D, inches.

3.9.9.6.7 Length of influent pipe required, LP, ft.

3.9.9.6.8 Operation manpower required, OMH, MH/yr.

3.9.9.6.9 Maintenance manpower required, MMH, MH/yr.

AD-A115 314 PROCESS DESIGN AND COST ESTIMATING ALGORITHMS FOR THE COMPUTER ASSISTED P..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS R W HARRIS ET AL. JAN 82 17/18

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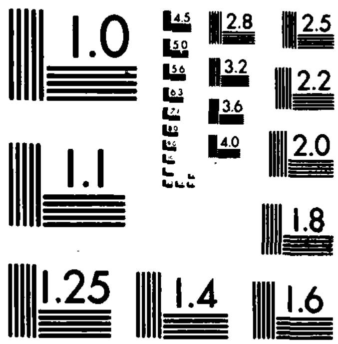

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

3.9.9.6.10 O&M material and supply costs as percent of sand media cost, OMMP, %.

3.9.9.6.11 Correction factor for other construction cost items, CF.

3.9.9.7 Unit Price Input Required.

3.9.9.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

3.9.9.7.2 Unit price input for sand media installed COSAND, \$/cu yd.

3.9.9.7.3 Unit price input for gravel media installed, COGRVL, \$/cu yd.

3.9.9.7.4 Cost of 6" Ø perforated clay pipe installed, COSTCP, \$/ft.

3.9.9.7.5 Cost of standard size pipe (12" diameter), \$/ft.

3.9.9.8 Cost Calculations.

3.9.9.8.1 Calculate cost of earthwork.

$$COSTE = \frac{VEW}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

VEW = volume of earthwork, \$/cu yd.

UPIEX = unit price input for earthwork.

3.9.9.8.2 Calculate cost of sand media.

$$COSM = \frac{VS}{27} (COSAND)$$

where

COSM = cost of sand media installed, \$.

VS = volume of sand required, cu ft.

COSAND = unit price for sand media installed, \$/cu yd.

3.9.9.8.3 Calculate cost of gravel media.

$$COGM = \frac{V_{GL}}{27} (COGRVL)$$

where

COGM = cost of gravel media installed, \$.

V_{GL} = volume of gravel media required, cu ft.

COGRVL = unit price for gravel media installed, \$/cu yd.

3.9.9.8.4 Calculate cost of underdrain pipe.

$$COSTU = (LDP)\ (0.8)\ (COSTCP)$$

where

COSTU = cost of underdrain system, \$.

LDP = length of drain pipe, ft.

COSTCP = cost of 6" ϕ perforated clay pipe installed, \$/ft.

0.8 = correction factor for 4" ϕ pipe.

3.9.9.8.5 Calculate cost of influent pipe.

3.9.9.8.5.1 Calculate installed cost of influent pipe.

$$IPC = (LP)\ \frac{COSTRP}{100}\ (COSP)$$

where

IPC = installed pipe cost, \$.

LP = length of influent pipe required, ft.

COSTRP = cost of influent pipe of diameter D as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

3.9.9.8.5.2 Calculate COSTRP.

$$COSTRP = 6.842\ (D)^{1.2255}$$

where

COSTRP = cost of influent pipe of diameter D as percent of cost of standard size pipe, %.

D = influent pipe size, inches.

3.9.9.8.5.3 Calculate COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is $13.50 per foot in first quarter 1977.

3.9.9.8.6 Calculate O&M material and supply cost.

$$OMMC = \frac{OMMP}{100} COSM$$

where

OMMC = O&M material and supply cost, $/yr.

OMMP = O&M material and supply costs as percent of sand media cost, %.

COSM = cost of sand media installed, $.

3.9.8.7 Calculate total bare construction cost.

$$TBCC = [COSTE + COSM + COGM + COSTU + IPC] CF$$

where

TBCC = total bare construction cost, $.

COSTE = cost of earthwork, $.

COSM = cost of sand media installed, $.

COGM = cost of gravel media installed, $.

COSTU = cost of underdrain system, $.

IPC = installed pipe cost, $.

CF = correction factor for other construction cost.

3.9.9.9 Cost Calculations Output Data.

3.9.9.9.1 Total bare construction cost, TBCC, $.

3.9.9.9.2 O&M material and supply cost, OMMC, $.

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3.11 LAND TREATMENT

3.11.1 Background.

3.11.1.1 Land treatment of wastewater involves the use of plants, the soil surface and the soil matrix for wastewater treatment. Although there are some differences in the use and definition of tems, there are three principal processes of land treatment. The "slow rate process" (also called crop irrigation) couples wastewater management with recycling of nutrients in crop production. "Rapid infiltration" (also known as infiltration/percolation) emphasizes water reclamation rather than direct nutrient recycling. The product water from rapid infiltration may be reused for crop production, returned to surface waters, or allowed to recharge groundwaters. "Overland flow" also emphasizes water reclamation. Unlike rapid infiltration, however, the product water from overland flow is almost always discharged directly to surface waters.

3.11.1.2 Selection of the appropriate land application method requires matching the management objectives and wastewater characteristics to the characteristics of potential sites, expected treatment efficiencies, and land requirements. Factors such as wastewater quality, climate, soil geology, topography, land availability and effluent quality requirements will determine which process is most suitable for a particular application.

3.11.1.3 Typical design features for the various land treatment processes are presented in Table 3.11-1. Site characteristics are compared in Table 3.11-2. Expected quality of treated water and removal efficiencies for the three principal land processes are summarized in Table 3.11-3 and 3.11-4, respectively.

TABLE 3.11-1

COMPARISON OF DESIGN FEATURES FOR LAND TREATMENT PROCESSES

Feature	Principal Processes: Slow Rate	Principal Processes: Rapid Infiltration	Principal Processes: Overland Flow	Other Processes: Wetlands	Other Processes: Subsurface
Application techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface	Sprinkler or Surface	Subsurface piping
Annual application rate, ft	2 to 20	20 to 560	10 to 70	4 to 100	8 to 87
Field area required, acres[b]	56 to 560	2 to 56	16 to 110	11 to 280	13 to 140
Typical weekly application rate, in.	0.5 to 4	4 to 120	2.5 to 6[c] 6 to 16[d]	1 to 25	2 to 20
Minimum preapplication treatment provided in United States	Primary Sedimentation[e]	Primary Sedimentation	Screening and Grit removal	Primary Sedimentation	Primary Sedimentation
Disposition of Applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evaporation with some percolation	Evapotranspiration, percolation, and runoff	Percolation with some evapotranspiration
Need for vegetation	Required	Optional	Required	Required	Optional

a. Includes ridge-and-furrow and border strip.
b. Field area in acres not including buffer area, roads, or ditches for 1 Mgal/d (43.8 L/S) flow.
c. Range for application of screened wastewater.
d. Range for application of lagoon and secondary effluent.
e. Depends on the use of the effluent and the type of crop.

1 in. = 2.54 cm
1 ft = 0.305 m
1 acre = 0.405 ha

TABLE 3.11-2

COMPARISON OF SITE CHARACTERISTICS FOR LAND TREATMENT PROCESSES

	Principal processes			Other processes	
Characteristics	Slow rate	Rapid infiltration	Overland flow	Wetlands	Subsurface
Slope	Less than 20% on cultivated land; less than 40% on noncultivated land	Not critical; excessive slopes require much earthwork	finish slopes 2 to 8%	Usually less than 5%	Not critical
Soil permeability	Moderately slow to moderately rapid	Rapid (sands, loamy sands)	Slow (clays silts, and soils with impermeable barriers)	Slow to moderate	Slow to rapid
Depth to groundwater	2 to 3 ft (minimum)	10 ft (less depths are acceptable where underdrainage is provided)	Not critical	Not critical	Not critical
Climatic restrictions	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often needed for cold weather	Storage may be needed for cold weather	None

1 ft = 0.305 m

TABLE 3.11-3

EXPECTED QUALITY OF TREATED WATER FROM LAND TREATMENT PROCESSES
MG/L

Constituent	Slow rate[a]		Rapid Infiltration[b]		Overland flow[c]	
	Average	Maximum	Average	Maximum	Average	Maximum
BOD	2	5	2	5	10	15
Suspended solids	1	5	2	5	10	20
Ammonia nitrogen as N	0.5	2	0.5	2	0.8	2
Total nitrogen as N	3	8	10	20	3	5
Total phosphorus as P	0.1	0.3	1	5	4	6

a. Percolation of primary or secondary effluent through 5 ft (1.5 m) of soil.

b. Percolation of primary or secondary effluent through 15 ft (4.5 m).

c. Runoff of comminuted municipal wastewater over about 150 ft (45 m) of slope.

TABLE 3.11-4

PERFORMANCE OF THE THREE PRINCIPAL LAND TREATMENT METHODS

Parameter	Removals (%)		
	Slow Rate	High Rate	Overland Flow
BOD_5	90-99+	95-99	80-95
TSS	90-99+	95-99	80-95
Total-N	50-95[a]	25-75	75-90
Total-P	80-99[b]	0-90[d]	30-60
Fecal Coliform	99.99[c]	99.9-99.99+	90-99.9

a. Depending on nitrogen uptake of vegetation.

b. Diminishes when P uptake exhausted.

c. When applied counts are more than 10^4 MPN/100 ml.

d. Until flooding exceeds adsorptive capacity.

3.11.2 <u>General Description Overland Flow.</u>

3.11.2.1 In overland flow land treatment, wastewater is applied over the upper reaches of sloped terraces and is treated as it flows across the vegetated surface to runoff collection ditches. The wastewater is renovated by physical, chemical and biological means as it flows in a thin film down the relatively impermeable slope. Typical hydraulic pathway is presented in Figure 3.11-1. As shown, there is relatively little percolation involved either because of an impermeable soil or a subsurface barrier to percolation.

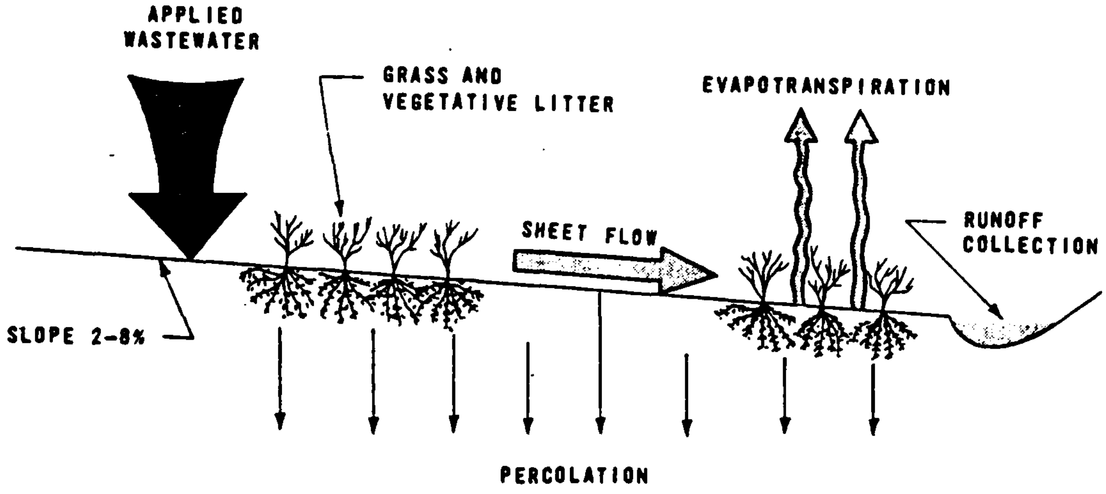
3.11.2.2 The primary objective of overland flow is wastewater treatment. A secondary objective of the system is for crop production. Perennial grasses (Reed, Canary, Bermuda, Red Top, Tall Fescue and Italian Rye) with long growing seasons, high moisture tolerance and extensive root formation are best suited to overland flow. Harvested grass is suitable for cattle feed.

3.11.2.3 Biological oxidation, sedimentation and grass filtration are the primary removal mechanisms for organics and suspended solids. Nitrogen removal is attributed mainly to nitrification/denitrification and plant uptake. Permanent nitrogen removal by plant uptake is only possible if the crop is harvested and removed from the field. Ammonia volatilization can be significant if the pH of the wastewater is above 7. Nitrogen removals normally range from 75-90% with runoff nitrogen being mostly in the nitrate form. Phosphorus is removed by adsorption, plant uptake and precipitation. Treatment efficiencies are somewhat limited because of the incomplete contact between the wastewater and the adsorption sites within the soil. Phosphorus removals usually range from 30-60%. Increased removals may be accomplished through preapplication treatment with aluminum or iron salts. Trace elements removal is relatively good with removal efficiencies ranging from 90-98%.

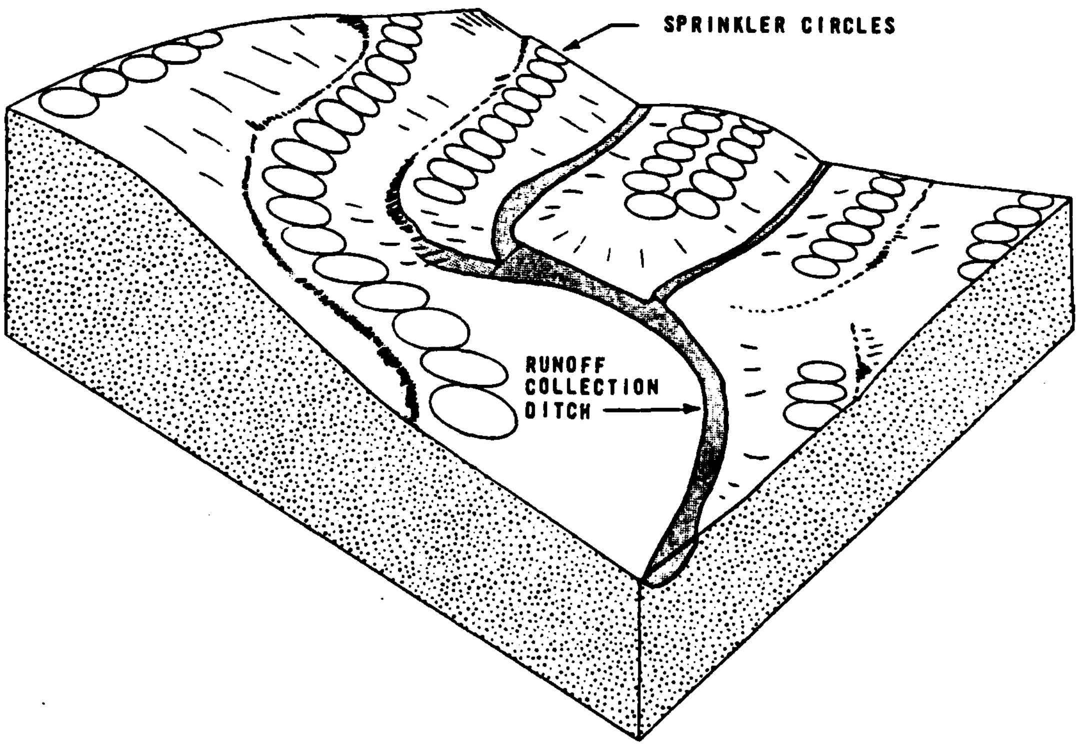
3.11.2.4 Loading rates and cycles are designed to maintain active microorganism growth on the soil surface. The operating principles are similar to a conventional trickling filter with intermittent dosing. The rate and length of application is controlled to minimize severe anaerobic conditions that result from overstressing the system. The resting period should be long enough to prevent surface ponding yet short enough to keep the microorganisms in an active state. Surface methods of distribution include the use of gated pipe or bubbling orifice. Slopes must be steep enough to prevent ponding of the runoff yet mild enough to prevent erosion and provide sufficient detention time for the wastewater on the slopes. Slopes must also have a uniform cross slope and be free from gullies to prevent channeling and allow distribution over the surfaces. Storage must be provided for nonoperating periods when air temperatures fall below freezing. Runoff is normally collected in open ditches.

FIGURE 3.11-1

OVERLAND FLOW



(a) HYDRAULIC PATHWAY



(b) PICTORIAL VIEW OF SPRINKLER APPLICATION

3.11.2.5 Typical design criteria include field area requirements of 35-100 acres/mgd; terraced slopes of 2 to 8%; terrace lengths of 120 to 140 ft; application rate of 11 to 32 ft/yr (2.5 to 16 in/wk); BOD_5 loading rate of 5 to 50 lb/acre-d; soil depth must be sufficient to form slopes that are uniform and to maintain a vegetative cover; soil permeability of 0.2 in/hr or less; hydraulic loading cycle of 6 to 8 hours application period followed by 16 to 18 hours resting period; operating period of 5 to 6 days/wk; soil texture of clay or clay loams.

3.11.2.6 Common preapplication treatment include screening or comminution for isolated sites with no public access, screening or comminution plus aeration to control odors during storage or application for urban locations with no public access. Wastewater high in metal content should be pretreated to avoid soil and plant contamination.

3.11.3 General Description Rapid Infiltration.

3.11.3.1 Rapid infiltration treats the wastewater with a minimum of land area. Wastewater is applied to deep and permeable deposits such as sand or sandy loam usually by distributing in basins or infrequently by sprinkling, and is treated as it travels through the soil matrix by filtration, adsorption, ion exchange precipitation and microbial action. Vegetation is not usually used, but crops may be grown to help maintain infiltration rates, however, harvest normally would not be an objective. Typical hydraulic pathway for rapid infiltration is shown in Figure 3.11-2. A much greater portion of the applied wastewater percolates to the groundwater than with slow rate land treatment. An underdrain system may be incorporated into the system to recover the renovated water for reuse, to control groundwater mounding, or to minimize trespass of wastewater onto adjoining property by horizontal subsurface flow. There is little or no consumptive use by plants and less evaporation in proportion to a reduced surface area. A cycle of flooding and drying maintains the infiltration and treatment capacity of the soil material.

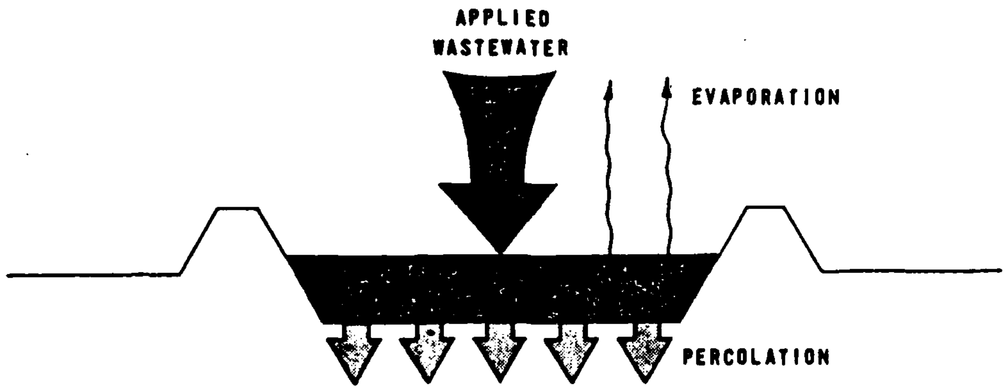
3.11.3.2 Removal of wastewater constituents by filtering and straining action of the soil are excellent. Suspended solids, BOD, and fecal coliforms are almost completely removed in most cases. Removal of BOD is primarily accomplished by aerobic bacteria that depend on the resting period to reaerate the soil. BOD loading rates have some effect on removals but too many other variables such as temperature, resting period, and soil type are involved to allow estimation of removals from loading rates alone.

3.11.3.3 Nitrogen removals are generally poor. The basic mechanisms for nitrogen removal include nitrification-denitrification and ammonium sorption. Denitrification may be enhanced through adjusting application cycles, supplying an additional carbon source, using vegetated basins, recycling of renovated water and reducing application rates. Rapid infiltration systems will produce a nitrified effluent at nitrogen loadings up to 60 lbs/acre-day. Nitrification below a temperature of 36°F and below a pH of 5 is minimal.

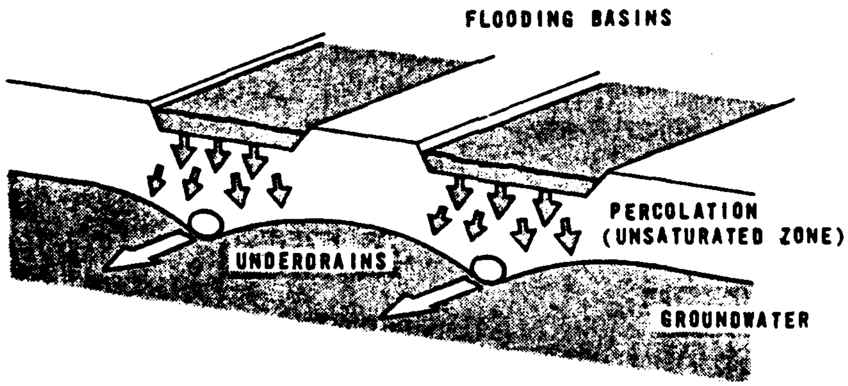
3.11.3.4 Phosphorus removals can range from 70-99% depending on the physical and chemical characteristics of the soil. The primary removal mechanism is adsorption with some chemical precipitation. Consequently, long-term capacity is limited by the mass of the soil in contact with the wastewater. Removals are also related to the residence time of the wastewater in the soil and travel distance.

FIGURE 3.11-2

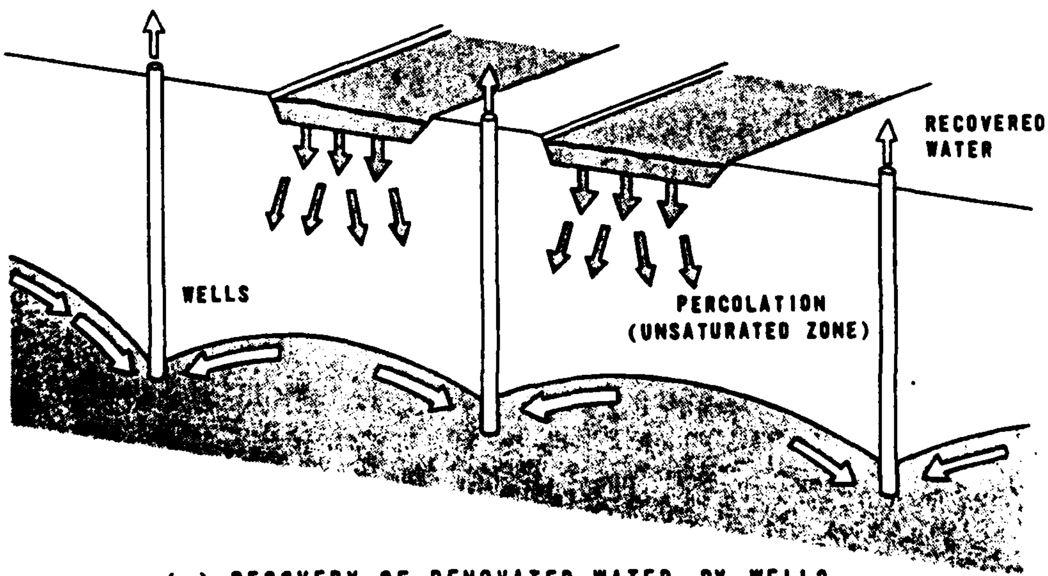
RAPID INFILTRATION



(a) HYDRAULIC PATHWAY



(b) RECOVERY OF RENOVATED WATER BY UNDERDRAINS



(c) RECOVERY OF RENOVATED WATER BY WELLS

3.11.3.5 Heavy metals are removed from solution by adsorptive process and by precipitation and ion exchange in the soil. The concern about heavy metals in rapid infiltration systems are related to the high rate of application and low adsorptive potential of the coarse soils. Microorganism removal is accomplished through sedimentation, predation and desiccation during preapplication treatment, desiccation and radiation during application; and straining, desiccation, radiation, predation and hostile environmental factors upon application to the land.

3.11.3.6 Typical design criteria include field areas of 3 to 56 acre/mgd; application rate of 20-400 ft/yr (4-92 in/wk); BOD_5 loading rate of 20 to 100 lb/acre-d; soil depth of 10 to 15 ft or more; soil permeability of 0.6 in/hr or more; hydraulic loading cycle of 9 hours to 2 weeks application period followed by 15 hours to 2 weeks resting period; soil texture - sand, sandy loams; basin size of 1 to 10 acres with a minimum of 2 basins/site; height of dikes of 4 ft; underdrains of 6 or more ft deep, application techniques -flooding or sprinkling.

3.11.3.7 Common preapplication treatment practices include: primary treatment for isolated locations with restricted public access; biological treatment for urban locations with controlled public access. Storage is sometimes provided for flow equalization and for non-operating periods. Environmental impacts include potential for contamination of groundwater by nitrites and nitrates. Heavy metals may be eliminated by pretreatment techniques as necessary.

3.11.4 General Description Slow Infiltration.

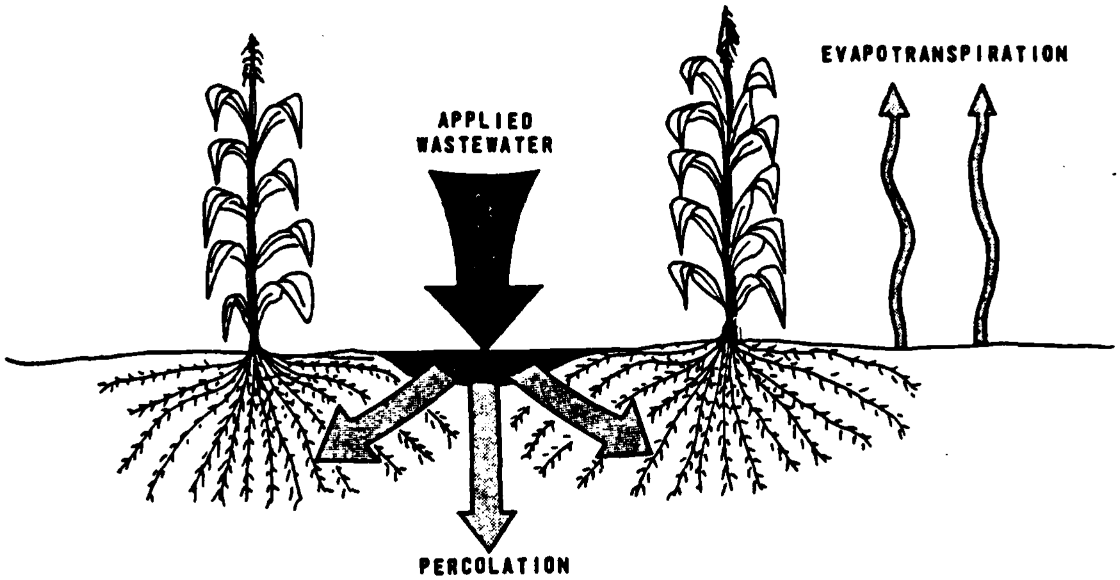
3.11.4.1 Slow rate infiltration is the most common method of treatment by land application. Wastewater is applied to vegetated soils that are slow to moderate in permeability (clay loams to sand loams) and is treated as it travels through the soil matrix by filtration, adsorption, ion exchange, precipitation, microbial action and by plant uptake. Part of the water is lost through evaporation and plant transpiration, part is stored in plant tissue, and the remainder either percolates to groundwater or is collected in an underdrainage system and reused (Figure 3.11-3a). Surface runoff is generally negligible.

3.11.4.2 Wastewater application techniques include sprinkling or surface distribution (Figure 3.11-3b and 3.11-3c). Sprinklers can be categorized as hand moved, mechanically moved and permanent set, the selection of which includes the following considerations: field conditions (shape, slope, vegetation and soil type), climate, operating conditions, and economics. Surface distribution employs gravity flow from piping systems or open ditches to flood the application area with several inches of water. Application techniques include ridge and furrow and surface flooding (border strip flooding). Ridge and furrow irrigation consists of running irrigation streams along small channels (furrows) bordered by raised beds (ridges) upon which crops are grown. Surface flooding irrigation consists of directing a sheet flow of water along border strips or cultivated strips of land bordered by small levees. The latter method is suited to close-growing crops such as grasses that can tolerate periodic inundation at the ground surface. A tail water return system for wastewater runoff from excess surface application is usually employed. Advantages of sprinkler application over surface distribution methods include: more uniform distribution of water and greater flexibility in range of application rates, applicability to most crops, less susceptibility to topographic constraints, and reduced operator skill and experience requirements. Limitations to sprinkling include adverse wind conditions and clogging of nozzles.

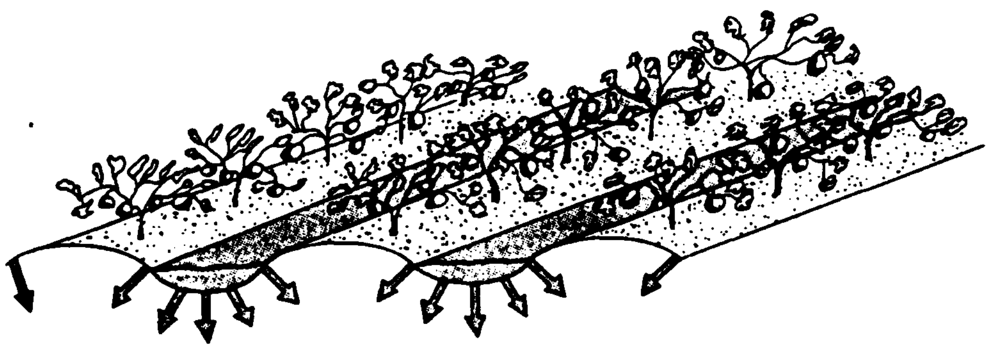
3.11.4.3 Slow rate treatment systems produce the best results of land treatment systems. Organics are removed substantially by biological oxidation within the top few inches of soil. Filtration and adsorption are the initial mechanisms in BOD removal but biological oxidation is the ultimate treatment mechanism. Suspended solids removals are not as well documented as BOD removals, but concentrations of 1 mg/l or less can generally be expected in the renovated water. Filtration is the major removal mechanism for suspended solids while volatile solids are biologically oxidized and fixed or mineral solids become part of the soil matrix.

FIGURE 3.11-3

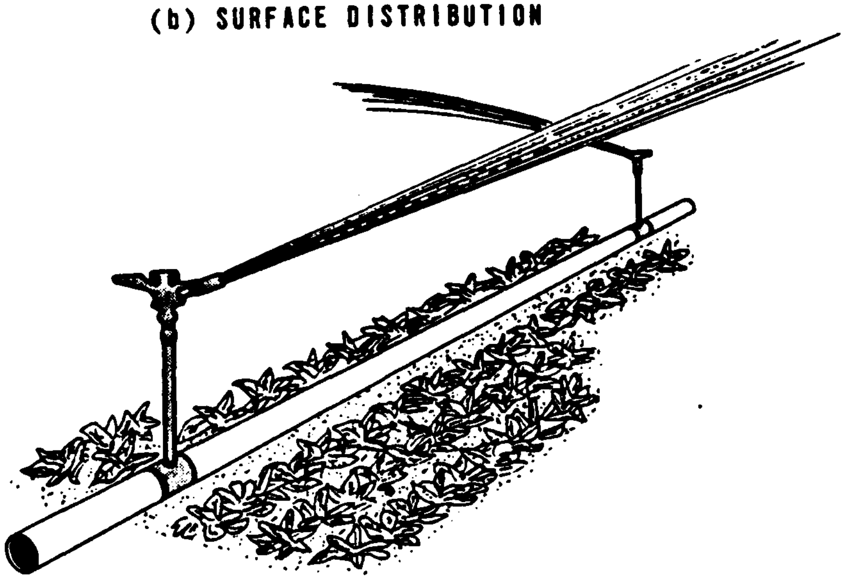
SLOW RATE LAND TREATMENT



(a) HYDRAULIC PATHWAY



(b) SURFACE DISTRIBUTION



(c) SPRINKLER DISTRIBUTION

3.11.4.4 Nitrogen is removed primarily by crop uptake, which varies with the type of crop grown and the crop yield. Crop nitrogen uptake values, based on typical yields under commercial fertilization, is presented in Table 2.16-1. Nitrogen removal can also be partially accomplished through biological denitrification and volatilization. Losses due to denitrification has been reported to range from 15% to 25% of the applied nitrogen. To protect groundwaters, the percolate nitrogen should be limited to 10 mg/l for design purposes.

3.11.4.5 Phosphorus removal is extremely effective as a result of soil adsoprtion, precipitation and crop uptake. Trace metals are removed from solution by adsorptive process and by precipitation and ion exchange in the soil.

3.11.4.6 Vegetation is a vital part of a slow rate system and serves to extract nutrients, reduce erosion and maintain soil permeability. Considerations for crop selection include: (1) Suitability to local climate and soil conditions, (2) consumptive water use and water tolerance, (3) nutrient uptake and sensitivity to wastewater constituents, (4) economic value and marketability, (5) length of growing season, (6) ease of management, and (7) public health regulations.

3.11.4.7 Typical design criteria include: field area 56 to 560 acres/mgd; application rate 2-20 ft/yr (0.5-4 in/wk); BOD_5 loading rate 0.2 to 5 lb/acre-d; soil depths 2-3 ft or more; soil permeability 0.06-2.0 in/hr; minimum preapplication treatment equivalent to primary; lower temperature limit 25°F.

3.11.4.8 Preapplication treatment include the following: primary treatment for isolated locations with restricted public access and when limited to crops not for direct human consumption; biological treatment plus control of coliform to 1000 MPN/100 ml for agricultural irrigation except for food crops to be eaten raw; secondary treatment plus disinfection for public access areas (parks). Wastewater high in metal content must also be pretreated to avoid soil and plant contamination.

3.11.5 <u>Overland Flow.</u>

3.11.5.1 <u>Input Data.</u>

3.11.5.1.1 Wastewater flow, Q, mgd.

3.11.5.1.1.1 Minimum flow, mgd.

3.11.5.1.1.2 Average flow, mgd.

3.11.5.1.1.3 Maximum flow, mgd.

3.11.5.1.2 Wastewater characteristics.

3.11.5.1.2.1 Suspended solids, mg/l.

3.11.5.1.2.2 Volatile suspended solids, % of suspended solids.

3.11.5.1.2.3 Settleable solids, mg/l.

3.11.5.1.2.4 BOD_5 (soluble and total), mg/l.

3.11.5.1.2.5 COD (soluble and total), mg/l.

3.11.5.1.2.6 Phosphorus (as PO_4), mg/l.

3.11.5.1.2.7 Total kjeldahl nitrogen (TKN), mg/l.

3.11.5.1.2.8 Ammonia-nitrogen, NH_3, mg/l.

3.11.5.1.2.9 Nitrite-nitrogen, NO_2, mg/l.

3.11.5.1.2.10 Nitrate-nitrogen, NO_3, mg/l.

3.11.5.1.2.11 Temperature, °C.

3.11.5.1.2.12 pH, units.

3.11.5.1.2.13 Oil and grease, mg/l.

3.11.5.1.2.14 Cations, mg/l.

3.11.5.1.2.15 Anions, mg/l.

3.11.5.2 <u>Design Parameters.</u>

3.11.5.2.1 Application rate.

3.11.5.2.1.1 Screened wastewater = 2.5 - 6 in/wk.

3.11.5.2.1.2 Lagoon or secondary effluent = 6 - 16 in/wk.

3.11.5.2.2 Precipitation rate, P_r, in/wk.

3.11.5.2.3 Evapotranspiration rate, ET, in/wk.

3.11.5.2.4 Runoff, R, in/wk (site dependent).

3.11.5.2.5 Wastewater generation period, WWGP, days/yr.

3.11.5.2.6 Field application period, FAP, wks/yr.

3.11.5.2.7 Spray evaporation (percent of application rate) = 2-8%.

3.11.5.2.8 Storage requirements (specify one).

3.11.5.2.8.1 No storage.

3.11.5.2.8.2 Minimum storage, days/yr.

3.11.5.2.9 Liner required (only used with storage).

3.11.5.2.10 Embankment protection (only used with storage).

3.11.5.2.11 Recovery system (specify one).

3.11.5.2.11.1 Gravity pipe.

3.11.5.2.11.2 Open channel recovery system.

3.11.5.2.12 Buffer zone width, ft (site dependent) = 0.0 - 500 ft.

3.11.5.2.13 Current ground cover.

3.11.5.2.13.1 Forest, %.

3.11.5.2.13.2 Brush, %.

3.11.5.2.13.3 Pasture, %.

3.11.5.2.14 Slope of land, % = 2.0 - 8.0%.

3.11.5.2.15 Monitoring wells.

3.11.5.2.15.1 Number.

3.11.5.2.15.2 Depth per well, ft.

3.11.5.2.16 Fraction denitrified, D, % = 75 - 90%.

3.11.5.2.17 Ammonia volatilization, AV, % = 0.0%.

3.11.5.2.18 Removal of phosphorus, % = 80%.

3.11.5.2.19 Hours per day operation, hrs/day.

3.11.5.2.20 Days per week operation, days/wk.

3.11.5.3 <u>Process Design Calculations</u>.

3.11.5.3.1 Calculate water loss due to evaporation, E, in/wk.

$$E = (E_f)(L_w)/(100)$$

E = water loss due to evaporation, in/wk.

E_f = percent of total applied wastewater lost through evaporation, %.

L_w = hydraulic application rate, in/wk.

3.11.5.3.2 Calculate percolation rate.

$$W_p = (W_p)_f \; (L_w)/(100).$$

W_p = percolating water rate, in/wk.

$(WP)_f$ = percent of applied wastewater lost to percolation, %.

3.11.5.3.3 Calculate runoff, R, in/wk, from water balance.

$$R = L_w + P_r - ET - W_p - E$$

R = runoff, in/wk.

P_r = design precipitation, in/wk.

ET = evapotranspiration, in/wk (crops consumptive use of water).

3.11.5.3.4 Calculate BOD_5 loading, $(L)_{BOD}$, lbs/acre-yr.

$$(L)_{BOD} = (TBOD)_i \text{ mg/l } (L_w) \text{ in/wk } \left(\frac{1 \text{ gm}}{1000 \text{ mg}}\right) \left(3.785 \frac{l}{\text{gal}}\right)$$

$$\left(\frac{1 \text{ lb}}{454 \text{ gm}}\right)\left(\frac{1 \text{ ft}}{12 \text{ in}}\right)\left(52 \frac{\text{wks}}{\text{yr}}\right)\left(7.48 \frac{\text{gal}}{\text{ft}^3}\right) (43{,}560 \text{ ft}^2/\text{acre})$$

$$= (11.77)\ (TBOD)_i(L_w)$$

$(L)_{BOD}$ = total BOD_5 loading, lbs/acre-yr.

$(TBOD)_i$ = total BOD_5 in applied wastewater, mg/l.

3.11.5.3.4 Calculate total BOD_5 in runoff, mg/l.

$$(TBOD_5)_R = (L)_{BOD}/(R)(11.77)$$

$(TBOD_5)_R$ = BOD_5 concentration in runoff, mg/l.

3.11.5.3.5 Calculate soluble BOD_5 loading, $(L)_{SBOD_5}$, lb/acre-yr.

$$(L)_{SBOD_5} = (11.77)\ (SBOD_5)_i\ (L_w)$$

$(L)_{SBOD_5}$ = soluble BOD_5 loading, lbs/acre-yr.

$(SBOD_5)_i$ = soluble BOD_5 in applied wastewater, mg/l.

3.11.5.3.6 Calculate soluble BOD_5 concentration in runoff, $(SBOD_5)_R$, mg/l.

$$(SBOD_5)_R = (L)_{SBOD_5}/(R)(11.77)$$

$(SBOD_5)_R$ = soluble BOD_5 concentration in runoff, mg/l.

3.11.5.3.7 Calculate total nitrogen concentration, C_n, in applied wastewater.

$$C_n = (TKN)_i + (NO_2)_i + (NO_3)_i$$

C_n = total nitrogen concentration in applied wastewater, mg/l.

$(TKN)_i$ = total kjeldahl nitrogen concentration in applied wastewater, mg/l.

$(NO_2)_i$ = nitrite-N concentration in applied wastewater, mg/l.

$(NO_3)_i$ = nitrate-N concentration in applied wastewater, mg/l.

3.11.5.3.8 Calculate wastewater nitrogen loading, lbs/acre-yr.

$$L_n = (C_n)(L_w)(11.77)$$

L_n = wastewater nitrogen loading, lbs/acre-yr.

3.11.5.3.9 Calculate total nitrogen loading, $(L_t)_N$, lbs/acre-yr.

$$(L_t)_N = L_n + (11.77)\ (P_r)\ (0.5)$$

$(L_t)_N$ = total nitrogen loading rate, lbs/acre-yr.

0.5 = assumed total nitrogen concentration in precipitation, mg/l.

3.11.5.3.10 Calculate crop nitrogen uptake rate, $(U)_N$, lbs/acre-yr; use forage grass for ground cover.

$$(U)_N = 0.891\ [(118.73) + (0.36)\ (L_t)_N]\ \text{lbs/acre-yr}$$

3.11.5.3.11 Calculate nitrogen loss through denitrification, D, lbs/acre-yr.

$$D = (D_f)(L_t)_N/(100)$$

D = nitrogen loss through denitrification, lbs/acres/yr.

D_f = percent of total applied nitrogen lost through denitrification, %.

3.11.5.3.12 Calculate nitrogen loss due to volatilization, AV, lbs/acre-yr.

$$AV = (AV)_f\ (L_t)_N/(100)$$

AV = nitrogen loss due to volatilization, lbs/acre-yr.

$(AV)_f$ = fraction of total nitrogen lost through volatilization, %.

3.11.5.3.13 Calculate sum of nitrogen losses, $(\Sigma N)_L$, lbs/acre-yr.

$$(\Sigma N)_L = (U)_N + (D) + (AV)$$

$(\Sigma N)_L$ = sum of nitrogen lost, lbs/acre-yr.

$$(\Sigma N)_L \leq (0.8)\ (L_t)_N$$

3.11.5.3.14 From nitrogen mass balance calculate nitrogen concentration in runoff, $(C_R)_N$, mg/l.

$$(C_R)_N = [(L_t)_N - (\Sigma N)_L]/(R)(11.77)$$

3.11.5.3.15 Calculate required field area, TA, acre.

$$TA = (36.83)(Q_{av})(WWGP)/(L_w)(FAP)$$

TA = required field area, acres.

Q = average wastewater flow, mgd.

WWGP = wastewater generation period, days/yr.

FAP = field application period, wks/yr.

3.11.5.3.16 Calculate phosphorus loading, L_p, lb/acre-yr.

$$L_p = (11.77)(TP)_i\ (L_w)$$

L_p = total phosphorus loading, lbs/acre-yr.

$(TP)_i$ = total phosphorus concentration in applied wastewater, mg/l.

3.11.5.3.17 Calculate soil removal of phosphorus, lbs/acre-yr.

$$(SRP) = (SRP)_f\ (L_p)/(100)$$

SRP = soil removal of phosphorus, lbs/acre-yr.

$(SRP)_f$ = percent of total phosphorus removed by the soil, %.

3.11.5.3.18 Calculate removal of phosphorus, U_p, lbs/acre-yr.

$$Up = 0.891\ [(83.386) - (0.0373)(L_p)] \quad \text{lbs/acre-yr.}$$

U_p = removal of phosphorus, lbs/acre-yr.

3.11.5.3.19 From phosphorus mass balance, calculate phosphorus concentration in runoff.

$$L_p = (U_p) + (C_p)_R\ (R)(11.77)$$

$$(C_p)_R = [(L_p) - (U_p)]/(11.77)(R)$$

$(C_p)_R$ = phosphorus content in runoff, mg/l.

$(0.01)(TP)_i$

3.11.5.3.20 Calculate volume of runoff, R, in mgd.

$$R(\text{mgd}) = [(R)\ \text{in/wk}\ (\frac{1\ \text{ft}}{12\ \text{in}})(\frac{1\ \text{wk}}{7\ \text{day}})]\ (TA)\ \text{acre}\ (43{,}560\ \frac{\text{ft}^2}{\text{acre}})$$

$$(7.48\ \frac{\text{gal}}{\text{ft}^3})(1/10^6)$$

3.11.5.3.21 Calculate suspended solids concentration in runoff, mg/l, assume 93% SS removal.

$$(SS)_R = (SS)_i(0.07)$$

$(SS)_R$ and $(SS)_i$ = suspended solids concentration in runoff and applied wastewater, respectively, mg/l.

3.11.5.3.22 Calculate total and soluble COD in runoff.

$$(TCOD)_R = [(TCOD)_i - (TBOD_5)_i]\ (0.9) + (TBOD_5)_R$$

$$(SCOD)_R = [(SCOD)_i - (SBOD_5)_i]\ (0.9) + (SBOD_5)_R$$

$(TCOD)_R$ and $(SCOD)_R$ = total and soluble COD in runoff, respectively, mg/l.

$(TCOD)_i$ and $(SCOD)_i$ = total and soluble COD in applied wastewater, respectively, mg/l.

3.11.5.3.23 Nitrite-N concentration in runoff = 0.

3.11.5.3.24 Nitrate-N concentration in runoff = $(0.75)(C_R)_N$.

3.11.5.3.25 Ammonia-N concentration in runoff = $(0.25)(C_R)_N$.

3.11.5.3.26 Oil and Grease in runoff = 0.0.

3.11.5.4 <u>Process Design Output Data.</u>

3.11.5.4.1 Hours per day operation, hours.

3.11.5.4.2 Days per week operation, days.

3.11.5.4.3 Application rate, in/week.

3.11.5.4.4 Runoff, in/week.

3.11.5.4.5 Percent denitrified, percent.

3.11.5.4.6 Percent ammonia volatilization, percent.

3.11.5.4.7 Removal of phosphorus, percent.

3.11.5.4.8 Spray evaporation, percent.

3.11.5.4.9 Wastewater generation period, days/yr.

3.11.5.4.10 Field application period, weeks/yr.

3.11.5.4.11 Buffer zone width, feet.

3.11.5.4.12 Current ground cover.

3.11.5.4.12.1 Forest, percent.

3.11.5.4.12.2 Brush, percent.

3.11.5.4.12.3 Pasture, percent.

3.11.5.4.13 Slope of site, percent.

3.11.5.4.14 Number of monitoring wells, wells,

3.11.5.4.15 Depth of Monitoring wells, feet.

3.11.5.4.16 Treatment area required, acres.

3.11.5.4.17 Volume of runoff, mgd.

3.11.5.4.18 Quality of runoff, mgd.

3.11.5.4.18.1 Suspended solids, mg/l.

3.11.5.4.18.2 Volatile solids, percent.

3.11.5.4.18.3 BOD_5, mg/l.

3.11.5.4.18.4 BOD_5 soluble, mg/l.

3.11.5.4.18.5 COD, mg/l.

3.11.5.4.18.6 COD soluble, mg/l.

3.11.5.4.18.7 PO_4, mg/l.

3.11.5.4.18.8 TKN, mg/l.

3.11.5.4.18.9 NO_2, mg/l.

3.11.5.4.18.10 NO_3, mg/l.

3.11.5.4.18.11 Oil and grease, mg/l.

3.11.5.5 <u>Quantities Calculations</u>.

3.11.5.5.1 Distribution Pumping.

3.11.5.5.1.1 Calculate the design flow.

$$FLOW = \frac{(Q_{avg})\ (WWGP)\ (24)}{(FAP)\ (DPW)\ (HPD)}$$

where

FLOW = actual daily flow to spray field, mgd.

Q_{avg} = average daily wastewater flow, mgd.

WWGP = wastewater generation period, days/yr.

FAP = field application period, wks/yr.

DPW = days per week treatment system is operated, days/wk.

HPD = hours per day treatment system is operated, hrs/day.

Using the flow calculated (FLOW), the distribution pumping will be sized and the cost estimated from the existing chapter entitled, "Intermediate Pumping".

3.11.5.5.2 Storage Requirements. Overland flow unlike rapid infiltration is dependent upon weather. Storage is required to hold the wastewater generated when application is not possible due to cold weather or heavy rains. This, of course, varies greatly for different parts of the country with different climates.

For purposes of this program the number of days of storage required will be assumed to be 50% of the number of days which wastewater cannot be applied to the field.

3.11.5.5.2.1 Calculate storage volume,

$$SV = (.5)\ [365-(FAP)(7)]\ (GFx10^6)$$

2here

SV = storage volume, gal.

FAP = field application period, wks/yr.

GF = generated flow, mgd.

3.11.5.5.2.2 Calculate size and number of storage lagoons.

3.11.5.5.2.2.1 The following assumptions are made in determining size and number of lagoons:

A minimum of 2 lagoon cells will always be used.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
The largest single lagoon cell will be 40 acres which represents approximately 85 million gallons storage volume.

3.11.5.5.2.2.2 If SV $\leq$ 170,000,000 gal.

$$NLC = 2$$

3.11.5.5.2.2.3 If SV > 170,000,000 gal. a trial and error solution for NLC will be used.

Assume NLC = 4; If $\frac{SV}{NLC} > 85{,}000{,}000$ gal.

Redesignate NLC = NLC+2 and repeat calculation until $\frac{SV}{NLC} \leq 85{,}000{,}000$ gal.

where

SV = storage volume, gal.

NLC = number of lagoon cells.

3.11.5.5.2.3 Calculate storage volume per cell.

$$SVC = \frac{SV}{(NLC)(7.48)}$$

where

SVC = storage volume per cell, ft^3.

SV = storage volume, gal.

NLC = number of lagoon cells.

7.48 = conversion from gal to ft^3.

3.11.5.5.2.4 Calculate lagoon cell dimensions.

The following assumptions are made concerning lagoon construction:

The lagoon cells will be square.
Common levee construction will be used where possible.
Lagoons will be constructed using equal cut and fill.
Lagoon depth will be 10 ft. with 8 ft. water depth and 2 ft. freeboard.
Minimum water depth will be 1.5 ft.
Side slopes will be 3 to 1.
A 30% shrinkage factor is used for fill.

$$L = \frac{(0.615\ SVC-1521)^{0.5} + 60}{2}$$

where

L = length of one side of lagoon cell, ft.

SVC = storage volume per cell, ft^3.

3.11.5.5.2.5 Calculate volume of earthwork required for lagoons.

3.11.5.5.2.5.1 The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = 10$$

$$VF = [\ 3(DF)^2 + 10DF\ ]\ [\ \frac{5NLC}{2} + 2\ ]\ (L)$$

$$VC = (1.3)(NLC)(DC)\ [\ L^2-(6)(DF)(L)+12\ DF^2 + 120\ DF - 60L + 1200]$$

3.11.5.5.2.5.2 Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If VC < VF then assume DC > 1 ft. and recalculate VC and VF.

If VC > VF then assume DC < 1 ft. and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the storage lagoon.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

3.11.5.5.3 Overland Flow Distribution System.

3.11.5.5.3.1 In an overland flow treatment system the wastewater is usually applied to the field with fixed sprinklers. The pipes are nomally buried and impact type irrigation sprinklers with flow from 6 to 15 gpm per sprinkler are used. There are many ways to layout a sprinkler field and where there are slopes of greater than 2% available, the natural topography of the site dictates the layout. Though every layout is different the amount and size of pipe and sprinklers used will not vary greatly.

3.11.5.5.3.2 Assume:

Field will be square.
Arrangement of headers and laterals will be as shown in Figure 2.35-4.
The treatment area will be increased by 15% to account for additional area for drainage ditches and service roads.

3.11.5.5.3.3 Calculate number of headers. The number of headers will be selected based on the following:

Flow $\leq$ 1 mgd; NH = 1
1 mgd < Flow $\leq$ 2 mgd; NH = 2
2 mgd < Flow $\leq$ 4 mgd; NH = 3
Flow > 4 mgd; NH = 4

where

FLOW = actual daily flow to spray field.

NH = number of headers.

3.11.5.5.3.4 Calculate flow per header.

$$FPH = \frac{(FLOW \times 10^6)}{(NH)(HPD)(60)}$$

where

FPH = flow per header, gpm.

FLOW = actual daily flow to spray field, mgd.

NH = number of headers.

HPD = hours per day of operation, hrs/day.

60 = number of munites per hour, min/hr.

3.11.5.5.3.5 Calculate flow per sprinkler (FPN).

$$FPN = \frac{4051.7\ (AR)}{(DPW)(HPD)(FAP)}$$

where

FPN = flow per sprinkler, gpm.

AR = application rate, in/wk.

DPW = days per week treatment system is operated, days/wk.

HPD = hours per day treatment system is operated, hrs/day.

FAP = field application period, wks/yr.

4051.7 = combined conversion factors.

3.11.5.5.3.6 Calculate number of sprinklers per header.

$$SPH = \frac{FPH}{FPN}$$

If SPH < 1 set SPH = 1. Note: Flow is not sufficient, reduce operating period specification.

where

FPH = flow per header, gpm.

FPN = flow per sprinkler, gpm.

SPH = sprinklers per header.

3.11.5.5.3.7 Calculate number of laterals per header.

$$LPH = SPH/q$$

LPH must be an integer.

If LPH < 1 set LPH = 1 and recalculate the number of required sprinklers.

$$NSL = \frac{FPH}{FPN}$$

where

LPH = laterals per header.

SPH = sprinklers per header.

NSL = number of sprinklers per lateral.

FPH = flow per header, gpm.

FPN = flow per sprinkler, gpm.

3.11.5.5.3.8 Calculate flow per lateral.

$$FPL = \frac{FPH}{LPH}$$

where

FPL = flow per lateral, gpm.

FPH = flow per header, gpm.

LPH = laterals per header.

3.11.5.5.3.9 Calculate lateral diamater (DIAL).

$$DIAL = 0.286\ (FPL)^{0.5}$$

DIAL must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48. Always use the next larger diameter above the calculated diameter.

where

DIAL = diameter of lateral pipe, inches.

FPL = flow per lateral, gpm.

3.11.5.5.3.10 Calculate header pipe sizes.

The header pipe nomally decreases in size due to decreasing volume of flow as each set of lateral pipes removes part of the flow from the header pipe. There will nomally be four laterals taken off from approximately the same location. The header size will be calculated after each group of laterals.

$$DIAHN = 0.286\ [\ FPH - (N)(4)(FPL)\ ]^{0.5}$$

where

DIAHN = diameter of header pipe, inches.

FPH = flow per header, gpm.

FPL = flow per lateral, gpm.

N = number of points at which flow is removed from header.

DIAHN must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

Begin calculation with N = 0. This will give the diameter (DIAHO) of the header before the first group of laterals remove any flow. Then set N = N + 1 and repeat the calculation. This will give the diameter (DIAH 1) of the header after flow has been removed by the first group of laterals. Repeat the calculation each time redesignating N until the expression FPH-(N)(4)(FPL) is equal to zero or yields a negative number. The "N" used in this last calculation will be the number of slopes which must be constructed.

3.11.5.5.3.11 Determine length of lateral pipe.

$$LDIAL = (SPH)\ (50)\ (NH)$$

where

LDIAL = length of lateral pipe required, ft.

SPH = sprinklers per header.

50 = distance between sprinklers, ft.

3.11.5.5.3.12 Determine length of header pipes. As can be seen from Figure 3.11-4 the spray field is laid out such that the distance between lateral take-off points is 400 ft. Therefore, to determine the length of each size pipe required for a single header, sum the number of points at which the diameter is the same and multiply by 400 ft. To determine the amount of each size header pipe for the entire field, multiply by the number of headers since they are identical.

$$LDIAHN = (NH)(400) \sum N\emptyset$$

where

NH = number of headers.

LDIAHN = length of header pipe of diameter DIAHN, ft.

N∅ = number of points where same diameter pipe was chosen.

For the length of pipe from pump station to the spray field use the number of headers times the width of the spray field. The diameter will be the diameter calculated before any flow is removed from the header.

3.11.5.5.3.13 Calculate number of valves for distribution system.

3.11.5.5.3.13.1 There will be a butterfly valve in each header for flow control. These valves will be in the header upstream from the spray field and will be the same size as the initial size calculated for the header.

$$NBV = NH$$
$$DBV = DIAHN$$

where

NBV = number of butterfly valves.

DBV = diameter of butterfly valves, inches.

NH = number of headers.

3.11.5.5.3.13.2 There will be a plug valve in each lateral pipe which will be automatic but will be either fully open or fully closed. They will be the same size as the size calculated for the lateral pipes.

$$NLV = (LPH)\ (NH)$$
$$DLV = DIAL$$

where

NLV = number of lateral valves.

LPH = laterals per header.

DLV = diameter of lateral valve, inches.

DIAL = diameter of lateral pipe, inches.

NH = number of headers.

3.11.5.5.4 Construction of Overland Flow Slopes. Overland flow systems must have slopes from 2% to 8%, must be clear of trees and brush, and must be leveled to a constant slope. Not many land areas meet this criteria, therefore, in many cases the area must be cleared, slopes formed, and leveled.

3.11.5.5.4.1 Clearing and Grubbing. The areas will be classified in three categories; heavy, meadium, and light. Heavy refers to wooded areas with mature trees. Medium refers to spotted mature trees with numerous small trees and bushes. Light refers to only small trees and bushes. The user must specify the type of clearing and grubbing required as well as the percent of the treatment area requiring clearing and grubbing.

$$CAGH = \frac{(PCAGH)}{100} (TA)$$

$$CAGM = \frac{(PCAGM)}{100} (TA)$$

$$CAGL = \frac{(PCAGL)}{100} (TA)$$

where

CAGH = area which requires heavy clearing, acres.

PCAGH = percentage of treatment area requiring heavy clearing, %.

CAGM = area which requires medium clearing, acres.

PCAGM = percentage of treatment area requiring medium clearing, %.

CAGL = area which requires light clearing, acres.

PCAGL = percentage of treatment area requiring light clearing, %.

TA = treatment area, acres.

3.11.5.5.4.2 Determine Earthwork Required.

3.11.5.5.4.2.1 For areas which are flat (0-2% slope) the overland flow slopes must be formed by moving earth. For areas where slopes of from 2% to 8% exist, very little earth moving is required. The following assumptions are made to estimate the quantities of earthwork required for slopes from 0-2%.

3.11.5.5.4.2.1.1 The slopes shall be as indicated in Figure 3.11-4.

3.11.5.5.4.2.1.2 Equal cut and fill will be used.

3.11.5.5.4.2.2 Calculate volume of earthwork.

$$VSEW = (55,100)(TA)$$

where

VSEW = volume of earthwork required for slope construction, ft^3.

TA = treatment area, acres.

55,100 = volume of earthwork required per acre, ft^3.

3.11.5.5.5 Runoff collection. The overland flow system does not depend on infiltration for treatment and much of the water runs off. This water must be collected and monitored. There are basically two types of collection systems: open ditch and buried drain pipe. With both systems, the runoff from each individual slope is carried to the main collection system by small ditches or terraces.

3.11.5.5.5.1 Determine earthwork required for terraces.

$$VET = (N+2)(NH)(1000)(5)$$

where

VET = volume of earthwork for terraces, ft^3.

N = number of points which flow is removed from header.

NH = number of headers.

1000 = length of individual slope.

5 = volume of earthwork required per foot of terrace length.

3.11.5.5.5.2 Main Runoff Collection System. As stated before there are two systems which may be used, open ditches or buried drain pipe. One or the other would be used but never both.

3.11.5.5.5.2.1 Buried drain pipe.

3.11.5.5.5.2.1.1 Since an overland flow facility would not nomally be operated when the site received a rainfall in excess of 0.5 inches in 24 hours, this criteria will be used to size the drainage system. The assumption of 100% runoff will be made for design purposes.

3.11.5.5.5.2.1.2 The pipe size will vary as runoff from each slope is added. Using the layout shown in Figure 3.11-4 and the assumed rainfall, the flow from one slope would be 0.149 ft^3/sec. The following assumptions are made:

Pipe will be concrete drain pipe.
Pipe will be flowing half full.
Pipe will be laid on a .2% slope.
Friction factor is 0.013.

3.11.5.5.5.2.1.3 Calculate pipe size.

$$CDIAN = 6.38(N)^{0.375}$$

where

CDIAN = diameter of concrete drain pipe, inches.

N = number of points at which flow is removed from header.

CDIAN must be one of the following 12, 15, 18, 21, 24, 27, 30, 33, 36, 42, 48, 54, 60, 66, 72, 78, 84, 90, or 96 inches.

3.11.5.5.5.2.1.4 As in sizing the header pipe, the collection pipe will vary in size. Start with N=1 and calculate pipe size. Then set N=N+1 and repeat the calculation. Redesignate N in this manner until N is equal to total number of take off points in header pipe calculation.

3.11.5.5.5.2.1.5 Again, the length of each size pipe required will be determined by summing the number of points at which the same diameter is calculated and multiply by 400 ft. To determine the amount of pipe for the entire field multiply by NH+1 since there will be NH+1 identical collection lines.

$$LCDIAN = (400)(NH+1) \sum NCDIAN$$

where

LCDIAN = length of drain pipe of given diameter, ft.

NCDIAN = number of points where same diameter pipe was chosen.

NH = number of headers.

3.11.5.5.5.2 Open ditches.

3.11.5.5.5.2.1 Assume that the ditches will be all cut and erosion control will be required in construction.

$$LDIT = (N)(400)(NH+1)$$

where

LDIT = total length of ditches for system, ft.

N = number of points at which flow is removed from header.

400 = length of ditch between slopes, ft.

NH = number of headers.

3.11.5.5.6 Calculate total land area required.

3.11.5.5.6.1 Ditches and service roads increase the area by 15%.

$$ADR = (.15)(TA)$$

where

ADR = area for ditches and roads, acres.

3.11.5.5.6.2 Area for storage lagoons.

$$ASL = \frac{1.2(NLC)(L)^2}{43,560}$$

where

ASL = area for storage lagoons, acres.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

1.2 = additional area required for cross levee.

3.11.5.5.6.3 Area for buffer zone.

Assume:

Buffer zone will be around entire treatment area.
Assume facility will be essentially square.

3.11.5.5.6.3.1 Calculate dimensions of treatment area.

$$LTA = \left(\frac{TA + ADR + ASL}{43,560}\right)^{0.5}$$

where

LTA = length of one side of treatment area, ft.

TA = treatment area, acres.

ADR = area for ditches and service roads, acres.

ASL = area for storage lagoons, acres.

3.11.5.5.6.3.2 Calculate area for buffer zone.

$$ABZ = 4WBZ\ (LTA + WBZ)$$

where

ABZ = area required for buffer zone, acres.

LTA = length of one side of treatment area, ft.

WBZ = width of buffer zone (input by user), ft.

3.11.5.5.6.4 Total land area.

$$TLA = TA + ADR + ASL + ABZ$$

where

TLA = total land area required, acres.

TA = treatment area, acres.

ADR = area required for ditches and service roads, acres.

ASL = area required for storage lagoons, acres.

ABZ = area required for buffer zones, acres.

3.11.5.5.7 Calculate fencing required

Assume

Entire facility is to be fenced.
Facility is square.

$$LF = 834.8\ (TLA)^{0.5}$$

where

LF = length of fence required, ft.

TLA = total land area required, acres.

834.8 = combined constants.

3.11.5.5.8 Calculate operation and maintenance manpower.

3.11.5.5.8.1 Distribution System.

3.11.5.5.8.1.1 If $TA \leq 70$;

$$OMMHD = 77.91\ (TA)^{0.5373}$$

3.11.5.5.8.1.2 If TA > 70;

$$OMMHD = 18.12\ (TA)^{0.8814}$$

where

OMMHD = operation and maintenance manpower for distribution, MH/yr.

TA = treatment area, acres.

3.11.5.5.8.2 Runoff collection by gravity pipe.

3.11.5.5.8.2.1 If TA ≤ 100;

$$OMMHP = 6.65\ (TA)^{0.4224}$$

3.11.5.5.8.2.2 If TA > 100;

$$OMMHP = 2.41\ (TA)^{0.6434}$$

where

OMMHP = operation and maintenance manpower for runoff collection by gravity pipe, MH/hr.

TA = treatment area, acres.

3.11.5.5.8.3 Runoff collection by open ditch.

3.11.5.5.8.3.1 If TA ≤ 150;

$$OMMHO = 36.9\ (TA)^{0.3578}$$

3.11.5.5.8.3.2 If TA > 150;

$$OMMHO = 8.34\ (TA)^{0.6538}$$

where

OMMHO = operation and maintenance manpower for runoff collection by open ditch, MH/yr.

TA = treatment area, acres.

3.11.5.5.9 Calculate operation and maintenance material costs.

3.11.5.5.9.1 Distribution System.

3.11.5.5.9.1.1 If TA ≤ 500;

$$OMMPD = 0.783(TA)^{-0.0673}$$

3.11.5.5.9.1.2 If TA > 500;

$$OMMPD = 9.46\ (TA)^{-0.47}$$

where

OMMPD = O&M material costs for distribution system as percent construction cost of distribution system, %.

TA = treatment area, acres.

3.11.5.5.9.2 Runoff collection by gravity pipe.

3.11.5.5.9.2.1 If TA ≤ 225;

$$OMMPP = 0.9566\ (TA)^{-0.2539}$$

3.11.5.5.9.2.2 If TA > 225;

$$OMMPP = .242\%$$

where

OMMPP = O&M material costs for runoff collection by gravity pipe as percent construction cost for gravity pipe system, %.

TA = treatment area, acres.

3.11.5.5.9.3 Runoff collection by open ditch.

3.11.5.5.9.3.1 If TA ≤ 60;

$$OMMPO = 25.4\ (TA)^{-0.1383}$$

3.11.5.5.9.3.2 If TA > 60;

$$OMMPO = 14.42$$

where

OMMPO = O&M material costs for runoff collection by open ditch as percent construction cost for open ditch system, %.

3.11.5.5.10 Other construction cost items. The quantities computed account for approximately 90% of the construction cost of the systems. Other miscellaneous costs such as final land leveling, connecting piping for lagoons, miscellaneous concrete structures, etc., make up the additional 10%.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction cost.

3.11.5.6 Quantities Calculation Output Data.

3.11.5.6.1 Volume of earthwork required for lagoon construction, VLEW ft^3.

3.11.5.6.2 Diameter of lateral pipes, DIAL, inches.

3.11.5.6.3 Diameters of header pipes, DIAHN, Inches.

3.11.5.6.4 Length of lateral pipes, LDIAL, ft.

3.11.5.6.5 Length of header pipes, LDIAHN, ft.

3.11.5.6.6 Number of nozzles, NON.

3.11.5.6.7 Diameter of butterfly valves, DBV, inches.

3.11.5.6.8 Number of butterfly valves, NBV.

3.11.5.6.9 Diameter of lateral valves, DLV, inches.

3.11.5.6.10 Number of lateral valves, NLV.

3.11.5.6.11 Area which requires heavy clearing, CAGH, acres.

3.11.5.6.12 Area which requires medium clearing, CAGM, acres.

3.11.5.6.13 Area which requires light clearing, CAGL, acres.

3.11.5.6.14 Volume of earthwork required for slope construction, VSEW, ft^3.

3.11.5.6.15 Volume of earthwork for terraces, VET, ft^3.

3.11.5.6.16 Diameter of concrete drain pipe, CDIAN, inches.

3.11.5.6.17 Length of drain pipe of size CDIAN, LCDIAN, ft.

3.11.5.6.18 Total length of ditches for system, LDIT, ft.

3.11.5.6.19 Total land area required, TLA, acres.

3.11.5.6.20 Length of fence required, LF, ft.

3.11.5.6.21 Operation and maintenance manpower for distribution system, OMMHD, MH/yr.

3.11.5.6.22 Operation and maintenance manpower for runoff collection by gravity pipe, OMMHP, MH/yr.

3.11.5.6.23 Operation and maintenance manpower for runoff collection by open ditch, OMMHO, MH/yr.

3.11.5.6.24 Operation and maintenance material costs for distribution system as percent construction cost of distribution system, OMMPD, %.

3.11.5.6.25 Operation and maintenance material costs for runoff collection by gravity pipe as percent construction cost of gravity pipe system, OMMPP, %.

3.11.5.6.26 Operation and maintenance cost for runoff collection by open ditch as percent construction cost for open ditch system, OMMPO, %.

3.11.5.6.27 Correction factor for other construction cost, CF.

3.11.5.6.28 Number of headers, NH.

3.11.5.7 Unit Price Input Required.

3.11.5.7.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.11.5.7.2 Cost of standard size pipe (12"Ø), COSP, $/ft.

3.11.5.7.3 Cost of standard size valve (12"Ø butterfly), COSTSV, $.

3.11.5.7.4 Cost per sprinkler, COSTEN, $.

3.11.5.7.5 Cost of standard size drain pipe (24"Ø R.C. pipe), COSTCP, $/ft.

3.11.5.7.6 Unit price input for heavy clearing and grubbing, UPICG, $/acre.

3.11.5.7.7 Unit price input for fencing, UPIF, $/ft.

3.11.5.8 Cost Calculations.

3.11.5.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW+VSEW+VET}{27} \; UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

VSEW = volume of earthwork required for slope construction, ft^3.

VET = volume of earthwork for terraces, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

3.11.5.8.2 Cost of Header pipes.

3.11.5.8.2.1 Calculate total installed cost of header pipes.

$$TICHP = ICHPN$$

where

TICHP = total installed cost of header pipes, \$.

ICHPN = installed cost of various size header pipes, \$.

3.11.5.8.2.2 Calculate installed cost of each size header pipes.

$$ICHPN = (LDIAHN)\frac{(COSTPN)}{100}(COSTSP)$$

where

ICHPN = installed cost of various size header pipes, \$.

LDIAHN = length of header pipes of size DIAHN, ft.

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

3.11.5.8.2.3 Calculate COSTPN

$$COSTPN = 6.842\ (DIAHN)^{1.2255}$$

where

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

DIAHN = diameters of header pipes, inches.

3.11.5.8.2.4 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is $13.50 per foot in 4th quarter of 1977.

3.11.5.8.3 Cost of lateral pipes.

3.11.5.8.3.1 Calculate total installed cost of lateral pipes.

$$TICLP = (LDIAL) \frac{(COSTP)}{100} (COSP)$$

where

TICLP = total installed cost of lateral pipes, $.

LDIAL = length of lateral pipes, ft.

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), $/ft.

3.11.5.8.3.2 Calculate COSTP.

$$COSTP = 6.842 \; (DIAL)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

DIAL = diameter of lateral pipes, inches.

3.11.5.8.3.3 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is $13.50 per foot in 4th quarter of 1977.

3.11.5.8.4 Calculate cost of butterfly valves.

3.11.5.8.4.1 Calculate installed cost of butterfly valves.

$$COSTBV = \frac{(COSTRV)(COSTSV)(NBV)}{100}$$

where

COSTBV = installed cost of butterfly valves, $.

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

COSTSV = cost of standard size valve (12"$\emptyset$), \$.

NBV = number of butterfly valves.

3.11.5.8.4.2 Calculate COSTRV.

$$COSTRV = 3.99\ (DBV)^{1.395}$$

where

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

DBV = diameter of butterfly valves, inches.

3.11.5.8.4.3 Determine COSTSV. COSTSV is the cost of a 12" $\emptyset$ butterfly valve suitable for water service. This cost is \$1004 for the 4th quarter of 1977.

3.11.5.8.5 Calculate cost of lateral valves.

3.11.5.8.5.1 Calculate installed cost of lateral valves.

$$COSTLV = \frac{(COSTRL)(COSTSV)(NLV)}{100}$$

where

COSTLV = installed cost of lateral valves, \$.

COSTRL = cost of lateral valve of size DLV as percent of cost of standard valve, %.

COSTSV = cost of standard size valve (12"$\emptyset$ butterfly), \$.

NLV = number of lateral valves.

3.11.5.8.5.2 Calculate COSTRL.

$$COSTRL = 15.33\ (DLV)^{1.053}$$

where

COSTRL = cost of lateral valve of size DLV as percent of cost of standard valve, %.

DLV = diameter of lateral valve, inches.

3.11.5.8.5.3 Determine COSTSV. COSTSV is the cost of a 12" $\emptyset$ butterfly valve suitable for water service. This cost is \$1004 for the 4th quarter of 1977.

3.11.5.8.6 Calculate cost of sprinklers.

3.11.5.8.6.1 Calculate installed cost of sprinklers.

$$COSTN = (1.2)(NON)(COSTEN)$$

where

COSTN = installed cost of nozzles, \$.

NON = number of nozzles.

COSTEN = cost per nozzle, \$.

1.2 = 20% cost of installation.

3.11.5.8.6.2 Determine COSTEN. The cost of an impact type rotor pop-up full circle sprinkler with a flow from 6 to 15 gpm, for the 4th quarter of 1977 is \$65.00.

$$COSTEN = \$65.00.$$

For better cost estimation, COSTEN should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTEN = \$65.00 \frac{MSECI}{518.4}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

518.4 = Marshall and Swift Cost Index 4th quarter 1977.

3.11.5.8.7 Calculate total cost of distribution system.

$$TCDS = TICHP + TICLP + COSTBV + COSTLV + COSTN$$

where

TCDS = total cost of distribution system, \$.

TICHP = total installed cost of header pipe, \$.

TICLP = total installed cost of lateral pipe, \$.

COSTBV = installed cost of butterfly valves, \$.

COSTLV = installed cost of lateral valves, \$.

COSTN = installed cost of nozzles, \$.

3.11.5.8.8 Detemine cost of runoff collection by open ditch.

$$COSTOD = (.57)(LDIT)UPIEX$$

where

COSTOD = cost of runoff collection by open ditch, \$.

LDIT = total length of ditches for system, ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.11.5.8.9 Detemine cost of runoff collection by gravity pipe.

3.11.5.8.9.1 Calculate total installed cost of collection system.

$$TICRC = \sum ICRCN$$

where

TICRC = total installed cost of runoff collection by gravity pipe, \$.

ICRCN = installed cost of each size drain pipe, \$.

3.11.5.8.9.2 Calculate installed cost of each size drain pipe.

$$ICRCN = \frac{(LCDIAN)(COSTRN)(COSTSC)}{100}$$

where

ICRCN = installed cost of each size drain pipe, \$.

LCDIAN = length of drain pipe of size CDIAN, ft.

COSTRN = cost of drain pipe of size CDIAN as percent of cost of standard size (24"Ø) drainage pipe, %.

COSTCP = cost of standard size (24"Ø R.C. pipe), \$/ft.

3.11.5.8.9.3 Calculate COSTRN.

$$COSTRN = 0.7044\ (CDIAN)^{1.6587}$$

where

COSTRN = cost of gravity pipe of size CDIAN as percent of cost of standard size (24"Ø) gravity pipe, %.

CDIAN = diameter of concrete drain pipe, inches.

3.11.5.8.9.4 Determine COSTCP. COSTCP is the cost per foot of 24"Ø reinforced concrete drain pipe with gasket joints. This cost for the 4th quarter of 1977 is $10.76/ft.

$$COSTSC = \$10.76/ft.$$

3.11.5.8.10 Calculate cost for clearing and grubbing.

$$COSTCG = (CAGH + 0.306\ CAGM + 0.092\ CAGL)\ UPICG$$

where

COSTCG = cost for clearing and grubbing site, $.

CAGH = area which requires heavy clearing, acres.

CAGM = area which requires medium clearing, acres.

CAGL = area which requires light clearing, acres.

UPICG = unit price input for heavy clearing and grubbing, $/acre.

3.11.5.8.11 Calculate cost of fencing.

$$COSTF = (LF)(UPIF)$$

where

COSTF = installed cost of fencing, $.

LF = length of fencing required, ft.

UPIF = unit price input for fencing, $/ft.

3.11.5.8.12 Calculate O & M material costs.

$$OMMC = \frac{(OMMPD)(TCDS) + (OMMPP)(TICRC) + (OMMPO)(COSTOD)}{100}$$

where

OMMC = total O&M material cost, $.

OMMPD = O&M material costs for distribution system as percent construction cost for distribution system, %.

TCDS = total cost of distribution system, $.

OMMPP = O&M material costs for runoff collection by gravity pipe as percent of construction cost of gravity pipe system, %.

TICRC = total installed cost of runoff collection by gravity pipe, $.

OMMPO = O&M material cost for runoff collection by open ditch as percent construction cost for open ditch system, %.

COSTOD = cost of runoff collection by open ditch, $.

3.11.5.8.13 Total bare construction cost.

$$TBCCOF = (1.11)(TCDS+TICRC+COSTOD+COSTE+COSTCG+COSTF+COSTL)$$

where

TBCCOF = total bare construction cost for overland flow, $.

TCDS = total cost of distribution system, $.

TICRC = total installed cost of runoff collection by gravity pipe, $.

COSTOD = cost of runoff collection by open ditch, $.

COSTE = cost of earthwork, $.

COSTCG = cost of clearing and grubbing site, $.

COSTF = installed cost of fencing, $.

COSTL = cost of land for facility, $.

3.11.5.9 <u>Cost Calculations Output Data</u>.

3.11.5.9.1 Total bare construction cost for overland flow, TBCCOF, $.

3.11.5.9.2 O&M material cost, OMMC, $/yr.

3.11.6 Rapid Infiltration.

3.11.6.1 Input Data.

3.11.6.1.1 Wastewater flow, Q, mgd.

3.11.6.1.1.1 Minimum flow, mgd.

3.11.6.1.1.2 Average flow, mgd.

3.11.6.1.1.3 Maximum flow, mgd.

3.11.6.1.2 Wastewater characteristics.

3.11.6.1.2.1 Suspended solids, mg/l.

3.11.6.1.2.2 Volatile suspended solids, % of suspended solids.

3.11.6.1.2.3 Settleable solids, mg/l.

3.11.6.1.2.4 BOD_5 (soluble and total), mg/l.

3.11.6.1.2.5 COD (soluble and total), mg/l.

3.11.6.1.2.6 Phosphorus (as PO_4), mg/l.

3.11.6.1.2.7 Total kjeldahl nitrogen (TKN), mg/l.

3.11.6.1.2.8 Ammonia-nitrogen, NH_3, mg/l.

3.11.6.1.2.9 Nitrite-nitrogen, NO_2, mg/l.

3.11.6.1.2.10 Nitrate-nitrogen, NO_3, mg/l.

3.11.6.1.2.11 Temperature, °C.

3.11.6.1.2.12 pH, units.

3.11.6.1.2.13 Oil and grease, mg/l.

3.11.6.1.2.14 Cations, mg/l.

3.11.6.1.2.15 Anions, mg/l.

3.11.6.2 Design Parameters.

3.11.6.2.1 Application rate, L_w, in/wk = 4.0 - 150 in/wk.

3.11.6.2.2 Precipitation rate, P_r, in/wk.

3.11.6.2.3 Evapotranspiration, ET, in/wk.

3.11.6.2.4 Runoff, R, in/wk = 0.0.

3.11.6.2.5 Wastewater generation period, WWGP, days/yr.

3.11.6.2.6 Field application period, FAP, wks/yr.

3.11.6.2.7 Recovery systems (specify one).

3.11.6.2.7.1 Recovery wells (number; diameter, in., and depth, ft).

3.11.6.2.7.2 Underdrains.

3.11.6.2.7.3 No recovery.

3.11.6.2.8 Buffer width, ft (site dependent) = 0.0 - 500 ft.

3.11.6.2.9 Monitoring wells.

3.11.6.2.9.1 Number.

3.11.6.2.9.2 Depth per well, ft.

3.11.6.2.10 Fraction denitrified, D, % = 30-60%.

3.11.6.2.11 Ammonia volatilization, AV, % = 0.0%.

3.11.6.2.12 Removal of phosphorus, % = 90%.

3.11.6.3 <u>Process Design Calculations</u>.

3.11.6.3.1 Calculate total nitrogen concentration, C_n, in the applied wastewater.

$$C_n = (TKN)_i + (NO_2)_i + (NO_3)_i$$

where

C_n = total nitrogen concentration in applied wastewater, mg/l.

$(TKN)_i$ = total Kjeldahl nitrogen concentration in applied wastewater, mg/l.

$(NO_2)_i$ = nitrite-N concentration in applied wastewater, mg/l.

$(NO_3)_i$ = nitrate-N concentration in applied wastewater, mg/l.

3.11.6.3.2 Calculate wastewater nitrogen loading, L_n, lbs/acre-d.

$$L_n = 11.77\ C_n L_w$$

where

L_n = wastewater nitrogen loading, lbs/acre-yr.

L_w = wastewater hydraulic loading rate, in/wk.

3.11.6.3.3 From water balance, calculate percolating water rate, W_p, in/wk.

$$W_p = L_w + (P_r - ET) - R$$

where

W_p = percolating water rate, in/wk.

P_r = design precipitation, in/wk.

ET = evapotranspiration (or crops consumptive use of water), in/wk.

R = net runoff, in/wk.

3.11.6.3.4 Calculate total nitrogen loading, $(L_t)_N$, lb/acre-yr.

$$(L_t)_N = L_n + 11.77\ (P_r)(0.5)$$

where

$(L_t)_N$ = total nitrogen loading rate, lb/acre-yr.

0.5 = assumed nitrogen concentration in precipitation water, mg/l.

3.11.6.3.5 Assume crop nitrogen uptake, $(U)_N$, lb/acre-yr.

3.11.6.3.6 Calculate nitrogen loss through denitrification, D, lb/acre-yr.

$$D = (D_f)(L_t)_N/(100)$$

where

D = nitrogen loss through denitrification, lb/acre-yr.

D_f = nitrogen loss as a percent of total applied nitrogen, %.

3.11.6.3.7 Calculate nitrogen loss due to volatilization, AV, lb/acre-yr.

$$AV = (AV)_f \ (L_t)_N/(100)$$

where

AV = nitrogen loss due to volatilization, lb/acre-yr.

$(AV)_f$ = percent of total nitrogen applied lost to volatilization, %.

3.11.6.3.8 Calculate sum of nitrogen losses, $(\Sigma N)_L$, lb/acre-yr.

$$(\Sigma N)_L = (U)_N + D + AV$$

where

$(\Sigma N)_L$ = sum of total nitrogen lost, lb/acre-yr.

3.11.6.3.9 Check total nitrogen against 0.8 $(L_t)_N$.

$$(\Sigma N)_L \quad 0.8 \ (L_t)_N$$

$$\text{if } (\Sigma N)_L \quad 0.8 \ (L_t)_N$$

$$\text{set } (\Sigma N)_L = 0.8 \ (L_t)_N$$

3.11.6.3.10 From nitrogen balance, calculate nitrogen concentration in percolate, $(C_p)_N$, mg/l.

$$(L_t)_N = (11.77)(W_p)(C_p)_N + (\Sigma N)_L$$

$$(C_p)_N = [(L_t)_N - (\Sigma N)_L]/(11.77)(W_p)$$

where

$(C_p)_N$ = nitrogen concentration in percolate, mg/l.

3.11.6.3.11 Calculate required treatment acre, TA, acres.

$$TA = (258.5)(Q)/(L_w)$$

where

TA = required field area, acres.

Q = average wastewater flow, mgd.

3.11.6.3.12 Calculate phosphorus loading, L_p, lb/acre-yr.

$$L_p = 11.77\ (TP)_i(L_w)$$

where

L_p = total phosphorus loading, lbs/acre-yr.

$(TP)_i$ = total phosphorus concentration in applied wastewater, mg/l.

3.11.6.3.13 Calculate soil removal of phosphorus, U_p, lb/acre-yr.

$$(U_p) = 0.891\ [(94.544 - 0.0041)(L_p)]$$

where

U_p = soil removal of phosphorus, lbs/acre-yr.

3.11.6.3.14 From phosphorus mass balance, calculate phosphorus concentration of percolate water.

$$(C_p)_p = [(L_p) - (SRP) - (U_p)]/(11.77)(W_p)$$

where

$(C_p)_p$ = phosphorus content of percolate water, mg/l. $(0.10)\ (TP)_i$

3.11.6.3.15 Calculate percolate rate, W_p, mgd.

$$W_p\ (\text{mgd}) = [(W_p)\ \text{in/wk}\ (\frac{1\ \text{ft}}{12\ \text{in}})(\frac{1\ \text{wk}}{7\ \text{day}})]\ (TA)\ \text{acre}\ (43{,}560\ \frac{\text{ft}^2}{\text{acre}})(7.48\ \frac{\text{gal}}{\text{ft}^3})(1/10^6)$$

3.11.6.3.16 Calculate suspended solids concentration in percolate, mg/l, assume 99% removal.

$$(SS)_p = (0.01)(SS)_i$$

where

$(SS)_p$ = suspended solids concentration in percolate, mg/l.

$(SS)_i$ = suspended solids concentration in applied wastewater, mg/l.

3.11.6.3.17 Calculate total and soluble BOD_5 concentration in percolate, mg/l, assume 99% removal of total BOD_5.

$$(TBOD_5)_p = (TBOD_5)_i \ (0.01)$$

$$(SBOD_5)_p = (SBOD_5)_i \ (0.01)$$

where

$(TBOD_5)_p$ = total BOD_5 concentration in percolate, mg/l.

$(TBOD_5)_i$ = total BOD_5 concentration in applied wastewater, mg/l.

$(SBOD_5)_p$ = soluble BOD_5 in percolate, mg/l.

$(SBOD_5)_i$ = soluble BOD_5 in applied wastewater, mg/l.

3.11.6.3.18 Calculate total and soluble COD concentration in percolate, mg/l.

$$(TCOD)_p = [(TCOD)_i - (TBOD_5)_i] \ (0.5) + 0.01 \ (TBOD_5)_i$$

$$(SCOD)_p = [(SCOD)_i - (SBOD_5)_i] \ (0.5) + 0.01 \ (SBOD_5)_i$$

where

$(TCOD)_p$ and $(TCOD)_i$ = total COD concentration in percolate and applied wastewater, respectively, mg/l.

$(SCOD)_p$ and $(SCOD)_i$ = soluble COD concentration in percolate and applied wastewater, respectively, mg/l.

3.11.6.3.19 Nitrite-N concentration in percolate = 0.0.

3.11.6.3.20 Nitrate-N concentration in percolate = $(C_p)_N$.

3.11.6.3.21 Ammonia-N concentration in percolate = 0.0. Total Kjeldahl-nitrogen concentration in percolate = 0.0.

3.11.6.3.22 Oil and grease concentration in percolate = 0.0.

3.11.6.4 Process Design Output Data.

3.11.6.4.1 Application rate, in/week.

3.11.6.4.2 Evapotranspiration rate, in/week.

3.11.6.4.3 Precipitation rate, in/week.

3.11.6.4.4 Runoff, in/week.

3.11.6.4.5 Percent denitrified, percent.

3.11.6.4.6 Percent ammonia volatilization, percent.

3.11.6.4.7 Removal of phosphorus, percent.

3.11.6.4.8 Wastewater generation period, days/yr.

3.11.6.4.9 Field application period, weeks/yr.

3.11.6.4.10 Surface flooding.

3.11.6.4.11 Buffer zone width, feet.

3.11.6.4.12 Number of monitoring wells, wells,

3.11.6.4.13 Depth of monitoring wells, feet.

3.11.6.4.14 Treatment area required, acres.

3.11.6.4.15 Volume of percolate, mgd.

3.11.6.4.16 Quality of percolate.

3.11.6.4.16.1 Suspended solids, mg/l.

3.11.6.4.16.2 Volatile solids, percent.

3.11.6.4.16.3 BOD_5, mg/l.

3.11.6.4.16.4 BOD_5 soluble, mg/l.

3.11.6.4.16.5 COD, mg/l.

3.11.6.4.16.6 COD soluble, mg/l.

3.11.6.4.16.7 PO_4, mg/l.

3.11.6.4.16.8 TKN, mg/l.

3.11.6.4.16.9 NO_2, mg/l.

3.11.6.4.16.10 NO_3, mg/l.

3.11.6.4.16.11 Oil and grease, mg/l.

3.11.6.5 <u>Quantities Calculations</u>.

3.11.6.5.1 Distribution Pumping. Distribution pumping will be taken from the section entitled "Intemediate Pumping".

3.11.6.5.2 Detemine number and size of basins required.

3.11.6.5.2.1 Assume:

Use minimum depth of 4 feet.
Use a minimum of 4 infiltration basins.
Infiltration basins will be a maximum of 10 acres in area and will be square.

3.11.6.5.2.2 If TA $\leq$ 4 acres

$$NIB = 2$$

$$IBA = \frac{TA}{2}$$

If IBA .1 set IBA = .1

where

TA = treatment area, acres.

NIB = number of infiltration basins.

IBA = area of individual infiltration basins, acres.

3.11.6.5.2.3 If TA is less than or equal to 40 acres use 4 equal sized basins.

$$\text{If } 4 < TA \leq 40;\ NIB = 4$$

$$IBA = \frac{TA}{4}$$

where

TA = treatment area

NIB = number of infiltration basins

IBA = area of individual infiltration basins

3.11.6.5.2.4 If TA is greater than 40 acres.

$$TA > 40;\ NIB = \frac{TA}{10}$$

NIB must be an integer.

$$IBA = \frac{TA}{NIB}$$

3.11.6.5.3 Calculate volume of earthwork for basins.

3.11.6.5.3.1 Assume:

Levees will be built on top of natural ground with fill hauled in from off the site.
Levee side slopes will be 3 to 1.
Top of the levee will be 10 feet wide.
Basins will be 4 feet deep.
Basins will be square.

3.11.6.5.3.2 Calculate dimensions of basins.

$$L = 208.7\ (IBA)^{0.5}$$

where

L = Length of one side of the basin.

3.11.6.5.3.3 Volume of earthwork.

$$V_{ew} = NIB\ (352L + 11{,}968)$$

where

V_{ew} = volume of fill required to construct levees.

L = length of one side of the basin.

NIB = number of infiltration basins.

3.11.6.5.4 Calculate header size to feed infiltration basins.

If GF $\leq$ 40 mgd calculate PIPE using GF if GF > 40 mgd calculate PIPE using GF/2.
Assume velocity (V) = 4 fps.

$$PIPE = 8.42\ (GF)^{0.5}$$

Pipe must be one of the following: 2,3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

Check velocity (V) using selected pipe size.

$$V = \frac{(283.6)\ (GF)}{(PIPE)^2} \text{ or } \frac{(283.6)\ (GF/2)}{(PIPE)^2}$$

If $V \leq 1$ fps use next smallest diameter.

If $V > 5$ fps use next largest diameter.

where

PIPE = diameter of pipe (inches).

GF = generated flow (mgd).

V = velocity of water in pipe (fps).

3.11.6.5.5 Calculate quantity of header pipe required.

If $GF \leq 40$ mgd LPIPE = (NIB)(L)

If $GF > 40$ mgd LPIPE = 2(NIB)(L)

where

LPIPE = length of header pipe required, ft.

3.11.6.5.6 Calculate pipe size for lateral to each infiltration basin.

3.11.6.5.6.1 Calculate flow.

$$FLOW = (.012)(AR)(IBA)$$

If FLOW ≤ 62 ft^3/sec calculate DIA using FLOW.

If FLOW > 62 ft^3/sec calculate DIA using FLOW/2.

3.11.6.5.6.2 Calculate diameter.

Assume velocity (V) = 4 fps.

$$DIA = 6.77 \ (FLOW)^{0.5}$$

DIA must be one of the following 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 30, 36, 42, 48.

3.11.6.5.6.3 Select diameter closest to the calculated diameter.

3.11.6.5.6.4 Check the velocity (V) using the selected diameter.

$$V = \frac{(183.3)\ FLOW}{(DIA)^2}$$

If $V \leq 1$ fps use next smallest diameter.

If $V > 5$ fps use next largest diameter.

where

FLOW = the wastewater flow to each basin (ft^3/sec).

DIA = diameter of lateral pipe (inches).

V = velocity of water in pipe (ft/sec).

AR = application rate, (in/wk).

3.11.6.5.7 Determine size and number of valves for distribution system.

3.11.6.5.7.1 Assume:

Each lateral will have a valve to cut off flow to that infiltration basin.
Valves will be butterfly valves suitable for use in water service.

Valves will be the same size as lateral pipe (DIA).

3.11.6.5.7.2 If FLOW ≤ 62 ft^3/sec.

NBV = NIB

3.11.6.5.7.3 If FLOW > 62 ft^3/sec

NBV = 2 NIB

where

NBV = Number of butterfly valves.

NIB = Number of infiltration basins.

FLOW = the wastewater flow to each basin, ft^3/sec

DIA = diameter of lateral pipe, inches.

3.11.6.5.8 Calculate quantity of lateral pipe required.

If FLOW $\leq$ 62 ft^3/sec; LLAT = (100) (NIB)

If FLOW $>$ 62 ft^3/sec; LLAT = (2) (100) (NIB)

where

FLOW = the wastewater flow to each basin (ft^3/sec).

LLAT = length of lateral pipe of diameter DIA, (ft).

NIB = number of infiltration basins.

3.11.6.5.9 Recovery of renovated water.

Two recovery systems are commonly used, underdrains and recovery wells. The user must designate which system is to be used if the water is to be recovered.

3.11.6.5.9.1 Underdrain system.

3.11.6.5.9.1.1 The following assumptions are made:

Perforated PVC pipe 6 inches in diameter will be used for underdrain laterals in basins.
100% of the applied wastewater will be recovered.
The 6 inch pipe will be laid on 1% slope and assumed to flow 1/2 full.
Concrete sewer pipe will be used as collection headers.

3.11.6.5.9.1.2 Calculate quantity of underdrain pipe required.

$$DPIPE = (.0105)(L)(IBA)(AR)(NIB)$$

where

DPIPE = length of 6" drain pipe required (ft).

L = length of one side of infiltration basin, (ft).

IBA = area of individual infiltration basins, (acres).

AR = application rate, (inches/wk).

NIB = number of infiltration basins.

0.0105 = accumulated constants.

3.11.6.5.9.1.3 Calculate size and quantity of collection header pipe.

Assume:

Class III concrete sewer pipe will be used.
Pipe will be laid on 1% slope.
Pipe will be sized by Manning formula assumed flowing half full with "N" factor 0.013.

$$CDIA = 9.56\ (FLOW)^{0.375}$$

where

CDIA = diameter of collection header pipe, inches.

FLOW = the wastewater flow to each basin, ft^3/sec.

9.56 = accumulated constants.

$$LDCH = (L)(NIB)$$

where

LDCH = length of drain collection header pipe of diameter CDIA, ft.

L = length of one side of infiltration basins, ft.

NIB = number of infiltration basins.

3.11.6.5.9.2 Recovery wells.

User must specify number of wells (NW), size of wells (WDIA), and depth of wells (DW).

where

NW = number of recovery wells required.

WDIA = diameter of recovery wells, inches.

DW = depth of recovery wells, ft.

3.11.6.5.10 Monitoring System.

Monitoring wells shall be 4" in diameter. User must specify the number of monitoring wells, (NMW) and depth of monitoring wells (DMW).

where

NMW = number of monitoring wells.

DMW = depth of monitoring wells, (ft).

3.11.6.5.11 Operation and maintenance manpower requirements.

3.11.6.5.11.1 Distribution System.

3.11.6.5.11.1.1 If $TA \leq 15$

$$OMMHD = 128.5\ (TA)^{0.6285}$$

3.11.6.5.11.1.2 If $TA > 15$

$$OMMHD = 78.8\ (TA)^{0.8092}$$

where

TA = treatment area, acres.

OMMHD = operation and maintenance manpower for distribution, MH/yr.

3.11.6.5.11.2 Water recovery by wells.

$$OMMHW = 384.64\ (GF)^{0.5981}$$

where

OMMHW = operation and maintenance manpower for water recovery by wells, MH/yr.

GF = generated flow, mgd.

3.11.6.5.11.3 Water recovery by underdrains.

3.11.6.5.11.3.1 If $TA \leq 80$

$$OMMHU = 54.71\ (TA)^{-0.2414}$$

3.11.6.5.11.3.2 If $TA > 80$

$$OMMHU = 10.12\ (TA)^{0.6255}$$

where

OMMHU = operation and maintenance manpower for water recovery by underdrains, MH/yr.

TA = treatment area, acres.

3.11.6.5.11.4 Monitoring wells.

$$OMMHM = 6.39\,(NMW)\,(DMW)^{0.2760}$$

where

OMMHM = operation and maintenance manpower for monitoring wells, MH/yr.

NMW = number of monitoring wells.

DMW = depth of monitoring wells, ft.

3.11.6.5.12 Operation and Maintenance Materials Cost. This item includes repair and replacement material costs. It is expressed as a percentage of the capital costs for the various areas of construction of the rapid infiltration system.

3.11.6.5.12.1 Distribution System.

3.11.6.5.12.1.1 If $TA \leq 19$

$$OMMPD = 2.64 \, (TA)^{-0.2101}$$

3.11.6.5.12.1.2 IF $TA > 19$

$$OMMPD = 1.59 \, (TA)^{-0.0399}$$

where

OMMPD = O&M material cost for distribution system as percentage of construction cost of distribution system.

TA = treatment area, acres.

3.11.6.5.12.2 Water recovery by wells.

3.11.6.5.12.2.1 If $GF \leq 5$

$$OMMPW = 1.53 \, (GF)^{0.6570}$$

3.11.6.5.12.2.2 If $5 < GF \leq 10$

$$OMMPW = 2.76 \, (GF)^{0.2894}$$

3.11.6.5.12.2.3 If $GF > 10$

$$OMMPW = 4.55 \, (GF)^{0.0715}$$

where

OMMPW = O&M material cost for water recovery wells as percentage of construction cost of recovery wells.

GF = generated flow, mgd.

3.11.6.5.12.3 Water recovery by underdrains.

3.11.6.5.12.3.1 If T $\leq$ 200

$$OMMPU = 14.13\ (TA)^{-0.1392}$$

3.11.6.5.12.3.2 If TA >200

$$OMMPU = 30.95\ (TA)^{-0.2860}$$

where

OMMPU = O&M material costs for water recovery by underdrains as percentage of construction cost of underdrains.

TA = Treatment area, acres.

3.11.6.5.12.4 Monitoring wells.

$$OMMPM = 2.28\ (DMW)^{0.0497}$$

where

OMMPM = O&M material cost for monitoring wells as percentage of construction cost of monitoring wells.

DMW = depth of monitoring wells.

3.11.6.5.13 Electrical energy requirements.

3.11.6.5.13.1 Recovery wells.

Assume:

Pump efficiency of 60%.
Motor efficiency of 90%.
Total head is equal to the well depth plus 40 ft.

$$KWH = (12.6)\ (GF)\ (DW+40)\ (DPW)\ (HPD)$$

where

KWH = energy required, kwhr/yr.

GF = generated flow, mgd.

DW = depth of well, ft.

DPW = days per week of operations, day/wk.

HPD = hours per day of operation, hr/day.

3.11.6.5.14 Other construction cost items.

The quantities and items computed account for approximately 85% of the cost of the systems. Other miscellaneous items such as concrete head walls, pneumatic piping, etc., make up the other 15%.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other construction costs.

3.11.6.6 Quantities Calculations Output Data.

3.11.6.6.1 Area of individual infiltration basins, IBA, acres.

3.11.6.6.2 Number of infiltration basins, NIB.

3.11.6.6.3 Length of side of basin, L, ft.

3.11.6.6.4 Volume of earthwork for infiltration basins, V_{ew}, cu.ft.

3.11.6.6.5 Diameter of distribution header, PIPE, inches.

3.11.6.6.6 Length of distribution header pipe required, LPIPE, ft.

3.11.6.6.7 Diameter of lateral pipe, DIA, inches.

3.11.6.6.8 Length of lateral pipe of diameter DIA, LLAT, ft.

3.11.6.6.9 Length of 6 inch diameter drain pipe required, DPIPE, ft.

3.11.6.6.10 Number of recovery wells, NW.

3.11.6.6.11 Diameter of recovery wells, WDIA, inches.

3.11.6.6.12 Depth of recovery wells, DW, ft.

3.11.6.6.13 Number of monitoring wells, NMW.

3.11.6.6.14 Depth of monitoring wells, DMW, ft.

3.11.6.6.15 Operation and Maintenance manpower for distribution, OMMHD, MH/yr.

3.11.6.6.16 Operation and maintenance manpower for recovery wells, OMMHU, MH/yr.

3.11.6.6.17 Operation and maintenance manpower for underdrain system, OMMHU, MH/yr.

3.11.6.6.18 Operation and maintenance manpower for monitoring wells, OMMHM, MH/yr.

3.11.6.6.19 O&M material costs for distribution system, OMMPD, %.

3.11.6.6.20 O&M material costs for recovery wells, OMMPW, %.

3.11.6.6.21 O&M material cost for underdrain system, OMMPU, %.

3.11.6.6.22 O&M material costs for monitoring wells, OMMPM, %.

3.11.6.6.23 Diameter of drain collection header pipe, CDIA, inches.

3.11.6.6.24 Length of drain collection header pipe, LDCH, ft.

3.11.6.6.25 Number of butterfly vlaves, NBV.

3.11.6.6.26 Energy required for recovery wells, KWH, Kw hr/yr.

3.11.6.6.27 Correction factor for other construction costs, CF.

3.11.6.7 <u>Unit Price Input Required</u>.

3.11.6.7.1 Unit price input for earthwork assuming hauled from offsite and compacted, UPIEW, $/cu yd.

3.11.6.7.2 Cost of standard size steel pipe (12"Ø), COSP, $/ft.

3.11.6.7.3 Cost of standard size butterfly valve (12"Ø) COSTSV, $.

3.11.6.7.4 Unit price input for 6" PVC perforated drain pipe, UPIPP, $/ft.

3.11.6.7.5 Cost of standard size (24"Ø) reinforced concrete drain pipe (Class III) COSTCP, $/ft.

3.11.6.8 <u>Cost Calculations</u>.

3.11.6.8.1 Cost of earthwork.

$$COSTE = \frac{(V_{ew})\ (UPIEW)}{27}$$

where

COSTE = Cost of earthwork for levees, \$.

V_{ew} = Volume of earthwork for infiltration basins, cu ft

UPIEW = unit price input for earthwork assuming hauled from offsite and compacted, \$/cu.yd.

3.11.6.8.2 Cost of header pipe.

3.11.6.8.2.1 Calculate installed cost header pipe (excluding trenching and backfilling).

$$COSTP = \frac{(COSP)\ (COSTRP)}{100}$$

where

COSTP = cost of pipe of diameter PIPE, \$/ft.

COSP = cost of standard size pipe (12" diameter), \$/ft.

COSTRP = cost of pipe of diameter PIPE as percent of cost of standard size pipe, percent.

3.11.6.8.2.2 Calculate COSTRP.

$$COSTRP = 5.48\ (PIPE)^{1.1655}$$

where

PIPE = diameter of header pipe, inches.

COSTRP = cost of pipe of diameter PIPE as percent of cost of standard size pipe, percent.

3.11.6.8.2.3 Determine COSP.

COSP is the cost per foot of 12" diameter welded steel pipe in place (excluding cost for trenching and backfilling).

3.11.6.8.2.4 Calculate cost for trenching and backfilling. This cost is computed as a fraction of the cost of the pipe.

If PIPE 12" $EBF = 0.334\ (PIPE)^{-0.6840}$

If PIPE 12" $EBF = 0.061$

where

EBF = fraction of pipe cost for trenching and backfilling.

PIPE = diameter of header pipe, inches.

3.11.6.8.2.5 Total installed cost of header pipe.

$$TICHP = (1 + EBF)\ (COSTP)\ (LPIPE)$$

where

TICHP = total installed cost of header pipe, \$.

EBF = fraction of pipe cost for trenching and backfilling.

COSTP = cost of pipe of diameter PIPE, \$/ft.

LPIPE = length of header pipe required, ft.

3.11.6.8.3 Cost of lateral piping to infiltration basins.

3.11.6.8.3.1 Calculate installed cost of lateral piping (excluding trenching and backfilling).

$$COSTLP = \frac{(COSP)\ (COSTRL)}{100}$$

where

COSTLP = cost of lateral pipe of diameter DIA, \$/ft.

COSP = cost of standard size pipe (12" diameter), \$/ft.

COSTRL = cost of lateral pipe of diameter DIA as percent of cost of standard size pipe, percent.

3.11.6.8.3.2 Calculate COSTRL

$$COSTRL = 5.48\ (DIA)^{1.1655}$$

where

DIA = diameter of lateral pipe, inches.

COSTRL = cost of lateral pipe of diameter DIA as percent of cost of standard size pipe, percent.

3.11.6.8.3.3 Calculate cost of trenching and backfilling. This cost is computed as a fraction of the cost of the pipe.

If $DIA \leq 12"$ $EBFL = 0.334\ (DIA)^{-0.6840}$

If $DIA > 12"$ $EBFL = 0.061$

where

EBFL = fraction of pipe cost for trenching and backfilling.

DIA = diameter of lateral pipe, inches.

3.115.6.8.3.4 Total installed cost of lateral pipe.

$$TICLP = (1 + EBFL)\ (COSTLP)\ (LLAT)$$

where

TICLP = total installed cost of lateral pipe, \$.

EBFL = fraction of pipe cost for trenching and backfilling.

COSTLP = cost of lateral pipe of diameter DIA, \$/ft.

LLAT = length of lateral pipe required, ft.

3.11.6.8.4 Cost of butterfly valves.

3.11.6.8.4.1 Calculate installed cost of butterfly valves.

$$COSTBV = \frac{(COSTSV)\ (COSTRV)\ (NBV)}{100}$$

where

COSTBV = installed cost of butterfly valves, \$.

COSTSV = cost of standard size valve (12" ∅), \$.

COSTRV = cost of valve of size DIA as a percent of the cost of the standard size valve, %.

NBV = number of butterfly valves.

3.11.6.8.4.2 Calculate COSTRV

$$COSTRV = 3.99\ (DIA)^{1.395}$$

3.11.6.8.4.3 Determine COSTSV

COSTSV is the installed cost of a 12"∅ butterfly valve suitable for water service.

3.11.6.8.5 Total cost of distribution system.

$$TCDS = COSTE + TICHP + TICLP + COSTBV$$

where

TCDS = total cost of distribution system, \$.

3.11.6.8.6 Cost of underdrain system.

3.11.6.8.6.1 Cost of underdrain laterals.

$$COSTUL = (DPIPE)\ (UPIPP)\ (1.1)$$

where

COSTUL = installed cost of underdrain laterals, \$.

DPIPE = length of 6" drain pipe required, ft.

UPIPP = unit price input for 6" PVC perforated drain pipe, \$/ft.

1.1 = 10% adjustment for trenching and backfilling.

3.11.6.8.6.2 Cost of underdrain collection header pipe.

$$ICUCH = \frac{(COSTRC)\ (COSTSC)\ (LDCH)\ (1 + EBFD)}{100}$$

where

ICUCH = installed cost of underdrain collection header.

COSTRC = cost of underdrain collection header pipe of diameter CDIA as percent of standard size pipe, %.

COSTCP = cost of standard size pipe (24"Ø reinforced concrete pipe Class III), $/ft.

LDCH = length of underdrain collection header required, ft.

EBFD = cost for trenching and backfilling as fraction of pipe cost.

3.11.6.8.6.2.1 Calculate COSTRC.

$$COSTRC = 0.489\ (CDIA)^{1.686}$$

where

CDIA = diameter of underdrain collection header pipe, inches.

3.11.6.8.6.2.2 Determine COSTCP.

COSTCP is the cost per foot of 24"Ø Class III reinforced concrete sewer pipe with gasket joints.

3.11.6.8.6.2.3 Calculate EBFD

$$EBFD = 0.392\ (CDIA)^{-0.2871}$$

where

EBFD = cost for trenching and backfilling as a fraction of pipe cost.

3.11.6.8.6.3 Calculate total cost of underdrain system.

$$TCUS = COSTUL + ICUCH$$

where

TCUS = total cost of underdrain system, $.

3.11.6.8.7 Calculate cost of recovery wells and pump.

3.11.6.8.7.1 Calculate cost of well.

3.11.6.8.7.1.1 Calculate COSTW.

$$COSTW = RWC\ (DW)\ (NW)\ (COSP)$$

where

COSTW = cost of recovery wells, \$.

RWC = recovery well cost as fraction of cost of standard pipe.

COSP = cost of standard size pipe (12"Ø welded steel), \$/ft.

3.11.6.8.7.1.2 Calculate RWC.

3.11.6.8.7.1.2.1 If $4" \leq WDIA \leq 10"$

$$RWC = 160.4\ (DW)^{-0.7033}$$

3.11.6.8.7.1.2.2 If $12" \leq WDIA \leq 20"$

$$RWC = 159.8\ (DW)^{-0.6209}$$

3.11.6.8.7.1.2.3 If $24" \leq WDIA \leq 34"$

$$RWC = 142.7\ (DW)^{-0.5286}$$

3.11.6.8.7.1.2.4 If $36" \leq WDIA \leq 42"$

$$RWC = 206.5\ (DW)^{-0.445}$$

where

WDIA = diameter of the well, inches.

RWC = recovery well cost as fraction of cost of standard size pipe.

DW = depth of recovery wells, ft.

3.11.6.8.7.1.3 Determine COSP.

COSP is the cost per foot of 12"Ø welded steel pipe. This cost in the 1st quarter of 1977 is \$13.50/ft. For the best cost estimation COSP should be a current price input from a vendor, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSP = \$13.50 \frac{MSECI}{491.6}$$

where

COSP = cost of standard size pipe (12"Ø welded steel), \$/ft.

MSECI = current value for Marshall and Swift Equipment Cost Index.

3.11.6.8.7.2 Calculate cost of pump for recovery wells.

3.11.6.8.7.2.1 Calculate COSTWP.

$$COSTWP = (COSTPS)\ (WPR)\ (NW)$$

where

COSTWP = cost of pumps for recovery wells, \$.

COSTPS = cost of standard size pump (3000 gpm), \$.

WPR = cost of well pump as fraction of cost of standard pump.

NW = number of recovery wells.

3.11.6.8.7.2.2 Calculate WPR.

$$WPR = 0.00048\ (WDIA)^{1.791}\ (DW)^{0.658}$$

where

WPR = cost of well pump as fraction of cost of standard pump.

WDIA = diameter of recovery wells, inches.

DW = depth of recovery wells, ft.

3.11.6.8.7.2.3 Detemine COSTPS.

COSTPS is the cost of a 3000 gpm pump. This cost is \$17,250 for the first quarter of 1977. For the best cost estimate the user should input a current value for COSTPS, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250 \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump (3000 gpm), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

3.11.6.8.7.3 Calculate total cost of recovery wells.

$$COSTRW = COSTW + COSTWP$$

where

COSTRW = total cost of recovery wells, \$.

COSTW = cost of recovery well, \$.

COSTWP = cost of pumps for recovery wells, \$.

3.11.6.8.8 Cost of monitoring wells and pumps.

3.11.6.8.8.1 Calculate cost of wells.

3.11.6.8.8.1.1 Calculate COSTM.

$$COSTM = (RMWC)(DMW)(NMW)(COSP)$$

where

COSTM = cost of monitoring wells, \$.

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

NMW = number of monitoring wells.

COSP = cost of standard size pipe (12" welded steel), \$/ft.

3.11.6.8.8.1.2 Calculate RMWC.

$$RMWC = 160.4\ (DMW)^{-0.7033}$$

where

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

3.11.6.8.8.1.3 Determine COSP.

COSP is the cost per foot of 12"Ø welded steel pipe. This cost in first quarter of 1977 is $13.50/ft. For the best cost estimate COSP should be a current price input from a vendor, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSP = \$13.50 \frac{MSECI}{491.6}$$

where

COSP = cost of standard size pipe (12"Ø welded steel), $/ft.

MSECI = current Marshall and Swift Equipment Cost Index.

3.11.6.8.8.2 Calculate cost of pumps for monitoring wells.

3.11.6.8.8.2.1 Calculate COSTMP.

$$COSTMP = (COSTPS)(MWPR)(NMW)$$

where

COSTMP = cost of pumps for monitoring wells, $.

COSTPS = cost of standard size pump (3000 gpm), $.

MWPR = cost of well pump as fraction of cost of standard pump.

NMW = number of monitoring wells.

3.11.6.8.8.2.2 Calculate MWPR.

$$MWPR = 0.0551\ (DMW)^{0.658}$$

where

MWPR = cost of well pump as fraction of cost of standard size pump.

DMW = depth of monitoring wells, ft.

3.11.6.8.8.2.3 Determine COSTPS.

COSTPS is the cost of a 3000 gpm pump. The cost is $17,250 for the first quarter of 1977. For the best cost estimate the user should input a current value of COSTPS, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250 \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump (3000 gpm), $.

MSECI = current Marshall and Swift Equipment Cost Index.

3.11.6.8.8.3 Calculate total cost of monitoring wells.

$$COSTMW = COSTM + COSTMP$$

where

COSTMW = total cost of monitoring wells, $.

COSTM = cost of monitoring wells, $.

COSTMP = cost of pumps for monitoring wells, $.

3.115.6.8.9 Operation and maintenance material cost.

$$OMMC = \frac{(TCDS)(OMMPD) + (TCUS)(OMMPW) + (COSTRW)(OMMPW) + (COSTMW)(OMMPM)}{100}$$

where

OMMC = O&M material costs, $/yr.

3.115.6.8.10 Total bare construction cost.

$$TBCCRI = (TCDS + TCUS + COSTRW + COSTMW)(1.18)$$

where

TBCCRI = total bare construction cost for rapid infiltration, $.

TCDS = total cost of distribution system, $.

TCUS = total cost of underdrain system, $.

COSTRW = installed cost of recovery wells, $.

COSTMW = installed cost of monitoring wells, $.

3.11.6.9 Cost Calculations Output Data.

3.11.6.9.1 Total bare construction cost for rapid infiltration, TBCCRI, $.

3.11.6.9.2 Operation and maintenance material cost, OMMC, $/yr.

3.11.7 Slow Infiltration.

3.11.7.1 Input Data.

3.11.7.1.1 Wastewater flow, Q, mgd.

3.11.7.1.1.1 Minimum flow, mgd.

3.11.7.1.1.2 Average flow, mgd.

3.11.7.1.1.3 Maximum flow, mgd.

3.11.7.1.2 Wastewater characteristics.

3.11.7.1.2.1 Suspended solids, mg/l.

3.11.7.1.2.2 Volatile suspended solids, % of suspended solids.

3.11.7.1.2.3 Settleable solids, mg/l.

3.11.7.1.2.4 BOD_5 (soluble and total), mg/l.

3.11.7.1.2.5 COD (soluble and total), mg/l.

3.11.7.1.2.6 Phosphorus (as PO_4), mg/l.

3.11.7.1.2.7 Total Kjeldahl Nitrogen (TKN), mg/l.

3.11.7.1.2.8 Ammonia-Nitrogen, NH_3, mg/l.

3.11.7.1.2.9 Nitrite-Nitrogen, NO_2, mg/l.

3.11.7.1.2.10 Nitrate-Nitrogen, N_2O_3, mg/l.

3.11.7.1.2.11 Temperature, °C.

3.11.7.1.2.12 pH, units.

3.11.7.1.2.13 Oil and Grease, mg/l.

3.11.7.1.2.14 Cations, mg/l.

3.11.7.1.2.15 Anions, mg/l.

3.11.7.2 Design Parameters.

3.11.7.2.1 Crops classification (specify).

3.11.7.2.1.1 Forage grass.

3.11.7.2.1.2 Corn.

3.11.7.2.2 Application rate, L_w.

3.11.7.2.2.1 Average application rate, in/wk = (0.5 - 4 in/wk).

3.11.7.2.2.2 Maximum application rate, in/hr = (0.1-0.5 in/hr).

3.11.7.2.3 Precipitation rate, P_r, in/wk.

3.11.7.2.4 Evapotranspiration rate, ET, in/wk.

3.11.7.2.5 Runoff, R, in/wk.

3.11.7.2.6 Wastewater generation period, WWGP, days/yr.

3.11.7.2.7 Field application period, FAP, wks/yr.

3.11.7.2.8 Piping classification (specify one).

3.11.7.2.8.1 Solid set piping.

3.11.7.2.8.2 Center pivot piping.

3.11.7.2.9 Storage requirements, days/yr (specify one).

3.11.7.2.9.1 Minimum storage, days/yr.

3.11.7.2.9.2 No storage.

3.11.7.2.10 Liner required (liner should only be used with storage).

3.11.7.2.11 Embankment protection (should only be used with storage).

3.11.7.2.12 Recovery system (specify one).

3.11.7.2.12.1 Underdrains recovery system.

3.11.7.2.12.2 No recovery system.

3.11.7.2.13 Buffer zone width, ft (site dependent) = 0.0-500 ft.

3.11.7.2.14 Current ground cover, %.

3.11.7.2.14.1 Forest, % (require heavy clearing).

3.11.7.2.14.2 Brush, % (require medium clearing).

3.11.7.2.14.3 Pasture, % (require light clearing).

3.11.7.2.15 Slope, %.

3.11.7.2.15.1 Cultivated land 20%.

3.11.7.2.15.2 None cultivated land 40%.

3.11.7.2.16 Monitoring wells.

3.11.7.2.16.1 Number.

3.11.7.2.16.2 Depth per well, ft.

3.11.7.2.17 Fraction denitrified, D, % = 15 - 25%.

3.11.7.2.18 Ammonia volatilization, AV, % = 0.0%.

3.11.7.2.19 Soil removal of phosphorus, % = 80%.

3.11.7.2.20 Hours per day operation, HPD, hrs.

3.11.7.2.21 Days per week operation, DPW, days.

3.11.7.3 Process Design Calculations.

3.11.7.3.1 Calculate total nitrogen concentration, C_n, in the applied wastewater.

$$C_n = (TKN)_i + (NO_2)_i + (NO_3)_i$$

where

C_n = total nitrogen concentration in applied wastewater, mg/l.

$(TKN)_i$ = total Kjeldahl nitrogen concentration in applied wastewater, mg/l.

$(NO_2)_i$ = nitrite-N concentration in applied wastewater, mg/l.

$(NO_3)_i$ = nitrate-N concentration in applied wastewater, mg/l.

3.11.7.3.2 Calculate wastewater nitrogen loading, L_n, lbs/acre-d.

$$= 11.77\ C_n L_w$$

where

L_n = wastewater nitrogen loading, lbs/acre-yr.

L_w = wastewater hydraulic loading rate, in/wk.

3.11.7.3.3 From water balance, calculate percolating water rate, W_p, in/wk.

$$W_p = L_w + (P_r - ET) - R$$

where

W_p = percolating water rate, in/wk.

P_r = design precipitation, in/wk.

ET = evapotranspiration (or crops consumptive use of water), in/wk.

R = net runoff, in/wk.

3.11.7.3.4 Calculate total nitrogen loading, $(L_t)_N$, lb/acre-yr.

$$(L_t)_N = L_n + 11.77\ (P_r)(0.5)$$

where

$(L_t)_N$ = total nitrogen loading rate, lb/acre-yr.
water, mg/l.

3.11.6.3.5 Assume crop nitrogen uptake, $(U)_N$, lb/acre-yr = 0.9.

3.11.7.3.5.1 For forage grass.

$$(U)_N = 0.891\ [(118.73) + 0.36\ (L_t)_N]\ \text{lb/acre-yr}$$

3.11.7.3.5.2 For corn.

$$(U)_N = 0.891[(176.53) - 0.0476(L_t)_N]\ \text{lb/acre-yr}$$

where

$(U)_N$ = crop nitrogen uptake, lb/acre-yr.

3.11.7.3.6 Calculate nitrogen loss through denitrification, D, lb/acre-yr.

$$D = (D_f)(L_t)_N/(100)$$

where

D = nitrogen loss through denitrification, lb/acre-yr.

D_f = nitrogen loss as a percent of total applied nitrogen, %.

3.11.7.3.7 Calculate nitrogen loss due to volatilization, AV, lb/acre-yr.

$$AV = (AV)_f\ (L_t)_N/(100)$$

where

AV = nitrogen loss due to volatilization, lb/acre-yr.

$(AV)_f$ = percent of total nitrogen applied lost to volatilization, %.

3.11.7.3.8 Calculate sum of nitrogen losses, $(\ N)_L$, lb/acre-yr.

$$(\ N)_L = (U)_N + D + AV$$

where

$(\ N)_L$ = sum of total nitrogen lost, lb/acre-yr.

3.11.7.3.9 Check total nitrogen against $(0.8\ (L_t)_N$.

$$(\ N)_L \quad 0.8\ (L_t)_N$$

$$\text{if } (\ N)_L \quad 0.8\ (L_t)_N$$

$$\text{set } (\ N)_L = 0.8\ (L_t)_N$$

3.11.7.3.10 From nitrogen balance, calculate nitrogen concentration in percolate, $(C_p)_N$, mg/l.

$$(C_p)_N = [(L_t)_N - (\ N)_L]/(11.77)(W_p)$$

where

$(C_p)_N$ = nitrogen concentration in percolate, mg/l.

3.11.7.3.11 Calculate required treatment acre, TA, acres.

$$TA = (258.5)(Q)/(L_w)$$

where

TA = required field area, acres.

Q = average wastewater flow, mgd.

3.11.7.3.12 Calculate volume of required storage, acre-ft.

$$(SV) = (SR)(Q_{ave})(10^6)/(7.48) \quad (43{,}560)$$

where

SV = volume of required storage, acre-ft.

3.11.7.3.13 Calculate phosphorus loading, L_p, lb/acre-yr.

$$L_p = 11.77\ (TP)_i(L_w)$$

where

L_p = total phosphorus loading, lbs/acre-yr.

$(TP)_i$ = total phosphorus concentration in applied wastewater, mg/l.

3.11.7.3.14 Calculate soil removal of phosphorus, lb/acre-yr.

$$(SRP) = (SRP)_f \, (L_p)/(100)$$

where

SRP = soil removal of phosphorus, lb/acre-yr.

$(SRP)_f$ = percent of total applied phosphorus removed by the soil, %.

3.11.7.3.15 Calculate plant uptake of phosphorus, U_p, lb/acre-yr.

$$U_p = 0.891 \, [215.54 - 37.11 \log_e (L_p)]$$

where

U_p = plant uptake of phosphorus, lbs/acre-yr.

3.11.6.3.16 From phosphorus mass balance, calculate phosphorus concentration of percolate water.

$$(C_p)_p = [(L_p) - (SRP) - (U_p)]/(11.77)(W_p)$$

where

$(C_p)_p$ = phosphorus content of percolate water, mg/l. $(0.01)\ (TP)_i$

3.11.7.3.17 Calculate percolate rate, W_p, mgd.

$$W_p(\text{mgd}) = [(W_p)\text{in/wk}(\frac{1\ \text{ft}}{12\ \text{in}})(\frac{1\ \text{wk}}{7\ \text{day}})]\ (TA)\ \text{acre}\ (43{,}560\ \frac{\text{ft}^2}{\text{acre}})\ (7.48\ \frac{\text{gal}}{\text{ft}^3})(1/10^6)$$

3.11.7.3.18 Calculate suspended solids concentration in percolate, mg/l, assume 97% removal.

$$(SS)_p = (0.03)(SS)_i$$

where

$(SS)_p$ = suspended solids concentration in percolate, mg/l.

$(SS)_i$ = suspended solids concentration in applied wastewater, mg/l.

3.11.7.3.19 Calculate total and soluble BOD_5 concentration in percolate, mg/l, assume 95% removal of total BOD_5.

$$(TBOD_5)_p = (TBOD_5)_i\ (0.05)$$

$$(SBOD_5)_p = (SBOD_5)_i\ (0.05)$$

where

$(TBOD_5)_p$ = total BOD_5 concentration in percolate, mg/l.

$(TBOD_5)_i$ = total BOD_5 concentration in applied wastewater, mg/l.

$SBOD_5$ = soluble BOD_5, mg/l.

3.11.7.3.20 Calculate total and soluble COD concentration in percolate, mg/l, assume COD removal of 95%.

$$(TCOD)_p = (TCOD)_i\ (0.05)$$

$$(SCOD)_p = (SCOD)_i\ (0.05)$$

where

$(TCOD)_p$ and $(TCOD)_i$ = total COD concentration in percolate and applied wastewater, respectively, mg/l.

$(SCOD)_p$ and $(SCOD)_i$ = soluble COD concentration in percolate and applied wastewater, respectively, mg/l.

3.11.7.3.21 Nitrite-N concentration in percolate = 0.0.

3.11.7.3.22 Nitrate-N concentration in percolate = $(C_p)_N$.

3.11.7.3.23 Ammonia-N concentration in percolate = 0.0. Total Kjeldahl-nitrogen concentration in percolate = 0.0.

3.11.7.3.24 Oil and grease concentration in percolate = 0.0.

3.11.7.4 Process Design Output Data.

3.11.7.4.1 Hours per day operation, hours.

3.11.7.4.2 Days per week operation, days.

3.11.7.4.3 Forage grasses.

3.11.7.4.4 Application rate, in/week.

3.11.7.4.5 Maximum application rate, in/hour.

3.11.7.4.6 Evapotransportation rate, in/week.

3.11.7.4.7 Precipitation rate, in/week.

3.11.7.4.8 Runoff, in/week.

3.11.7.4.9 Percent denitrified, percent.

3.11.7.4.10 Percent ammonia volatilization, percent.

3.11.7.4.11 Removal of phosphorus, percnet.

3.11.7.4.12 Wastewater generation period, days/yr.

3.11.7.4.13 Field application period, weeks/yr.

3.11.7.4.14 Solid set piping and pumping.

3.11.7.4.15 Calculated storage required, acre-ft.

3.11.7.4.16 Buffer zone width, feet.

3.11.7.4.17 Current ground cover.

3.11.7.4.17.1 Forest, percent.

3.11.7.4.17.2 Brush, percent.

3.11.7.4.17.3 Pasture, percent.

3.11.7.4.18 Slope of site, percent.

3.11.7.4.19 Number of monitoring wells, wells.

3.11.7.4.20 Depth of monitoring wells, feet.

3.11.7.4.21 Treatment area required, acres.

3.11.7.4.22 Volume of percolate, mgd.

3.11.7.4.23 Quality of percolate.

3.11.7.4.23.1 Suspended solids, mg/l.

3.11.7.4.23.2 Volatile solids, percent.

3.11.7.4.23.3 BOD_5, mg/l.

3.11.7.4.23.4 BOD_5 soluble, mg/l.

3.11.7.4.23.5 COD, mg/l.

3.11.7.4.23.6 COD soluble, mg/l.

3.11.7.4.23.7 PO_4, mg/l.

3.11.7.4.23.8 TKN, mg/l.

3.11.7.4.23.9 NO_2, mg/l.

3.11.7.4.23.10 NO_3, mg/l.

3.11.7.4.23.11 Oil and grease, mg/l.

3.11.7.5 <u>Quantities Calculations</u>.

3.11.7.5.1 Distribution pumping. User must input the operating schedule, days per week (DPW) and hours per day (HPD) of operation.

3.11.7.5.1.1 Calculate the design flow.

$$FLOW = \frac{(Q_{avg})\ (WWGP)\ (24)}{(FAP)\ DPW)\ (HPD)}$$

where

FLOW = actual daily flow to spray field, mgd.

Q = average daily flow, mgd.

WWGP = wastewater generation period, days/yr.

FAP = field application period, wks/yr.

DPW = days per week treatment system is operated, days/wk.

HPD = hours per day treatment system is operated, hrs/day.

24 = conversion from days to hours, hrs/day.

Using the flow calculated (FLOW), the distribution pumping will be sized and the cost estimated from the existing section entitled "Intemediate Pumping".

3.11.7.5.2 Storage requirements. The slow rate system, like overland flow, is dependent upon weather. Also if crops are grown it is dependent upon growing seasons. The user must input the number of days of storage required based on anticipated crops, and climatic data for the particular area.

3.11.7.5.2.1 Calculate storage volume.

$$SV = (SR)\ (Q \times 10^6)$$

where

SV = storage volume, gal.

SR = storage required, days/yr.

Q = average daily flow, mgd.

3.11.7.5.2.2 Calculate size and number of storage lagoons.

3.11.7.5.2.2.1 The following assumptions are made in determining size and number of lagoons:

A minimum of 2 lagoon cells will always be used.
An even number of lagoon cells will be used, such as 2, 4, 6, 8, etc.
The largest single lagoon cell will be 40 acres which represents approximately 85 million gallons storage volume.

3.11.7.5.2.2.2 If $SV \leq 170{,}000{,}000$ gal.

$$NLC = 2$$

3.11.7.5.2.2.3 If $SV > 170{,}000{,}000$ gal. a trial and error solution for NLC will be used.

Assume $NLC = 4$; if $\frac{SV}{NLC} > 85{,}000{,}000$ gal.

Redesignate $NLC = NLC + 2$ and repeat calculation until $\frac{SV}{NLC} \leq 85{,}000{,}000$ gal.

where

SV = storage volume, gal.

NLC = number of lagoon cells.

3.11.7.5.2.3 Calculate storage volume per cell.

$$SVC = \frac{SV}{(NLC)\ (7.48)}$$

where

SVC = storage volume per cell, ft^3.

SV = storage volume, gal.

NLC = number of lagoon cells.

7.48 = conversion from gal. to ft^3, gal/ft^3.

3.11.7.5.2.4 Calculate lagoon cell dimensions. The following assumptions are made concerning lagoon construction:

The lagoon cells will be square.
Common levee construction will be used where possible.
Lagoons will be constructed using equal cut and fill.
Lagoon depth will be 10 ft with 8 ft water depth and 2 ft freeboard.
Minimum water depth will be 1.5 ft.
Side slopes will be 3 to 1.
A 30% shrinkage factor will be used for fill.

$$L = \frac{(0.615\ SVC - 1521)^{0.5} + 60}{2}$$

where

L = length of one side of lagoon cell, ft.

SVC = storage volume per cell, ft^3.

3.11.7.5.2.5 Calculate volume of earthwork required for lagoons. The volume of earthwork must be determined by trial and error using the following equations:

$$DC + DF = 10$$

$$VF = [\ 3\ (DF)^2 + 10DF\ ]\ [\ \frac{5NLC}{2} + 2]\ \ (L)$$

$$VC = (1.3)(NLC)(DC)\ [\ L^2 - (6)(DF)(L) + 12\ (DF)^2 + 120\ DF - 60L + 1200]$$

Assume that the depth of cut (DC) is equal to 1 ft. From the equations calculate the volume of fill (VF) required and the volume of cut (VC) required. Compare VC and VF.

If $VC < VF$ then assume $DC > 1$ ft and recalculate VC and VF.

If $VC > VF$ then assume $DC < 1$ ft and recalculate VC and VF.

Repeat this procedure until VC = VF. This is the volume of earthwork required for the storage lagoon.

$$VC = VF = VLEW$$

where

DC = depth of cut, ft.

DF = depth of fill, ft.

VF = volume of fill, ft^3.

VC = volume of cut, ft^3.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

VLEW = volume of earthwork required for lagoon construction, ft^3.

3.11.7.5.3 Slow rate distribution system. In a slow rate land treatment system the wastewater is usually applied to the field in one of two ways, buried solid set sprinklers or center pivot sprinkler systems. Both of these distribution methods will be addressed.

3.11.7.5.3.1 Buried solid set sprinklers. The selection of the optimum sprinkler type, size, spacing is very dependent on the site conditions. Certain assumptions will be made on these parameters to simplify the calculations. While these assumptions, if used to design some systems, would drastically affect performance, they will have little affect on the overall costs.

Assume:

The treatment area will be square.
The spacing between laterals will be 50 ft.
The spacing between sprinklers will be 50 ft.
Sprinklers will be arranged in square patterns.

3.11.7.5.3.1.2 Calculate dimensions of treatment area.

$$LTA = [(TA)\ (43,560)]^{0.5}$$

where

LTA = length of one side of treatment area, ft.

TA = treatment area, acres.

43,560 = conversion from acres to ft^2, ft^2/acre.

3.11.7.5.3.1.3 Calculate flow per sprinkler.

$$FPS = \frac{2,500\ (MAR)}{96.3}$$

where

FPS = flow per sprinkler, gpm.

MAR = Maximum application rate, inches/hr. (Must be input by user based on crop and infiltration rate)

2500 = application area for each sprinkler, ft^2.

96.3 = combined conversion factors.

3.11.7.5.3.1.4 Calculate number of headers. This will be a trial and error process. The governing assumption will be that the header pipes will be less than 48"Ø. Assume NH = 1.

3.11.7.5.3.1.4.1 Calculate the length of laterals.

$$LL = \frac{LTA}{2NH}$$

where

LL = length of laterals, ft.

LTA = length of one side of treatment area, ft.

NH = number of headers.

3.11.7.5.3.1.4.2 Calculate number of sprinklers per lateral.

$$NSL = \frac{LL}{50}$$

where

NL = number of laterals per header.

LTA = length of one side of treatment area.

3.11.7.5.3.1.4.3 Calculate number of laterals per header.

$$NL = \frac{LTA}{50}$$

where

NL = number of laterals per header.

LTA = length of one side of treatment area, ft.

50 = spacing of laterals on header, ft.

3.11.7.5.3.1.4.4 Calculate number of sprinklers per header.

$$NSH = (NL)\ (NSL)$$

where

NSH = number of sprinklers per header.

NL = number of laterals per header.

NSL = number of sprinklers per lateral.

3.11.7.5.3.1.4.5 Calculate flow per header.

$$FPH = (FPS)\ (NSH)$$

If FPH   16,000 gpm; assume NH = NH + 1 and recalculate FPH until FPH   16,000 gpm.

where

FPH = flow per header, gpm.

FPS = flow per sprinkler, gpm.

NSH = number of sprinkler per header.

3.11.3.1.5 Calculate pipe size for laterals.

$$DIAL = 0.286\ [(NSL)\ (FPS)]^{0.5}$$

DIAL must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48. Always use the next larger diameter above the calculated diameter.

where

DIAL = diameter of lateral, inches.

NSL = number of sprinklers per lateral.

FPS = flow per sprinkler, gpm.

0.286 = combined conversion factors.

3.11.7.5.3.1.6 Calculate quantity of lateral pipe required.

$$TLL = (LL)\ (NLH)\ (NH)$$

where

TLL = total length of lateral pipe, ft.

LL = length of laterals, ft.

NLH = number of laterals per header.

NH = number of headers.

3.11.7.5.3.1.7 Calculate pipe sizes for headers. The header pipe normally decreases in size due to decreasing volume of flow as each lateral pipe removes part of the flow from the header pipe. The header size will be calculated after each lateral on the header.

$$DIAHN = 0.286\ [FPH - (N)\ (FPL)]^{0.5}$$

Begin calculation with N = 0. This will give the diameter of the header (DIAHO) before any flow is removed. Then set N = N + 1 and repeat the calculation. This will give the diameter (DIAH 1) of the header after the first lateral has removed a part of the flow. Repeat the calculation each time redesignating N until N = NLH.

DIAHN must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48.

where

DIAHN = diameter of header pipe, inches.

FPH = flow per header, gpm.

N = number of laterals.

FPL = flow per lateral, gpm.

0.286 = combined conversion factors.

3.11.7.5.3.1.8 Calculate length of header pipe.

$$LDIAHN = (50)\ (SUM)\ (NH)$$

where

LDIAHN = length of header pipe of diameter DIAHN, ft.

SUM = the number of points with the same diameter.

50 = spacing between laterals, ft.

3.11.7.5.3.1.9 Calculate number of butterfly valves for distribution system. There will be a butterfly valve in each header for flow control. These valves will be in the header upstream from the spray field and will be the same as the initial size calculated for the header.

$$NBV = NH$$

$$DBV = DIAHN$$

where

NBV = number of butterfly valves.

NH = number of headers.

DBV = diameter of butterfly valves, inches.

DIAHN = diameter of header calculated when N = 0, inches.

3.11.7.5.3.1.10 Calculate number of valves for lateral lines. There will be a plug valve in each lateral line which will be automatic but will be either fully open or fully closed. They will be the same size as the size calculated for the lateral pipes.

$$NLV = (NLH)\ (NH)$$

$$DLV = DIAL$$

where

NLV = number of lateral valves.

NLH = number of laterals per header.

NH = number of headers.

DLV = diameter of lateral valves, inches.

DIAL = diameter of lateral pipes, inches.

3.11.7.5.3.1.11 Calculate number of sprinklers.

$$NS = (NSL)\ (NLH)\ (NH)$$

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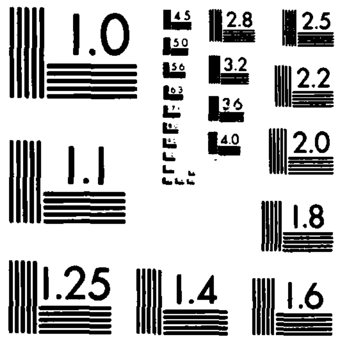

MICROCOPY RESOLUTION TEST CHART
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where

NS = number of sprinklers

NSL = number of sprinklers per lateral.

NLH = number of laterals per header.

NH = number of headers.

3.11.7.5.3.2 Center pivot system.

3.11.7.5.3.2.1 Determine size and number of center pivot systems. Center pivot systems are available in sizes which cover from 2 to 450 acres. Because of weight and structural consideration, the largest pipe available in the system is 8 inches. For this reason, hydraulics sometimes control the sizing rather than area of coverage.

3.11.7.5.3.2.1.1 The following assumptions will be made:

The systems will operate 24 hours per day, 7 days per week.
A minimum of 2 units will be used.
Ten percent of the treatment area will not be irrigated because of the circular configuration.

$$SCP = \frac{(TA)\ (1.1)}{N}$$

Begin with $N = 2$ and if $SCP > 450$ redesignate $N = N + 1$ and recalculate.

3.11.7.5.3.2.1.2 Because of hydraulic considerations check system velocity.

$$V = (.017)\ (AR)\ (SCP)$$

If $V > 10$ fps redesignate $N = N + 1$ and recalculate SCP and V. When $V \leq 10$ fps use the calculated SCP and N.

where

SCP = size of center pivot system, acres.

TA = treatment area, acres.

N = number of center pivot systems.

V = velocity in system, ft/sec.

AR = application rate, in/hr.

2.89 = combined constants and conversion factors.

3.11.7.5.3.2.2 Determine size of header pipe. Assume each center pivot system takes flow from header consecutively.

$$CDIAHN = 0.832\ [(N)\ (AR)\ (SCP)]^{0.5}$$

This gives the header size to the first unit. DIAHN must be one of the following: 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 24, 30, 36, 42, 48 inches. Always use next higher pipe size.

Set N = N - 1 and repeat the calculation. This gives the diameter of the header pipe between the first and second unit.

Redesignate N after each calculation until N = 0.

where

CDIAHN = diameter of segments of header, inches.

N = number of center pivot systems.

AR = application rate, in/hr.

SCP = size of center pivot system, acres.

3.11.7.5.3.2.3 Determine length of segments of header pipe.

$$LCDIAN = 235.5\ (SCP)^{0.5}$$

Each segment of header pipe is approximately the same length and is essentially equal to the diameter of the center pivot system.

where

LCDIAN = length of segment of header pipe of diameter CDIAHN, ft.

SCP = size of center pivot system, acres.

235.5 = combined constants and conversion factors.

3.11.7.5.4 Underdrain system for groundwater control. Practical drainage systems for wastewater applications will be at depths of 4 to 8 ft and spaced 200 ft apart. The following assumptions will be made concerning the drainage system.

6" diameter perforated PVC pipe will be used.
Spacing will be 200 ft.
Depth of burial will be 4 to 8 ft.

$$LDP = [\frac{LTA}{200} + 2] \quad LTA$$

where

LDP = length of drain pipe, ft.

LTA = length of one side of treatment area, ft.

3.11.7.5.5 Land preparation. Unlike overland flow there is very little land forming required for slow rate. The land will, however, require clearing and grubbing.

For clearing and grubbing the areas will be classified in three categories: heavy, medium and light. Heavy refers to wooded areas with mature trees. Medium refers to spotted mature trees with numerous small trees and bushes. Light refers to only small trees and bushes. The user must specify the type of clearing and grubbing required as well as the percent of the treatment area requiring clearing and grubbing.

$$CAGH = \frac{PCAGH}{100} (TA)$$

$$CAGM = \frac{PCAGM}{100} (TA)$$

$$CAGL = \frac{PCAGL}{100} (TA)$$

where

CAGH = area which requires heavy clearing, acres.

PCAGH = percentage of treatment area requiring heavy clearing, %.

CAGM = area which requires medium clearing, acres.

PCAGM = percentage of treatment area requiring medium clearing, %.

CAGL = area which requires light clearing, acres.

PCAGL = percentage of treatment area requiring light clearing, %.

TA = treatment area, acres.

3.11.7.5.6 Determine total land requirement. The land requirement will be different depending on which application method is used center pivot, or fixed sprinklers.

3.11.7.5.6.1 Total treatment area.

3.11.7.5.6.1.1 Center pivot. The actual treatment area for the center pivot system will be increased by 10% because there will be unwetted areas due to the circular configuration.

$$TTA = (TA)\ (1.1)$$

where

TTA = total treatment area, acres.

TA = treatment area, acres.

1.1 = factor for unwetted area.

3.11.7.5.6.1.2 Buried solid set sprinklers. The treatment area will be increased by approximately 5% for service roads.

$$TTA = (1.05)\ (TA)$$

where

TTA = total treatment area, acres.

TA = treatment area, acres.

1.05 = factor for service roads.

3.11.7.5.6.2 Area for storage lagoons.

$$ASL = \frac{(1.2)\ (NLC)\ (L)^2}{43,560}$$

where

ASL = area for storage lagoons, acres.

NLC = number of lagoon cells.

L = length of one side of lagoon cell, ft.

1.2 = additional area required for cross levee.

3.11.7.5.6.3 Area for buffer zone.

Assume:

Buffer zone will be around entire treatment area.
Facility will be square.

$$ABZ = \frac{4\ WBZ\ [\ (43{,}560\ TTA)^{0.5} + WBZ\ ]}{43{,}560}$$

where

ABZ = area required for buffer zone, acres.

WBZ = width of buffer zone (must be input by user), ft.

TTA = total treatment area, acres.

3.11.7.5.6.4 Total land area

$$TLA = TTA + ASL + ABZ$$

where

TLA = total land area required, acres.

TTA = total treatment area, acres.

ASL = area required for storage lagoons, acres.

ABZ = area required for buffer zone, acres.

3.11.7.5.7 Calculate fencing required.

$$LF = 834.8\ (TLA)^{0.5}$$

where

LF = length of fence required, ft.

TLA = total land area required, acres.

834.8 = combined conversion factors and constants.

3.11.7.5.8 Calculate operation and maintenance manpower.

3.11.7.5.8.1 Distribution system.

3.11.7.5.8.1.1 Solid set sprinklers.

3.11.7.5.8.1.1.1 If $TA \leq 60$;

$$OMMHD = 158.32\ (TA)^{0.4217}$$

3.11.7.5.8.1.1.2 If $TA > 60$;

$$OMMHD = 26.73\ (TA)^{0.8561}$$

3.11.7.5.8.1.2 Center pivot.

3.11.7.5.8.1.2.1 If $TA \leq 100$;

$$OMMHD = 209.86\ (TA)^{0.4467}$$

3.11.7.5.8.1.2.2 If $TA > 100$;

$$OMMHD = 32.77\ (TA)^{0.8481}$$

where

TA = treatment area, acres.

OMMHD = O&M manpower for distribution system, MH/yr.

3.11.7.5.8.2 Underdrain system.

3.11.7.5.8.2.1 If $TA \leq 80$;

$$OMMHU = 54.71\ (TA)^{0.2414}$$

3.11.7.5.8.2.2 If $TA > 80$;

$$OMMHU = 10.12\ (TA)^{0.6255}$$

where

TA = treatment area, acres.

OMMHU = O&M manpower for underdrain system, MH/yr.

3.11.7.5.8.3 Monitoring wells.

$$OMMHM = 6.39(NMW)(DMW)^{0.2760}$$

where

OMMHM = operation and maintenance manpower for monitoring wells, MH/yr.

NMW = number of monitoring wells.

DMW = depth of monitoring wells, ft.

3.11.7.5.9 Calculate O&M material costs.

3.11.7.5.9.1 Distribution system.

3.11.7.5.9.1.1 Solid set sprinklers.

$$OMMPD = 0.906\ (TA)^{-0.0860}$$

3.11.7.5.9.1.2 Center pivot.

3.11.7.5.9.1.2.1 If TA 175;

$$OMMPD = 1.06\ (TA)^{0.0696}$$

3.11.7.5.9.1.2.2 If TA 175;

$$OMMPD = 2.92\ (TA)^{-0.1261}$$

where

TA = treatment area, acres.

OMMPD = O&M material cost as percent of construction cost of distribution system, %.

3.11.7.5.9.2 Monitoring wells.

$$OMMPM = 2.28\ (DMW)^{0.0497}$$

where

OMMPM = O&M material cost as percent of construction cost of monitoring wells.

DMW = depth of monitoring wells, ft.

3.11.7.5.9.3 Underdrain system.

3.11.7.5.9.3.1 If TA 200;

$$OMMPU = 14.13\ (TA)^{-0.1392}$$

3.11.7.5.9.3.2 If TA 200;

$$OMMPU = 30.95\ (TA)^{-0.2860}$$

where

OMMPU = O&M material cost as percent of construction cost of underdrain system, %.

TA = treatment area, acres.

3.11.7.5.10 Other construction cost items. The quantities computed account for approximately 85% of the construction cost of the systems. Other miscellaneous costs such as connecting piping for lagoons, lagoon influent and effluent structures, miscellaneous concrete structures, etc., make up the additional 15%.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other construction costs.

3.11.7.6 Quantities Calculations Output Data.

3.11.7.6.1 Volume of earthwork required for lagoon construction, VLEW, ft^3.

3.11.7.6.2 Diameter of lateral pipes, DIAL, inches.

3.11.7.6.3 Total length of lateral pipe, TLL, ft.

3.11.7.6.4 Diameters of header pipes, DIAHN, inches.

3.11.7.6.5 Length of header pipes of diameters DIAHN, LDIAHN, ft.

3.11.7.6.6 Number of butterfly valves, NBV.

3.11.7.6.7 Diameter of butterfly valves, DBV, inches.

3.11.7.6.8 Number of lateral valves, NLV.

3.11.7.6.9 Diameter of lateral valves, DLV, inches.

3.11.7.6.10 Number of sprinklers, NS.

3.11.7.6.11 Number of center pivot systems, N.

3.11.7.6.12 Size of center pivot system, SCP, acres.

3.11.7.6.13 Diameter of segments of header pipe for center pivot, CDIAHN, inches.

3.11.7.6.14 Length of segment of header pipe of diameter CDIAHN for center pivot, LCDIAN, ft.

3.11.7.6.15 Length of drain pipe, LDP, ft.

3.11.7.6.16 Area which requires heavy clearing, CAGH, acres.

3.11.7.6.17 Area which requires medium clearing, CAGM, acres.

3.11.7.6.18 Area which requires light clearing, CAGL, acres.

3.11.7.6.19 Total land area required, TLA, acres.

3.11.7.6.20 Length of fencing required, LF, ft.

3.11.7.6.21 O&M manpower for distribution system, OMMHD, MH/yr.

3.11.7.6.22 O&M manpower for underdrain system, OMMHU, MH/yr.

3.11.7.6.23 O&M material costs as percent of construction cost of distribution system, OMMPD, %.

3.11.7.6.24 O&M material costs as percent of construction cost of underdrain system, OMMPU, %.

3.11.7.6.25 Correction factor for other construction costs, CF.

3.11.7.7 Unit Price Input Required.

3.11.7.7.1 Unit price input for earthwork, UPIEX, \$/cu yd.

3.11.7.7.2 Cost of standard size pipe (12"Ø), COSP, \$/ft.

3.11.7.7.3 Cost of standard size valve (12"Ø butterfly), COSTSV, \$.

3.11.7.7.4 Cost per sprinkler, COSTEN, \$.

3.11.7.7.5 Unit price input for 6" PVC perforated drain pipe, UPIPP, \$/ft.

3.11.7.7.6 Unit price input for heavy clearing and grubbing, UPICG, \$/acre.

3.11.7.7.7 Unit price input for fence, UPIF, \$/ft.

3.11.7.8 Cost Calculations.

3.11.7.8.1 Cost of earthwork.

$$COSTE = \frac{VLEW}{27}\ UPIEX$$

where

COSTE = cost of earthwork, \$.

VLEW = volume of earthwork required for lagoon construction, ft^3.

UPIEX = unit price input for earthwork, \$/cu yd.

3.11.7.8.2 Cost of distribution system for solid set sprinklers.

3.11.7.8.2.1 Cost of header pipes.

3.11.7.8.2.1.1 Calculate total installed cost of header pipe.

$$TICHP = \sum ICHPN$$

where

TICHP = total installed cost of header pipes, \$.

ICHPN = installed cost of various size header pipes, \$.

3.11.7.8.2.1.2 Calculate installed cost of each size header pipe.

$$ICHPN = (LDIAHN) \frac{COSTPN}{100} (COSP)$$

where

ICHPN = installed cost of various size header pipes, \$.

LDIAHN = length of header pipes of size DIAHN, ft.

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

3.11.7.8.2.1.3 Calculate COSTPN.

$$COSTPN = 6.842\ (DIAHN)^{1.2255}$$

where

COSTPN = cost of pipe of diameter DIAHN as percent of cost of standard size pipe, %.

DIAHN = diameters of header pipes, inches.

3.11.7.8.2.1.4 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot 4th quarter, 1977.

3.11.7.8.2.2 Cost of lateral pipes.

3.11.7.8.2.2.1 Calculate total installed cost of lateral pipe.

$$TICLP = (TLL) \frac{(COSTP)}{100} (COSP)$$

where

3.11.7.8.2.2.1 Calculate total installed cost of lateral pipe.

$$TICLP = (TLL) \frac{(COSTP)}{100} (COSP)$$

where

TICLP = total installed cost of lateral pipe, \$.

TLL = total length of lateral pipe.

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

3.11.7.8.2.2.2 Calculate COSTP.

$$COSTP = 6.842\ (DIAL)^{1.2255}$$

where

COSTP = cost of pipe of diameter DIAL as percent of cost of standard size pipe, %.

DIAL = diameter of lateral pipes, inches.

3.11.7.8.2.2.3 Detemine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot in 4th quarter, 1977.

3.11.7.8.2.3 Calculate cost of butterfly valves.

3.11.7.8.2.3.1 Calculate installed cost of butterfly valves.

$$COSTBV = \frac{(COSTRV)\ (COSTSV)\ (NBV)}{100}$$

where

COSTBV = installed cost of butterfly valves, \$.

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

COSTSV = cost of standard size valve, \$.

NBV = number of butterfly valves.

3.11.7.8.2.3.2 Calculate COSTRV.

$$COSTRV = 3.99(DBV)^{1.395}$$

where

COSTRV = cost of butterfly valve of size DBV as percent of standard size valve, %.

DBV = diameter of butterfly valves, inches.

3.11.7.8.2.3.3 Determine COSTSV. COSTSV is the cost of a 12" ϕ butterfly valve suitable for water service. This cost is $1,004 for 4th quarter, 1977.

3.11.7.8.2.4 Calculate cost of lateral valves.

3.11.7.8.2.4.1 Calculate installed cost of lateral valves.

$$COSTLV = \frac{(COSTRL)\ (COSTSV)\ (NLV)}{100}$$

where

COSTLV = installed cost of lateral valves, $.

COSTRL = cost of lateral valve of size DLV as percent of cost of standard size valve, %.

COSTSV = cost of standard size valve (12" ϕ butterfly), $.

NLV = number of lateral valves.

3.11.7.8.2.4.2 Calculate COSTRL.

$$COSTRL = 15.33\ (DLV)^{1.053}$$

where

COSTRL = cost of lateral valve of size DLV as percent of cost of standard size valve, %.

DLV = diameter of lateral valves, inches.

3.11.7.8.2.4.3 Determine COSTSV. COSTSV is the cost of a 12" ϕ butterfly valve suitable for water service. This cost is $1004 for the 4th quarter of 1977.

3.11.7.8.2.5 Calculate cost of sprinklers.

3.11.7.8.2.5.1 Calculate installed cost of sprinklers.

$$COSTS = 1.2\ (NS)\ COSTEN$$

COSTS = installed cost of sprinklers, $.

NS = number of sprinklers.

COSTEN = cost per sprinkler, \$.

1.2 = 20% for cost of installation.

3.11.7.8.2.5.2 Determine COSTEN. COSTEN is the cost of an impact type rotary pop-up, full circle sprinkler with a flow from 6 to 15 gpm. This cost is \$65.00 for the 4th quarter of 1977.

$$COSTEN = \$65.00$$

For better cost estimation, COSTEN should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COSTEN = \$65.00 \frac{MSECI}{518.4}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

518.4 = Marshall and Swift Equipment Cost Index 4th quarter of 1977.

3.11.7.8.2.6 Calculate total cost of distribution system for solid set sprinklers.

$$TCDSS = TICHP + TICLP + COSTBV + COSTLV + COSTS$$

where

TCDSS = total cost of distribution system, \$.

TICHP = total installed cost of header pipes, \$.

TICLP = total installed cost of lateral pipes, \$.

COSTBV = installed cost of butterfly valves, \$.

COSTLV = installed cost of lateral valves, \$.

COSTS = installed cost of sprinklers, \$.

3.11.7.8.3 Cost of distribution system for center pivot system.

3.11.7.8.3.1 Cost of center pivot systems.

3.11.7.8.3.1.1 Calculate cost of center pivot systems.

$$COSTCP = \frac{(N)\ (COSTRC)\ (COCP)}{100}$$

where

COSTCP = total cost of center pivot systems, \$.

N = number of center pivot systems required.

COSTRC = cost of center pivot system of size SCP as percent of standard size system, %.

COCP = cost of standard size system (200 acres), \$.

3.11.7.8.3.1.2 Calculate COSTRC.

$$COSTRC = 12.25\ (SCP)^{0.4559}$$

where

COSTRC = cost of center pivot system of size SCP as percent of standard size system, %.

SCP = size of center pivot system, acres.

3.11.7.8.3.1.3 Determine *COCP*. *COCP* is the cost of a center pivot sprinkler system capable of irrigating 200 acres. The cost is \$29,200 for the 4th quarter of 1977.

$$COCP = \$29,200$$

For better cost estimation, COCP should be obtained from an equipment vendor and treated as a unit price input. Otherwise, for future escalation, the equipment cost should be adjusted by using the Marshall and Swift Equipment Cost Index.

$$COCP = \$29,200\ \frac{MSECI}{518.4}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

518.4 = Marshall and Swift Cost Index 4th quarter of 1977.

3.11.7.8.3.2 Cost of header pipe for center pivot.

3.11.7.8.3.2.1 Total cost header pipe.

$$TCHPC = \sum CHPCN$$

where

TCHPC = total cost header pipe for center pivot, \$.

CHPCN = cost of various size header pipe for center pivot, \$.

3.11.7.8.3.2.2 Calculate cost of each size header pipe.

$$CHPCN = \frac{(LCDIAN)\ (COSTPN)\ (COSP)}{100}$$

where

CHPCN = installed cost of various size header pipe, \$.

LCDIAN = length of header pipes of diameter CDIAHN, ft.

COSTPN = cost of pipe of diameter CDIAHN as percent of cost of standard size pipe, %.

COSP = cost of standard size pipe (12" diameter), \$/ft.

3.11.7.8.3.2.3 Calculate COSTPN.

$$COSTPN = 6.842\ (CDIAHN)^{1.2255}$$

where

COSTPN = cost of pipe of diameter CDIAHN as percent of cost of standard size pipe, %.

CDIAHN = diameter of segments of header pipe, inches.

3.11.7.8.3.2.4 Determine COSP. COSP is the cost per foot of 12" diameter welded steel pipe. This cost is \$13.50 per foot in 4th quarter of 1977.

3.11.7.8.3.3 Calculate total cost of distribution system for center system.

$$TCDCP = COSTCP + TCHPC$$

where

TCDCP = total cost of distribution system for center pivot system, \$.

COSTCP = cost of center pivot systems, \$.

TCHPC = total cost of header pipe for center pivot, \$.

3.11.7.8.4 Cost of underdrain system.

$$COSTU = (LDP)(UPIPP)(1.1)$$

where

COSTU = cost of underdrain system, \$.

LDP = length of drain pipe, ft.

UPIPP = unit price input for 6" PVC perforated drain pipe, \$/ft.

1.1 = 10% adjustment for trenching and backfilling.

3.11.7.8.5 Calculate cost of clearing and grubbing.

$$COSTCG = (CAGH + 0.306\ CAGM + 0.092\ CAGL)\ UPICG$$

where

COSTCG = cost for clearing and grubbing site, \$.

CAGH = area which requires heavy clearing, acres.

CAGM = area which requires medium clearing, acres.

CAGH = area which requires light clearing, acres.

UPICG = unit price input for heavy clearing and grubbing \$/acres.

3.115.7.8.6 Calculate cost of fencing.

$$COSTF = (LF)\ (UPIF)$$

where

COSTF = installed cost of fencing, \$.

LF = length of fencing required,ft.

UPIF = unit price input for fencing, \$/ft.

3.11.7.8.7 Cost of monitoring wells and pumps.

3.11.7.8.7.1 Calculate cost of wells.

3.11.7.8.7.1.1 Calculate COSTM.

$$COSTM = (RMWC)(DMW)(NMW)(COSP)$$

where

COSTM = cost of monitoring wells, \$.

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

NMW = number of monitoring wells.

COSP = cost of standard size pipe (12" welded steel), \$/ft.

3.11.7.8.7.1.2 Calculate RMWC.

$$RMWC = 160.4\ (DMW)^{-0.7033}$$

where

RMWC = cost of well as fraction of cost of standard pipe.

DMW = depth of monitoring wells, ft.

3.11.7.8.7.1.3 Determine COSP.

COSP is the cost per foot of 12"Ø welded steel pipe. This cost in first quarter of 1977 is \$13.50/ft. For the best cost estimate COSTSP should be a current price input from a vendor, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSP = \$13.50\ \frac{MSECI}{491.6}$$

where

COSP = cost of standard size pipe (12"Ø welded steel), \$/ft.

MSECI = current Marshall and Swift Equipment Cost Index.

3.11.7.8.7.2 Calculate cost of pumps for monitoring wells.

3.11.7.8.7.2.1 Calculate COSTMP.

$$COSTMP = (COSTPS)(MWPR)(NMW)$$

where

COSTMP = cost of pumps for monitoring wells, \$.

COSTPS = cost of standard size pump (3000 gpm), \$.

MWPR = cost of well pump as fraction of cost of standard pump.

NMW = number of monitoring wells.

3.11.7.8.7.2.2 Calculate MWPR.

$$MWPR = 0.0551\ (DMW)^{0.658}$$

where

MWPR = cost of well pump as fraction of cost of standard size pump.

DMW = depth of monitoring wells, ft.

3.11.7.8.7.2.3 Determine COSTPS.

COSTPS is the cost of a 3000 gpm pump. The cost is $17,250 for the first quarter of 1977. For the best cost estimate the user should input a current value of COSTPS, however, if this is not done the cost will be updated using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17{,}250\ \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump (3000 gpm), $.

MSECI = current Marshall and Swift Equipment Cost Index.

3.11.7.8.7.3 Calculate total cost of monitoring wells.

$$COSTMW = COSTM + COSTMP$$

where

COSTMW = total cost of monitoring wells, $.

COSTM = cost of monitoring wells, $.

COSTMP = cost of pumps for monitoring wells, $.

3.11.7.8.8 Calculate O&M material costs.

$$OMMC = \frac{(OMMPD)\ (TCDSS) + (OMMHD)\ (TCDCP) + OMMPU(COSTU) + OMMPM(COSTMW)}{100}$$

where

OMMC = total O&M material cost, $.

OMMPD = O&M material cost for distribution system as percent of construction cost for distribution system, $.

TCDSS = total cost of distribution system for solid set sprinklers, \$.

OMMPM = O&M material cost for monitoring well as percent of construction cost of monitoring well, %.

COSTMW = total cost of monitoring wells, \$.

TCDCP = total cost of distribution system for center pivot, \$.

OMMPU = O&M material cost for underdrain system as percent of construction cost of underdrain system, \$.

COSTU = cost of underdrain system, \$.

3.11.7.8.9 Total bare construction cost.

TBCCSR = (1.18)(TCDSS+TCDCP+COSTU+COSTE+COSTCG+COSTF+COSTMW)

where

TBCCSR = total bare construction cost for slow rate land treatment, \$.

TCDSS = total cost of distribution system for solid set sprinklers, \$.

TCDCP = total cost of distribution system for center pivot, \$.

COSTU = cost of underdrain system, \$.

COSTE = cost of earthwork, \$.

COSTCG = cost of clearing and grubbing, \$.

3.11.7.9 <u>Cost Calculations Output Data.</u>

3.11.7.9.1 Total bare construction cost for slow rate land treatment, TBCCSR, \$.

3.11.7.9.2 O&M material cost, OMMC, \$.

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3.13 OXIDATION DITCH

3.13.1 Background. The oxidation ditch, developed in the Netherlands, is a variation of the extended aeration process that has been used in small towns, isolated communities, and institutions in Europe and the United States. The typical oxidation ditch is equipped with aeration rotors or brushes that provide aeration and circulation. The sewage moves through the ditch at 1 to 2 fps. The ditch may be designed for continuous or intermittent operation. Because of this feature, this process may be adaptable to the fluctuations in flows and loadings associated with recreation area wastewater production.

3.13.2 Input Data.

3.13.2.1 Wastewater flow.

3.13.2.1.1 Average, mgd.

3.13.2.1.2 Peak hourly, mgd.

3.13.2.2 Wastewater characteristics.

3.13.2.2.1 BOD_5 (average and peak).

3.13.2.2.2 COD (average and peak).

3.13.2.2.3 SS (average and peak).

3.13.2.2.4 VSS (average and peak).

3.13.2.2.5 Nondegradable VSS (average and peak).

3.13.3 Design Parameters.

3.13.3.1 MLSS, mg/l, (Range 4000-8000; mean 5000).

3.13.3.2 Organic loading (F/M ratio), lb BOD_5/lb MLVSS/day, (0.067).

3.13.3.3 Volumetric loading, lb BOD_5/1000 ft^3/day, (12.5).

3.13.3.4 Recycle ratio, (50-100 percent).

3.13.3.5 Oxygen requirement, lb O_2 removed, (2.35).

3.13.3.6 Wasted sludge, lb/lb BOD_5 removed, (0.68).

3.13.3.7 Effluent quality, excellent, approximately 90-95% BOD and S.S. reduction.

3.13.4 Process Design Calculations.

3.13.4.1 Calculate Ditch Volume. The design of oxidation ditch activated sludge system is based on the volumetric BOD_5 loading.

$$V = \frac{Q_{avg} \times S_o \times 8.34 \times 1000}{12.5}$$

where

V = volume of ditch, cu ft.

Q_{avg} = average daily flow, mgd.

S_o = BOD_5 in influent, mg/l.

12.5 = loading rate, lb BOD_5/1000 cu ft/day.

3.13.4.2 Calculate Oxygen Requirements.

$$O_2 = 2.35 \times Q_{avg} \times S_o \times 8.34$$

where

O_2 = oxygen required, lb/day.

2.35 = oxygen utilization, lb O_2/lb BOD_5 applied.

3.13.4.3 Effluent Characteristics.

3.13.4.3.1 Suspended Solids.

$$SSE = SS\ (1 - \frac{SSR}{100})$$

where

SSE = effluent suspended solids concentration, mg/l.

SS = influent suspended solids concentration, mg/l.

SSR = suspended solids removal rate, %.

3.13.4.3.2 BOD_5.

$$BODE = BOD\ (1 - \frac{BODR}{100})$$

If $BODE < S_e$, set $BODE = S_e$

where

BODE = effluent BOD_5 concentration, mg/l.

BOD = influent BOD_5 concentration, mg/l.

BODR = BOD_5 removal rate, %.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.13.4.3.3 COD.

$$CODE = 1.5\ BODE$$

$$CODSE = 1.5\ S_e$$

If CODE < CODSE, set CODE = CODSE

where

CODE = effluent COD concentration, mg/l.

CODSE = effluent soluble COD concentration, mg/l.

BODE = effluent BOD_5 concentration, mg/l.

S_e = effluent soluble BOD_5 concentration, mg/l.

3.13.4.3.4 Nitrogen.

$$TKNE = TKN\ (1 - \frac{TKNR}{100})$$

$$NH3E = TKNE$$

where

TKNE = effluent Kjedahl nitrogen concentration, mg/l.

TKN = influent Kjedahl nitrogen concentration, mg/l.

TKNR = Kjedahl nitrogen removal rate, mg/l.

NH3E = effluent ammonia concentration, mg/l.

3.13.4.3.5 Phosphorus.

$$PO4E = PO4\ (1 - \frac{PO4R}{100})$$

where

PO4E = effluent phosphorus concentration, mg/l.

PO4 = influent phosphorus concentration, mg/l.

PO4R = phosphorus removal rate, %.

3.13.4.3.6 Oil and Grease.

$$OAGE = 0.0$$

where

OAGE = effluent oil and grease concentration, mg/1.

3.13.4.3.7 Settleable Solids.

$$SETSO = 0.0$$

where

SETSO = effluent settleable solids concentration, mg/1.

3.13.4.4 Sludge Production. For municipal wastewater only.

biomass: 0.18 lb per lb of BOD_5 removed

$$X_v = 0.18 \times (S_o - S_e) \times Q_{avg} \times 8.34$$

Inert Mass: 0.50 lb per lb of BOD_5 removed

$$X_o = 0.50 \times (S_o - S_e) \times Q_{avg} \times 8.34$$

Total sludge produced

$$X_a = X_v + X_o = 0.68 \times (S_o - S_e) \times Q_{avg} \times 8.34$$

where

X_a = total sludge production, lb/day.

X_v = biomass wasted per day, lb/day.

X_o = inert mass wasted per day, lb/day.

S_o = influent BOD_5, mg/1

S_e = effluent BOD_5, mg/1

Q_{avg} = averaged daily flow, mgd.

3.13.5 <u>Process Design Output Data</u>.

3.13.5.1 Volume of aeration basin required, V, cu ft.

3.13.5.2 Oxygen requirement, O_2, lb/day.

3.13.5.3 Effluent BOD_5, S_e, mg/1.

3.13.6 Quantities Calculations.

3.13.6.1 Assumptions.

3.13.6.1.1 In this flow range only one oxidation ditch will ever be required.

3.13.6.1.2 Two size rotors will be used: 42" ⌀ and 27½" ⌀.

3.13.6.1.3 The following parameters will be used for the 42" ⌀ and 27½" ⌀ rotors.

	42" ⌀	27½" ⌀
Water Depth	12 ft	5 ft
Oxygen Transfer lb/hr ft	3.74 lb/hr ft	2.85
Rotor Speed	72 RPM	91 RPM
Rotor Immersion	8 inches	6 inches
Mixing Requirement	21,000 gal/ft	16,000 gal/ft
Motor Requirement	1.126 BHP/ft	0.838 BHP/ft

3.13.6.1.4 The ditch will be constructed as shown in Figure 3.13-1.

3.13.6.1.5 Two rotors will always be used.

3.13.6.2 Rotor selection. Rotor length will be calculated based on oxygen requirements and mixing requirements and the rotor of the greatest length will be used. The length of 42" ⌀ rotor required will be calculated first.

3.13.6.2.1 Length required based on oxygen requirement for 42" ⌀ rotor.

$$LRTO = \frac{O_2}{(3.74)\ (24)}$$

where

LRTO = length of rotor required for oxygenation, ft.

O_2 = oxygen required, lb/day.

3.74 = oxygen transfer of 42 inch diameter rotor with 8 inch submergence, O_2 lb/hr ft.

24 = conversion, hrs/day.

3.13.6.2.2 Length required based on complete mix criteria for 42" ⌀ rotor.

$$LRTM = \frac{(7.48)(V)}{21,000}$$

where

LRTM = length of rotor required for mixing, ft.

V = volume of basin, cu ft.

7.48 = conversion factor, gal/cu ft.

21,000 = mixing capacity of 42" ϕ rotor, gal/ft.

3.13.6.2.3 Design length of rotor. If either LRTO or LRTM is greater than or equal to 12 then select the larger of the two values for the design length of rotor, LRT and select the rotor diameter, DRT, to be 42". The number of rotors will be 2 and the length per rotor will be:

$$LRTK = \frac{LRT}{2}$$

LRTK must be an integer.

However, if LRT is less than 12 the 42" ϕ rotor is too large. Select the rotor diameter, DRT, of 27½" and recalculate LRTK.

3.13.6.2.4 Length required based on oxygen requirement for 27½" ϕ rotor.

$$LRTO = \frac{O_2}{(2.85)(24)}$$

where

LRTO = length of rotor required for oxygenation, ft.

O_2 = oxygen required, lb/day.

2.85 = oxygen transfer of 27½" rotor with 6" submergence, O_2 lb/hr ft.

24 = conversion, hrs/day.

3.13.6.2.5 Length required based on complete mix criteria for 27½" ϕ rotor

$$LRTM = \frac{(7.48)(V)}{16,000}$$

where

LRTM = length of rotor required for mixing, ft.

V = volume of basin, cu ft.

7.48 = conversion, gal/cu ft.

16,000 = mixing capacity of 27½" ϕ rotor, gal/ft.

3.13.6.2.6 Design length of rotor. Select the larger of the two values for the design length of rotor, LRT. If LRT is less than 6 ft, then set LRT = 6 ft. The number of rotors will always be 2 and the length per rotor, LRTK, will be:

$$LRTK = \frac{LRT}{2}$$

LRTK must be an integer.

where

LRTK = length of each rotor, ft.

LRT = design length of rotor, ft.

3.13.6.3 Determine basin design and dimensions.

3.13.6.3.1 Calculations when 42" ϕ rotor is selected.

If $LRTK \leq 15.5$ ft; then $W_b = LRTK + 1$

If $LRTK > 15.5$ ft; then $W_b = LRTK + 4$

where

W_b = ditch bottom width, ft.

LRTK = length of each rotor, ft.

3.13.6.3.1.1 Basin configuration

Basin water depth = 12 ft.
Side walls are 45 degrees.
Basin water surface width, $W_s = W_b + 12$
Freeboard = 1.0 ft
Median strip width = 1.0 ft.

3.13.6.3.1.2 Calculate volume of circular ends.

$$V_e = 37.71\ W_b^{\ 2} + 490.2\ W_b + 2045.8$$

where

V_e = volume of circular ends, cu ft.

W_b = ditch bottom width, ft.

3.13.6.3.1.3 Calculate volume of straight section.

$$V_s = (24\ W_b + 144)\ L_s$$

where

V_s = volume of straight section, cu ft.

L_s = length of straight section, ft.

W_b = ditch bottom width, ft.

3.13.6.3.1.4 Calculate length of straight section.

$$L_s = \frac{V-Ve}{24\ W_b + 144}$$

where

L_s = length of straight section, ft.

V = volume required, cu ft.

V_e = volume of circular ends

3.13.6.3.1.5 Calculate total length, L_t, and width, W_t, including freeboard.

$$L_t = L_s + 2W_b + 26$$

$$W_t = 2\ W_b + 27$$

where

L_t = total length of ditch, ft.

W_t = total width of ditch, ft.

L_s = length of straight section, ft.

W_b = ditch bottom width, ft.

3.13.6.3.1.6 Calculate volume for excavation.

$$Vex = 42.4\ W_b^2 + 636.3\ W_b + 27\ W_b L_s + 209.2 L_s + 2983.5$$

where

V_{ex} = volume of excavation required for ditch, cu ft.

W_b = ditch bottom width, ft.

L_s = length of straight section, ft.

3.13.6.3.2 Calculations when 27½" ⌀ rotor is selected.

If LRTK $\leq$ 15.5 ft; then W_b = LRTK + 1

If LRTK > 15.5 ft; then W_b = LRTK + 4

where

W_b = ditch bottom width, ft.

LRTK = length of each rotor, ft.

3.13.6.3.2.1 Basin Configuration.

Basin water depth = 5 ft
Side walls are 45 degrees
Basin water surface width, $W_s = W_b + 5$
Freeboard = 1.0 ft
Median strip width = 1.0 ft

3.13.6.3.2.2 Calculate volume of circular ends.

$$V_e = 15.72\ W_b^{\,2} + 94.3\ W_b + 174.2$$

where

V_e = volume of circular ends, cu ft.

W_b = ditch bottom width, ft.

3.13.6.3.2.3 Calculate volume of straight section.

$$V_s = (10\ W_b + 25)\ L_s$$

where

V_s = volume of straight section, cu ft.

L_s = length of straight section, ft.

W_b = ditch bottom width, ft.

3.13.6.3.2.4 Calculate length of straight section.

$$L_s = \frac{V - Ve}{10\ Wb + 25}$$

where

V = volume required, cu ft.

L_s = length of straight section, ft.

W_b = ditch bottom width, ft.

V_e = volume of circular ends, cu ft.

3.13.6.3.2.5 Calculate total length, L_t, and width, W_t, including freeboard.

$$L_t = L_s + 2\ W_b + 12$$

$$W_t = 2\ W_b + 13$$

where

L_t = total length of ditch, ft.

W_t = total width of ditch, ft.

L_s = length of straight section, ft.

W_b = ditch bottom width, ft.

3.13.6.3.2.6 Calculate volume for excavation.

$$V_{ex} = 20.43\ W_b^{\ 2} + 163.44\ W_b + 13\ W_bL_s + 55.25\ L_s + 388.17$$

where

V_{ex} = volume of excavation required for ditch, cu ft.

W_b = ditch bottom width, ft.

L_s = length of straight section, ft.

3.13.6.4 Determine quantity of concrete required.

3.13.6.4.1 Volume of concrete wall required.

$$V_{cw} = (D_w + 1)\ (L_s)$$

where

V_{cw} = volume of concrete wall, cu ft.

D_w = depth of water in ditch, ft.

L_s = length of straight section, ft.

3.13.6.4.2 Volume of concrete slab required.

$$V_{cs} = 4.4\ D_wW_b + 4.4\ W_b + 2.2\ D_w^{\ 2} + 4.4\ D_w + L_sW_b + 1.4\ L_s\ D_w + 1.4\ L_s + 2.2$$

where

V_{cs} = volume concrete slab, cu ft.

D_w = depth of water in ditch, ft.

W_b = ditch bottom width, ft.

L_s = length of straight section, ft.

3.13.6.5 Determine quantity of handrail required.

$$LHR = 2\ L_s + 6.28\ W_b + 6.28\ D_w + 6.28$$

where

LHR = length of handrail, ft.

L_s = length of straight section, ft.

W_b = ditch bottom width, ft.

D_w = depth of water in ditch, ft.

3.13.6.6 Operation and maintenance manpower requirement.

$$OMMH = 2386.3\ (Q_{avg})^{0.4693}$$

where

OMMH = operation and maintenance manpower, MH/yr.

Q_{avg} = average daily flow, mgd.

3.13.6.7 Energy Requirement for Operation. The energy requirement depends on the rotor chosen. The 42" ∅ rotor at 72 RPM and 8" submergence requires 1.126 BHP/ft. The 27½" ∅ rotor at 91 RPM and 6" submergence requires 0.838 BHP/ft.

3.13.6.7.1 Calculate BHP.

3.13.6.7.1.1 For 42" ∅ rotor.

$$BHP = (1.126)\ (LRTK)$$

3.13.6.7.1.2 For 27½" ∅ rotor.

$$BHP = (0.838)\ (LRTK)$$

where

BHP = brake horsepower required per rotor.

LRTK = length of each rotor, ft.

1.126 = horsepower required per foot of 42" ∅ rotor, BHP/ft.

0.838 = horsepower requirement per foot of 27½" ∅ rotor, BHP/ft.

3.13.6.7.2 Calculate KWH.

$$KWH = BHP \times 2 \times 0.85 \times 24 \times 365$$

where

KWH = energy requirement for operation, Kwhr/yr.

2 = number of rotors.

BHP = brake horsepower required per rotor.

0.85 = conversion factor and efficiencies.

24 = hours per day.

365 = days per year.

3.13.6.8 Determine total installed horsepower. The total installed horsepower (THP) will be greater than the brake horsepower since motors nomally available come in only certain sizes. Using BHP just calculated the motor size will be one of the following: 2, 3, 5, 7½, 10, 15, 20, 25, 30, 40, 50. Select the motor size which is nearest the calculated BHP but larger. This will be the motor horsepower, MHP.

$$THP = (MHP)\ (2)$$

where

THP = total installed horsepower.

MHP = motor horsepower.

2 = number of rotors.

3.13.6.9 Operation and maintenance material and supply costs. O&M material and supply costs include such items as lubrication oil, paint, and repair materials. These costs are estimated as a percentage of installed costs for the aeration equipment.

$$OMMP = 4.225 - 0.975 \log (THP)$$

where

OMMP = O&M material supply cost as percent of installed cost of aeration equipment, %.

THP = total installed horsepower.

3.13.6.10 Other construction cost items. From the calculations approximately 95% of the construction cost has been accounted for. Other minor costs such as effluent weir, influent and effluent pipe, etc., would be about 5%.

$$CF = \frac{1}{0.95} = 1.052$$

where

CF = correction factor for construction cost.

3.13.7 Quantities Calculations Output Data.

3.13.7.1 Length of each rotor, LRTK, ft.

3.13.7.2 Diameter of rotor, DRT, inches.

3.13.7.3 Ditch bottom width, W_b, ft.

3.13.7.4 Depth of water in ditch, D_w, ft.

3.13.7.5 Total ditch length, L_t, ft.

3.13.7.6 Total ditch width, W_t, ft.

3.13.7.7 Volume of excavation required for ditch, V_{ex}, cu ft.

3.13.7.8 Volume of concrete wall, V_{cw}, cu ft.

3.13.7.9 Volume of concrete slab, V_{cs}, cu ft.

3.13.7.10 Length of handrail, LHR, ft.

3.13.7.11 Operation and maintenance manpower, OMMH, MH/yr.

3.13.7.12 Energy requirement for operation, KWH, Kwhr/yr.

3.13.7.13 O&M material and supply cost as percent installed cost aeration equipment , OMMP, %.

3.13.7.14 Correction factor for construction cost, CF.

3.13.8 Unit Price Input Required.

3.13.8.1 Unit price input for earthwork, UPIEX, \$/cu yd.

3.13.8.2 Unit price input for concrete wall, UPICW, \$/cu yd.

3.13.8.3 Unit price input for concrete slab, UPICS, \$/cu yd.

3.13.8.4 Unit price input for handrail, UPIHR, \$/ft.

3.13.8.5 Standard size rotor cost, COSTDS, \$ (Optional).

3.13.8.6 Installation labor rate, LABRI, \$/hr.

3.13.8.7 Current Marshall and Swift Equipment Cost Index, MSECI.

3.13.9 Cost Calculations.

3.13.9.1 Cost of earthwork.

$$COSTE = \frac{(V_{ex})\ (UPIEX)}{27}$$

where

COSTE = cost of earthwork, \$.

V_{ex} = volume of excavation required for ditch, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.13.9.2 Cost of concrete walls.

$$COSTCW = \frac{(V_{cw})\ (UPICW)}{27}$$

where

COSTCW = cost of concrete walls, \$.

V_{cw} = volume of concrete walls, cu ft.

UPICW = unit price input for concrete walls, \$/cu yd.

3.13.9.3 Cost of concrete slab.

$$COSTCS = \frac{(V_{cs})\ (UPICS)}{27}$$

where

COSTCS = cost of concrete slab, \$

V_{cs} = volume of concrete slab, cu ft.

UPICS = unit price input for concrete slab, \$/cu yd.

3.13.9.4 Cost of installed equipment.

3.13.9.4.1 Purchase cost of rotors.

$$COSTRK = \frac{(COSTDS)\ (COSTRO)}{100}$$

where

COSTRK = purchase cost of rotor of LRTK feet, \$.

COSTDS = purchase cost of standard size rotor, 42" ⌀ and 20 ft long, \$.

COSTRO = cost of rotor of length LRTK as percent of cost of standard size rotor, %.

3.13.9.4.2 Calculate COSTRO.

3.13.9.4.2.1 For rotor diameter of 42 inches.

If LRTK $\leq$ 18 ft

$$COSTRO = 39.51\ (LRTK)^{0.2790}$$

If LRTK $>$ 18 ft

$$COSTRO = 16.59\ (LRTK)^{0.5792}$$

3.13.9.4.2.2 For rotor diameter of 27½" inches.

If LRTK $\leq$ 9.0 ft

$$COSTRO = 44.23\ (LRTK)^{0.1781}$$

If LRTK $>$ 9.0 ft

$$COSTRO = 27.63\ (LRTK)^{0.3932}$$

3.13.9.4.3 Cost of Standard Size Rotor, COSTDS. The cost of a 42" diameter rotor 20 feet in length for the first quarter of 1977 is \$15,340. For better cost estimation, COSTDS, should be obtained from an equipment vendor and treated as a unit cost input. However, if this is not done the Marshall and Swift Equipment Cost Index will be used to update the cost.

$$COSTDS = \$15{,}340 \times \frac{MSECI}{491.6}$$

where

COSTDS = purchase cost of standard size rotor, \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = MSECI for 1st quarter 1977.

3.13.9.4.4 Equipment Installation Cost.

3.13.9.4.4.1 Calculate installation labor required.

$$IMH = 10 + 2.667\ (LRTK)$$

where

IMH = installation labor required, MH.

LRTK = length of each rotor, ft.

3.13.9.4.4.2 Calculate installation cost.

$$ICOST = (IMH)\ (LABRI)$$

where

ICOST = installation cost, \$.

IMH = installation labor, MH.

LABRI = installation labor rate, \$/MH.

3.13.9.4.5 Other minor costs associated with installation of equipment. These costs include electrical work, foundations, bridge, painting, etc. It is expressed as percentage of the equipment purchase cost.

$$PMINC = 52.04 - 0.34\ (LRTK)$$

PMINC is always greater than 40.

3.13.9.4.6 Installed aeration equipment cost.

$$IEC = [(COSTRK)\ (1 + \frac{PMINC}{100}) + ICOST](2)$$

where

IEC = installed equipment cost, \$.

COSTRK = purchase cost of rotor of length LRTK, \$.

PMINC = minor costs as percent of purchase cost of equipment, %.

ICOST = installation cost, \$.

3.13.9.5 Cost of handrails.

$$COSTHR = (LHR)\ (UPIHR)$$

where

COSTHR = installed handrail cost, \$.

LHR = length of handrail required, ft.

UPIHR = unit price input for handrails, \$/ft.

3.13.9.6 Total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + IEC + COSTHR)\ (CF)$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earthwork, \$.

COSTCW = cost of concrete wall, \$.

COSTCS = cost of concrete slab, \$.

IEC = installed equipment cost, \$.

COSTHR = installed handrail cost, \$.

CF = correction factor for other cost items.

3.13.9.7 Operation and maintenance material and supply costs.

$$OMMC = (IEC)\ (\frac{OMMP}{100})$$

where

OMMC = O&M material and supply cost, $/yr.

OMMP = O&M material and supply costs as percent of installed equipment costs, %.

IEC = installed equipment cost, $.

3.13.10 Cost Calculations Output Data.

3.13.10.1 Total bare construction cost, TBCC, $.

3.13.10.2 O&M material and supply cost, OMMC $/yr.

3.13.11 Bibliography.

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3.13.11.7 Parker, H.W., Wastewater Systems Engineering, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1975.

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3.15 PUMPING

3.15.1 Background. There are several situations throughout a sewage treatment facility which require pumping. Typically at the head of the treatment facility pumping of the raw waste is required. Other points in the treatment facility which might require pumping are prior to trickling filters, tertiary filters, carbon adsorption units, or any treatment process which creates relatively high head losses. Generally speaking two different type pumps are used for raw waste pumping and pumping for other processes in the treatment facility. For this reason the pumping has been divided into raw waste pumping and intermediate pumping.

3.15.2 Intermediate Pumping.

3.15.2.1 In intermediate pumping the wastewater is relatively clean and free from large solids so that more efficient pumps can be used for these processes than for raw waste pumping.

3.15.3 Raw Waste Pumping.

3.15.3.1 Pumping of raw sewage at the head of a treatment facility is often required to produce the head required for sewage to flow through the plant. This pumping is accomplished with relatively inefficient low-head pumps which are capable of passing large solids without damage to the pumps.

3.15.4 Intermediate Pumping.

3.15.4.1 Input Data.

3.15.4.1.1 Average daily wastewater flow, Q_{avg}, mgd.

3.15.4.1.2 Peak wastewater flow, Q_{pk}, mgd.

3.15.4.2 Design Parameters.

3.15.4.2.1 Wastewater flow, mgd.

3.15.4.2.2 Number of pumps.

3.15.4.2.3 Total head on pumps, ft.

3.15.4.3 Process Design Calculations.

3.15.4.3.1 Calculate design capacity of pumps.

3.15.4.3.1.1 Intemediate pumping.

$$GPM = \frac{(Q_{avg})\ (2)\ (10^6)}{1440}$$

where

GPM = design capacity of pumps, gpm.

2 = excess capacity factor to handle peak flows.

3.15.4.3.1.2 Return sludge pumping.

$$GPM = \frac{(Q_{avg})\ (RSR)\ (10^6)}{1440}$$

where

GPM = design capacity of pumps, gpm.

RSR = return sludge ratio to average wastewater flow, from Table 3.15-1.

Table 3.15-1

Activated Sludge Processes	RSR
Conventional	1.0
Complete-Mix	1.0
Step-Aeration	1.0
Modified-Aeration	1.0
Contact-Stabilization	1.0
Extended-Aeration	1.5
Kraus-Process	1.0
High-Rate Aeration	5.0
Pure-Oxygen System	1.0

3.15.4.3.2 Determine the type, number and size of pumps required. For the purposes of this program it has been assumed the pumps will be horizontal single-stage, single-suction, split casing centrifugal pumps designed for sewage applications. Also the head required is assumed to be 40 ft for all applications. All pumps will be assumed to be the same size with variable speed drives and the convention of sparing the largest pump will be adhered to.

3.15.4.3.2.1 Number of batteries. The number of batteries will be calculated by trial and error; begin with NB = 1. If GPM/NB > 80,000, then go the NB = NB + 1 and repeat until GPM/NB ≤ 80,000. Then:

$$GPMB = \frac{GPM}{NB}$$

where

$GPMB$ = design flow per battery, gpm.

NB = number of batteries.

3.15.4.3.2.2 Number of pumps per battery. The number of pumps per battery will be calculated by trial and error. Start with N = 2. If GPMB/N > 20,000 gpm, go to N = N+1 and repeat until GPMB/N ≤ 20,000 gpm.

$$GPMP = \frac{GPMB}{N}$$

$$NP = N + 1$$

where

$GPMP$ = design capacity of the individual pumps, gpm.

N = number of pumps required to handle design flow.

NP = total number of pumps per battery, including spare.

3.15.4.4 Process Design Output Data.

3.15.4.4.1 Design capacity of pumps, GPM, gpm.

3.15.4.4.2 Number of batteries, NB.

3.15.4.4.3 Design flow per battery, GPMB, gpm.

3.15.4.4.4 Number of pumps required to handle design flow, N.

3.15.4.4.5 Total number of pumps per battery, including spare, NP.

3.15.4.4.6 Design capacity of the individual pumps, GPMP, gpm.

3.15.4.5 Quantities Calculations.

3.15.4.5.1 Determine area of pump building.

$$PBA = [0.0284\ (GPMB) + 100]\ NB$$

where

PBA = pump building area, sq ft.

3.15.4.5.2 Calculate volume of earthwork required. The pumping building is usually a bilevel building with the pumps below ground and all electrical and control facilities above ground. It is assumed that the average depth of excavation would be 8 ft. The volume of earthwork will be estimated by:

$$V_{ew} = (8)(PBA)$$

where

V_{ew} = volume of earthwork required, cu ft.

3.15.4.5.3 Calculate operation manpower required. The operation manpower can be related to the firm pumping capacity.

3.15.4.5.3.1 Calculate firm pumping capacity.

$$FPC = \frac{(GPM)(1440)}{10^6}$$

where

FPC = firm pumping capacity, mgd.

3.15.4.5.3.2 If $0 < FPC \leq 7$ mgd:

$$OMH = 440\ (FPC)^{0.1285}$$

3.15.4.5.3.3 If $7 < FPC \leq 30$ mgd:

$$OMH = 294.4\ (FPC)^{0.3350}$$

3.15.4.5.3.4 If $30 < FPC \leq 80$ mgd:

$$OMH = 40.5\ (FPC)^{0.8661}$$

3.15.4.5.3.5 If $FPC > 80$ mgd:

$$OMH = 21.3\ (FPC)^{1.012}$$

where

OMH = operating manpower required, man-hours/yr.

3.15.4.5.4 Calculate maintenance manpower.

3.15.4.5.4.1 If $0 < FPC \leq 7$ mgd:

$$MMH = 360\ (FPC)^{0.1478}$$

3.15.4.5.4.2 If $7 < FPC \leq 30$ mgd:

$$MMH = 255.2\ (FPC)^{0.3247}$$

3.15.4.5.4.3 If $30 < FPC \leq 80$ mgd:

$$MMH = 85.7\ (FPC)^{0.6456}$$

3.15.4.5.4.4 If FPC > 80 mgd:

$$MMH = 30.6\ (FPC)^{0.8806}$$

where

MMH = maintenance manpower requirement, man-hours/yr.

3.15.4.5.5 Calculate electrical energy required.

$$KWH = 67{,}000\ (Q_{avg})^{0.9976}$$

where

KWH = electric energy required, kwhr/yr.

3.15.4.5.6 Calculate operation and maintenance material and supply costs. This item covers the cost of lubrication oils, paint, repair and replacement parts, etc. It is expressed as a percent of the total bare construction costs.

$$OMMP = 0.7\%$$

where

OMMP = operation and maintenance material and supply costs, as percent of the total bare construction cost, percent.

3.15.4.5.7 Other minor construction cost items. From the calculations approximately 85 percent of the construction costs have been accounted for. Other minor items such as piping, overhead crane, site cleaning, seeding, etc., would be 15 percent.

$$CF = \frac{1}{0.85} = 1.18$$

where

CF = correction factor for other minor construction costs.

3.15.4.6 <u>Quantities Calculations Output Data</u>.

3.15.4.6.1 Pump building area, PBA, sq ft.

3.15.4.6.2 Volume of earthwork required, V_{ew}, cu ft.

3.15.4.6.3 Firm pumping capacity, FPC, mgd.

3.15.4.6.4 Operating manpower required, OMH, man-hours/yr.

3.15.4.6.5 Maintenance manpower required, MMH, man-hours/ yr.

3.15.4.6.6 Electrical energy required, KWH/kwhr/yr.

3.15.4.6.7 Operation and maintenance material and supply costs, OMMP, percent.

3.15.4.6.8 Correction factor for other minor construction costs, CF.

3.15.4.6.9 Design capacity of the individual pumps, GPMP, gpm.

3.15.4.7 <u>Unit Price Input Required</u>.

3.15.4.7.1 Unit price input for building cost, UPIBC, $/sq ft.

3.15.4.7.2 Unit price input for earthwork, UPIEX, $/cu ft.

3.15.4.7.3 Cost of standard size pump equipment, COSTPS, $ (optional).

3.15.4.7.4 Marshall and Swift Equipment Cost Index, MSECI.

3.15.4.8 <u>Cost Calculations</u>.

3.15.4.8.1 Cost of earthwork.

$$COSTE = \frac{V_{ew}}{27} UPIEX$$

where

COSTE = cost of earthwork, \$.

V_{ew} = volume of earthwork, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.15.4.8.2 Cost of pump building.

$$COSTPB = (PBA)(UPIBC)$$

where

COSTPB = cost of pump building, \$.

PBA = pump building area, sq ft.

UPIBC = unit price input for building cost, \$/sq ft.

3.15.4.8.3 Purchase cost of pumps and drivers.

3.15.4.8.3.1 Calculate COSTP.

$$COSTP = \frac{COSTRO}{100}\ (COSTPS)(NP)(NB)$$

where

COSTP = cost of pumps and drivers, \$.

COSTRO = cost of pumps and drivers of capacity GPMP, as percent of cost of standard size pump, percent.

COSTPS = cost of standard size pump (3000 gpm), \$.

NP = total number of pumps per battery.

NB = number of batteries.

3.15.8.3.2 Calculate COSTRO.

If $0 < GPMP \leq 5000$ gpm, COSTRO is calculated by:

$$COSTRO = 2.93\ (GPMP)^{0.4404}$$

If $GPMP > 5000$ gpm, COSTRO is calculated by:

$$COSTRO = .0064\ (GPMP)^{1.16}$$

3.15.4.8.3.3 Purchase cost of standard size pump and driver. A 3000 gpm pump was selected as a standard. The cost of a 3000 gpm pump and driver for the first quarter of 1977 is:

$$COSTPS = \$17,250$$

For better estimation, COSTPS should be obtained from the vendor and treated as a unit price input. If this is not done, the cost will be adjusted using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$17,250 \frac{MSECI}{491.6}$$

where

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index.

3.15.4.8.4 Installed equipment costs. Typically, the installation cost of pumps is approximately 100 percent of the equipment cost. This includes cost of piping, concrete, steel, electrical, paint, and installation labor.

$$IPC = (COSTP)(2.0)$$

where

IPC = installed pumping equipment cost, \$.

3.15.4.8.5 Total bare construction cost.

$$TBCC = [COSTE + COSTPB + IPC]\ CF$$

where

TBCC = total bare construction cost, \$.

CF = correction factor minor construction costs

3.15.4.8.6 Operation and maintenance material and supply costs.

$$OMMC = (TBCC)\left(\frac{OMMP}{100}\right)$$

where

OMMC = operation and maintenance material and supply cost, \$.

OMMP = operation and maintenance material and supply costs, as a percent of total bare construction cost, percent.

3.15.4.9 Cost Calculations Output Data.

3.15.4.9.1 Total bare construction cost, TBCC, $.

3.15.4.9.2 Operation and maintenance material and supply costs, OMMC, $/yr.

3.15.5 Raw Waste Pumping.

3.15.5.1 Input Data.

3.15.5.1.1 Wastewater flow.

3.15.5.1.1.1 Average wastewater flow, Q_{avg}, mgd.

3.15.5.1.1.2 Peak wastewater flow, Q_p, mgd.

3.15.5.1.2 Pump head, PH, ft.

3.15.5.2 Process Design Calculations.

3.15.5.2.1 Determine the number of pumps. In this flow range a minimum of two pumps will always be used. One pump must be capable of handling the average wastewater flow, the other pump will be used as a spare and also to handle the additional flow when the peak wastewater flow occurs.

$$N = \frac{Q_p}{Q_{avg}}$$

$\frac{Q_p}{Q_{avg}}$ must be an integer.

where

N = number of pumps.

3.15.5.2.2 Determine size of pumps.

$$\frac{(Q_{avg})\ (10^6)}{1440}$$

where

GPM = size of each pump, gpm.

Q_p = peak wastewater flow, mgd.

3.15.5.2.3 Determine wetwell dimensions.

3.15.5.23.1 Assumptions for wetwell.

Wetwell will be circular.
Three (3) minutes pumping time will be provided between low water and high water control levels.
Low water cut-off will be a minimum of 2 ft above bottom of wetwell.
A minimum of 3 ft will be provided between high water and low water levels.

A 2 ft freeboard will be required above the high water level.
Minimum diameter for access purposes will be 4 ft.

3.15.5.2.3.2 Calculate diameter.

$$D = .4125\ (GPM)^{0.5}$$

D must be an integer.
If D < 4 ft set D = 4 ft

where

D = diameter of wet well, ft.

GPM = size of each pump, gpm.

3.15.5.2.3.3 Depth of wetwell. From the assumptions the total depth of the wetwell is fixed.

$$DW = 7\ ft$$

where

DW = depth of wetwell, ft.

3.15.5.3 Process Design Output Data.

3.15.5.3.1 Wastewater flow.

3.15.5.3.1.1 Average wastewater flow, Q_{avg}, mgd.

3.15.5.3.1.2 Peak wastewater flow, Q_p, mgd.

3.15.5.3.2 Pump head, PH, ft.

3.15.5.3.3 Number of pumps, N.

3.15.5.3.4 Size of each pump, GPM, gpm.

3.15.5.3.5 Diameter of wetwell, D, ft.

3.15.5.3.6 Depth of wetwell, Dw, ft.

3.15.5.4 Quantities Calculations.

3.15.5.4.1 Determine size of pump building. For flows in this range we will assume that a building with 100 sq ft is sufficient for the entire flow range.

$$PBA = 100\ ft^2$$

where

PBA = area of the pump building, ft^2.

3.15.5.4.2 Calculate volume of earthwork.

$$V_{ew} = 5.5D^2 + 98.96D + 535$$

where

V_{ew} = volume of earthwork required, cu ft.

D = diameter of wetwell, ft.

3.15.5.4.3 Calculate volume of concrete required.

Assume wall thickness of 9 inches.

$$V_{cw} = .59D^2 + 20.7D + 7.2$$

where

V_{cw} = volume of R.C. wall required, cu ft.

D = diameter of wetwell, ft.

3.15.5.4.4 Calculate firm pumping capacity.

$$FPC = (GPM)\ (1440)\ (10^{-6})\ (N - 1)$$

where

FPC = firm pumping capacity, mgd.

GPM = size of each pump, gpm.

3.15.5.4.5 Calculate operation manpower required.

$$OMH = 519.2\ (FPC)^{0.1133}$$

where

OMH = operation manpower required, MH/yr.

FPC = firm pumping capacity, mgd.

3.15.5.4.6 Calculate maintenance manpower required.

$$MMH = 440.3\ (FPC)^{0.1386}$$

where

MMH = maintenance manpower required, MH/yr.

FPC = firm pumping capacity, mgd.

3.15.5.4.7 Calculate operation andmaintenance material and supply costs.

$$OMMP = 0.6$$

where

$OMMP$ = O&M material and supply costs as percent of total bare construction cost, %.

3.15.5.4.8 Calculate operation energy costs.

$$KWH = 2121.4\ (Q_{avg})\ (PH)$$

where

KWH = operation energy costs, kw hr/yr.

Q_{avg} = average wastewater flow, mgd.

PH = pump head, ft.

3.15.5.4.9 Other construction cost items. The above calculations would account for approximately 80% of the total cost of the pumping station. The other 20% would be for such items as piping, valves, painting, etc.

$$CF = \frac{1}{.8} = 1.25$$

where

CF = correction factor for other construction cost items.

3.15.5.5 <u>Quantities Calculations</u>.

3.15.5.5.1 Area of pump building, PBA, ft^2.

3.15.5.5.2 Volume of earthwork required, V_{ew}, cu ft.

3.15.5.5.3 Volume of R.C. wall required, V_{cw}, cu ft.

3.15.5.5.4 Operation manpower required, OMH, MH/yr.

3.15.5.5.5 Maintenance manpower required, MMH, MH/yr.

3.15.5.5.6 O&M material and supply costs as percent of total bare construction costs, OMMP, %.

3.15.5.5.7 Correction factor for other construction cost items, CF.

3.15.5.6 <u>Unit Price Input Required</u>.

3.15.5.6.1 Unit price input for building cost, UPIBC, $\$/ft^2$.

3.15.5.6.2 Unit price input for earthwork, UPIEX, $/cu yd.

3.15.5.6.3 Unit price input for R.C. wall in-place, UPICW, $/cu yd.

3.15.5.6.4 Cost of standard size pump and drive, COSTPS, $. (optional).

3.15.5.6.5 Marshall and Swift Equipment Cost Index, MSECI.

3.15.5.7 <u>Cost Calculations</u>.

3.15.5.7.1 Calculate cost of pump building.

$$PBC = (PBA)\ (UPIBC)$$

where

PBC = cost of pump building, $.

PBA = area of pump building, ft^2.

UPIBC = unit price input for building cost, $\$/ft^2$.

3.15.5.7.2 Calculate cost of earthwork.

$$COSTE = \frac{V_{ew}}{27}\ (UPIEX)$$

where

COSTE = cost of earthwork, $.

V_{ew} = volume of earthwork required, ft^3.

UPIEX = unit price input for excavation, \$/cu yd.

3.15.5.7.3 Calculate cost of concrete.

$$COSTC = \frac{V_{cw}}{27} (UPICW)$$

where

COSTC = cost of concrete in-place, \$.

V_{cw} = volume of R.C. wall required, ft^3.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.15.5.7.4 Calculate purchase cost of pumps and drivers.

$$COSTP = \frac{COSTRO}{100} (COSTPS)\ (N)$$

where

COSTP = purchase cost of pumps and drivers, \$.

COSTRO = cost of pump and driver of capacity GPM, as percent of cost of standard size pump, %.

COSTPS = cost of standard size pump and driver (100 gpm), \$.

N = number of pumps.

3.15.5.7.4.1 Calculate COSTRO.

$$COSTRO = 13.44\ (GPM)^{0.4466}$$

where

COSTRO = cost of pump and driver of capacity GPM, as percent of cost of standard size pump, %.

GPM = size of each pump, gpm.

3.15.5.7.4.2 Determine COSTPS. COSTPS is the cost of a standard size pump and driver. A pump and driver capable of pumping 100 gpm at 40 ft of head. The cost for the first quarter of 1977 is:

$$COSTPS = \$2,800$$

For better cost estimation, COSTPS should be obtained from the vendor and treated as a unit price input. If this is not done, the cost will be adjusted using the Marshall and Swift Equipment Cost Index.

$$COSTPS = \$2,800 \frac{MSECI}{491.6}$$

where

COSTPS = cost of standard size pump and driver (100 gpm), \$.

MSECI = current Marshall and Swift Equipment Cost Index.

491.6 = Marshall and Swift Equipment Cost Index first quarter of 1977.

3.15.5.7.5 Calculate installed equipment cost. Typically, the installation cost of pumps is approximately 100 percent of the equipment cost.

$$IEC = (COSTP)\ (2.0)$$

where

IEC = installed equipment cost, \$.

COSTP = purchase cost of pumps and drivers, \$.

3.15.5.7.6 Calculate total bare construction cost.

$$TBCC = (IEC + PBC + COSTE + COSTC)\ CF$$

where

TBCC = total bare construction cost, \$.

IEC = installed equipment cost, \$.

PBC = cost of pump building, \$.

COSTE = cost of earthwork, \$.

COSTC = cost of concrete in-place, \$.

3.15.5.7.7 Calculate O&M material and supply costs.

$$OMMC = \frac{OMMP}{100} (TBCC)$$

where

OMMC = O&M material and supply costs, \$.

OMMP = O&M material and supply costs as percent of total bare construction cost, %.

TBCC = total bare construction cost, \$.

3.15.5.8 Cost Calculations Output Data.

3.15.5.8.1 Total bare construction costs, TBCC, \$.

3.15.5.8.2 O&M material and supply costs, OMMC, \$.

3.15.6 Bibliography.

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3.17 SEPTIC TANKS AND TILE FIELDS

3.17.1 Background. Septic tanks and soil absorption systems have been used extensively for treating sewage in non-sewered areas from single and multiple residences, businesses, recreational areas, and small communities. The concept of the septic tank system appears simple but it is actually a complex physical, chemical and biological system. The system is popular because once it is constructed, assuming adequate design, it functions without controls with almost no maintenance.

3.17.2 Input Data.

3.17.2.1 Average daily flow, mgd.

3.17.2.2 Soil percolation rate, min/in.

3.17.3 Design Parameters.

3.17.3.1 Flow, mgd.

3.17.3.2 Soil percolation rate, min/in.

3.17.4 Process Design Calculations.

3.17.4.1 Calculate septic tank volume.

3.17.4.1.1 If $Q_{avg} \leq 0.0015$ the septic tank volume is given by:

$$V_N = (1.5)\ (Q_{avg})\ (10^6)$$

3.17.4.1.2 If $Q_{avg} > 0.0015$ the septic tank volume is given by:

$$V_N = 1125 + 0.75\ (Q_{avg})\ (10^6)$$

where

V_N = net volume of septic tank, gal.

Q_{avg} = average daily flow, mgd.

3.17.4.2 Calculate the length of tile for absorption field. Assume a 2 ft wide trench will be used with 4 inch perforated pipe.

$$L = Q_{avg} \frac{(10^6)\ (t)^{0.5}}{10}$$

where

L = length of tile, ft.

Q_{avg} = average daily flow, mgd.

t = percolation rate, min/inch.

3.17.4.3 Determine dosing tank requirement.

3.17.4.3.1 If $L \leq 500$ no dosing tank is required.

3.17.4.3.2 If $L > 500$ the recommended volume of the dosing tank is 75 percent of the capacity of the tile.

$$VD = (0.75)\ (.65)\ (L)$$

where

VD = volume of dosing tank, gal.

L = length of tile required.

0.65 = interior capacity of 1 ft of 4" pipe in gal.

3.17.5 <u>Process Design Output Data</u>.

3.17.5.1 Average daily flow, Q_{avg}, mgd.

3.17.5.2 Soil percolation rate, t, min/inch.

3.17.5.3 Net volume of septic tank, V_N, gal.

3.17.5.4 Length of tile, L, ft.

3.17.5.5 Volume of dosing tank, VD, gal.

3.17.6 <u>Quantities Calculations</u>.

3.17.6.1 Calculate dimensions of septic tank.

3.17.6.1.1 The following assumptions are made concerning septic tank dimensions.

The depth of liquid will be 5 ft.
The distance from the top of the tank to the liquid will be 20% of the liquid depth.
The baffles at the inlet and outlet will extend from the top of the tank to a depth equal to 40% of the liquid depth.
The outside walls and top of the septic tank will be 6" reinforced concrete.
The length of the septic tank is twice the width.

3.17.6.1.2 Calculate septic tank length and width.

$$LS = (0.231)\ (V_N)^{0.5}$$

$$WS = L/2$$

where

LS = length of septic tank, ft.

WS = width of septic tank, ft.

V_N = net volume of septic tank, gal.

3.17.6.2 Calculate dimensions of dosing tank. Assume dosing tank is same depth as septic tank and is square.

$$LD = (0.163)\ (VD)^{0.5}$$

where

LD = length of dosing tank, ft.

VD = volume of dosing tank, gal.

3.17.6.3 Calculate volume of earthwork required. Assume 2 ft over-excavation on all sides of structure. Also assume the top of the septic tank will be 2 ft below natural ground.

3.17.6.3.1 Calculate volume of earthwork for septic tank.

$$VES = (LS+3)\ (WS+3)(9.0)$$

where

VES = volume of earthwork for septic tank, cu ft.

LS = length of septic tank, ft.

WS = width of septic tank, ft.

3.17.6.3.2 Calculate volume of earthwork for dosing tank.

$$VED = (9.0)(LD+3)^2$$

where

VED = volume of earthwork for dosing tank, cu ft.

LD = length of dosing tank, ft.

3.17.6.3.3 Calculate total volume of earthwork required.

$$VEW = VES + VED$$

where

VEW = total volume of earthwork required, cu ft.

VES = volume of earthwork for septic tank, cu ft.

VED = volume of earthwork for dosing tank, cu ft.

3.17.6.4 Calculate volume of reinforced concrete required for septic tank.

3.17.6.4.1 Calculate quantity of R.C. wall required for septic tank.

$$VS_{cw} = (6.5)(LS) + (8.0)(WS) + (0.5(LS)(WS) + 0.5$$

where

VS_{cw} = volume of R.C. wall required for septic tank, cu ft.

LS = length of septic tank, ft.

WS = width of septic tank, ft.

3.17.6.4.2 Calculate quantity of R.C. slab required for septic tank.

$$VS_{cs} = (0.5)(LS)(WS)$$

where

VS_{cs} = volume of R.C. slab required for septic tank, cu ft.

LS = length of septic tank, ft.

WS = width of septic tank, ft.

3.17.6.5 Calculate quantity of reinforced concrete required for dosing tank.

3.17.6.5.1 Calculate R.C. wall required for dosing tank. Assume dosing tank has one common wall with septic tank.

$$VD_{cw} = (0.5)(LD)^2 + (10)(LD) + 0.5$$

where

VD_{cw} = volume of R.C. wall required for dosing tank, cu ft.

LD = length of dosing tank, ft.

3.17.6.5.2 Calculate R.C. slab required for dosing tank.

$$VD_{cs} = (0.5)(LD)^2 + LD + 0.5$$

where

VD_{cs} = volume of R.C. slab required for dosing tank, cu ft.

LD = length of dosing tank, ft.

3.17.6.6 Calculate total volume R.C. concrete required.

$$V_{cw} = VS_{cw} + VD_{cw}$$

$$V_{cs} = VS_{cs} + VD_{cs}$$

where

V_{cw} = total volume of R.C. concrete wall required, cu ft.

V_{cs} = total volume of R.C. concrete slab required, cu ft.

VS_{cw} = volume of R.C. wall required for septic tank, cu ft.

VS_{cs} = volume of R.C. slab required for septic tank, cu ft.

VD_{cw} = volume of R.C. wall required for dosing tank, cu ft.

VD_{cs} = volume of R.C. slab required for dosing tank, cu ft.

3.17.6.7 Calculate volume of gravel for absorption field. Assume the tile will have 9" of gravel below it and 3" above it.

$$VG = (3)\ (L)$$

where

VG = volume of gravel required, cu ft.

L = length of tile, ft.

3.17.6.8 Operation and maintenance manpower. Septic tanks require virtually no operation or maintenance. The only requirement is that the sludge accumulation in the tank should be checked once a year. If the accumulation is excessive the tank should be pumped out. This probably would not be required on a yearly basis.

$$MMH = 8$$

where

MMH = maintenance manpower required, MH/yr.

3.17.6.9 Other construction cost items. The preceding calculations account for approximately 90% of the construction cost. The remaining 10% would include costs for items such as miscellaneous piping, dosing siphons, etc.

$$CF = \frac{1}{0.9} = 1.11$$

where

CF = correction factor for other construction costs.

3.17.7 <u>Quantities Calculations Output Data.</u>

3.17.7.1 Length of septic tank, LS, ft.

3.17.7.2 Width of septic tank, WS, ft.

3.17.7.3 Length of dosing tank, LD, ft.

3.17.7.4 Total volume of earthwork required, VEW, cu ft.

3.17.7.5 Total volume of R.C. wall required, V_{cw}, cu ft.

3.17.7.6 Total volume of R.C. slab required, V_{cs}, cu ft.

3.17.7.7 Volume of gravel required, VG, cu ft.

3.17.7.8 Maintenance manpower required, MMH, MH/yr.

3.17.7.9 Correction factor for other construction costs, CF.

3.17.8 <u>Unit Price Input Required.</u>

3.17.8.1 Unit price input for earthwork, UPIEX, $/cu yd.

3.17.8.2 Unit price input for R.C. wall in-place, UPICW, $/cu yd.

3.17.8.3 Unit price input for R.C. slab in-place, UPICS, \$/cu yd.

3.17.8.4 Cost of 6" Ø perforated clay pipe installed, COSTCP, \$/ft.

3.17.8.5 Unit price for gravel media installed, COGRVL, \$/cu yd.

3.17.9 Cost Calculations.

3.17.9.1 Calculate cost of earthwork.

$$\text{COSTE} = \frac{\text{VEW}}{27}\ \text{UPIEX}$$

where

COSTE = cost for earthwork, \$.

VEW = total volume of earthwork required, cu ft.

UPIEX = unit price input for earthwork, \$/cu yd.

3.17.9.2 Calculate cost of R.C. wall in-place.

$$\text{COSTCW} = \frac{V_{cw}}{27}\ \text{UPICW}$$

where

COSTCW = cost of R.C. wall in-place, \$.

V_{cw} = total volume of R.C. wall required, cu ft.

UPICW = unit price input for R.C. wall in-place, \$/cu yd.

3.17.9.3 Calculate cost of R.C. slab in-place.

$$\text{COSTCS} = \frac{V_{cs}}{27}\ \text{UPICS}$$

where

COSTCS = cost of R.C. slab in-place, \$.

V_{cs} = total volume of R.C. slab required.

UPICS = unit price input for R.C. slab in-place, \$/cu yd.

3,17.9.4 Calculate cost of pipe for absorption field.

$$COSTDP = (L)\ (0.8)\ (COSTCP)$$

where

COSTDP = cost of drain pipe for absorption field, \$.

L = length of tile, ft.

COSTCr - cost of 6" ϕ perforated pipe installed, \$/ft.

0.8 = correction factor for 4" ϕ pipe.

3.17.9.5 Cost of gravel bedding.

$$COSTGV = \frac{VG}{27}\ COGRVL$$

where

COSTGV = cost of gravel bedding, \$.

VG = volume of gravel required, cu ft.

COGRVL = unit price for gravel media installed, \$/cu yd.

3.17.9.6 Calcuiate total bare construction cost.

$$TBCC = (COSTE + COSTCW + COSTCS + COSTDP + COSTGV)\ CF$$

where

TBCC = total bare construction cost, \$.

COSTE = cost of earhtwork, \$.

COSTCW = cost of R.C. wall in-place, \$.

COSTCS = cost of R.C. slab in-place, \$.

COSTDP = cost of drain pipe for absorption field, \$.

COSTGV = cost of gravel bedding, \$.

3.17.10 Cost Calculations Output Data.

3.17.10.1 Total bare construction cost, TBCC, \$.

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3.19 SLUDGE DRYING BEDS

3.19.1 Background.

3.19.1.1 Sludge drying beds are a common method for dewatering digested sludge, especially in small plants. Drying beds are usually constructed using 4-9 inches of sand over 8-18 inches of graded gravel. The beds are usually divided into at least three sections for operational purposes. An underdrain system usually of vetrified clay pipe, spaced 9-20 ft apart, is used to remove water.

3.19.1.2 The design of sludge beds is influenced by many factors, such as weather conditions, sludge characteristics, land value, proximity of residences and use of sludge conditioning aids.

3.19.2 Input Data.

3.19.2.1 Sludge flow, gpd.

3.19.2.2 Solids in thickened sludge, %.

3.19.2.3 Solids desired, %.

3.19.3 Design Parameters.

3.19.3.1 Depth of sludge applied (8-12 in).

3.19.3.2 Days (T) in which drainage is the primary drying mechanism (1-8 days).

3.19.3.3 Solids after T days (15-25%).

3.19.3.4 Clearwater evaporation rate (available from the U. S. Weather Bureau).

3.19.3.5 Correction of evaporation rate for sludge ($\approx$ 0.75).

3.19.3.6 Average rainfall in wet month (available from the U. S. Weather Bureau).

3.19.3.7 Fraction of rainfall absorbed by the sludge ($\approx$ 0.57).

3.19.3.8 Number of sections desired.

3.19.4 Process Design Calculations.

3.19.4.1 Fill several columns with thickened/digested sludge to desired application depth. The bottom of the column should contain sand and gravel in similar depths to that expected in the bed. Check the bed daily until drainage from the bottom has essentially ceased. Record the number of days of drainage (t_d) and the percent solids in the sludge.

3.19.4.2 Calculate the required drying time.

$$T = \frac{30 \times H \times S_o}{aE - bR} \; \frac{1}{S_1} - \frac{1}{S_2} + t_d$$

where

T = total drying time, days.

H = depth to which sludge is applied, in.

S_o = initial solids, percent.

a = correction of evaporation rate for sludge, ≈ 0.75.

E = clearwater evaporaation rate sludge, in/month.

b = fraction of water absorbed by sludge, ≈ 0.57.

R = rainfall during wet month, in/month.

S_1 = solids content after t_d days, percent.

S_2 = final solids content, percent.

t_d = time during which drainage is significant, days.

3.19.4.3 Calculate the surface area.

$$SA = \frac{Q_s \times T(12)}{H(7.48)}$$

where

SA = surface area required, ft^2.

Q_s = volumetric sludge flow, gpd.

The drying bed is divided into several sections which are filled in turn so that one is always available to accept additional sludge.

$$SA = SA \frac{N}{N - 1}$$

where

N = number of sections.

The area of each section can be given by:

$$AS = \frac{SA}{N}$$

where

AS = the area of each section, ft^2.

3.19.4.4 Calculate the solids produced.

$$DTPY = Q_s S_o \left(\frac{8.34 \text{ lb/gal}}{100} \right) \quad (365 \text{ days/yr}) \quad \left(\frac{\text{ton}}{2000 \text{ lb}} \right)$$

where

DTPY = tons of dry solids per year.

3.19.4.5 Calculate weight of solids removed.

$$TPY = \frac{(DTPY)(100)}{S_2}$$

where

TPY = total tons per year removed.

S_2 = final solids content, %.

3.19.5 Process Design Output Data.

3.19.5.1 Area required, ft^2.

3.19.5.2 Depth of sludge application, in.

3.19.5.3 Number of sections.

3.19.5.4 Area of each section, ft^2.

3.19.5.5 Drying time in bed, days.

3.19.6 Quantities Calculations.

3.19.6.1 Uncovered drying beds with truck tracks for ease of sludge pickup (See Figure 2.53-1) will be utilized in this manual. A minimum of three beds will be provided for alternate sludge application, drying and cleanup. A typical width of 20 feet is utilized. This limitation is imposed by the constraint of manual sludge pickup and transfer to trucks. The length of bed is limited to less than 300 feet. This is due to the hydraulic constraint of the drainage pipes.

3.19.6.2 Selection of the number of beds. The selection of the number of drying beds is dependent on the total surface area required. The following rule will be followed in this selection.

$$N = 3 + \frac{(SA - 18{,}000)}{6000}$$

N must be an integer and should always be larger than 3.

where

N = number of beds.

SA = total drying bed surface area required, sq ft.

3.19.6.3 Dimensions of drying beds.

3.19.6.3.1 Surface area of each individual bed, SAN, sq ft.

$$SAN = \frac{SA}{N}$$

3.19.6.3.2 Length of each bed, LN, ft.

$$LN = \frac{SAN}{20}$$

where

20 = width of bed, ft.

3.19.6.4 Earthwork required for drying bed construction. It is assumed that the depth of cut would be 4 feet and the slope of cut would be 1:1. Thus the total volume of cut can be expressed by:

$$V_{ew} = [48\ N\ (LN) + 224\ N + \sqrt{560\ N^2\ (LN)^2 + 4480\ N^2\ (LN)}]\ \frac{4}{3}$$

where

V_{ew} = volume of earthwork in cu ft.

3.19.6.5 Quantity of reinforced concrete required.

3.19.6.5.1 This item includes the concrete dividing walls and the truck tracks.

3.19.6.5.2 The typical cross section of the dividing wall is shown in Figure 3.19-1. The quantity of wall would be:

$$V_{cw} = 5.06\ [40 \cdot N + (N)\ (LN) + (LN)]$$

where

V_{cw} = volume of reinforced concrete in-place, cu ft.

3.19.6.5.3 Volume of concrete required for the construction of truck tracks:

$$V_{cs} = 3\ (N)\ (LN)$$

where

V_{cs} = volume of R.C. in-place, cu ft.

3.19.6.6 Quantity of sand and gravel.

3.19.6.6.1 It is assumed that a 12-inch depth of gravel and a 9-inch depth of sand will be utilized in drying bed construction.

3.19.6.6.2 Volume of sand.

$$V_{ds} = 20\ (N)\ (LN)\ \frac{9}{12} = 15\ (N)\ (LN)$$

where

V_{ds} = volume of sand required, cu ft.

3.19.6.6.3 Volume of gravel.

$$V_{dg} = 20\ (N)\ (LN)$$

where

V_{dg} = volume of gravel, cu ft.

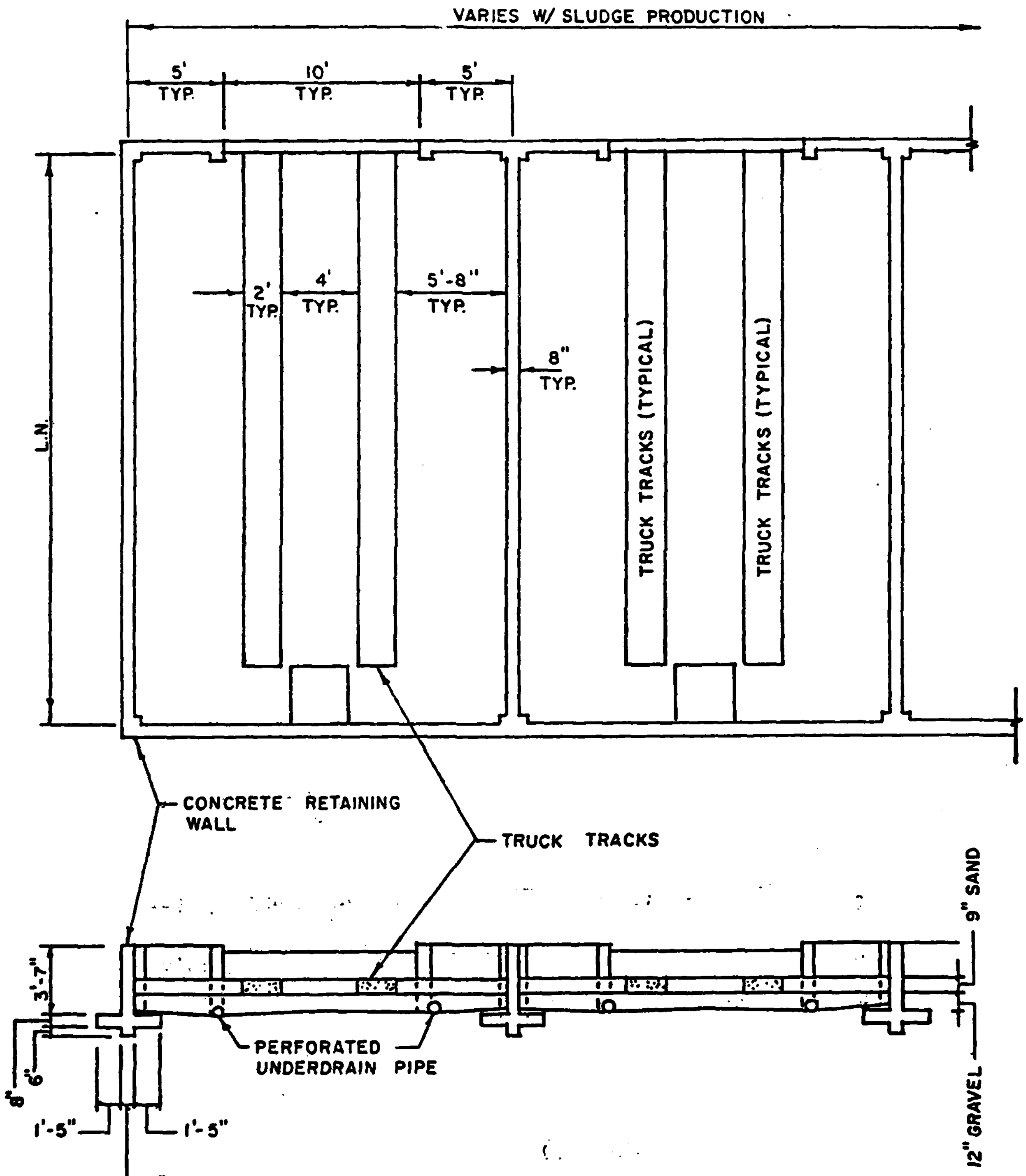

FIGURE 3.19-1. SCHEMATIC OF A TYPICAL SLUDGE DRYING BED

3.19.6.7 Perforated drain pipe required.

3.19.6.7.1 It is assumed that only the vitrified clay pipe will be used for this purpose.

3.19.6.7.2 Size of pipe to be used. Only 4-inch, 6-inch, and 8-inch diameter will be utilized. Selection of pipe size depends on length of bed. The following rule will be used:

Length of Bed, LN	Clay Pipe Diameter (DICLP) In
< 100'	4"
100 - 200	6"
> 200	8"

3.19.6.7.3 Total length of pipe, CPL.

$$CPL = 2\ (N)\ (LN)$$

3.19.6.8 Operation and maintenance manpower requirement.

3.19.6.8.1 Operation man-hours required, OMH.

When TPD < 0.09 (tons/day)

$$OMH = 360$$

When $0.09 \leq TPD \leq 0.8$ (tons/day)

$$OMH = 964\ (TPD)^{0.409}$$

When TPD > 0.8 (tons/day)

$$OMH = 1066.4\ (TPD)$$

where

TPD = sludge solids applied per day, tons/day.

OMH = operation man-hour requirement, man-hours/yr.

3.19.6.8.2 Maintenance man-hour requirement.

When TPD < 0.09 (tons/day)

$$MMH = 160$$

When $0.09 \leq TPD \leq 0.8$ (tons/day)

$$MMH = 432.8\ (TPD)^{0.409}$$

When TPD 0.8 (tons/day)

MMH = 532.8 (TPD)

where

MMH = maintenance man-hour requirement, man-hour/yr.

3.19.6.9 Operation and material costs. This item is principally related to replacement of sand removal in the bed cleaning process. It is expressed as a percent of total bare construction costs of the drying bed.

OMMP = 0.9%

where

OMMP = percent of the installed equipment costs for the operation and maintenance material costs, percent.

3.19.6.10 Other construction cost items. From the above calculations it can be seen that approximately 90 percent of the construction costs have been accounted for. Other items such as inlet piping, filtrate collection system, etc., would be 10 percent. Thus, the correction factor would be:

$$CF = \frac{1}{0.90} = 1.11$$

where

CF = correciton factor for minor construction costs.

3.19.7 Quantities Calculations Output Data.

3.19.7.1 Number of beds, N.

3.19.7.2 Surface area of each bed, SAN, sq ft.

3.19.7.3 Length of each bed, LN, ft.

3.19.7.4 Quantity of earthwork required, V_{ew}, cu ft.

3.19.7.5 Quantity of concrete wall, V_{cw}, cu ft.

3.19.7.6 Quantity of concrete slab, V_{cs}, cu ft.

3.19.7.7 Quantity of sand, V_{ds}, cu ft.

3.19.7.8 Quantity of gravel, V_{dg}, cu ft.

END

FILMED

DATE: 8-90

DTIC