

Design of Sewage Treatment Plants, Course #407

Presented by:

PDH Enterprises, LLC
PO Box 942
Morrisville, NC 27560
www.pdhsite.com

This course discusses design of sewage treatment plants and explains the procedures and standards required for preliminary, primary, secondary (biological) and advanced treatment methods for domestic sewage.

The course describes preliminary treatment units such as bar screen, comminutor, and grit chamber with illustrations and design particulars.

Primary sedimentation is explained with illustration and design features. Numerical example with solution is provided for better understanding.

Secondary (biological) treatment is explained in considerable length. Variety of secondary treatment process units such as trickling filter (all forms), activated sludge units (all modifications), and different types of stabilization ponds are described. Design standards are furnished for each unit. Numerical problems with solution are provided for easy understanding.

Disinfection theory is explained. Different types of disinfectant such as chlorine, ozone, and chlorine dioxide are described with their design and procedures for use.

Various methods of treated effluent disposal such as disposal in water bodies, disposal on land, recreational, and municipal reuse are described.

Sludge treatment steps such as thickening (mechanical and flotation), digestion (aerobic and anaerobic), conditioning, and dewatering (drying bed, vacuum filter, centrifuge) are explained with illustrations. Sludge disposal by land filling, lagooning, incineration, and ocean disposal is described.

To receive credit for this course, each student must pass an online quiz consisting of twenty-five (25) questions. A passing score is 70% or better. Completion of this course and successfully passing the quiz will qualify the student for **four (4)** hours of continuing education credit.

Course Author:

JN Ramaswamy, PhD, PE

DESIGN OF SEWAGE TREATMENT PLANTS

By

J.N. Ramaswamy, Ph.D., P.E.

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I. INTRODUCTION

Sewage treatment plants, also called domestic waste water treatment plants, are designed to convert a raw sewage into an accessible final effluent, and to dispose of the solids removed in the process. It is therefore required to determine the characteristics of the raw sewage and the required characteristics of the effluent or the required treatment, before proceeding with the design of the treatment plant. It is generally necessary to obtain the approval of a regulatory body before proceeding with construction of any sewage treatment plant. The regulations of the agency usually establish many of the basic design considerations. Many states have established classifications for various streams within their boundaries. These classifications generally establish “treatment standards” or “effluent standards” which limit the pollution material in the effluent. The “treatment standard” or the “effluent standards” are established taking into account the ability of the receiving waters to assimilate the waste and the uses to which the receiving waters are put.

Periods of design for treatment plants vary. A normal design period would require treatment units to be designed for population and sewage flows anticipated some 15 to 20 years after completion of construction. Units are designed to be readily expandable as the population increases.

Water consumption records, where available, are a good basis for determining domestic flow rates. About 70 to 80% of domestic water consumption may be expected to reach the sewer. In the absence of any better basis, many regulatory agencies accept a rate of 100 gallons per capita per day (gpcd). If commercial sewage flow is quite small in communities, the commercial flow is included as domestic flow. The design average flow rate is the average flow during some maximum significant period such as 4, 8, 12, or 16 hr, depending on circumstances.

Determination of important characteristics of sewage is essential to the proper design of treatment works. Where only population data are available, acceptable equivalents for design of treatment works are 0.20 lb of suspended solids (SS) per day per capita or 250 parts per million (ppm) and 0.17 lb of biochemical oxygen demand (BOD) per day per capita or 200 ppm. Sewage treatment processes may be classified as “preliminary”, “primary”, “secondary” or “advanced” (tertiary). The purpose of preliminary treatment is to remove deleterious materials which would damage equipment, interfere with the satisfactory operation of a process or equipment, or cause objectionable shore-line conditions. Primary treatment can usually be expected to remove 50 to 60% suspended solids and 25 to 35% BOD. Secondary treatment using conventional biological processes may remove up to 90% of suspended solids and 75 to 90% BOD. Different biological process units are deployed in secondary treatment. Tertiary or advanced treatment may be expected to remove over 95% of both BOD and SS in addition to reducing some undesirable chemicals.

Prior to disposing of the effluent, it is subjected to disinfection by injecting chlorine or ozone in to the effluent or passing ultra-violet rays in to the effluent.

The effluent disposal methods in use are: discharge to streams and rivers, land disposal to irrigate certain crops, deep well disposal, and submarine outfalls extending into the ocean.

Sludge is collected and subjected to the following treatment prior to disposal: thickening (either gravity or flotation), digestion (aerobic or anaerobic), and dewatering using sand beds or equipment such as vacuum filter or centrifuge.

Dewatered sludge is disposed of on land, processed as compost and sold to farmers, deposited in sanitary land fill, or incinerated.

II. PRELIMINARY TREATMENT

Preliminary treatment of sewage includes screening, grinding and grit removal.

II.1 Screening – The first unit operation encountered in sewage treatment plants is the filtering operation or screening. A screen is a device with openings, generally of uniform size, used to retain coarse sewage solids. The screening element may consist of parallel bars, rods or wires, grating, wire mesh, or perforated plate, and the openings may be of any shape, generally circular or rectangular slots. A screen composed of parallel bars or rods is called a rack or a bar screen. The material removed by the screening devices is known as screenings or rakings.

According to the method of cleaning, racks and screens are designated as hand cleaned or mechanically cleaned. According to the size of openings, screens are designated as coarse, or fine.

II.1.1 Racks – These are classified under coarse screen and are made of bars of steel welded in to a frame that fits across the channel with opening between bars ranging from 3 to 6 in. These are mainly used in sewage treatment plants to protect pumps, valves, pipe lines, and other appurtenances from damage or clogging by rags and large objects. The bars run vertically or at a slope varying 30° to 80° with the horizontal. Large objects are caught on the rack, carried up by traveling rakes, and scraped and collected. The approach velocity of the sewage in the raking or screening channel shall not be below a self cleaning value (1.25 ft/sec) or rise to a magnitude at which the rakings or screenings will be discharged from the bars or screens (3.0 ft/sec) or the loss of head through the rack or screen shall be such as not to back up the flow to place the entrant sewer under pressure. Figure II.1 shows a hand-cleaned and a mechanically-cleaned rack.

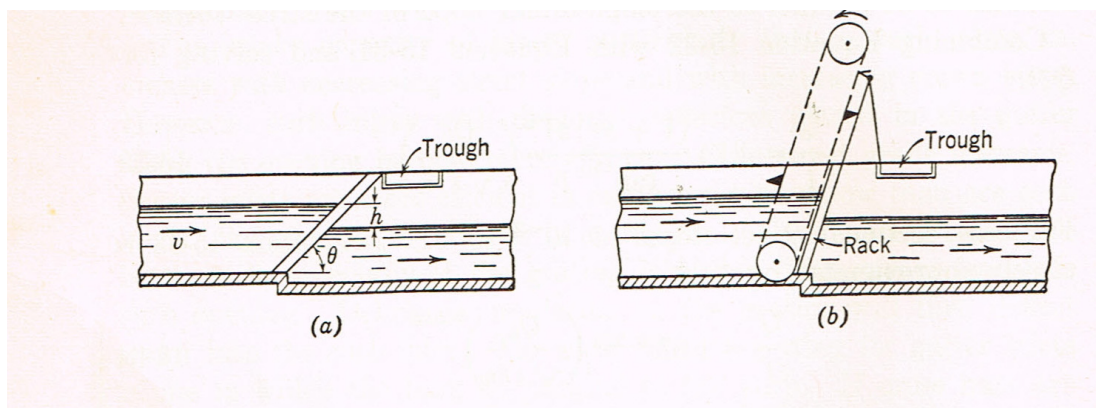


Figure II.1 (a) Hand-cleaned rack

(b) Mechanically-cleaned Rack

Hydraulic loss through bar racks is a function of bar shape and the velocity head of the flow between the bars. Velocities of 2 to 4 ft/sec through the open area have been used satisfactorily. The following equation is used to calculate the head loss.

$$h_L = \beta(w/b)^{1.33} h_v \sin \theta \dots \dots \dots \text{Eq. (II.1)}$$

where h_L = head loss, ft

β = a bar-shape factor

w = maximum cross sectional width of bars facing direction of flow, ft

b = minimum clear spacing of bars, ft

h_v = velocity head of flow approaching rack, ft

θ = angle of rack with horizontal

The head loss calculated using the above equation is applicable only if the bars are clean. Head loss increases with the degree of clogging. A minimum allowance for head loss through hand-cleaned screen is 6 in. For mechanically cleaned screens manufacturer's literature provides the allowance for head loss. Values of β for several shapes of bars are given in Table II.1 below.

Table II.1 Values of β

Bar type	β
Sharp-edged rectangular	2.42
Rectangular with semicircular upstream face	1.83
Circular	1.79
Rectangular with semicircular upstream and downstream faces	1.67

II.1.2. Fine screen – These are mechanically cleaned devices using a medium of perforated plate, woven-wire cloth, or closely placed bars through which the sewage flows. The openings are usually 3/16 in or less. One variety of fine screens used is the drum type. In this screen the filter medium is a cylinder, furnished with a mechanical means of rotation, and with self-cleaning devices. The drum is approximately 1/3 to 2/3 submerged in the sewage. The liquid passes through the screen and flows out at one end. The solids which are removed from the liquid are raised above the liquid level as the drum rotates and are removed by brushes, scrapers, and/or a backwash. The backwash may utilize water, air, or steam.

Another variety of fine screen is the disk-type screen. These screens consist of a round flat plate revolving on an axis inclined 10° to 25° from the vertical. The sewage flows through the lower two-thirds of the plate. As the plate rotates, the retained solids are brought above the liquid where brushes remove them for disposal. Commonly a motor is used to provide the rotation. Head loss through fine

screens may be obtained from manufacturers' rating tables or may be calculated by means of the common orifice formula:

$$h_L = (1/2g) (Q/CA)^2 \dots\dots\dots \text{Eq. (II.2)}$$

Where C = coefficient of discharge

Q = discharge through screen, cfs

A = effective submerged open area, ft²

g = acceleration due to gravity, ft/sec²

h_L = head loss, ft

Figure II.2 shows a brush-cleaned disk screen and a brush-cleaned drum screen.

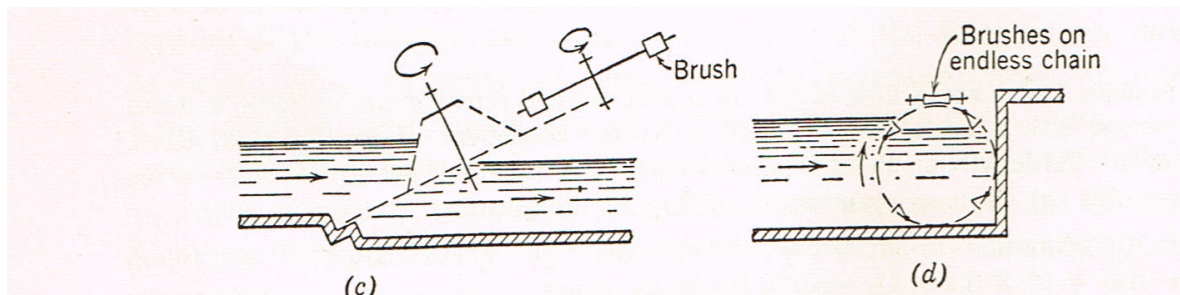


Figure II.2 © Brush-cleaned disk screen

(d) Brush –cleaned drum screen

II.1.3. Comminuting devices – A comminuting device is a mechanically cleaned screen which incorporates a cutting mechanism that cuts the retained material without removal from the sewage flow. This tends to reduce odors, flies, and unsightliness often found around sewage screenings handled by other means. A comminuting device has a submerged revolving drum with openings varying from ¼ to 3/8 in. Coarse material is cut by cutting teeth and shear bars at the revolving drum which passes through a stationary cutting comb. The comminuted solids then pass, with sewage liquor, out of the bottom opening and back into the down stream channel. This requires a special volute-shaped basin to give proper hydraulic conditions for satisfactory operation. The basin shape makes its installation more expensive. A comminuting device is often used in locations where the removal of screenings would be difficult such as in a very deep pit. Figure II.3 shows plan and cross sectional views of a comminuter.

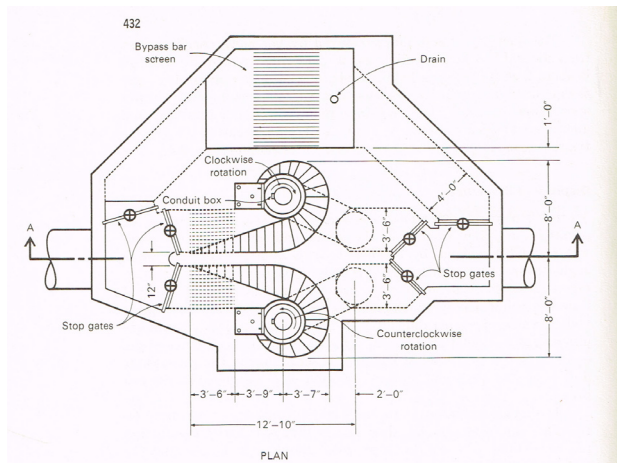


Figure II.3 Plan and cross sectional views of a comminuter

II.1.4. Disposal of screenings – Large variation is reported in the volume of screenings removed per million gallons of sewage. The factors affecting the quantity of screenings are as follows:

1. Clear opening between bars
2. Percentage of combined sewers in the tributary system
3. Character of industrial waste treated, and
4. Habits of tributary population

Incineration has been found to be a satisfactory means of screenings disposal.

Screenings grinders have been used for disposal of screenings. The material is reduced in size and returned to the raw sewage. The grinders are located near the source of screenings to be processed. Grinders used are the hammer-mill type or the disintegrator type. A comminuting device is not a substitute for a grinder. Screenings from a grinder are usually disposed of as raw sludge.

Another method of disposing of screenings is by burial. If this method is chosen, suitable and sufficient area must be available.

II.1.5. **Grit removal** – Minute pieces of mineral matter like sand, and gravel, and materials that are not of mineral origin like coffee grounds, seeds, and similar material constitute grit. Grit in sewage has two characteristics: (1) They are non-putrescible and (2) they have subsiding velocities substantially greater than those of organic putrescible solids.

Grit chambers are located downstream of screen chambers. The purpose of a grit chamber is three fold: (1) the protection of moving mechanical equipment from abrasion and accompanying abnormal wear, (2) the reduction of pipe clogging caused by deposition of grit particles or heavy sludge in pipes and channels, particularly at changes in direction of conduits, and (3) reduction of frequency of digester and settling tank cleaning required as a result of excessive accumulation of grit in these units.

There are two types of grit chambers: horizontal flow and aerated. In the horizontal flow type, the flow passes through the chamber in a horizontal direction. A constant velocity of flow through the grit chamber must be maintained at 1 ft/sec for all depths of flow in order to prevent settling of organic solids. This is accompanied by means of providing a sutro weir or a proportional flow weir.

Figure II.4 shows cross section of the two weirs.

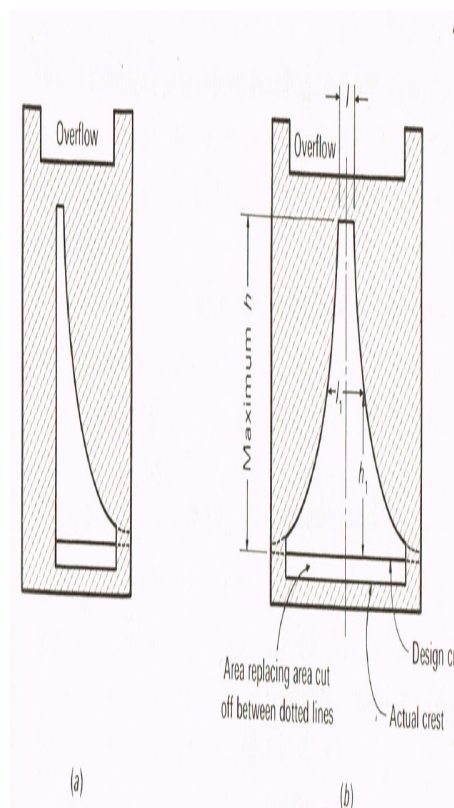


Figure II.4 Cross section of (a) sutro weir (b) proportional flow weir

The aerated type consists of a spiral-flow aeration tank, the spiral velocity being controlled by the dimensions and the quantity of air supplied to the unit. The detention provided is 3 minutes at the maximum flow rate.

The grit solids are raked by a rotating mechanism to a sump at the side of the tank, from which they are moved by a reciprocating rake mechanism. The quantities of grit vary from one location to another depending on the type of sewerage system, the characteristics of the drainage area, the condition of the sewers, the frequency of street sanding, the type of industrial wastes, the number of garbage grinders served, and the proximity and use of sandy bathing beaches. There is a wide range in the quantity of grit varying from $1/3 \text{ ft}^3$ to 24 ft^3 per million gallon of sewage treated. Because of the wide variation, a factor of safety must be used in calculations concerning the actual storage, handling, or disposal of the grit.

Common method of grit disposal is as fill, covered if necessary to prevent objectionable conditions. Grit also is incinerated with sludge. In coastal cities grit and screenings are barged to sea and dumped. Generally the grit must be washed before removal.

II.1.6. Pretreatment – Pretreatment is used to remove material such as grease and scum, from sewage prior to primary sedimentation to improve treatability. Pretreatment may include skimming, grease traps, pre-aeration and flotation.

A skimming tank is a chamber so arranged that floating matter rises and remains on the surface until removed while the liquid flows out continuously through deep outlets. This may be accomplished in a separate tank or combined with primary sedimentation. The object is to separate the lighter floating substances from sewage. The material removed includes oil, grease, soap, pieces of cork, and vegetable debris and fruit skins.

Grease traps are small skimming tanks. They are situated close to the source of grease, which may be an industry, a house sewer, or a small treatment plant. The inlet is situated just below the surface and the outlet at the bottom. Detention times of 10 to 30 min are used. They must be cleaned periodically.

Pre-aeration of sewage prior to primary sedimentation, if practiced, is classified as pretreatment. The objective of pre aerating sewage is to improve treatability and to control odor. Detention times of pre-aeration tanks range from 10 to 45 min. Tank depths are generally 15 ft and air requirements range from 0.1 to $0.4 \text{ ft}^3/\text{gal}$ of sewage.

III. PRIMARY TREATMENT

Primary treatment consists of settling the sewage in a sedimentation tank. Whenever a liquid containing solids in suspension is placed in a relatively quiescent state, those solids having a higher specific gravity than the liquid will tend to settle, and those with lower specific gravity will tend to rise. These principles are utilized in the design of sedimentation tanks. The objective of treatment by sedimentation is to reduce the suspended solids content by removing readily settleable solids and floating material.

Efficiently designed and operated primary sedimentation tanks should remove from 50 to 65% of SS and 25 to 40% of BOD. Sedimentation tanks are normally designed on the basis of a surface-loading rate at the average rate of flow, expressed as gallons/day/ft² of horizontal area. The effect of surface-loading rate and detention time on SS removal varies widely depending on the character of the sewage, proportion of settleable solids, concentration of solids, and other factors. When the area of the tank has been established, the detention period in the tank is governed by water depth.

Surface settling rates not followed by secondary treatment shall not exceed 600 gallons per day per square foot (gpd/ft²) for design flow of 1 mgd or less. Higher rates may be permitted for larger plants. Normally, primary detention tanks are designed to provide 90 to 150 min of detention based on the average rate of sewage flow. Weir loadings should not exceed 10,000 gallons/linear ft/day for plants designed for average flows of 1 MGD or less. For plants designed for higher flows, the weir loading rate can be increased up to a maximum of 15,000 gallons/linear ft/day. Weir rates have been found to have less effect on efficiencies of removal than over flow rates. A minimum water depth of 7 ft is recommended.

III.1. Tank type, size and shape – Almost all sedimentation tanks are designed as rectangular or circular tanks with mechanical cleaning mechanism. The selection of the shape is governed by the size of the installation, by rules and regulations of permitting authorities, by local site conditions and the estimate of cost. Two or more tanks should be provided in order that the process may remain in operation while one tank is out of service for maintenance and repair work.

III.1.1. Rectangular tanks - The length of rectangular tanks is restricted to 300 ft. Tank widths may not be more than 80 ft, but it should be divided in to 4 bays so that the cleaning mechanism can be installed in a 20-foot width bay. A rectangular tank is shown in Figure III.1.

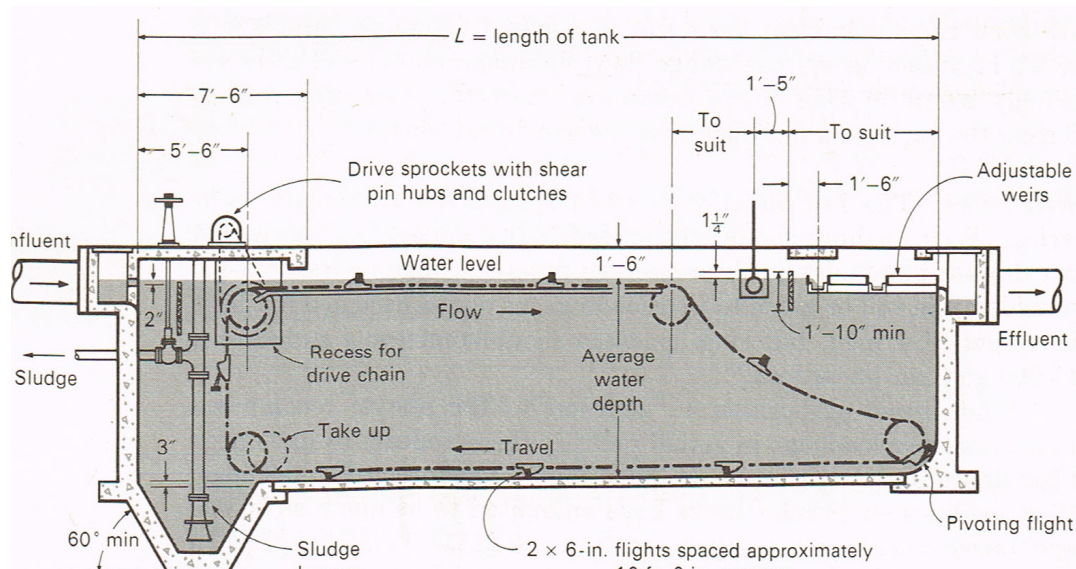


Figure III.1 Rectangular sedimentation tank

Attached to the chain at intervals of 10 ft, are 2 in thick cross pieces of wood, or flights, 6 to 8 in deep, extending the full width of the tank or bay. Linear conveyor speeds of 2 to 4 ft/min are common. The solids settling in the tank are scraped to sludge hoppers in small tanks and to transverse troughs in large tanks. These, in turn, are equipped with collecting mechanisms (cross collectors), of the same type as the longitudinal collectors, which convey solids to one or more sludge hoppers. Screw conveyors may also be used instead of cross collectors. Where cross collectors are not provided, multiple hoppers must be installed. If a common withdrawal line is used, provision is made to isolate and control the withdrawal from each hopper individually. It is desirable to locate the sludge pumping facilities close to the hoppers.

Rectangular tanks are used where ground area is at a premium. They are also used where tank roofs or covers are required.

The inlet arrangement is an important element in the design of rectangular tanks. Influent channels must be provided across the inlet end. With multiple units, the flow is distributed to each unit as uniformly as possible to obtain maximum efficiency. One effective method is the use of distribution boxes or chambers ahead of the sedimentation units with gates or orifices to adjust the flow between the units. Baffle boards in front of the inlets are used to distribute sewage flows laterally and vertically and to prevent short circuiting. Baffles are installed approximately 2 to 3 ft in front of the inlets and submerged 18 to 24 in.

Outlet structures include effluent channels and weirs located near the effluent end of the tank. Effluent weirs are adjustable for leveling and sufficiently long to avoid high heads which result in updraft currents. The crest is frequently provided with 90° V notches to provide uniform distribution at low flows.

Scum is usually collected at the effluent end of rectangular tanks. The scum removal method consists of a chain-and-flight type of collector that collects the scum at one side of the tank and scraps it up a short distance for deposit in scum hoppers, whence it is usually disposed of with the sludge produced at the plant.

III.1.2. **Circular tanks** – The diameter of round tanks varies from 10 to 180 ft with no single factor influencing the selection other than the size of the plant. The side wall depth varies from 7 to 14 ft. Floors are deepest at the center and slope radially upwards to the tank walls at a rate of 1 in per ft. The slope facilitates sludge withdrawal and drainage of the tank.

In one type of circular tanks, the sewage is carried to the center of the tank in a pipe suspended from a bridge or encased in concrete beneath the tank floor. At the center of the tank, sewage enters a circular well designed to distribute the flow equally in all directions. The removal mechanism moves continuously at a peripheral speed of 5 to 8 ft/min and may have two or four arms equipped with scrapers. The arms also support blades for scum removal. In the second type, a suspended circular aluminum baffle at a short distance from the tank wall forms an annular space into which the sewage is distributed in a tangential direction. The sewage flows spirally around the tank and underneath the baffle, the clarified liquid being skimmed off over weirs on both sides of a centrally located weir trough. Grease and scum are confined to the surface of the annular space. Intervals of pumping the sludge vary from once in 30 min to once in 12 hours depending upon the volume to be pumped and the plant operating schedules.

The volume of sludge produced depends upon:

1. Characteristics of the raw sewage
2. Period of sedimentation
3. Conditions of the deposited solids, and
4. Period between sludge-removal operations.

Example III.1 shows a typical design of a primary sedimentation tank.

Example III.1

Design a primary sedimentation tank given the following data: Sewage flow = 5 mgd, surface overflow rate = 600 gpd/ft², depth of tank = 10 ft, removal efficiency = 60%, SS in raw sewage = 200 mg/L, specific gravity of the sludge = 1.03, and moisture content of sludge = 95%, weir loading rate = 15,000 gpd/ft. Sludge is pumped out of the hoppers 3 times a day for 30 minutes duration each time.

Solution

Surface area = $5,000,000/600 = 8,333$ or use 8,340 ft²

Total volume = $10 \times 8,340 = 83,400$ ft³

Provide 4 rectangular tanks each 120 ft long and 20 ft wide which gives a length to width ratio of 6:1 and a total volume of 96,000 ft³.

Two 75-ft diameter circular tanks with a total volume of 88,313 ft³ also will be suitable.

Design flow/tank = 5,000,000/4 = 1,250,000 gpd

Weir length/tank = 1,250,000/15,000 = 83 linear ft

Weight of dry solids removed/million gallons = 200 x 8.34 x 1 x (60/100) = 1,000 lb

Volume of sludge/million gallon of sewage = 1,000/{8.34 x 1.03 x (5/100)} = 2,330 gallons

Volume of sludge/5 mgd of sewage = 2,330 x 5 = 11,650 gpd

Sludge volume pumped each time = 11,650/3 = 3,883 or use 3,890 gallons

Adding 10% for scum, volume to be pumped = 3,890 x 1.1 = 4,668 gallons

Pumping rate = 4,668/30 = 155.6 or use 160 gpm

Check for detention time

Using rectangular tanks: Flow = 5,000,000 gallons/day = 208,333 gal/hr

Volume of tank = 96,000 ft³ = 718,000 gal

Detention time = 718,080/208,333 = 3.45 hrs

Using circular tanks: Flow as before = 208,333 gal/hr

Volume of tank = 660,581; Detention time = 660,581/208,333 = 3.17 hr

Square tanks are also used but they are fewer in number. The design features are same as for circular tanks.

Figure III.2 shows a typical circular sedimentation tank.

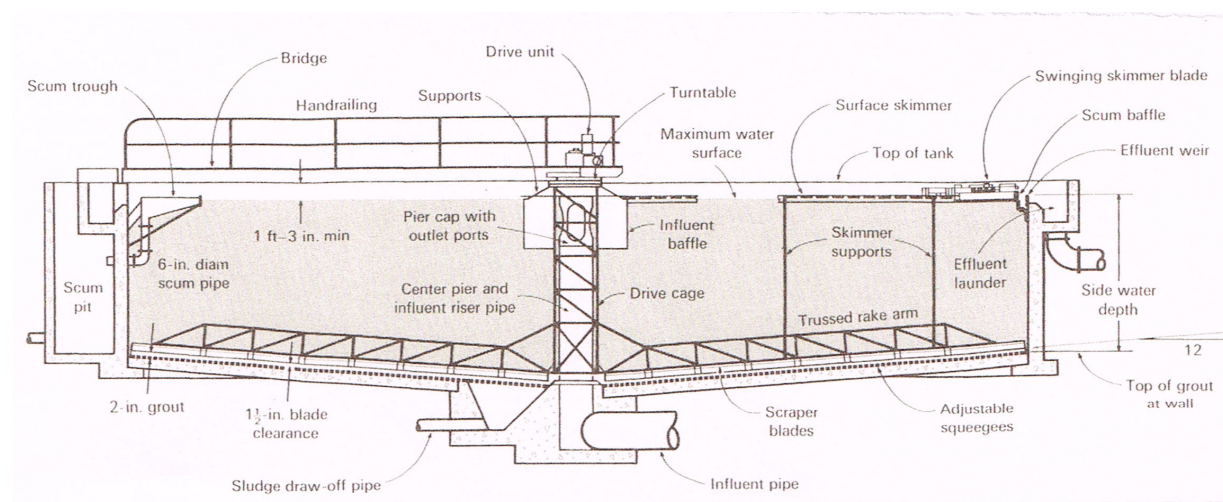


Figure III.2 Typical circular sedimentation tank

IV. SECONDARY TREATMENT

Secondary treatment of sewage involves biological processes that convert the finely divided and dissolved organic matter into flocculent settleable solids that can be removed in sedimentation tanks.

The common biological processes are:

1. Trickling filter
2. Activated sludge
3. Aerated lagoons
4. Stabilization ponds

IV.1. Trickling filter – A trickling filter, considered as attached growth system, consists of a bed with highly permeable media to which microorganisms are attached and through which sewage is percolated. The filter media usually consists of, rocks, varying in size from 1 to 4 in. in diameter. The depth of rock varies with each particular design, usually from 3 to 8 ft; an average depth is 6 ft. Trickling filters employing a plastic media have been built with depths of 30 to 40 ft. The filter bed is usually circular, and the sewage is distributed over the top of the bed by a rotary distributor. Each filter has an under drain system for collecting the treated effluent and any biological solids that have become detached from the media. The under drain system has two functions: one as a collecting unit for the effluent and the other as a porous structure through air can circulate. Figure IV.1 shows a cutaway view of a trickling filter.

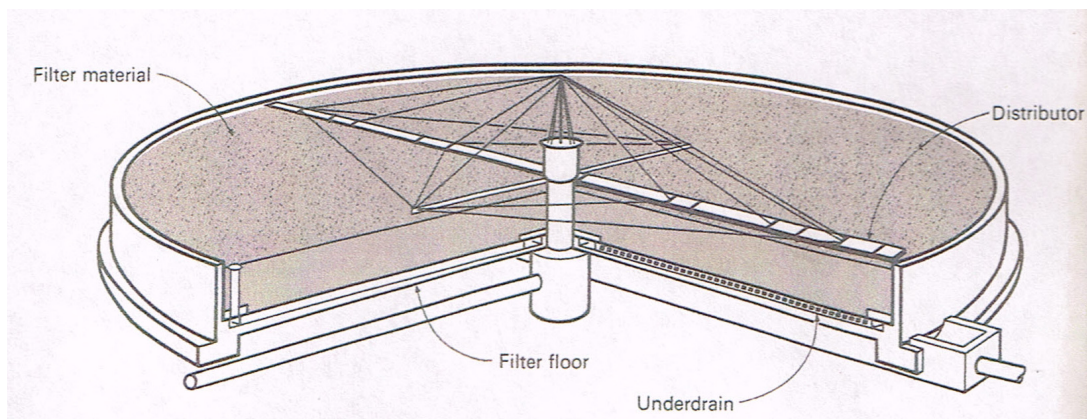


Figure IV.1 Cutaway view of a trickling filter

The trickling process depends on biochemical oxidation of complex organic matter in the sewage. Soon after a filter is placed in operation, the surface of the media becomes coated with zooglea , a viscous jelly-like substance containing bacteria and other biota. Under favorable conditions the zooglea absorbs

and utilize suspended, colloidal, and dissolved organic matter from the sewage which passes in a relatively thin film over its surface. Eventually population equilibrium is reached. As biota die, they, together with the more or less partly decomposed organic matter, are discharged from the filter. This discharge is termed *sloughing*. The sloughing may occur periodically or continuously. Secondary settling is provided to retain the settleable solids sloughed from the filter.

Trickling filters are expected to remove 70 to 80 % of BOD. They predominate in smaller plants. They have the ability to recover from shock loads and to provide good performance with a minimum of skilled technical supervision. They are classified by hydraulic or organic loading as high rate and low rate. The hydraulic loading is the total volume of liquid, including recirculation, per day per square unit of the filter area. The general practice is to use million gallons per acre per day (mgad). Organic loading is the pounds of 5-day, 20° C, BOD per day per cubic unit of the filter media. The Ten-State Standards has sponsored pounds per day per 1,000 ft³. The range of loadings encountered and other operational characteristics for the high rate and low rate filters are shown in Table IV.1

Table IV.1 Operational characteristics of high rate and low rate trickling filters

Factor	Low-rate filter	High-rate filter
Hydraulic loading, mgad	1 to 4	10 to 40
Organic loading, lb BOD ₅ /acre-ft-day	300 to 1,000	1,000 to 5,000
Depth, ft	6 to 10	3 to 8
Recirculation	None	1:1 to 4:1
Rock volume	5 to 10 times	1
Power requirements	None	10 to 50 hp/mg
Filter flies	Many	Few, larvae are washed away
Sloughing	Intermittent	Continuous
Operation	Simple	Some skill
Dosing interval	Not more than 5 min (generally intermittent)	Not more than 15 sec (continuous)
Effluent	Fully nitrified	Nitrification at low loadings

A low rate filter is also called a standard-rate or a conventional rate filter and is relatively simple device and is highly dependable, producing a consistent effluent quality with varying influent strength. A large population of nitrifying bacteria is prevalent. Head loss through the filter may be 5 to 10 ft. Odors are a common problem, especially if the sewage is stale or septic. Nuisance causing filter flies (*Psychoda*) may breed in the filters unless control measures are employed.

Filter effluent or final effluent is re-circulated in high rate filters resulting in higher organic loadings. Flow diagrams for various high rate trickling filter configurations are shown in Figure IV.2.

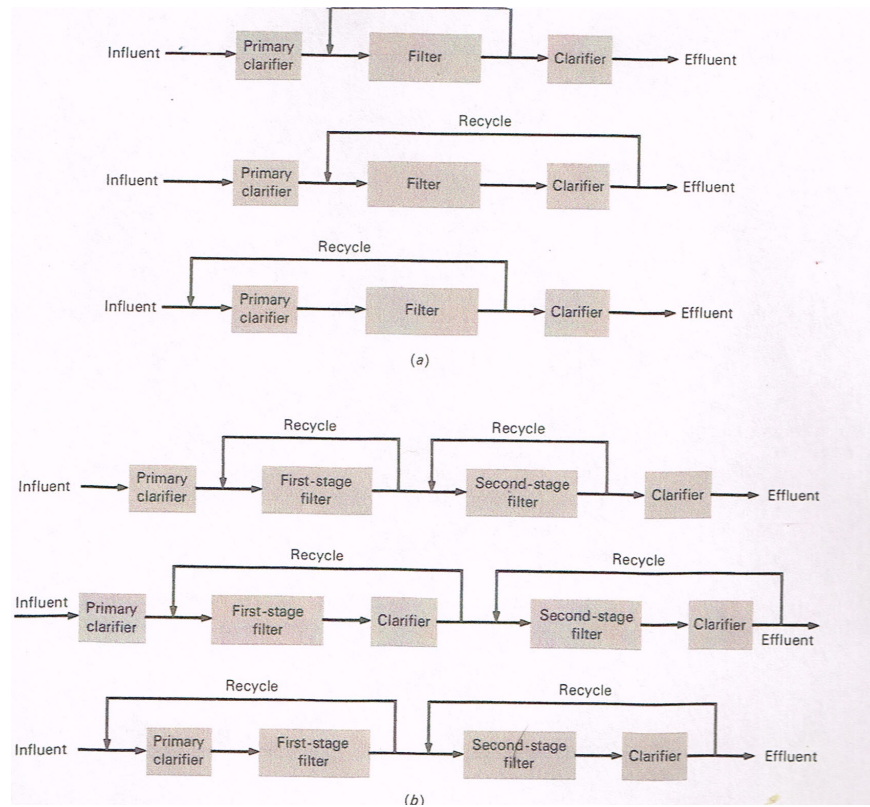


Figure IV.2 High rate trickling filter flow sheets with various recirculation patterns

(a) single-stage filters (b) two-stage filters

Recirculation of filter effluent around the filter results in the return of viable organisms and improves treatment efficiency. Recirculation also aids in preventing ponding in the filter and in reducing the nuisance due to odors and psychoda flies.

Equations are available to predict the BOD removals in trickling filters. Most commonly used equation is by The National Research Council, an empirical formula, which is shown below for the first stage filter:

$$E_1 = 1 / \{1 + 0.0085 (W/VF)^{0.5}\} \dots\dots\dots \text{Eq. (IV.1)}$$

where E_1 = fractional efficiency of BOD removal for process, including recirculation and sedimentation

W = BOD loading to filter, lb/day

V = volume of filter media, acre-ft.

F = recirculation factor

The recirculation factor is calculated by means of the following formula:

$$F = (1 + R) / \{1 + (R/10)\}^2 \dots\dots\dots \text{Eq. (IV.2)}$$

where R = recirculation ratio Q_r/Q

The recirculation factor represents the average number of passes of the influent organic matter through the filter

Example IV.1 illustrates the use of the NRC formulas in the design of trickling filters.

Example IV.1

A town is considering the use of a trickling filter for treatment of its sewage. A two stage filter is contemplated. The influent flow is 2 mgd with a settled BOD content of 200 mg/liter. The desired BOD in the effluent quality is 30 mg/liter. If the filter depths are 6 ft and the recirculation ratio is 4:1, what are the filter diameters assuming the fractional BOD removal efficiencies are the same in both the filters.

Solution

1. Compute E_1 and E_2 , the BOD removal efficiencies

$$\text{Overall efficiency} = (200-30)/200 = 85 \%$$

$$E_1 + E_2(1 - E_1) = 0.85$$

$$E_1 = E_2 = 0.615$$

2. Compute the recirculation factor

$$F = (1 + R) / \{1 + (R/10)\}^2 = (1 + 4) / 1.4^2 = 5 / 1.96 = 2.55$$

3. Compute the BOD loading for the first filter

$$W = 200 \times 8.34 \times 2 = 3,334 \text{ lb/day}$$

4. Compute the volume for the first stage filter

$$E_1 = 1 / \{1 + 0.0085(W/VF)^{0.5}\}$$

$$0.615 = 1 / \{1 + 0.0085 (3,334/2.55V)^{0.5}\}$$

$$V = 0.1 \text{ acre-ft}$$

5. Compute the diameter of the first filter

$$\text{Area} = 0.1/6 = 0.017 \text{ acres} = 726 \text{ ft}^2$$

$$\text{Diameter} = \{(726 \times 4)/3.14\}^{0.5} = 30 \text{ ft}$$

6. Compute the BOD loading for the second filter

$$W' = (1 - E_1) W = 0.385 \times 3,334 = 1,284 \text{ lb/day}$$

$$E_2 = 1 / [1 + \{0.0085/(1 - E_1)\} \times (W'/VF)^{0.5}]$$

$$0.615 = 1/\{1 + [0.0085/(1 - 0.615)] \times (1284/2.55V)^{0.5}\}$$

$V = 1.39$ acre-ft., Area = $1.39/6 = 0.23$ acres or $10,020 \text{ ft}^2$; diameter = 113 ft.

IV.1.1. **Physical facilities** – Factors that must be considered in the design of trickling filters include:

(1) the type and dosing characteristic of the distribution system, (2) the type of filter media to be used, (3) the configuration of the under drainage system, (4) provision for adequate ventilation, and (5) the design of the adequate settling tanks.

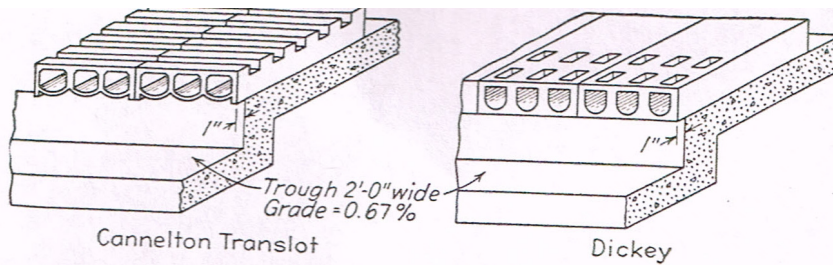
IV.1.1.1. **Distribution systems** – The common arrangement of distribution system is to provide two or more of rotary arms. They are mounted on a pivot in the center of the filter and revolve in a horizontal plane. The arms are hollow and contain nozzles through which the sewage is discharged over the filter bed. The distributor assembly is driven by the dynamic reaction of the sewage discharging from the nozzles or by an electric motor. The speed of revolution normally is 1 revolution in 10 minutes or less. Clearance of 6 to 9 in should be allowed between the bottom of the distributor arm and the top of the bed. Nozzles are spaced unevenly so that greater flow per unit of length is achieved at the periphery than at the center. The head loss through the distributor will be in the range of 2 to 5 ft.

Dosing tanks providing intermittent operation or recirculation by pumping may be employed to ensure that the minimum flow will be adequate to rotate the distributor and discharge the sewage from all nozzles.

Fixed-nozzle distribution system is also in use. It consists of a series of spray nozzles located at the points of equilateral triangles covering the filter bed. A system of pipes placed in the filter distributes the sewage uniformly to the nozzles. Special nozzles having a flat spray pattern are used.

IV.1.1.2. **Filter media** – The ideal filter media should have high surface area per unit of volume, should be low in cost, has a high durability, and does not clog easily. The most suitable material is crushed rock or gravel graded to a uniform size of 1 to 3 in. Other materials such as slag, cinders, or hard coal have also been used. Stones less than 1 in diameter must be avoided as they do not provide sufficient pore space between the stones for free flow of sewage and sloughed solids. Plugging of the media and *ponding* inside the filter will occur.

IV.1.1.3. **Under drains** – Under drains are part of the collection in a trickling filter. The collection system consists of filter floor, collection channel, and under drains. The under drains are specially designed vitrified-clay blocks with slotted tops that admit the sewage and support the media. The under drains are laid directly on the filter floor, which are sloped to the collection channel at a 1 to 2 percent gradient. Under drains may be open at both ends to facilitate easy inspection and flushing in the event of clogging. They also ventilate the floor, providing air for microorganisms that live in the filter slime. Figure IV.3 provides an under drain system.



(From G. S. Russell, *Sewage and Ind. Wastes Eng.*, May, 1950, p. 258.)

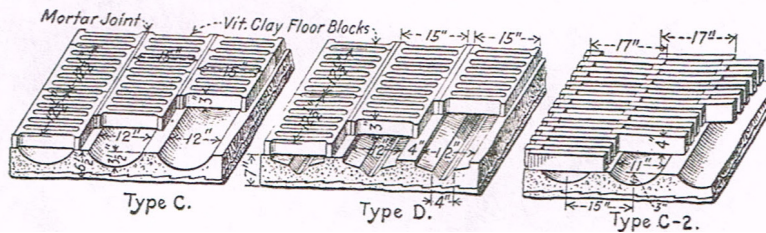


Figure IV.3 Under drain blocks for trickling filters

IV.1.1.4. **Ventilation** – Natural ventilation occurs by gravity within the filter and it is considered adequate if the trickling filter is properly designed, constructed, and operated. Forced ventilation is practiced at a rate of 1 ft^3 per ft^2 of filter area in deep or heavily loaded filters. During periods of extremely cold temperatures the air flow must be restricted to 0.1 ft^3 per ft^2 in order to prevent freezing of the filter.

Filters should be designed such that the entire media can be flooded with sewage and then drained without causing any overflows. Flooding is an effective method for flushing a filter to correct ponding and to control filter fly larvae.

IV.2. **Activated sludge process** - Activated sludge is defined as sludge floc produced in a raw or settled sewage by the growth of zoogeleal bacteria and other organisms in the presence of dissolved oxygen, and accumulated in sufficient concentration by returning floc previously formed. This is considered as a disperse growth system.

Activated sludge process is defined as a biological sewage treatment process in which a mixture of sewage and activated sludge is agitated and aerated. The activated sludge is subsequently separated from the treated sewage (mixed liquor) by sedimentation, and wasted or returned to the process as needed. The treated sewage overflows the weir of the settling tank in which separation from the sludge takes place.

Activated sludge flocs are composed of a synthetic gelatinous matrix in which filamentous and unicellular bacteria are imbedded, and on which protozoa and some metazoan crawl and feed. Activated sludge differs from other sludge in appearance, physical characteristics, and biological composition. Good activated sludge has a distinctive musty, earthy odor while in circulation in the

aeration basin. It is a light brown, flocculant precipitate that settles rapidly in its mother liquor, leaving a supernatant liquid that is clear, colorless, odorless and, sparkling.

The advantages of this process are producing a clear, sparkling and, non-putrescible effluent, freedom from offensive odors during operation, removing more than 90 % of BOD and SS, relatively low installation cost, some commercial value in the sludge and, the requirement of hydraulic head and surface area for the plant is less. The disadvantages include uncertainty concerning the results to be expected under all conditions, sensitivity to changes in the quality of the influent, high cost of operation, the necessity for constant skilled attendance, and difficulty in dewatering and disposing of the large volume of sludge proposed.

The effluent from the activated sludge process is normally clear, odorless, sparkling, high in dissolved oxygen, and low in BOD. It can be expected, in general, that the effluent will contain from 10 to 20 mg/l of BOD and SS.

The conventional activated sludge process together with the six modifications are listed below and they are described in detail: Design parameters for these processes are furnished in Table IV.2

1. Conventional activated sludge process
2. Complete –mix activated process
3. Tapered aeration activated sludge process
4. Step –aeration activated sludge process
5. Modified-aeration activated sludge process
6. Contact stabilization activated sludge process
7. Extended aeration activated sludge process

Table IV.2 Design parameters for activated sludge processes

Process	<u>BOD loading</u>				
	lb BOD/1000 ft ³ per day	lb BOD/day/ lb MLSS	Sludge age days	Aeration period hours	Return sludge percent
Conventional	20 – 40	0.2 -0.4	5 – 15	4 – 8	25 – 50
Complete mix	50 – 120	0.2 –0.6	5 – 15	3 – 5	25 – 100
Tapered aeration	30 -40	0.2 –0.5	5 – 15	6 – 7.5	30
Step aeration	30 -50	0.2 –0.5	5 – 15	5 – 7	50
Modified aeration	75 – 100	1.5 – 5.0	0.2 – 0.5	1.5 – 3	5 – 15
Contact stabilization	30 – 50	0.2 – 0.5	5 – 15	6 – 9	100

Extended aeration	10 – 30	0.6 – 1.5	5 – 10	20 – 30	100
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IV.2.1. **Conventional activated sludge process** – This is the earliest activated sludge system. The flow diagram for this process is shown in Figure IV.4

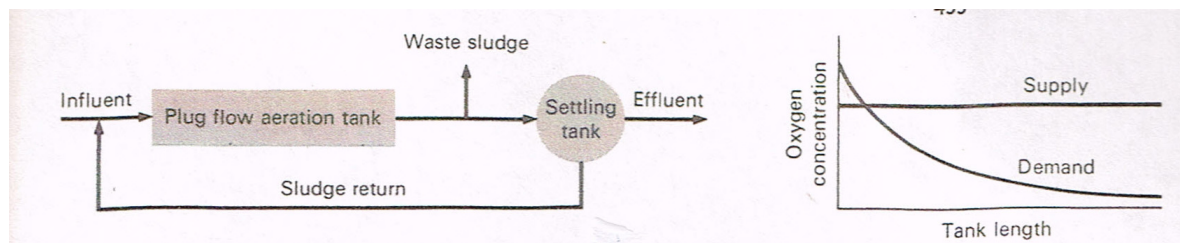


Figure IV.4 Flow diagram plus oxygen demand and supply for conventional activated sludge process

The aeration basin is a long rectangular tank with air diffusers on one side of the tank bottom to provide aeration and mixing. Settled sewage and return activated sludge enter the head of the tank, get aerated for about 6 hours and flow down its length in a spiral flow pattern. Constant aeration is provided by diffused air or mechanical means. During this period, adsorption, flocculation, and oxidation of the organic matter take place. The mixed liquor is settled in the activated-sludge settling tank, and sludge is returned at a rate of approximately 25 to 50 percent of the influent flow rate. The above process is illustrated in Example IV.2.

Example IV.2

Data: Volume of aeration tank = 120,000 ft³ or 0.898 mg

Settled sewage flow = 3.67 mgd

Return sludge flow = 1.27 mgd

Waste sludge flow = 18,900 gpd or 0.0189 mgd

MLSS in aeration tank = 2,350 mg/l

SS in waste sludge = 11,000 mg/l

Influent sewage BOD = 128 mg/l

Effluent BOD = 22 mg/l

Effluent SS = 26 mg/l

Using the above data calculate the loading and operational parameters.

Solution

BOD load = $3.67 \times 128 \times 8.34 = 3,920 \text{ lb/day}$

MLSS in aeration tank = $0.898 \times 2,350 \times 8.34 = 17,600 \text{ lb}$

BOD loading = $3,920/120 = 32.7 \text{ lb/day/1000 ft}^3$

BOD loading = $3,920/17,600 = 0.22 \text{ lb/day/lb of MLSS}$

Sludge age = $(2,350 \times 0.898)/(26 \times 3.67 + 11,000 \times 0.0189) = 7 \text{ days}$

Aeration period = $(0.898 \times 24)/3.67 = 5.9 \text{ hr}$

Return sludge rate = $(1.27 \times 100)/3.67 = 35 \%$

BOD removal = $\{(128 - 22) \times 100\}/128 = 83 \%$

Sludge production = $(0.0189 \times 11,000 \times 8.34)/3,920 = 0.44 \text{ lb SS wasted/lb BOD applied}$

IV.2.2. Complete-mix activated process – Process flow diagram for this process is shown in Figure IV.5

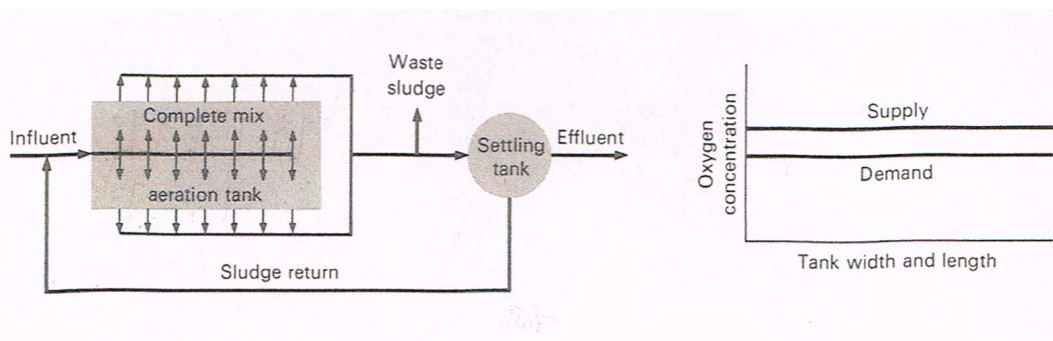


Figure IV.5 Flow diagram plus oxygen demand and supply for complete mix activated sludge process

The settled sewage influent and the return sludge flow are introduced at several points in the aeration tank from a central channel. The mixed liquor is aerated as it passes from the central channel to the effluent channels at both sides of the aeration tank. The aeration tank effluent is collected and settled in the activated sludge settling tank. The organic load on the aeration tank and the oxygen demand are uniform from one end to the other. As the mixed liquor passes across the aeration tank from the influent ports to the effluent channel, it is completely mixed by diffused or mechanical aeration.

IV.2.3. Tapered aeration activated sludge process – The objective of tapered aeration is to match the quantity of air supplied to the demand exerted by the microorganisms, as the liquor traverses the aeration tank. Thus only the arrangement of the diffusers and the amount of air consumed are affected in this process. At the inlet of the aeration tank where fresh settled sewage and return activated sludge first come in contact, the oxygen demand is very high. The diffusers are spaced close together to achieve a high oxygenation rate and thus satisfy the demand. As the mixed liquor traverses the tank, synthesis of new cells occurs, increasing the number of microorganisms and decreasing the concentration of available food. This results in a lower food/microorganism (U) ratio and a lowering of the oxygen demand. The spacing of diffusers is increased toward the tank outlet, to reduce the oxygenation rate. This results in two advantages: lowering of aeration cost and avoidance of over aeration creating inhibition of growth of nitrifying organisms.

IV.2.4. Step aeration activated sludge process – In this process, the settled sewage is introduced at several points in the aeration tank to equalize the U ratio, thus lowering the peak oxygen demand. A typical flow sheet for this process is shown in Figure IV.6

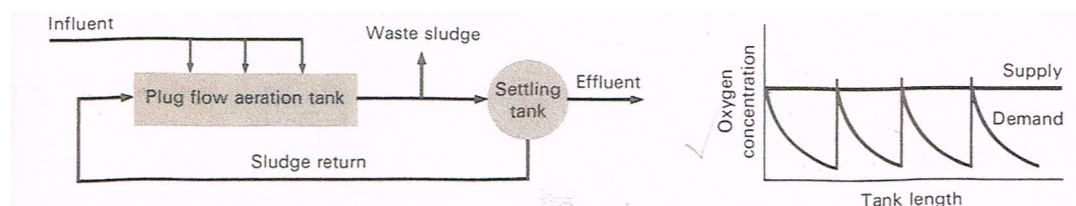


Figure IV.6 Flow diagram plus oxygen demand and supply for step aeration activated sludge process

The aeration tank is subdivided into four or more parallel channels through the use of baffles. Each channel comprises a separate step, and the several steps are linked together in series. Return activated sludge enters the first step of the aeration tank along with a portion of the settled sewage. The piping is so arranged that an increment of sewage is introduced into the aeration tank at each step. Flexibility of operation is an advantage in this process. Other advantages are: higher BOD loadings per 1,000 ft³ of aeration tank volume, soluble organics removal in a short period, and better utilization of the oxygen supplied. Example IV.3 shows the design of a treatment plant using the above process.

Example IV.3

Data:

Settled sewage flow = 7.40 mgd (989,000 ft³/day)

BOD content = 7,900 lbs

Design maximum BOD loading = 40 lbs/1000 ft³/day

Design minimum aeration period = 6 hr

Number of aeration tanks required = 4

Minimum operating MLSS = 2,000 mg/l

Number of final circular clarifiers = 4

Determine (1) the dimensions of the aeration tanks, and (2) the dimensions of the clarifiers.

Solution

Volume of tank based on BOD loading = $7,900 / (40 / 1000) = 198,000 \text{ ft}^3$

Volume of tank based on aeration period = $(7,400,000 \times 6) / (24 \times 7.48) = 247,000 \text{ ft}^3$

Use the higher value of 247,000 ft³

Now BOD loading = $7,900 / 247 = 31 \text{ lb} / 1000 \text{ ft}^3 / \text{day}$

Assume each aeration tank to have a width of 24 ft and liquid depth of 13 ft

Length of each tank = $247,000 / (4 \times 13 \times 24) = 198 \text{ ft}$

Size of each aeration tank = 198 ft x 24 ft x 13 ft

Assume over flow rate of 800 gpd/ft² for clarifiers

Surface area of each clarifier = $7,400,000/4 \times 800 = 2310 \text{ ft}^2$ and diameter = 54 ft

Detention time = $(2,310 \times 13 \times 24) / (989,400/4) = 2.7 \text{ hr}$

IV.2.5. Modified aeration activated sludge process – The flow diagram for this process is similar to that of conventional process except that this process uses shorter aeration times, usually 1.5 to 3 hours, and a high food to microorganism ratio. The MLSS concentration is relatively low, whereas the organic loading is high. BOD removal is in the range of 60 to 75 percent. The sludge has poor settling characteristics and the effluent contains high suspended solids.

IV.2.6. Contact stabilization activated sludge process – Flow sheet for this process is shown in Figure IV.7.

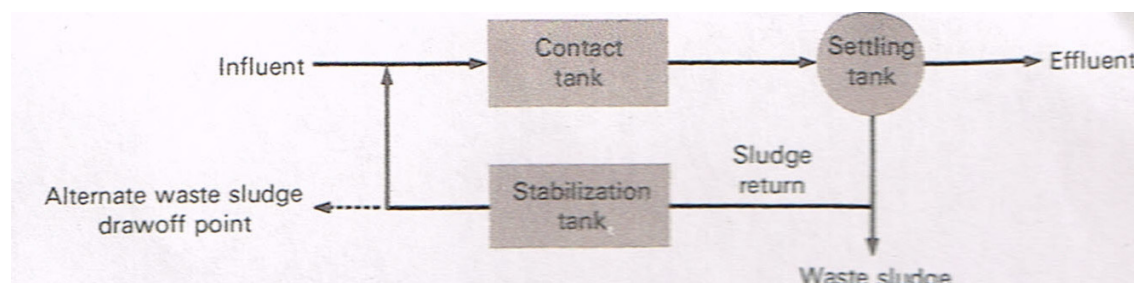


Figure IV.7 Flow sheet for contact stabilization tank

This process contains two aeration tanks; one for aerating the mixture of settled sewage and return sludge for a period of 30 to 90 min called the contact tank and the other is a separate aeration tank to aerate the return sludge from the final clarifier for 3 to 6 hours called stabilization tank. BOD removal occurs by adsorption in the contact tank and by absorption in stabilization tank. A portion of the return sludge is wasted prior to recycle to maintain a constant mixed liquor volatile suspended solids (MLVSS) concentration. The aeration tank volume requirements are approximately 50 % of conventional process. By converting an existing conventional plant in to a contact stabilization plant with minor modification to piping, the plant capacity can be even doubled with a little additional cost. This process is excellent for treating sewage not containing industrial wastes. Example IV.4 shows the design of a contact stabilization plant.

Example IV.4

A city with a population of 2,000 persons has built a contact stabilization plant for treating its sewage with the following data:

Volume of aeration tank = $2,500 \text{ ft}^3$

Volume of re-aeration tank = $5,000 \text{ ft}^3$

Volume of aerobic digester = $4,500 \text{ ft}^3$

Volume of sedimentation tank = $3,660 \text{ ft}^3$ and surface area = 300 ft^2

Calculate the BOD loading, aeration periods, and detention times.

Solution

Hydraulic load = $2,000 \times 100 \text{ gal/person/day} = 200,000 \text{ gal/day}$

BOD load = $2,000 \times 0.2 \text{ lb/person/day} = 400 \text{ lb/day}$

BOD loading on aeration tanks = $400 / (2,500 + 5,000) = 0.05 \text{ lb/ft}^3/\text{day}$

Aeration period in aeration tank = $(2,500 \times 7.48 \times 24) / 200,000 = 2.25 \text{ hr}$

Aeration period in stabilization tank = $(5,000 \times 7.48 \times 24) / 200,000 = 4.5 \text{ hr}$

Detention time in sedimentation tank with 100% recirculation = $(3,660 \times 24 \times 7.48) / (2 \times 200,000)$
= 1.64 hr

IV.2.7. Extended aeration activated sludge process – Flow sheet for this process is shown in Figure IV.8

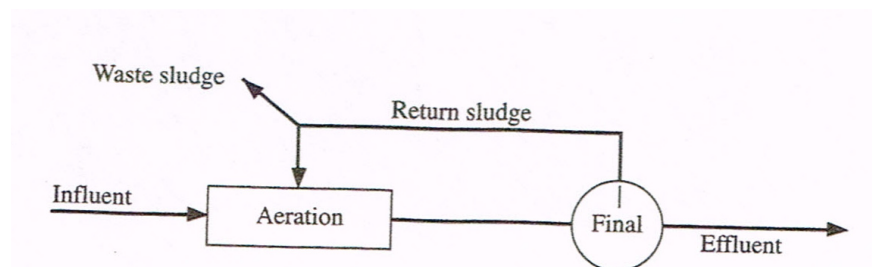


Figure IV.8 Flow sheet for extended aeration tank

This process operates in the endogenous phase of the growth curve, which necessitates a low organic loading and long aeration time of 24 hr or greater. Hence it is applicable to small treatment plant less than 1- mgd capacity. The process is stable and can accept variable loading. Final settling tanks are designed for a long detention time and a low overflow rate varying from 200 to 600 gpd/ft². The process is extensively used for prefabricated package plants. Primary sedimentation is omitted and separate sludge wasting is generally not provided.

IV.2.8. Aeration devices – There are two methods of providing aeration, one is dispersing diffused air and the other is using mechanical means. In diffused aeration, bubble air diffusers are used and they are set at a depth of 8 ft or more to provide adequate oxygen transfer and deep mixing. The diffusers are made of hollow porous stainless steel tubes 1 – 2 ft in length or hollow porous disks about 6 in. in diameter. The individual diffusers are attached along a submerged air header about 10 ft in length attached to an air supply hanger pipe which is designed with rotating joints. From data obtained from existing plants, the average power consumption is found to be 0.563 kwhr per lb of BOD removed. Figure IV.9 shows a cross section of an aeration tank with fine bubble diffuser system.

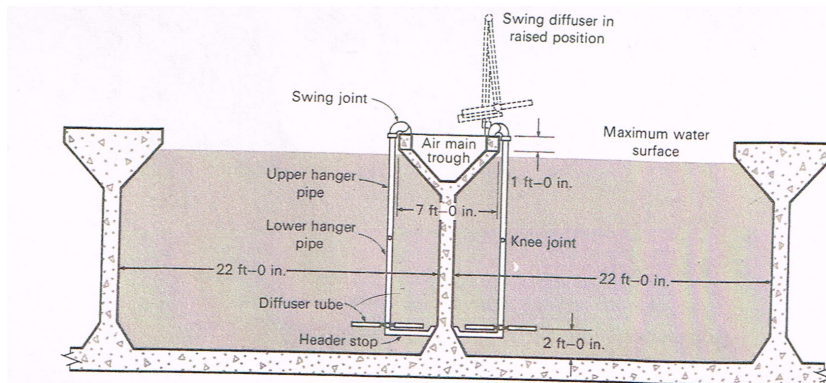


Figure IV.9 Cross section of an activated sludge aeration tank with diffuser

Mechanical aerators are of vertical draft-tube type. Flow through the draft tube is induced by a motor-driven propeller, cone or other rotary device. These aerators are designed for installation in 14 to 30 ft², hexagonal, or square tanks 8 to 18 ft deep. From data obtained from existing plants, the average power consumption is found to be 0.446 kwhr per lb of BOD removed. Figure IV.10 shows three varieties of mechanical aerators.

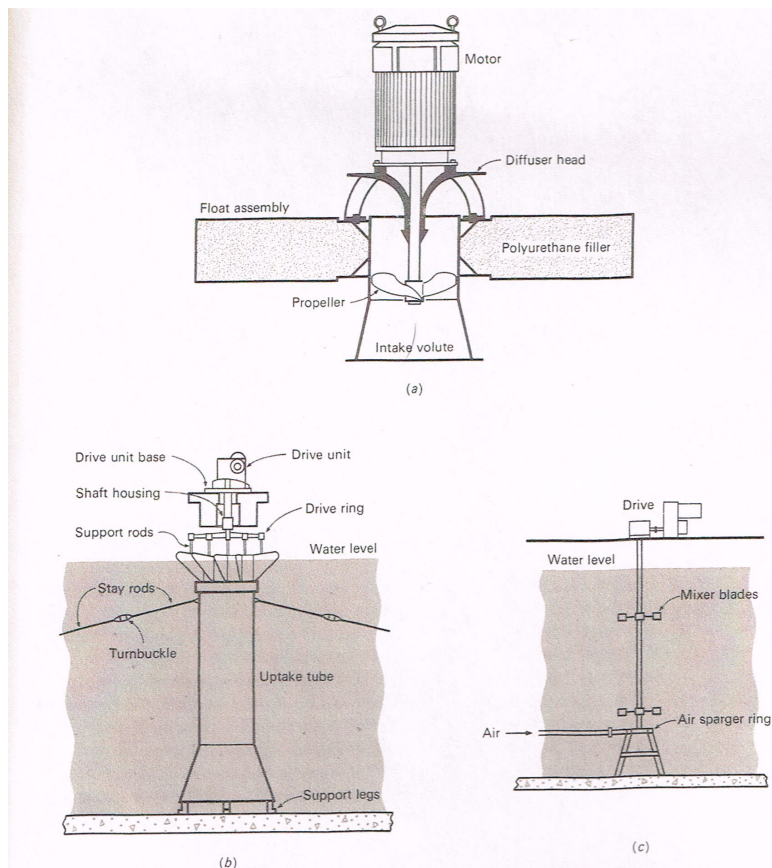


Figure IV.10 Mechanical aerators (a) surface aerator, (b) simplex cone, (c) turbine aerator

IV.2.9. **Operational characteristics and design parameters** – The operational characteristics for the different activated sludge processes are shown in Table IV.3

Table IV.3 Operational characteristics of activated sludge processes

Process modification	Flow model	Aeration system	BOD removal efficiency, %	Application
Conventional	Plug-flow	Diffused-air, mechanical aerators	85–95	Low-strength domestic wastes, susceptible to shock loads
Complete-mix	Complete-mix	Diffused-air, mechanical aerators	85–95	General application, resistant to shock loads, surface aerators
Step-aeration	Plug-flow	Diffused-air	85–95	General application to wide range of wastes
Modified-aeration	Plug-flow	Diffused-air	60–75	Intermediate degree of treatment where cell tissue in the effluent is not objectionable
Contact-stabilization	Plug-flow	Diffused-air, mechanical aerators	80–90	Expansion of existing systems, package plants, flexible
Extended-aeration	Complete-mix	Diffused-air, mechanical aerators	75–95	Small communities, package plants, flexible, surface aerators
Kraus process	Plug-flow	Diffused-air	85–95	Low-nitrogen, high-strength wastes
High-rate aeration	Complete-mix	Mechanical aerators	75–90	Use with turbine aerators to transfer oxygen and control the floc size, general application
Pure-oxygen systems	Complete-mix reactors in series	Mechanical aerators	85–95	General application, use where limited volume is available, use near economical source of oxygen, turbine or surface aerators

IV.2.10. **Operational difficulties** – Two most common operating problems in activated sludge plants are *rising sludge* and *bulking sludge*. Occasionally, de-nitrification of good settling sludge takes place in a sedimentation tank after relatively a short settling period. The nitrites and nitrates in the sewage are converted to nitrogen gas much of which is trapped in the sludge mass. If enough gas is formed, the sludge mass becomes buoyant and rises or floats to the surface. Rising sludge problem can be overcome by (1) increasing the rate of return activated sludge, (2) decreasing the rate of flow of aeration liquor, (3) increasing the speed of sludge-collecting mechanism in the sedimentation tanks, and (4) decreasing the mean cell residence time by increasing the sludge wasting rate.

Bulked sludge has poor settling characteristics and poor compactability. Two types of sludge bulking have been identified. One is caused by the growth of filamentous microorganisms such as *Sphaerotilus* and the other is caused by bound water in which the bacterial cells composing the floc swell through the addition of water to the extent that their density is reduced and they will not settle. Bulking of sludge is caused by fluctuations in flow and strength, pH, temperature, nutrient content, air supply capacity, sedimentation tank design, return-sludge pumping capacity limitation, short circuiting or poor mixing, low dissolved oxygen in the aeration tank, and overloading the aeration tanks. To control the bulking, at

least 2 mg/l dissolved oxygen must be maintained in the aeration tank, food/microorganism ratio must be maintained from 0.2 to 0.4 per day

Another problem encountered in sewage treatment plants is foam formation due to the presence of soap, detergents and other surfactants. Large quantities of foam may be produced during start-up of the process, when the MLSS are low, or whenever high concentrations of surfactants are present in the sewage. The foaming action produces a froth that contains sludge solids, grease, and microorganisms. The froth, besides being unsightly, is a hazard to workmen, because it is slippery even after collapse. To control foam formation, screened effluent or clear water is sprayed through nozzles mounted along the top of the edge of the aeration tank continuously or intermittently by a clock-controlled process. Another approach is to add a small quantity of anti foaming chemical at the inlet of the aeration tank or into the spray water.

IV.3. Oxidation ditch – This is an extended aeration process in a closed loop reactor and is good for small communities. A flow sheet for a typical oxidation ditch is shown in Figure IV.11

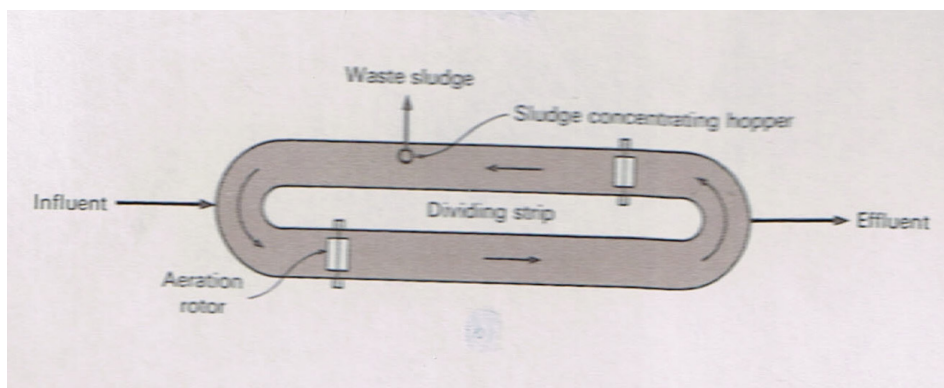


Figure IV.11 Flow sheet for an oxidation ditch

It consists of an elongated oval channel about 3 ft deep with vertical walls and a center dividing wall. Horizontal brush rotors are placed across the ditch to provide aeration and circulation. The screened sewage enters the ditch, is aerated by the rotors, and circulates at about 1 to 2 ft/sec. The operation can be either intermittent or continuous.

IV.4. Stabilization ponds – A stabilization pond, also called an oxidation pond, is a relatively shallow body of water contained in an earthen basin of controlled shape. Ponds are popular with small communities. Stabilization ponds are classified as aerobic, aerobic-anaerobic, and anaerobic.

IV.1. Aerobic ponds – Aerobic stabilization ponds contain aerobic bacteria and algae in suspension. Aerobic conditions prevail throughout the depth. There are two types of aerobic ponds. In the first type the depth is limited to 6 to 18 in. in order to provide maximum production of algae. In the second variety, the depth may be up to 5 ft so that maximum oxygen can be produced. The BOD removal is up to 95 percent. Aerobic ponds are used primarily for the treatment of soluble organic wastes and effluents from wastewater plants. Stabilization ponds may be employed in parallel or series arrangement to achieve special objectives. Parallel units provide better distribution of settled solids.

Recirculation of pond effluent has been used effectively to improve the performance of pond systems in series. The pond is designed on the basis of 100 persons per acre per day with a detention period of 200 days.

IV.2. Aerobic-Anaerobic ponds – Three zones exist in these ponds: (1) a surface zone where aerobic bacteria and algae exist in a symbiotic relationship, (2) an anaerobic bottom zone in which accumulated solids are actively decomposed by anaerobic bacteria, and (3) an intermediate zone that is partly aerobic and partly anaerobic, in which the decomposition of organic waste is carried out by facultative bacteria. Because of this, these ponds are also referred to as facultative ponds. The SS in the waste water are allowed to settle to the bottom and algae presence is not a requirement. These ponds also can be operated in series or parallel. The design parameters are same as for aerobic ponds. In cold climates during the winter months, a portion of the incoming BOD is stored in the accumulated sludge. As the temperature increase in spring and summer, the accumulated is an-aerobically converted, and the oxygen demand of acids and gases produced may exceed the oxygen resources of the aerobic surface layer of the pond. In situations where BOD storage will be a problem, surface aerators are recommended. The aerators should have a capacity adequate to satisfy from 175 to 275 % of incoming BOD.

IV.3. Anaerobic ponds – These ponds are anaerobic throughout their depth. To maintain anaerobic conditions, ponds are constructed with depths up to 20 ft. Stabilization is brought about by a combination of precipitation and the anaerobic conversion of organic wastes to CO₂, CH₄, and other gaseous products, organic acids, and cell tissues. BOD removal is up to 70 %.

IV.4. Aerated lagoons – An aerated lagoon is a basin in which sewage is treated on a flow-through basis. Oxygen is supplied by means of surface aerators or diffused aeration units. Depending on the amount of mixing, lagoons are classified as aerobic or aerobic-anaerobic. Depending upon the detention time, the effluent will contain about 1/3 to 1/2 the value of the incoming BOD in the form of cell tissues. Before the effluent is discharged, solids must be removed by settling. A settling tank is a normal component of this system,

In the case of a aerobic-anaerobic lagoon, the contents of the basin are not completely mixed, and a large portion of the incoming solids and the biological solids produced from waste conversion settles to the bottom of the lagoon. As the solids begin to build up, a portion undergoes anaerobic decomposition. The mean cell residence time varies from about 3 to 6 days. The amount of oxygen required varies from 0.7 to 1.4 times the amount of BOD removed. Ice formation may be a problem in some part of the country in winter months. The problem caused by ice formation can be minimized by increasing the depth of the lagoon. If the depth is increased beyond 12 ft, draft-tube aerators must be used.

Figure IV.12 shows schematic of aerated lagoon and aerobic-anaerobic lagoons.

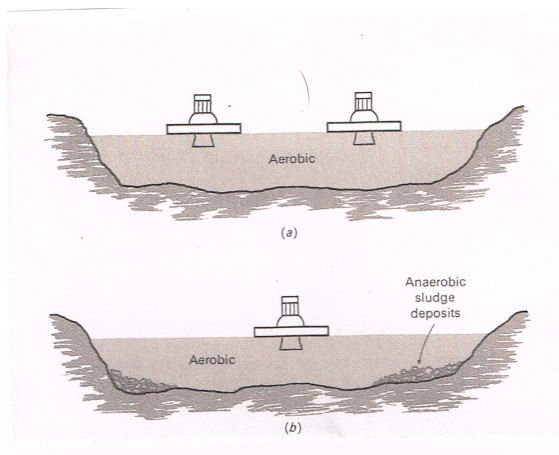


Figure IV.12 Schematic of (a) an aerated lagoon
(b) an aerobic-anaerobic lagoon

Design parameters for the different forms of the stabilization pond are furnished in Table IV.4.

Table IV.4 Design parameters for stabilization ponds

Parameter	Type of pond				Aerated lagoons
	Aerobic*	Aerobic-anaerobic	Aerobic-anaerobic	Anaerobic	
Flow regime	Intermittently mixed		Mixed surface layer		Completely mixed
Pond size, acres	<10 multiples	2-10 multiples	2-10 multiples	0.5-2.0 multiples	2-10 multiples
Operation†	Series or parallel	Series or parallel	Series or parallel	Series	Series or parallel
Detention time, days‡	10-40	7-30	7-20	20-50	3-10
Depth, ft	3-4	3-6	3-8	8-15	6-20
pH	6.5-10.5	6.5-9.0	6.5-8.5	6.8-7.2	6.5-8.0
Temperature range, °C	0-40	0-50	0-50	6-50	0-40
Optimum temperature, °C	20	20	20	30	20
BOD ₅ loading, lb/acre/day‡	60-120	15-50	30-100	200-500	
BOD ₅ conversion	80-95	80-95	80-95	50-85	80-95
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	CO ₂ , bacterial cell tissue
Algal concentration, mg/liter	80-200	40-160	10-40		
Effluent suspended solids, mg/liter§	140-340	160-400	110-340	80-160	260-300

* Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produced.
† Depends on climatic conditions.
‡ Typical values (much higher values have been applied at various locations). Loading values are often specified by state control agencies.
§ Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD₅ of 200 mg/liter and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/liter.

IV.5. Design of physical facilities – The following must be considered while designing the physical facilities: (1) location of influent lines, (2) outlet structure design, (3) dike construction, (4) liquid depth, (5) treatment of lagoon bottom, and (6) control of surface runoff.

For small ponds, a center inlet is preferred. For ponds 10 acres or more, the inlet can be installed 400 ft from the dike. For large aerobic-anaerobic ponds, multiple inlets are desirable to distribute the settleable solids over a larger area.

The outlet structure(s) should permit lowering the water level at a rate less than 1 ft/week. It should be large enough to provide easy access for maintenance. Provision for complete drainage of the pond is desirable. Overflow structures must be provided.

Dikes must be constructed such that seepage is prevented. Compaction can be done by the use of conventional equipment. Vegetation must be removed, and the area upon which the embankment is to be placed should be scarified. The dike must be wide enough to accommodate mowing machines and other maintenance equipment. A width of 8 ft is adequate. For outer slopes a 3 horizontal to 1 vertical is satisfactory. For inner slopes 1 vertical to 3 to 4 horizontal is satisfactory. A free board of 3 ft above the maximum water level is adequate. Liquid depths up to 5 ft will have some advantage. Provision of larger depths is necessary for larger ponds.

The bottom of ponds must be made as flat as possible. The bottom should be well compacted to avoid excessive seepage.

Ponds should not receive significant amount of surface runoff. If necessary, provision must be made to divert the surface water around the pond.

V. ADVANCED TREATMENT

Many of the substances found in sewage are not affected by conventional treatment operations and processes. These substances range from simple ions such as calcium, potassium, sulfate, nitrate, and phosphate to complex synthetic organic compounds. It is anticipated treatment requirements will be more stringent thus requiring advance treatment facilities. Because of their importance in promoting aquatic growths, compounds containing nitrogen and phosphorous receive considerable attention.

Unit operation and processes adopted in advanced treatment are classified as:

- (1) Physical,
- (2) Chemical, and
- (3) Biological.

Selection of a particular unit process depends upon:

- (1) The use to be made of the treated effluent,
- (2) The nature of the waste water,
- (3) The compatibility of the various operations and processes,
- (4) Available means for disposing of the ultimate contaminants, and
- (5) The economic feasibility of the various combinations.

Table V.1 shows the unit processes, their application and removal of the substances.

Table V.1 Application data for advanced treatment processes

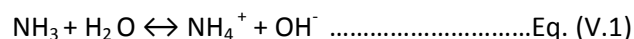
Description	Type of wastewater treated*	Removal efficiency, %								Waste for ultimate disposal
		SS	BOD	COD	NH ₃	Org N	NO ₃	PO ₄	TDS	
Physical unit operations										
Air stripping of ammonia	EBT	...	...	...	85-98	...	...	...	...	None
Filtration:										
Multimedium	EBT	80-90	50-70	40-60	...	20-40	...	...	...	Liquid and sludge
Diatomite bed	EBT	95-99	...	...	...	...	...	...	...	Sludge
Microstrainers	EBT	50-80	40-70	30-60	...	20-40	...	...	...	Sludge
Distillation	EBT nitrified + filtration	~99	98-99	95-98	...	90-98	~99	~99	95-99	Liquid
Flotation	EPT, EBT	60-80	...	...	...	20-30	...	...	...	Sludge
Foam fractionation	EBT	75-90	~70	60-70	...	...	...	...	...	Liquid
Freezing	EBT + filtration	95-98	95-99	90-99	...	90-99	~99	~99	95-99	Liquid
Gas-phase separation	EBT	...	...	...	50-70	...	...	...	...	None
Land application	EPT, EBT	95-98	90-95	80-90	60-80	80-95	5-15	60-90	...	None
Reverse osmosis	EBT + filtration	95-98	95-99	90-95	95-99	95-99	95-99	95-99	95-99	Liquid
Sorption	EBT	...	~50	~40	...	...	...	~99	~10	Liquid and sludge
Chemical unit processes										
Carbon adsorption	EPT, EBT	80-90	70-90	60-75	...	50-90	...	...	...	Liquid
Chemical precipitation	EBT	60-80	75-90	60-70	5-15	30-50	...	90-95	~20	Sludge
Chemical precipitation in activated sludge	EPT	80-95	90-95	85-90	30-40	30-40	30-40	30-40	~10	Sludge
Ion exchange	EBT + filtration	...	40-60	30-50	85-98	80-95	80-90	85-98	†	Liquid
Electrochemical treatment	Raw	80-90	50-60	40-50	80-85	80-85	...	80-85	...	Liquid and sludge
Electrodialysis	EBT + filtration + carbon adsorption	...	...	...	30-50	...	30-50	30-50	~40	Liquid
Oxidation (chlorine)	EBT	...	80-90	65-70	50-80	...	...	...	...	None
Reduction	EBT	...	...	...	...	...	NO ₃ → NH ₃	...	...	None
Biological unit processes										
Bacterial assimilation	EPT	80-95	75-95	60-80	30-40	30-40	30-40	10-20	...	Sludge
Denitrification	Agricultural return water	...	...	...	...	...	60-95	...	...	None
Harvesting of algae	EBT	...	50-75	40-60	50-90	50-90	50-90	~50	...	Algae
Nitrification-denitrification	EPT, EBT	...	...	...	...	...	60-95	...	...	None
* EPT is effluent from preliminary treatment and EBT is effluent from biological treatment. † Varies with type of resin.										

* EPT is effluent from preliminary treatment and EBT is effluent from biological treatment.

† Varies with type of resin.

V.1. Physical unit operations – Of the many physical operations that have been used in advanced treatment, removal of ammonia and nitrogen should be given considerable attention.

V.1.1. Air stripping of ammonia – Air stripping of ammonia is a modification of the aeration process used for the removal of gases dissolved in water. Ammonium ions in wastewater exist in equilibrium with ammonia, as shown in the following equation:



As the pH is increased to 7, the equilibrium is shifted to the left and the ammonium ion is converted to ammonia, which may be removed as a gas by agitating the wastewater in the presence of air.

V.1.2. Filtration – Filtration can be used to prepare the wastewater for subsequent treatment processes or for direct reuse as highly clarified water. It may be directly applied to the secondary treatment plant effluent or following coagulation-sedimentation process. The objective of filtration is to produce an effluent that consistently meets the established treatment criteria at minimum cost. The following types of filters are in use:

Dual or mixed-media filters - In recent years, sand filters have been replaced, in many cases by dual or mixed-media filters. These filters consist of different density media of varying size in an attempt to approximate reverse gradation. Two of the most commonly applied schemes in mixed-media filtration are:

- (1) Dual media, composed of a coarse anthracite coal approximately 12 in deep;
- (2) Mixed-media configuration which utilizes coal, silica sand, and garnet sand.

Both of these filters attempt to create a more ideal filtration mechanism by providing media in which the largest particles are on the top and the smallest particles on the bottom.

Granular media filters -These filters may be used with or without pretreatment (by coagulation and sedimentation) for removal of solids.

V.1.3. Other operations – The following operations are also practiced as appropriate:

- a. Distillation – This is a unit operation in which the components of a liquid solution are separated by vaporization and condensation. Volatile contaminants such as ammonia gas and low-molecular-weight organic acids can be removed by this process. Among the various distillation processes, multistage flash evaporation, multiple-effect evaporation, and vapor-compression distillation appear most feasible.
- b. Flotation - Flotation is used for removal of finely divided colloidal and suspended matter in treated sewage. Its use is increasing especially in conjunction with the use of polymers.
- c. Foam fractionation – This operation involves the separation of colloidal and suspended material by flotation and dissolved organics by adsorption.
- d. Freezing – This is an operation of physical separation similar to distillation. Wastewater is sprayed into a chamber operated under vacuum. A portion of the wastewater evaporates and the cooling effect produces contaminant-free ice crystals in the remaining liquid. The ice is then removed and melted by using the heat of condensation of the vapors from the evaporation stage.
- e. Reverse osmosis – This is a process in which water is separated from dissolved salts in solution by filtering through a semi-permeable membrane at a pressure greater than the osmotic pressure caused by the dissolved salts.

- f. Sorption – This is a process developed to remove various forms of phosphate. Activated alumina is used by passing a stream of water through the sorption column. Regeneration of the activated alumina for reuse is accomplished by using small amounts of caustic and nitric acids.

V.2. Chemical unit processes – A variety of advanced chemical unit processes have been applied to the treatment of sewage. Some have been used to both treated and untreated sewage while some others have been used to treated effluents. The following are the most common unit processes.

- a. Carbon adsorption –Following biological treatment, adsorption has been accomplished in fixed and expanded bed columns of granular carbon and in tanks using powdered carbon.
- b. Chemical precipitation – This process is used for precipitation of phosphorus by adding coagulants such as alum, lime, or iron salts, poly-electrolytes, and metal ions. Chemical precipitation may be carried out in primary or activated-sludge settling tanks or as a separate operation.
- c. Ion exchange – This is a unit process in which ions of a given species are displaced from an insoluble exchange material by ions of a different species in solution. Ion-exchange operations are either batch or continuous. Exchange material is placed in a packed column or bed and, water to be treated is passed through it.

V.2.1. Other processes – Other chemical treatment processes used include electro dialysis, oxidation, and reduction.

- a. Electro-dialysis - Ionic components of a solution are separated through the use of semi-permeable ion-selective membranes in this process. Application of an electrical potential between the two electrodes causes an electric current to pass through the solution, which in turn, causes a migration of anions towards the positive electrode and cations toward the negative electrode.
- b. Oxidation – Chemical oxidation can be used to remove ammonia, to reduce the concentration of residual organics, and to reduce the bacterial and viral content of wastewaters. Chlorine or hypochlorite can be added to remove ammonia by forming monochloramine and dichloramine as intermediate products and nitrogen gas and hydrochloric acid as end products.
- c. Reduction – Nitrate may be reduced electrolytically and by the use of strong reducing agents. When reducing agents are used, the reaction usually must be catalyzed. Other reducing agents have been tried. The use of the chemical depends on its availability at low cost and should not produce any toxic compounds.

V.3. Biological unit processes – The common biological process units employed are (a) bacterial assimilation and (b) nitrification de-nitrification. These processes have been used principally for the removal of nitrogen in various forms and indirectly for the removal of phosphorus.

- a. Bacterial assimilation – For cells production, nitrogen and phosphorus are required. About 0.13 lb of nitrogen and 0.0026 lb of phosphorus are required for each lb of cells produced. If

the food source is properly selected and adjusted, it should be possible to convert all soluble forms of nitrogen and phosphorus into organic forms contained in bacterial cells.

- b. Nitrification-de-nitrification – This process seems to be the most promising one for removal of nitrogen. If the wastewater contains nitrogen in the form of ammonia, first the ammonia is aerobically converted to nitrate nitrogen (nitrification) and subsequently the nitrates are converted an-aerobically into nitrogen gas (de nitrification). A plug-flow mixed reactor would be used for nitrification and de-nitrification. Mean cell residence time is a control factor and it varies from 2 to 4 days.

VI. DISINFECTION

Disinfection of treatment plant effluent involves specialized treatment for the destruction of harmful (pathogenic) and otherwise objectionable organisms. Disinfection has been practiced for destruction of pathogenic organisms, more particularly, bacteria of intestinal origin. The survival time of pathogenic organisms depends upon temperature, pH, oxygen and nutrient supply, dilution, competition with other organisms, resistance to toxic influences, ability to form spores, and others. Disinfection does not necessarily imply sterilization (complete destruction) of all living organisms. Elemental chlorine is commonly employed in municipal treatment applications. Wastewater disinfection is also practiced by the application of heat, irradiation by ultraviolet rays, and oxidants such as halogens, and ozone etc.

Chlorine is shipped in liquid form, in pressurized steel cylinders ranging in size from 100 lb to 1 ton. One volume of chlorine liquid yields 450 volumes of chlorine vapor. The moist gas is corrosive and so all piping and dosing equipment must be nonmetal or resistant to corrosion.

Chlorine gas is drawn from the pressurized cylinder through a solution feeder which controls the rate of application. The injector, in a solution feed chlorinator, dissolves the gas into the feed water. The concentrated solution is then applied to the process water. See Figure VI.1 for a chlorination flow diagram.

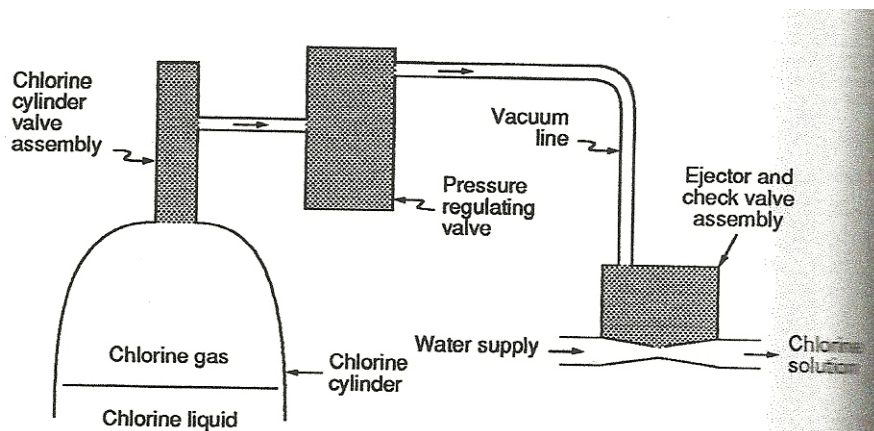


Figure VI.1 Chlorination flow diagram

VI.1. Chlorine- dioxide as a disinfectant – Chlorine dioxide may be produced from sodium chlorite and acid; from sodium chlorite and gaseous chlorine, or from sodium hypochlorite. After production, chlorine dioxide is fed through PVC pipe using a diaphragm pump. Safety features such as chlorine gas detectors, floor drains, and emergency gas masks should be available at the generation and application site. The major advantage of chlorine dioxide is in its use as a residual disinfectant. It does not produce measurable quantities of by-products such as trihalomethanes, because it does not react with many chlorine-demanding substances. Other advantages of chlorine dioxide include algae destruction; iron and manganese removal, and residual and general disinfection properties.

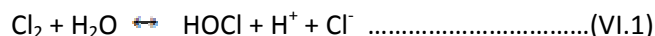
VI.2. Ozone as a disinfectant – Ozone is a strong oxidizing gas that reacts with most organic and many inorganic molecules. It is more reactive than chlorine. It does not react with effluent to produce disinfecting species but decomposes to produce oxygen and hydroxyl free radicals. The half-life of ozone is approximately 10 to 30 min and shorter if pH is above 8 and hence it must be generated at site. Ozone is rarely applied solely for disinfection because of the high cost relative to chlorine. In most cases its application is for inactivation of microorganisms. Ozone does not produce any health-related by-products. The ozonation system consists of four parts as follows:

1. A gas preparation system.
2. An electric power supply.
3. Ozone-generating equipment.
4. Contacting equipment.

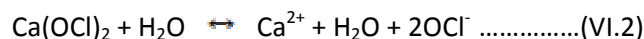
Two systems are available for ozone production – the Otto system and Welsback system. Ozone must be produced at the treatment plant. Pipes leading from the ozonator are usually stainless steel. Ozone is introduced into the effluent by injection through a filter head at the base of a column contactor or by jetting into an impeller at the base of a contact column or by diffusion through various media such as ceramic and stainless steel diffusers. Typically, the column provides 5 to 10 min of contact time between the ozone and the effluent.

VI.3. Chemistry of chlorination – Chlorine is used in the form of free chlorine or as hypochlorite. In either form it acts as a potent oxidizing agent and often dissipates itself in side reactions so rapidly that little disinfection is accomplished until amounts in excess of the chlorine demand have been added.

Reactions with water - Chlorine combines with water to form hypochlorous and hydrochloric acids as show in the following equation:



Hypochlorous is a weak acid and poorly dissociates at pH levels below 6. In dilute solution and at pH levels above 4, the equilibrium shown above is displaced greatly to the right and very little Cl_2 exists as such in solution. Hypochlorites are used largely in the form of calcium hypochlorites. When such compounds are dissolved in water, they ionize to yield hypochlorite ion as shown below:



This ion establishes equilibrium with hydrogen ions in accordance with the following equation:



The amounts of OCl^- ion and HOCl in the solution depend upon the pH as shown in Figure VI.2 below.

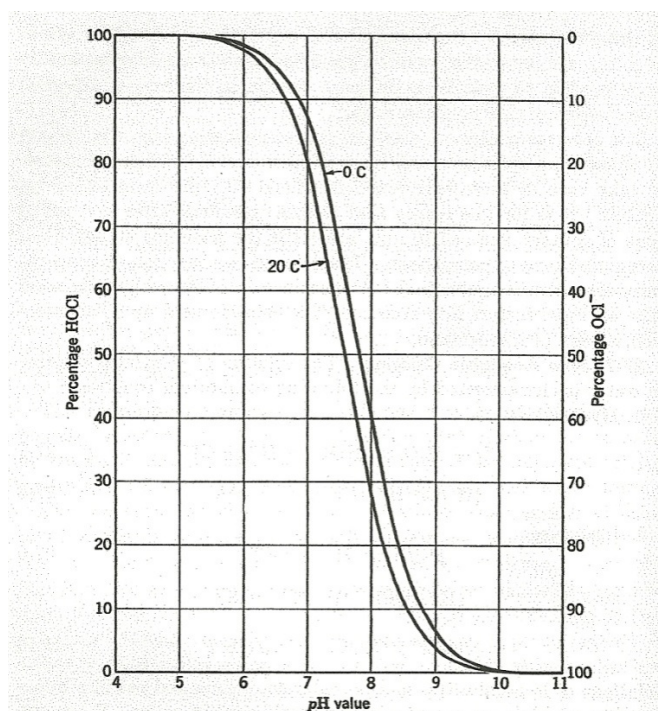
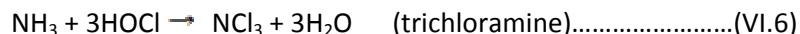
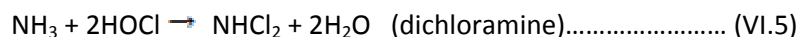
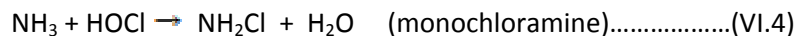


Figure VI.2 Distribution of HOCl and OCl^- at different pH_s & temperatures

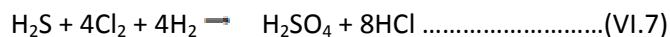
Reactions with Ammonia - Ammonium ions exist in equilibrium with ammonia and hydrogen ions. The ammonia reacts with chlorine or hypochlorous acid to form monochloramines, dichloramines, and trichloramines depending upon the relative amount of each and to some extent on the pH as follows:



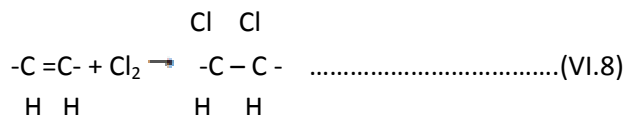
The mono- and dichloramines have significant disinfecting power and are, therefore, of interest in the measurement of chlorine residuals.

Chlorine combines with a wide variety of materials, particularly reducing agents. Many of the reactions are very rapid, while others are much smaller. These side reactions complicate the use of chlorine for

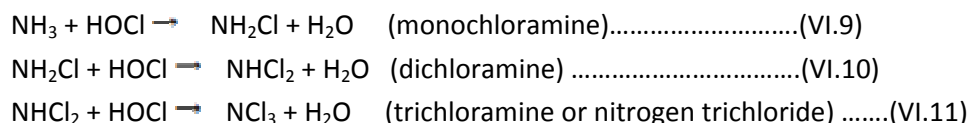
disinfecting purposes. Their demand for chlorine must be satisfied before chlorine becomes available to accomplish disinfection. The reaction between hydrogen sulfide and chlorine, as shown below, illustrates the type of reaction that occurs with reducing agents.



Fe^{2+} , Mn^{2+} , and NO_2^- are examples of other inorganic reducing agents present in effluents. A few organic reducing agents may be present, but their concentrations are very low. Organic compounds that possess unsaturated linkages will also need chlorine and increase the chlorine demand.



Chlorine-Ammonia reactions – The reactions of chlorine with ammonia are of great significance in disinfection. When chlorine is added to effluent containing natural or added ammonia, the ammonium reacts with HOCl to form various chloramines which, like HOCl, retains the oxidizing power of the chlorine. The reactions between chlorine and ammonia are shown below:



The distribution of reaction products is governed by the rates of formation of monochloramine and dichloramine, which are dependent on pH, temperature, time, and initial $\text{Cl}_2 : \text{NH}_3$ ratio. In general high $\text{Cl}_2 : \text{NH}_3$ ratios, low temperatures, and low pH levels favor dichloramine formation. It is evident some dichloramine can be anticipated at pH levels below 7. At pH levels below 7.5 some nitrogen trichloride can be expected. Depending on the free ammonia and organic nitrogen content, the level of free residual chlorination applied, contact time, and pH, nitrogen trichloride can pose a considerable problem which may be disposed by various means.

Chlorine Residuals: Time of contact and concentration of the disinfecting agent are extremely important in disinfection. Where other factors remaining constant, the disinfecting action may be represented by

$$\text{Kill} = C \times t$$

Where C = concentration of the disinfecting agent

t = time of contact

Kill = disinfecting effect

With long contact times, a low concentration of disinfectant suffices, whereas short contact times require high concentration to accomplish equivalent kills.

It has become common practice to refer to chlorine, hypochlorous acid, and hypochlorite ion as free chlorine residuals and chloramines are called combined chlorine residuals. The reaction rate between

ammonia and hypochlorous acid is most rapid at pH 8.3 and increases rapidly as the pH is decreased or increased. For this reason, it is common to find free chlorine and combined chlorine residuals coexisting after contact periods of 10, 15, or even 60 min.

With mole ratios of chlorine to ammonia up to 1:1, both monochloroamine and dichloroamine are formed, the relative amounts of each being a function of the pH. Further increases in the mole ratio of chlorine to ammonia result in formation of some trichloramine and oxidation of part of the ammonia to nitrogen gas. These reactions are essentially complete when 2 moles of chlorine have been added for each mole of ammonia nitrogen originally present in the water. Chloramines residuals usually reach a maximum when 1 mole of chlorine has been added for each mole of ammonia and then decline to a minimum value of chlorine to-ammonia ratio of 2:1. Further additions of chlorine produce free chlorine residuals. Chlorination to the extent that all the ammonia is converted to trichloramine or oxidized to free nitrogen or other gases is referred to as *break point chlorination* because of the peculiar character of the chlorine residual curve, as illustrated in Figure VI.3

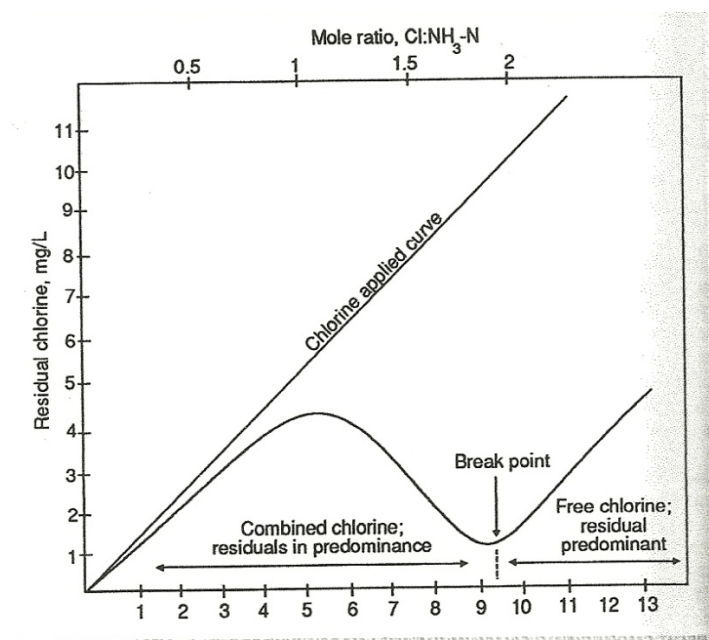
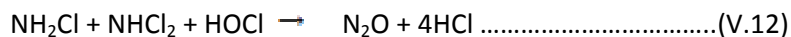


Figure VI.3 Residual chlorine curve

Theoretically, it should require 3 moles of chlorine for the complete conversion of 1 mole of ammonia to nitrogen trichloride (trichloramine). The fact that 2 moles of chlorine are required to reach the break point indicates that some unusual reactions occur. Nitrous oxide, nitrogen, and nitrogen trichloride have been identified among the gaseous products of the breakpoint reaction. The presence of nitrous oxide could be accounted for by the following reaction:



The total chlorine required for formation of monochloroamine, dichloramine, and the hypochlorous acid for the final oxidation step corresponds to 2 moles for each mole of ammonia. This would indicate that nitrous oxide is the major end product when ammonia is oxidized by chlorine in dilute solutions.

VI.4. Design criteria – The design criteria, as recommended by *the Recommended Standard for Sewage works, Great Lakes Upper Mississippi River Board of State Public Health & Environmental Managers* (Ten State Standards), are given below:

1. For normal domestic sewage, the following may be used as a guide in sizing chlorination facilities.
Trickling plant effluent.....10 mg/l
Activated plant effluent.....8 mg/l
Tertiary filtration effluent.....6 mg/l
Nitrified effluent.....6 mg/l
2. Standby equipment of sufficient capacity should be available to replace the largest unit during shutdowns.
3. An ample supply of water shall be available for operating the chlorinator.
4. The use of 1-ton containers should be considered where the average chlorine consumption is over 150 lbs.
5. Scales for weighing cylinders shall be provided at all plants using chlorine gas.
6. A bottle of 56% ammonium hydroxide solution shall be available for detecting chlorine leaks.
7. Piping systems should be as simple as possible.
8. A gas tight room shall separate the chlorination equipment from any other portion of the building.
9. A clear gas, gas-tight, window shall be installed in an exterior door or interior wall of the chlorinator room.
10. The temperature of the room where the chlorination equipment is installed must be kept at least 60⁰ F and forced mechanical ventilation shall be installed. Switches for fans and lights shall be outside the room.
11. Respiratory air-pac equipment shall be available and must be stored at a convenient location.
12. The chlorine contact tank should be constructed so as to reduce short-circuiting the flow.
13. The disinfectant shall be positively mixed as rapidly as possible, with the complete mix being effected in 3 seconds and a minimum contact time of 15 minutes provided at peak hourly flow
14. Facilities shall be included for sampling the disinfected effluent after contact and equipment shall be provided to measure the chlorine residual. Equipment shall also be provided for measuring fecal coliform using accepted test procedures.
15. Solution-feed vacuum-type chlorinators are generally preferred for large installations.

VII. EFFLUENT DISPOSAL

Ultimate disposal of waste water effluents will be by dilution in receiving waters, by discharge on land or desert areas, and by evaporation into the atmosphere as well as seepage into the ground.

Disposal by dilution in larger bodies of water, such as lakes, rivers, estuaries, or oceans is by far the most common method. The assimilative capacity or self-purification capacity of water bodies must be determined prior to discharging the effluent into them. Receiving water standards and effluent standards are established by regulatory agencies. One of the standards of receiving body of water is to maintain a minimum of 5.0 mg/l of dissolved oxygen.

VII.1. Disposal by dilution – Small lakes and reservoirs are completely mixed. A stream is a living thing capable of absorbing some pollution because of their ability to purify themselves through the action of living organisms. The sources of oxygen replenishment in a river are re-aeration from the atmosphere and photosynthesis of aquatic plants and algae. In most rivers, it is assumed that the effluent is evenly distributed over the cross section of the river. In river analysis the Streeter-Phelps equation is most commonly used.

The zone where the river meets the sea is called an estuary. The ebb and flow of tides may cause significant lateral mixing in the reaches of the rivers near the estuary. Estuarine waters are vertically stratified. In many estuarine channels, tidal action merely increases the amount and dispersion of the waste along the length of the channel.

Ocean disposal is typically accomplished by submarine outfalls that consist of a long section of pipe to transport the effluent some distance from shore. At the end of the outfall, the effluent is released in a simple stream or jetted through a manifold or multiple-port diffuser. The design of an outfall should meet applicable receiving-water standards. Bacterial, floatable material, nutrient, and toxicity requirements will govern the design and location of most outfalls.

VII.2. Disposal on land – Effluent disposal on land includes agricultural use, recreational use, ground water recharge, spraying, and containment (ponding). Spraying on irrigable land, wooded areas, and hill sides has been used. The amount of effluent disposal depends on the climatic conditions, the infiltration capacity of the soil, the types of grass or crops grown, and the quality standards imposed where runoff is allowed.

VII.3. Direct & Indirect reuse – The amount of effluent that can be reused is affected by the availability and cost of fresh water, transportation and treatment cost, water quality standards, and the reclamation potential of the effluent. It can be used as cooling water in industries. Agricultural use of effluent is practiced depending upon the crops. Field crops that are normally consumed in a raw state cannot be irrigated with the effluent.

VII.4. Recreational use – Recreational use includes golf course irrigation and park watering, establishment of ponds for boating and recreation, and maintenance of fish and wild life ponds.

VII.5 **Municipal use** – Reclaimed water can be used for lawn irrigation in addition to using for car washing, drive-way washing, toilet flushing, clothes washing etc. To accomplish this, there should be a dual municipal water system, one with fresh water for cooking and drinking purposes and the other with the reclaimed water for all uses other than drinking and cooking. In the construction of the two systems care should be taken to see that there is no chance of cross connection between the two systems.

VIII. SLUDGE TREATMENT AND DISPOSAL

For proper design of sludge treatment and disposal facilities, sources, quantities, and characteristics of the sludge must be known. Data on quantities of sludge produced from various processes and operations are presented in Table VIII.1

Table VIII.1 Sludge quantities produced from different treatment processes

Description	Type of wastewater treated*	Removal efficiency, %								Waste for ultimate disposal
		SS	BOD	COD	NH ₃	Org N	NO ₃	PO ₄	TDS	
Physical unit operations										
stripping of ammonia	EBT	...	...	...	85-98	...	...	...	...	None
multimedia	EBT	80-90	50-70	40-60	...	20-40	...	...	...	Liquid and sludge
diatomite bed	EBT	95-99	...	...	...	...	...	...	...	Sludge
microstrainers	EBT	50-80	40-70	30-60	...	20-40	...	...	...	Sludge
filtration	EBT nitrified + filtration	~99	98-99	95-98	...	90-98	~99	~99	95-99	Liquid
membrane filtration	EPT, EBT	60-80	...	...	...	20-30	...	...	...	Sludge
membrane fractionation	EBT	75-90	~70	60-70	...	...	...	...	...	Liquid
coagulation	EBT + filtration	95-98	95-99	90-99	...	90-99	~99	~99	95-99	Liquid
two-phase separation	EBT	...	...	...	50-70	...	...	...	...	None
solid application	EPT, EBT	95-98	90-95	80-90	60-80	80-95	5-15	60-90	...	None
reverse osmosis	EBT + filtration	95-98	95-99	90-95	95-99	95-99	95-99	95-99	95-99	Liquid
precipitation	EBT	...	~50	~40	...	...	...	~99	~10	Liquid and sludge
Chemical unit processes										
carbon adsorption	EPT, EBT	80-90	70-90	60-75	...	50-90	...	...	...	Liquid
chemical precipitation	EBT	60-80	75-90	60-70	5-15	30-50	...	90-95	~20	Sludge
chemical precipitation in activated sludge	EPT	80-95	90-95	85-90	30-40	30-40	30-40	30-40	~10	Sludge
ion exchange	EBT + filtration	...	40-60	30-50	85-98	80-95	80-90	85-98	†	Liquid
electrochemical treatment	Raw	80-90	50-60	40-50	80-85	80-85	...	80-85	...	Liquid and sludge
electrodialysis	EBT + filtration + carbon adsorption	...	...	...	30-50	...	30-50	30-50	~40	Liquid
oxidation (chlorine)	EBT	...	80-90	65-70	50-80	...	...	...	...	None
reduction	EBT	...	...	...	...	...	NO ₃ → NH ₃	...	...	None
Biological unit processes										
bacterial assimilation	EPT	80-95	75-95	60-80	30-40	30-40	30-40	10-20	...	Sludge
nitrification	Agricultural return water	...	...	...	...	...	60-95	...	...	None
harvesting of algae	EBT	...	50-75	40-60	50-90	50-90	50-90	~50	...	Algae
nitrification-denitrification	EPT, EBT	...	...	...	...	...	60-95	...	...	None
EPT is effluent from preliminary treatment and EBT is effluent from biological treatment. †varies with type of resin.										

The volume of sludge depends mainly on its water content and slightly on the solid matter. The characteristics of sludge vary depending on its origin, the amount of aging that has taken place, and the type of processing to which it has been subjected. Sludge from primary sedimentation tank is

usually grey and slimy and has an offensive odor. Sludge from chemical precipitation tanks is usually black and has objectionable odor. Activated sludge has a brown flocculent appearance. This sludge, when in good condition, has an inoffensive characteristic odor. Trickling filter humus is brownish, flocculent, and relatively inoffensive. Digested sludge is dark brown to black and contains a large amount of gas. It is not offensive and has odor like that of hot tar, burnt rubber, or sealing wax.

Sludge treatment includes the following treatment processes: thickening, digestion, conditioning, and dewatering.

VIII.1. Thickening – Waste activated sludge or mixture of primary and waste activated sludge are subjected to thickening. The aim of thickening is volume reduction. If a sludge is thickened from 1 to 4 percent solids, the volume will be reduced to 25 percent of the original volume. Mechanical (gravity) and dissolved-air flotation thickeners are commonly used to thicken sludge.

VIII.1.2. Mechanical thickener – Dilute raw primary or waste activated sludge is fed into the thickening tank continuously. Thickening tank is similar to a circular clarifier. Figure VIII.1 shows schematic of a mechanical thickener.

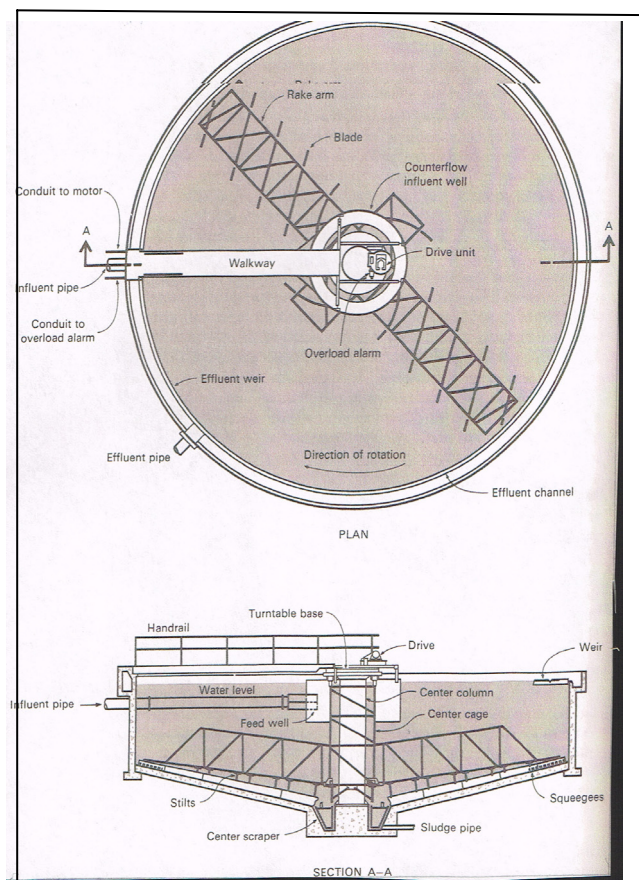


Figure VIII.1 Schematic of a mechanical thickener

The resulting continuous supernatant flow is returned to the primary settling tank. The thickened sludge collected at the bottom of the tank is pumped to the digesters. The thickeners are designed on the basis of hydraulic surface loading and solids loading. Typical surface loading rates are 400 to 800 gpd/ft². Solids loadings are shown in Table VIII.2

Table VIII.2 Solids loading rate for mechanical thickeners

Type of sludge	Sludge, percent solids		Solids loading for mechanical thickeners, lb/sq ft/day
	Unthickened	Thickened	
Separate sludges:			
Primary	2.5-5.5	8-10	20-30
Trickling filter	4-7	7-9	8-10
Modified aeration	2-4	4.3-7.9	7-18
Activated	0.5-1.2	2.5-3.3	4-8
Combined sludges:			
Primary and trickling filter	3-6	7-9	12-20
Primary and modified aeration	3-4	8.3-11.6	12-20
Primary and activated	2.6-4.8	4.6-9.0	8-16

Aerated mixed liquor or final effluent must be added to maintain aerobic conditions.

VIII.1.3. **Flotation thickener** – These are used normally with waste activated sludge. It will produce a sludge with approximately 4 percent solids. The solids loading rates are given in Table VIII.3

Table VIII.3 Solids loading rate for flotation thickener

Type of sludge	Loading, lb/sq ft/day
Activated (mixed liquor)	5-15
Activated (settled)	10-20
50% primary + 50% activated (settled)	20-40
Primary only	to 55

VIII.2. **Digestion**- Digestion is classified as anaerobic and aerobic. Although anaerobic digestion has been practiced for over a century, aerobic process has been growing in popularity for use.

VIII.2.1. **Anaerobic digestion** – Anaerobic digestion is classified as conventional or standard rate and high rate. Conventional digestion is carried out either as a single stage or two stage process. See Figures VIII.2 and VIII.3 for schematics.

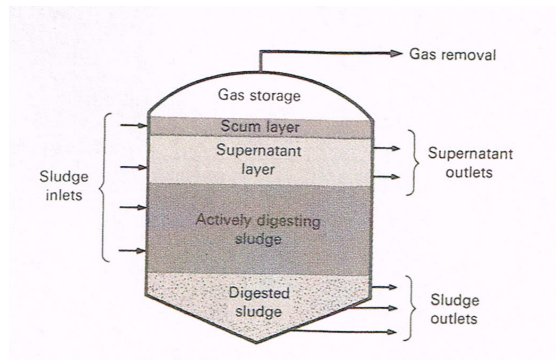


Figure VIII.2 Schematic of a conventional digester in single-stage process

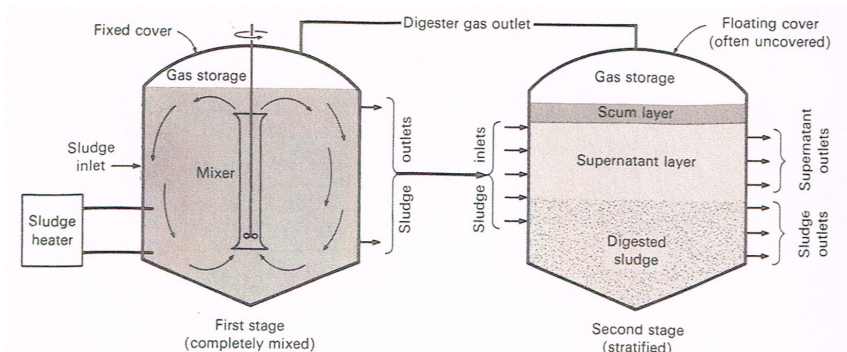


Figure VIII.3 Schematic of two-stage digestion process

The sludge is normally heated by means of coils located within the tank or an external heat exchanger. In single stage, the functions of digestion, thickening, and supernatant formation are carried out simultaneously. A cross section of a typical standard rate digester is shown in Figure VIII.4.

Due to the stratification and the lack of mixing, the volume of a standard-rate single stage digester is not more than 50 percent utilized. Recognizing these limitations, most conventional digesters are operated as two stage digesters.

In the two-stage process, the first tank is used for digestion. It is heated and equipped with mixing facilities. The second tank is used for storage and concentration of digested sludge and for formation of clear supernatant. Tanks may have fixed roof or floating covers. Tanks are usually circular and the diameter varies from 20 to 115 ft. Water depth should be minimum 25 ft at the center.

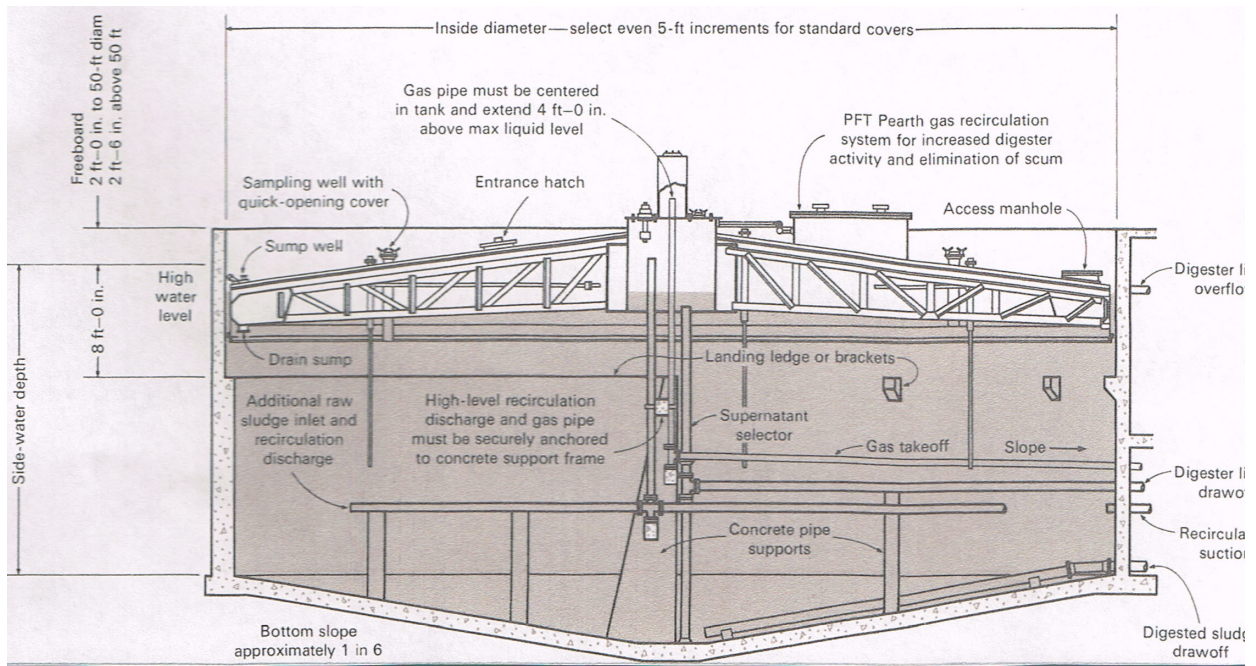


Figure VIII.4 Cross section of a standard rate digester

With the exception of higher loading rates and improved mixing, there are not many differences between a high rate digester and the first tank in a conventional two stage digester. Sludge should be pumped continuously. The incoming sludge displaces digested sludge to a holding tank. Typical volumes of digester gas (methane) produced in anaerobic digestion range from 8 to 12 ft³/lb of volatile solids added. Gas production varies from 0.6 to 0.8 ft³/capita in primary plants treating normal domestic sewage. In secondary treatment plants this is increased to about 1.0 ft³/capita. Heating value of digester gas is approximately 600 Btu/ft³

VIII.2.2. Aerobic digestion – Aerobic digesters are used to treat only waste activated sludge, mixtures of waste activated sludge or trickling filter sludge and primary sludge. Advantages of aerobic digestion are: (1) lower BOD concentrations in supernatant liquor, (2) production of an odorless, humus-like, biologically stable end product, (3) production of sludge with good dewatering characteristics, (4) recovery of basic fertilizer values, (5) fewer operational problems, and (6) lower capital cost. The disadvantages are (1) higher power cost, and (2) the useful by-product, methane gas, is not recovered.

Aerobic digestion is similar to activated sludge process. As the supply of available substrate (food) is depleted, the microorganisms will begin to consume their own protoplasm to obtain energy for cell maintenance. When this occurs the microorganisms are said to be in the endogenous phase or auto oxidation phase.

Factors that must be considered in designing aerobic digesters include hydraulic residence time, process loading criteria, oxygen requirements, energy requirements for mixing, environmental conditions, and process operation. Hydraulic residence time varies from 10 to 12 days. Volatile

solids removal ranges from 45 to 75 percent. Solids loading ranges from 0.1 to 0.2 ft³/day. Oxygen requirement for complete oxidation of BOD varies from 1.7 to 1.9 lb/lb of cell tissue destroyed. If mechanical aerators are used for mixing, horse power required is 0.5 to 1.0 hp/1,000 ft³ volume of the tank. In air mixing, air requirement is between 20 and 30 ft³/min/1,000 ft³ of tank volume. The system may perform poorly if the temperature and pH fall below 20°C and 5.5 respectively. The pH should be checked periodically and necessary adjustment made if necessary.

VIII.3. Conditioning – Conditioning is performed for the purpose of improving its dewatering characteristics. Addition of chemicals and heat treatment are the methods most commonly used. Elutriation, a physical washing operation, is employed to reduce the chemical requirement. The chemical dosage required is determined in the laboratory by filter-leaf test. Common chemicals used are Cao and FeCl₂.

VIII.4. Dewatering – Methods used for dewatering sludge include spreading on drying beds, vacuum filtration, and centrifugation. The choice among these methods depends on the characteristics of the sludge, the method of final disposal, the availability of land, and the economics involved.

VIII.4.1. Drying beds – Sludge is placed on the beds in 8 to 12-in layer and allowed to dry. After drying the sludge is removed and disposed in a landfill, or ground for use as a fertilizer. A typical sludge drying bed is shown in Figure VIII.4.

The drying area is partitioned into individual beds, approximately 20 ft wide and 20 to 100 ft long. The interior partitions consist of two or three creosoted planks, one on top of the other, to a height of 15 to 18 in stretching between slots in precast concrete posts. The area required for different types of sludge in northern United States is shown in Table VIII. 4.

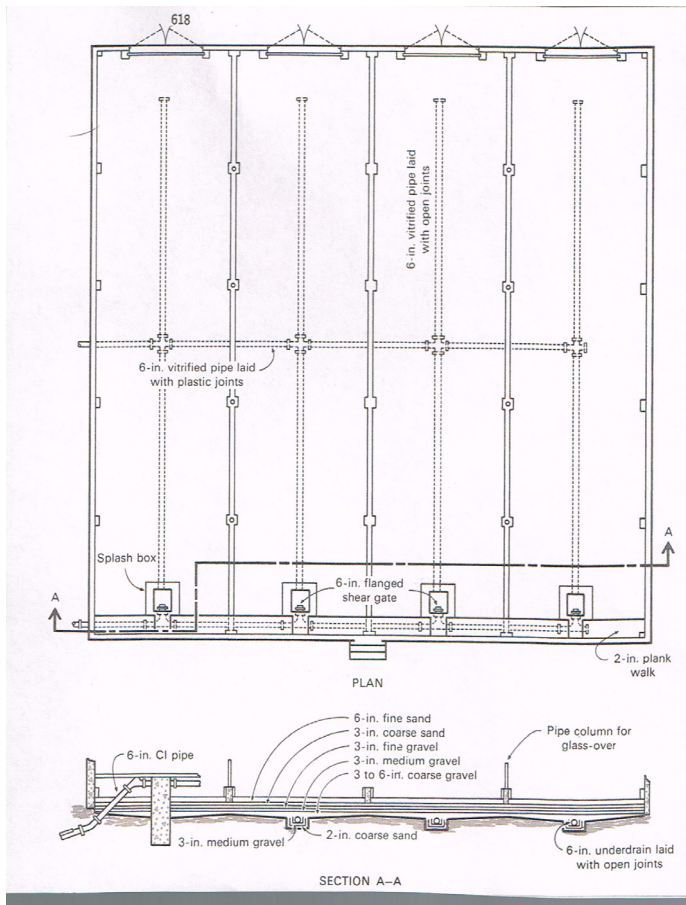


Figure VIII.4 Plan and section of a typical sludge drying bed

Table VIII.4 Area required for drying beds

Type of sludge	Open beds	Covered beds
Primary digested	1.0–1.5	0.75–1.0
Primary and humus digested	1.25–1.75	1.0–1.25
Primary and activated digested	1.75–2.5	1.25–1.5
Primary and chemically precipitated digested	2.0–2.5	1.25–1.5

The sand layer should be from 9 to 12 in deep. Sand should have uniformity coefficient of less than 4.0 and an effective size of 0.3 to 0.75 mm. Piping to the beds must be designed for a velocity of 2.5 ft/sec. The moisture content of the sludge is approximately 60 % after 10 to 15 days of drying under favorable conditions.

VIII.4.2. **Vacuum filters** – This may be the most widely used dewatering device using mechanical means. Surface areas of vacuum filters vary from 50 to 300 ft² and are equipped with various types of filter cloth. Filter cloths made of cotton, wool, nylon, Dacron are available. Filters are operated from 20 to 30 hours in a week and are designed for a yield of 3.5 lb/ft²/hr. Moisture content normally varies from 70 to 80 percent.

VIII.4.3. **Centrifuges** – centrifuges used in sewage treatment plants are of the solid-bowl type with electric-motor drive. Sludge is fed into the rotating bowl at a constant flow rate where it separates into a dense cake containing the solids and a little dilute stream called centrate. The sludge cake containing approximately 75 to 80 percent moisture is discharged from the bowl by a screw feeder into a hopper or on to a conveyor belt. Solids concentration in the cake will vary from 15 to 40 percent. The operation of centrifuges is clean, simple, and relatively inexpensive and does not require chemical conditioning. The centrate is returned to the head works of the treatment plant.

The area required for installation of a centrifuge is less than that required for a vacuum filter of equal capacity and the initial cost is lower. The operation cost will be relatively higher than that for the vacuum filter.

VIII.5. **Disposal** – The solids removed as sludge from sewage treatment plants should be disposed of. The method of final disposal determines the type of stabilization and the amount of volume reduction. The ultimate disposal of sludge is done on land and in sea. Also sludge is incinerated and the resulting ash is disposed of in a landfill.

VIII.5.1. **Land disposal** – The most common methods of land disposal include spreading on soil, lagooning, dumping, and landfilling.

VIII.5.1.1. **Spreading on soil** – Wet digested sludge may be disposed of by spreading over farm lands and plowing under after it has dried. Dried sludge can be disposed of by bagging it and selling it as a soil conditioner. The digested sludge may be heat dried, ground in a mill, and fortified with nitrogen to give it some fertilizer value.

VIII.5.2. **Lagooning** – This is a simple and economical method. A lagoon is an earth basin into which digested sludge is deposited. The sludge settles to the bottom and accumulates. Excess liquid from the lagoon, if any, should be returned to the treatment plant influent. The lagoons are shallow and the normal depth is 4 to 5 ft. They should be cleaned by scraping. Sludge may be stored indefinitely or it may be removed periodically after draining and drying.

VIII.5.3. **Dumping** – Digested sludge can be dumped in abandoned mine quarry. Along with this grit and incinerator residue can also be dumped in these quarries.

VIII.5.4. **Landfilling** – A sanitary landfill can be used for disposal of sludge, grease, and grit. Sanitary landfill is primarily used for disposing of domestic solid waste. In a sanitary landfill wastes are deposited in a designated area, compacted in place with a tractor or roller, and covered with a 12-in

layer of clean soil. After several years' time during which the wastes are decomposed and compacted, the land can be used for recreational or other purposes.

VIII.6. **Ocean disposal** – The sludge may be carried offshore in barges or sludge vessels and dumped, or it may be pumped to deep water through a submarine outfall. Sludge discharged through a separate outfall should remain submerged.

VIII.7. **Incineration** –Incineration is a process where sludge is converted into inert ash which can be disposed of easily. The process is usually self-sustaining with out the need for supplemental fuel. Multiple hearth furnace is a successful device. The furnace is a circular steel cylinder containing several hearths arranged in a vertical stack. About 1,800 to 2,500 Btu will be required to evaporate each pound of water in the sludge. Digested sludge has a heat content ranging from 2,500 to 5,500 Btu/lb of dry solids while raw sludge has a heat content of 6,500 to 9,500 Btu/lb of dry solids.