CHAPTER 10
PRELIMINARY TREATMENT

10-1. General considerations.
Preliminary treatment of wastewater includes screening, grinding, grit removal, flotation, equilization, and flocculation. Screens, grinders and grit removal are provided for the protection of other equipment in the treatment plant. Air flotation and flocculation aid in the removal of suspended solids and oil in the primary clarifier and reduce the biological loading on secondary treatment processes. Prechlorination or pre-aeration may be required to prevent odor problems and to eliminate septic conditions where wastewater has abnormally long runs to the plant. Equalization structures are used to dampen diurnal flow variations and to equalize flows to treatment facilities.

10-2. Bar screens.
  a. Description and function. The primary function of coarse screening is protection of downstream facilities rather than effective removal of solids from the plant influent. All screens used in sewage treatment plants or in pumping stations may be divided into the following classifications:

    (1) Trash racks, which have a clear opening between bars of 1½ to 4 inches and are usually cleaned by hand, by means of a hoist or; possibly, by a power-operated rake.

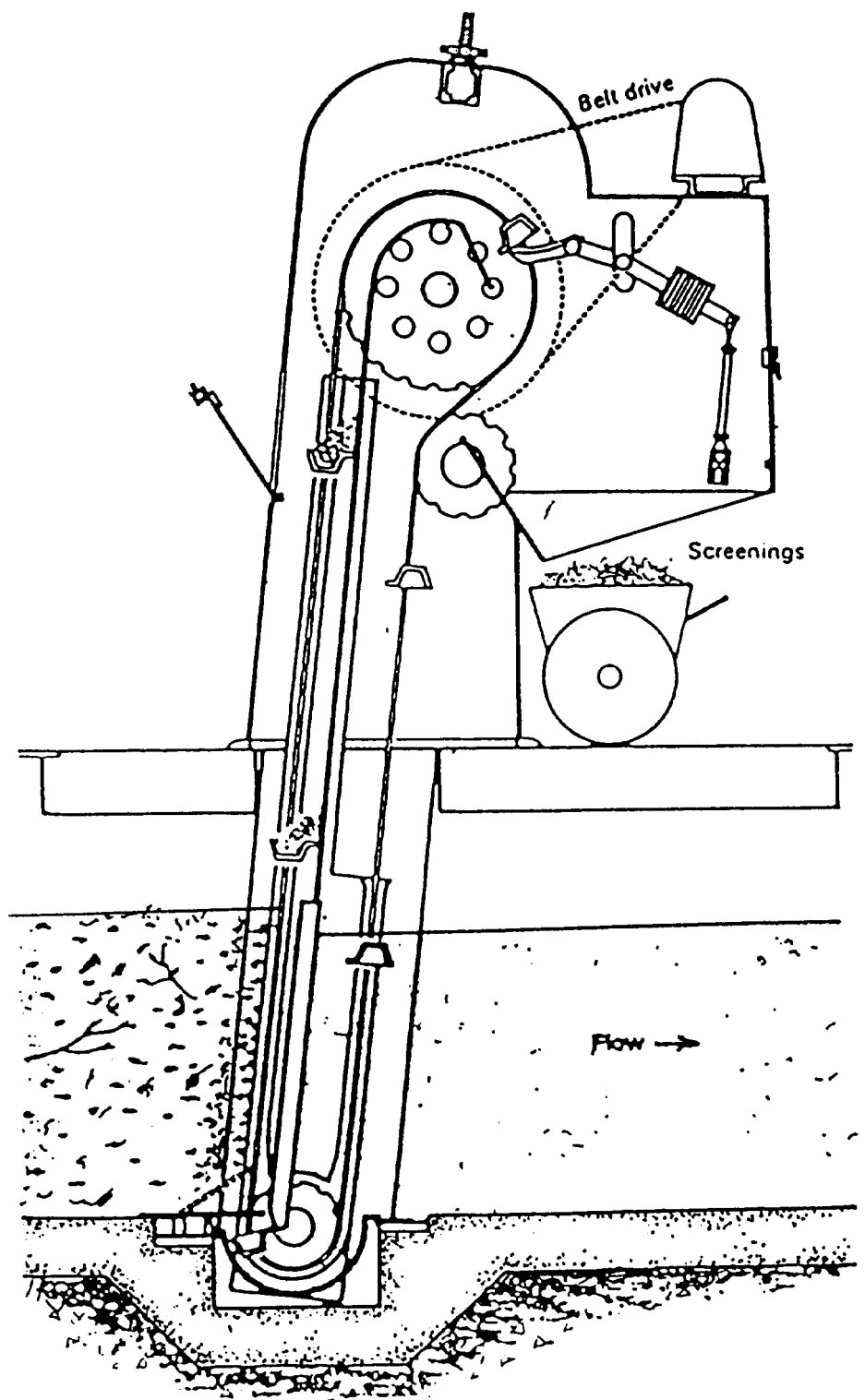   (2) Standard, mechanically cleaned bar screen with clear openings from ½ to 1½ inches (fig 10-1).

Figure 10-1. Standard mechanically cleaned bar screen.

(3) Fine screen with openings ¼ inch wide or smaller.

b. Design basis. Screens will be located where they are readily accessible. An approach velocity of 2 feet per second, based on average flow of wastewater through the open area, is required for manually cleaned bar screens. For a mechanically cleaned screen, the approach velocity will not exceed 3.0 feet per second at maximum flows.

(1) **Bar spacing**. Clear openings of 1 inch are usually satisfactory for bar spacing, but ½ to 1½-inch openings may be used. The standard practice will be to use 5/16-inch x 2-inch bars up to 6 feet in length and 3/8-inch x 2-inch or 3/8-inch x 2½-inch bars up to 12 feet in length. The bar will be long enough to extend above the maximum sewage level by at least 9 inches.

(3) **Size of screen channel**. The maximum velocity through the screen bars, based on maximum normal daily flow, will be 2.0 feet per second. For wet weather flows or periods of emergency flow, a maximum velocity of 3.0 feet per second will be allowed. This velocity will be calculated on the basis of the screen being entirely free from debris. To select the proper channel size, knowing the maximum storm flow and the maximum daily normal flow, the procedure is as follows: the sewage flow (million gallons per day) multiplied by the factor 1.547 will give the sewage flow (cubic feet per second). This flow (in cubic feet per second) divided by the efficiency factor obtained from table 10-1 will give the wet area required for the screen channel. The minimum width of the channel should be 2 feet and the maximum width should be 4 feet. As a rule, it is desirable to keep the sewage in the screen channel as shallow as possible in order to keep down the head loss through the plant; therefore, the allowable depth in the channel may be a factor in determining the size of the screen. In any event, from the cross-sectional area in the channel, the width and depth of the channel can be readily obtained by dividing the wet area by the depth or width, whichever is the known quantity.

Table 10–1. Efficiencies of bar spacing.

Bar Size inches	Openings inches	Efficiency
¼	1	0.800
5/16	1	0.768
3/8	1	0.728
7/16	1	0.696
½	1	0.667

(3) **Velocity check**. Although screen channels are usually designed on the basis of maximum normal flow or maximum storm flow, it is important to check the velocities which would be obtained through the screen for minimum or intermediate flows. The screen will be designed so that, at any period of flow, the velocities through the screen do not exceed 3 feet per second under any flow condition.

(4) **Channel configuration**. Considerable attention should be given to the design of the screen channel to make certain that conditions are as favorable as possible for efficient operation of the bar screen. The channel in front of the screen must be straight for 25 feet. Mechanical screens with bars inclined at an angle of 15 degrees from the vertical will be installed.

(5) **Screenings.** The graph shown in figure 10-2 will be used to predict the average amount of screenings that will be collected on the bar screen. The information required to make this estimate is flow and bar spacing. Grinding of the screenings (and returning them to the wastewater flow), incineration, and landfilling are satisfactory methods for disposal of the screenings.
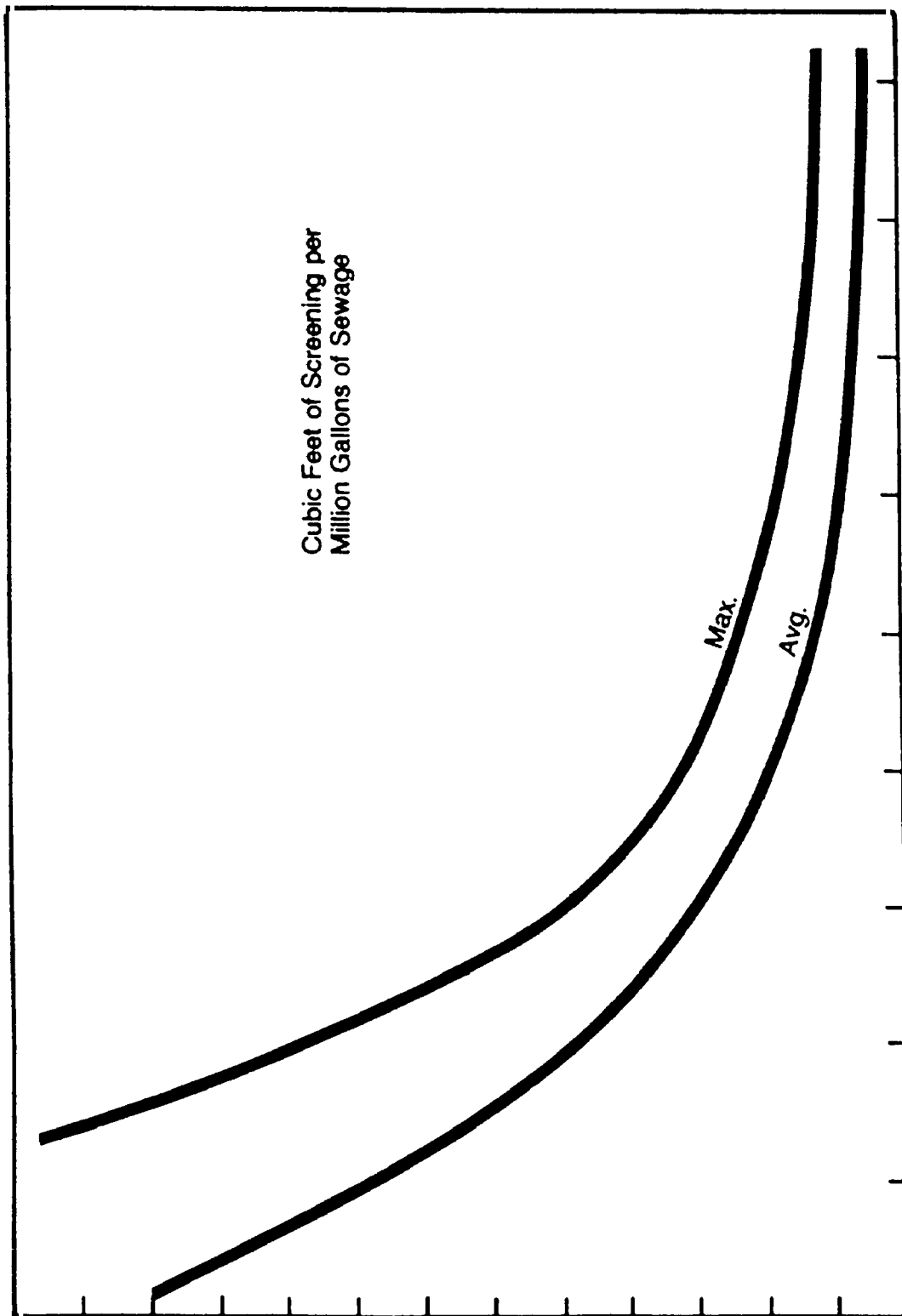
(6) **Design procedure**. Select bar size and spacing and determine efficiency factor. Determine number of units desired. Divide total maximum daily flow or total maximum storm flow by the number of screens desired to obtain maximum flow per screen. The procedure is then as follows:

Maximum daily flow in mgd $\times$ 1.547 Maximum daily flow in cfs.

Maximum storm flow in mgd $\times$ 1.547 = Maximum storm flow in cfs.

$\dfrac{\text{cfs}}{2}$ = Net area through bars for maximum daily flow.

$\dfrac{\text{cfs}}{3}$ = Net area through bars for maximum storm flow.

Whichever of the above gives the larger value shall be used for design.

$\dfrac{\text{Net area in sq.ft.}}{\text{Efficiency coefficient for bars}}$ = Gross area or channel cross‑section wet area.

Minimum width of bar rack = 2 ft; maximum width = 4 ft.

$\dfrac{\text{Channel cross‑section wet area}}{\text{Maximum desired width or depth}}$ = Corresponding depth or width.

The figures are based on recessing channel walls 6 inches each side for chain tracks and screen frame. The overall width of screen frame is 12 inches greater than width of bar rack. If it is not possible to recess the wall, the channel should be made 1 foot wider than calculated above.

10-3. Comminuting devices

a. Description and function. Comminuting devices are shredders which incorporate mechanisms that cut the retained material without removing it from the sewage flow. Comminutors are protective devices for the plant and also provide a means for reducing odors, flies and unsightliness often found around other coarse-screening devices. However; solids from a comminutor produce more scum at the digesters. Comminutors are generally located between grit chambers and the primary settling tanks.

b. Design basis. Comminutors will be required in locations where the removal of screenings will be difficult. Comminutors are available commercially; their design consists basically of screening device and cutting device (fig 10-3). More recently, "in-line" comminutors have been used to reduce the cost of structures for shredding solids. Each comminuting device must have a bypass for maintenance and repair purposes. The bypass will include a bar screen, described as coarse screens in paragraph 10-2a. Manufacturer's data and rating tables for these units will be consulted for recommended channel dimensions, capacity ranges, upstream and downstream submergence, and power requirements. Figure 10-4 illustrates a typical manufacturer's design data.
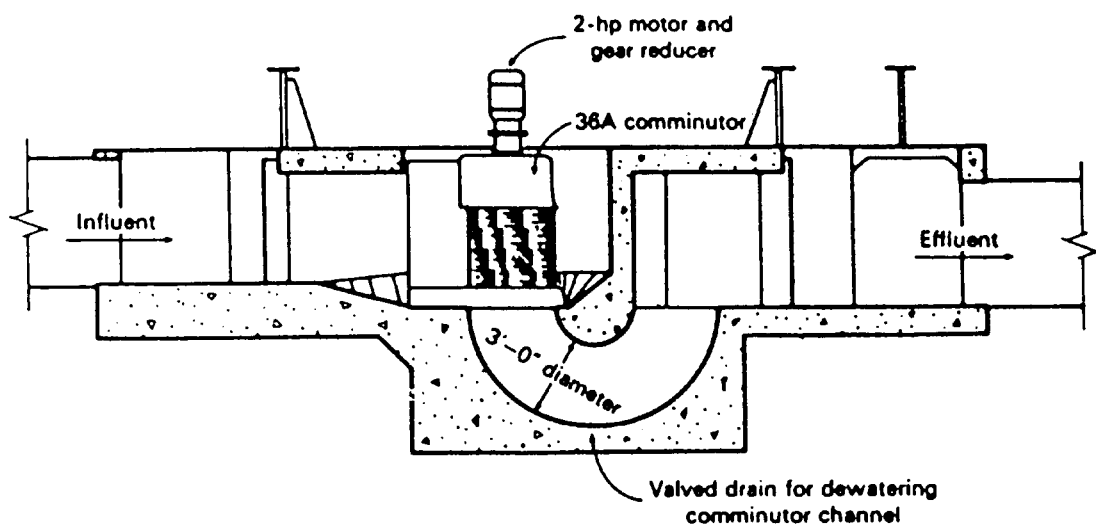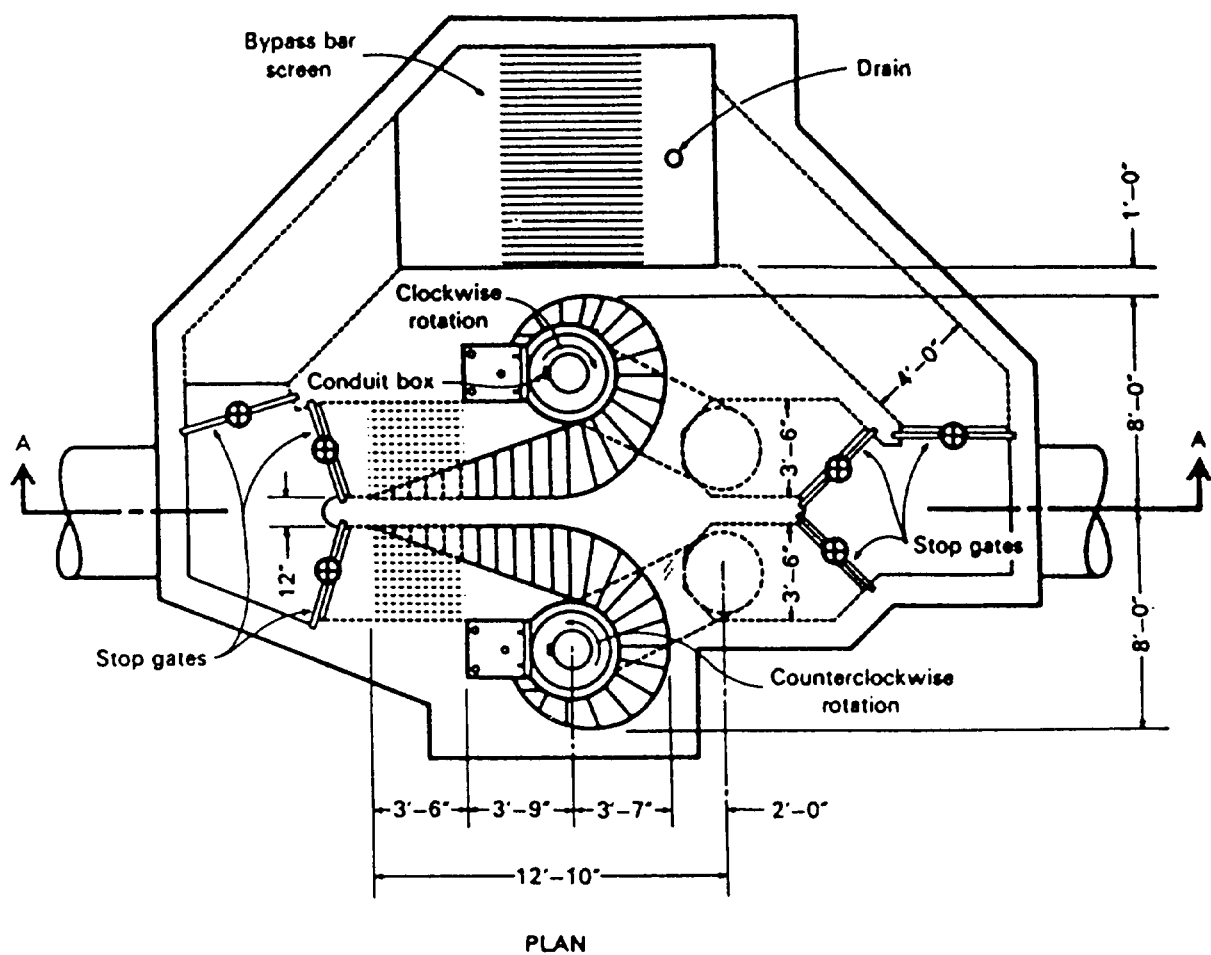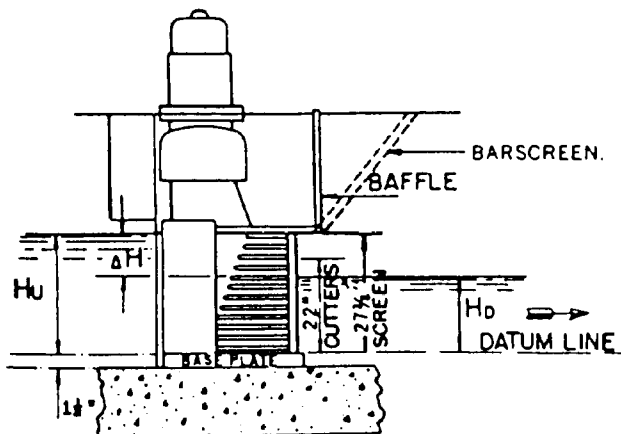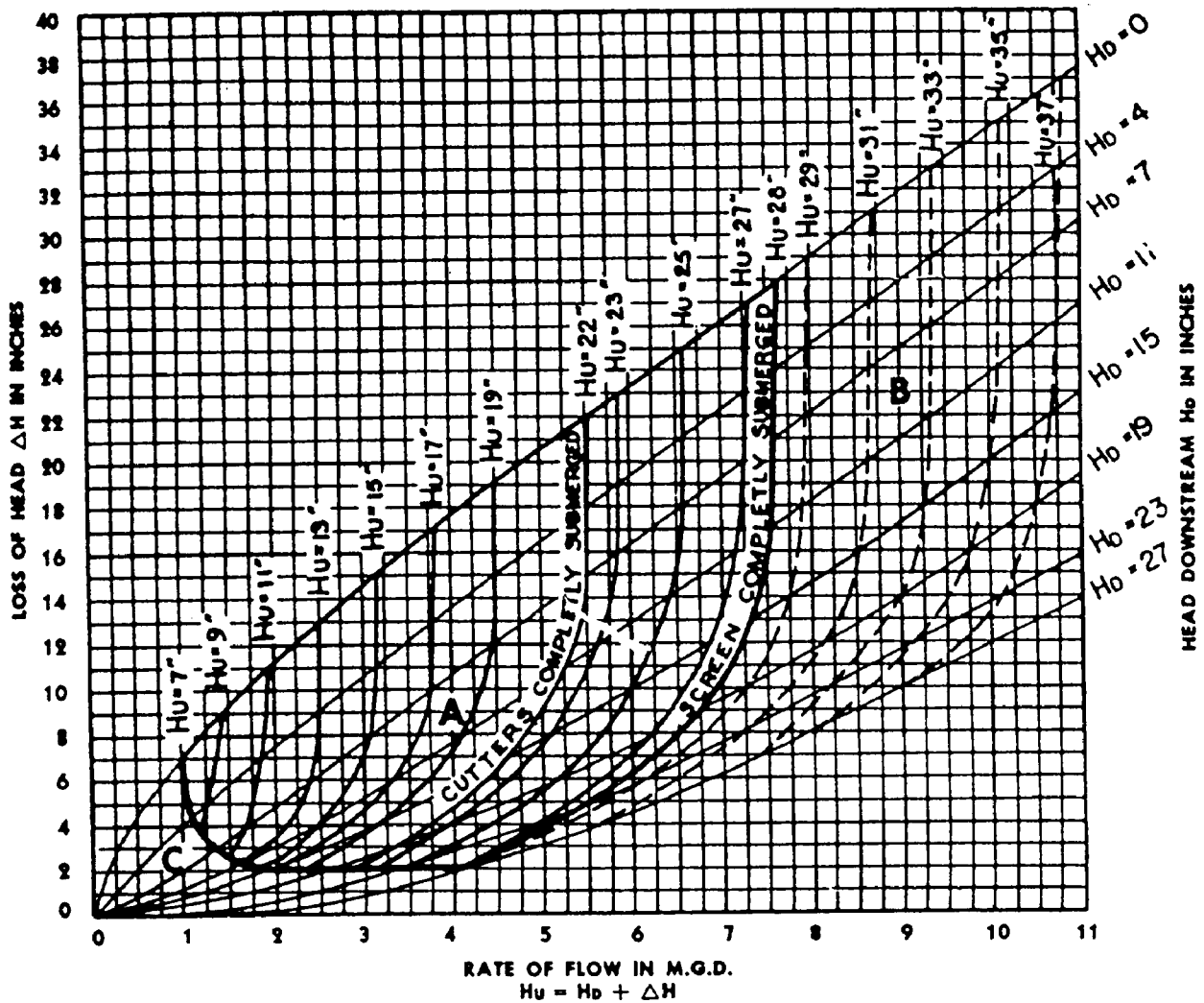
Figure 10-3. Plan and cross-sectional views of a comminutor installation.

RATE OF FLOW IN M.G.D.

$$H_u = H_D + \Delta H$$

AREA—A Desirable operating area for the average flow range. Establish comminutor datum elevation so that average flow is in this area.

AREA—B Reserve area for peak daily dry and wet weather flows provided by adding baffle or barscreen.

AREA—C Application is not recommended in this area for average flows. Carrying velocities are inadequate.

Figure 10-4. Typical communitor performance.

10-4.   Grit chambers.

a.   **Purpose**. The primary purpose of grit chambers is to protect pumps and other mechanical equipment. They may not be required if surface runoff is excluded from the sanitary sewer system; however; current policy is to include grit chambers for equipment protection regardless of the nature of the sewer system. Silting occurs through improper joints, broken manholes, and other openings in the system even without contributing surface runoff. Grit chambers will be located ahead of pumps and comminuting devices. Coarse bar racks will be placed ahead of mechanically cleaned grit-removal facilities. There are two types of grit chambers: horizontal-flow and aerated. The first attempts at controlling the wastewater velocity so that grit would settle out were the use of horizontal-flow chambers designed to maintain the velocity as close to 7 feet per second as practical. This velocity will carry most of the smaller organic particles through the chamber and will tend to resuspend those that settle but will allow the heavier inorganic grit to settle out. In recent years, the aerated grit chamber has been more widely used because introducing oxygen into the wastewater early in the treatment process is beneficial and there is minimal head loss through the chamber; however, the increased operational and energy costs must be included in evaluating this option.

b.   **Horizontal-flow grit chamber**.

(1) **Design basis.** Grit chambers will be designed for a controlled velocity of 1 foot per second (at the average rate of flow) in order to prevent settling of organic solids (at low rates of flow) and scouring (at high rates of flow). The velocities at these conditions will not vary more than 10 percent from the design velocity. A sample design is shown in appendix C.

(2) **Velocity control.** Control of velocity within a grit chamber will be provided by a control section paced by a weir, a Parshall flume, or a Venturi flume.

(a)   The weir will be either a proportional weir, Parshall flume or Venturi flume. Appendix C contains formulations and tables for design parameters applicable to such flow-control devices.

(b)   A Parshall flume is effective in controlling the velocity through a grit chamber within reasonable limits if the width of the flume throat is narrow enough to cause wide variations in the depth of water for the expected range of flow rates. Appendix C includes formulations and parameters of design for Parshall flumes. One advantage of Parshall flume control is that it can serve for both metering and velocity control.

(c)   A Venturi flume of rectangular cross-section is effective for the control of velocity in a rectangular grit chamber. The design of the effluent channel and other structures below the flume must be such that the head loss between the grit chamber and the effluent channel is not less than one-third the difference in elevations of the upstream crest and flume floor. Formulation and related design parameters for Venturi flume control are presented in appendix C.

(3) **Design factors**. Grit chambers will be designed for a controlled velocity of approximately 1 foot per second and a detention period of 45 seconds. The design of grit chambers and flow-control devices is discussed and illustrated in appendix C.

(a)   **Horizontal surface area**. To size a grit chamber with rectangular cross-section, first determine the horizontal surface area using equation 10-1:

$$A = QX, \tag{eq 10-1}$$

where:

A = horizontal surface area (sq ft);
Q = flow rate (cfs);
X = settling rate of grit particle (sec per ft).
    (Assume X = 16.7 sec/ft for all military installations.)

Assume a grit chamber width of 2 feet for flows less than 1 million gallons per day, and 2 to 4 feet for flows between 1 and 2 million gallons per day. For flows greater than 2 million gallons per day, divide the area calculated above by the assumed width to obtain the length of the channel. At a flow velocity of 1.0 foot per second, the depth of flow can be determined by equation 10-2:

$$D = \frac{L}{VX} \qquad \text{(eq 10-2)}$$

where:

D = depth of flow (ft);
L = length of channel (ft);
V = flow velocity (fps);
X = settling rate (sec/ft).

The rectangular cross-section channel will be used in treatment plants with capacity less than 2 million gallons per day. The parabolic channel is costly and best suited to larger plants. Square grit chambers are sized as rectangular cross-section channel chambers; horizontal area is the basis of design. These units are best suited to flows greater than 2 million gallons per day.

(b) **Channel size.** In channel grit chambers, the length will be designed to be 50 percent longer than theoretically required to allow for turbulence and outlet distribution. The floor of the chamber will be far enough below the weir crest to make allowance for the accumulation of about 2.5 cubic feet of grit every 10 days per million gallons of wastewater. This depth allowance will be not less than 2½ inches. The channels will be as narrow as possible without causing serious submergence of the crown of the inlet sewer. The effluent channel will be designed so that objectionable shooting velocities are not produced and submergence of the weir crest by tail water will not exceed permissible limits.

c. **Aerated grit chamber.** When wastewater flows into an aerated grit chamber, the grit will settle at rates dependent on the size, specific gravity, and the velocity of roll in the tank. The variable rate of air diffusion is a method of velocity control which can be easily adjustable to different field conditions. The following design criteria are to be used:

(1) **Air rates**. The air system should be designed to provide 8 cubic feet per minute per foot of grit chamber length. The design should allow the air rate to be controlled over a range.

(2) **Detention time.** The chamber should be designed to have a detention time of 3 minutes at the maximum flow rate.

(3) **Geometry.** The inlet and outlet should be placed to prevent short circuiting in the chamber. In addi tion, the inlet should introduce the wastewater directly into the circulation pattern caused by the air. The outlet should be at a right angle to the inlet with a baffle. A length to width ratio of 4:1 should be used.

d. **Quantity of grit.** The following design values should be used to provide storage for the collected grit. These values, along with the anticipated frequency of grit removal, will determine the storage volume required.

(1) **Combined sewer system**. For a sewer system carrying both stormwater and domestic wastewater, storage for 30 cubic feet of grit per million gallons of flow should be provided.

(2) **Separate sewer system**. Storage for 10 cubic feet of grit per million gallons of flow should be included.

e. **Disposal of grit.** Impervious surfaces with drains will be provided as grit-handling areas. If the grit is to be transported, the conveying equipment must be designed to minimize loss of material. Suitable drainage facilities must be provided for a screenings-collection platform and for storage areas. Grit disposal will be in a sanitary landfill.

10-5.   Dissolved air flotation.

Flotation is a unit process whereby particulate matter is separated from a wastewater, causing the matter to float to the liquid surface. Criteria are provided in the EPA Manual 625/1-74-006. Dissolved air flotation units will not be installed without permission from HQDA (DAEN-ECE-G) WASH DC 20314-1000 for Army projects or HQ USAF/LEEE WASH DC 20332 for Air Force projects. Permission will be granted only when adequate laboratory or pilot studies data are available and when adequate justification for the additional maintenance and operational labor requirements is provided.

10-6.   Wastewater flocculation.

Flocculation units will be used and will immediately precede clarification units when a chemical precipitation process is employed as part of primary, secondary, or advanced wastewater treatment schemes.

a.   Methods. Porous diffuser tubes or plates are commonly used for air agitation; but perforated pipes, impingement diffusers, jet diffusers, or helix-type diffusers may also be used for air flocculation. Mechanical flocculation is achieved by revolving or reciprocating paddles, radial-flow turbine impellers, or draft tubes. In typical situations, mechanical aerators (vertical draft tube type) will be used in single tanks arranged for cross flow.

b.   Design factors. Mechanical and air flocculation units for domestic wastewater will be designed for 30 minutes of flocculation detention time followed by a clarifier with surface settling rates of 800 gallons per day per square foot. The air requirement for flocculation is 0.1 standard cubic feet per gallon at 30 minutes of detention time. To insure proper agitation, air will be supplied at 2.5 standard cubic feet per minute per linear foot of tank channel. The number and size of air diffusers are determined by dividing the total air requirement by the optimum air diffusion rate per unit (4.0 cubic feet per minute per square foot of porous diffuser area). Water depths vary from 8 to 13 feet. In rectangular tanks, the ratio of length to width will be 3:1 For mechanical flocculation, revolving paddles may be either horizontal or vertical. Peripheral-paddle speed should be kept in the range of 1.0 to 3.0 feet per second to minimize deposition and yet avoid destruction of the flocs. Table 10-2 presents values for design factors to be used in designing typical sewage treatment plants.

Table 10-2. Flocculation tank design factors.

Design Factor	Flow Q (mgd)		
	Less than 0.1	0.1–1.0	Greater than 1.0
Detention time (min)	30	20	15
Depth (ft)	8	10	12
Power Factor G (sec^{-1})	20–40	30–50	40–60

c.   Power Requirements. To compute the power requirements and velocity gradient requirements, the following formula should be used:

$$D = \frac{C_D A p v^2}{2} \qquad \text{(eq 10–3)}$$

$$P = \frac{C_D A p v^3}{2} \qquad \text{(eq 10–4)}$$

$$G = \sqrt{\frac{P}{\mu V}} \qquad \text{(eq 10–5)}$$

where:

D = drag, lb;

C_D = coefficient of drag of flocculator paddles moving perpendicular to fluid;

A = area of paddles, sq ft;

p = mass fluid density, slugs/cu ft;

v = relative velocity of paddles in fluid, fps, usually about 0.7 to 0.8 of paddle–tip speed;

P = Power requirement, ft–lb/sec;

G = mean velocity gradient, ft/sec–ft = 1/sec;

V = focculator volume, cu ft;

μ = absolute fluid viscosity, lb force–sec/sq ft.

$$Gt_d = \frac{V}{Q} \sqrt{\frac{P}{\mu V}} = \frac{1}{Q} \sqrt{\frac{PV}{\mu}} \qquad \text{(eq 10–6)}$$

Typical values of G for a detention time of 15 to 30 minutes vary from 20 to 75 second[-1]. Figure 10-5 is supplied to aid the designer in determining the horsepower requirement for flocculation systems. To use the figure, select G and detention time desired. (**Note:** flocculation tank volume can be determined from the design flow rate and the detention time.) Enter chart at detention time and find power. Correct by multiplying by the appropriate factor contained in the table accompanying the figure.
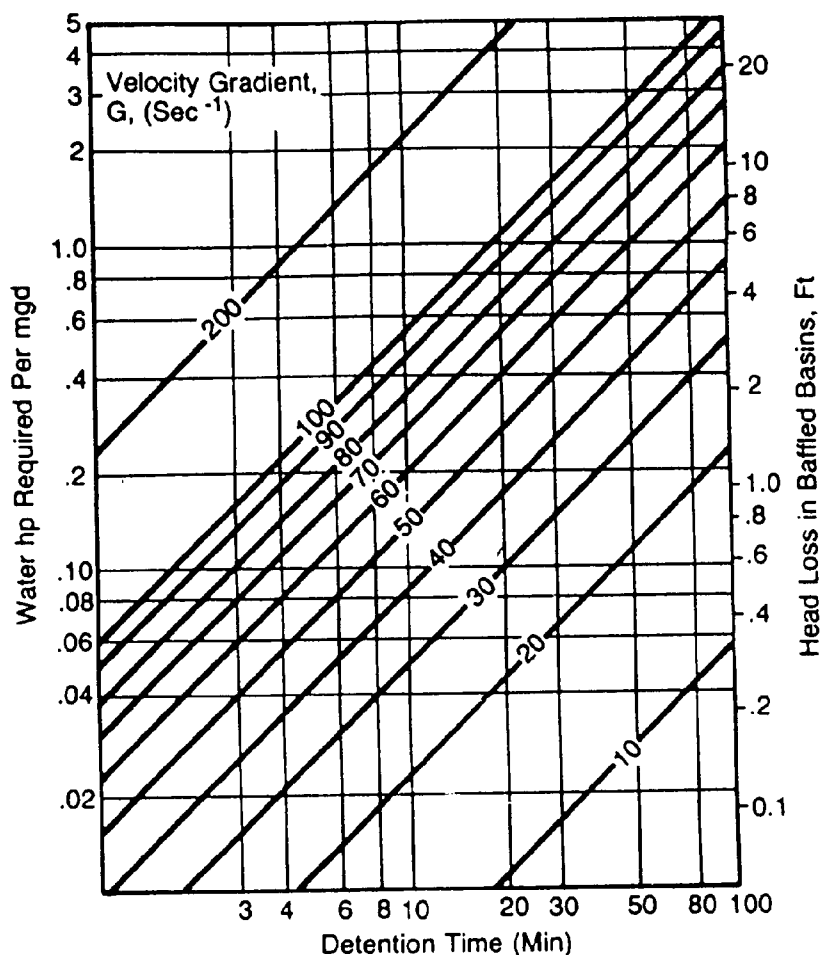
Power Required for Flocculation. 4°C

Temperature Correction

For Water Temp. Other than 4°C (39.2°F)
Multiply Curve hp by Factor:

°C	°F	Factor
2	35.6	1.070
4	39.1	1.000
5	41.0	0.970
6	42.8	0.940
7	44.6	0.915
8	46.4	0.885
9	48.2	0.860
10	50.0	0.835
12	53.6	0.790
15	59.0	0.730
20	68.0	0.640
25	77.0	0.570
30	86.0	0.510

Multiply by 1.25 (80% Drive Efficiency Assumed)
to Obtain BHP (Motor)/mgd

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Figure 10-5. Power required for flocculation.