CHAPTER 11

SCREENING [redacted]N

Screening and sedimentation are inexpensive physical processes that are widely incorporated into treatment operations for water, wastewater, and stormwater runoff. The basic laws of physics and fluid mechanics govern the processes.

11.1 SCREENS AND BAR RACKS

Screens (Fig. 11.1) and bar racks are located at intakes from rivers, lakes, and reservoirs for water treatment plants or at the wet well into which the main trunk sewer discharges for a wastewater treatment plant. They are also located before pumps in stormwater and wastewater pumping stations. They are almost always provided at these locations. These devices remove coarse debris (such as rags, solids, and sticks), which may damage pumps or clog downstream pipes and channels. The spacing of the bars may be coarse, with 50–150 mm (2–6 in.) openings; medium, with 20–50 mm (0.8–2 in.) openings; or fine screens, with openings of 10 mm (0.4 in.) or less (ASCE and WPCF, 1977; Degrémont, 1973).

To prevent the settling of coarse matter, the velocity in the approach channel to the screens should not be less than 0.6 m/s (2 ft/s). The ratio of the depth to width in the approach channel ranges from 1 to 2.

Coarse screens may be installed on an incline to facilitate the removal of debris. The headloss through the screens is a function of the flow velocity and the openings in the screens. A sketch of the water profile through a screen is given in Fig. 11.2. Bernoulli's equation is used to analyze the headloss.

$$h_1 + \frac{v^2}{2g} = h_2 + \frac{v_{sc}^2}{2g} + \text{losses} \tag{11.1}$$

where

h_1 is the upstream depth of flow
h_2 is the downstream depth of flow
g is the acceleration of gravity
v is the upstream velocity
v_{sc} is the velocity of flow through the screen

The losses are usually incorporated into a coefficient.

$$\Delta h = h_1 - h_2 = \frac{1}{2gC_d^2}(v_{sc}^2 - v^2) \tag{11.2}$$

where

C_d is a discharge coefficient (typical value = 0.84)

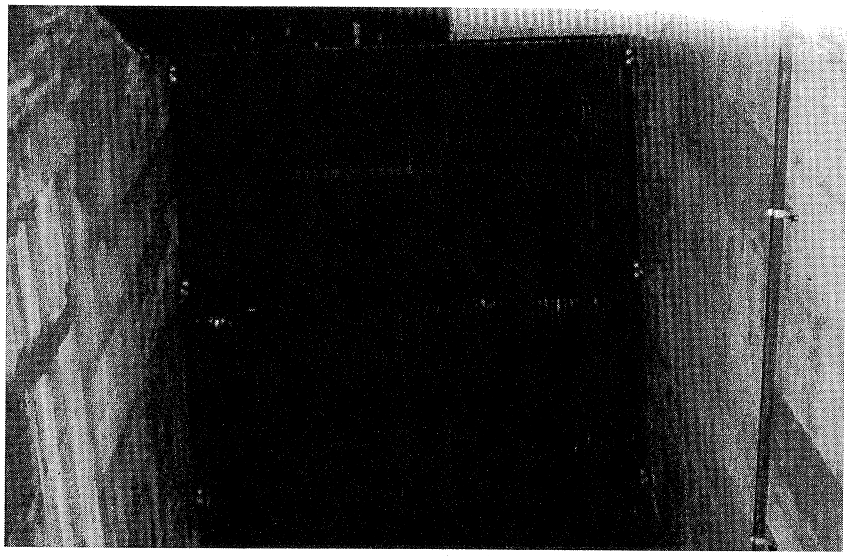

Figure 11.1 Screens at a wastewater treatment plant.

Alternatively, an orifice equation is often applied to the velocity through the screen.

$$\Delta h = \frac{v_{sc}^2}{2gC_d^2} = \frac{1}{2g}\left(\frac{Q}{C_d A}\right)^2 \tag{11.3}$$

where
Q is the volumetric flow rate
A is the area of the openings

The value of C_d in Eq. (11.3) will be supplied by the manufacturer or can be obtained from experimentation. The design value for the area is the open area of the screens for mechanically cleaned screens. The value of the discharge coefficient supplied by a manufacturer may take into account the open area of the screens. The open area of a screen may be considerably reduced by the space taken by the mesh. If the screens are to be manually cleaned, the open area should be taken as 50% of the open area (the half-clogged condition). The headloss is estimated at the maximum flow condition.

Screens or racks may be cleaned by hand or automatically. Screenings are generally nonputrescible. They are collected and hauled away to an incinerator or landfill disposal site.

Screens for Water Treatment Plants

When raw water is being withdrawn from the surface of a river, coarse screens (75 mm or 3 in. or larger) are installed to prevent the intake of small logs or other floating

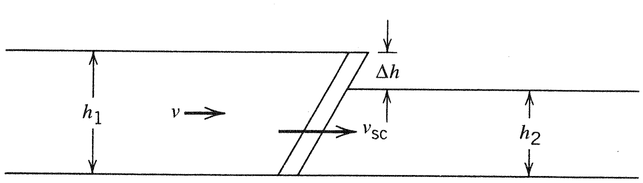

Figure 11.2 Water profile through a screen.

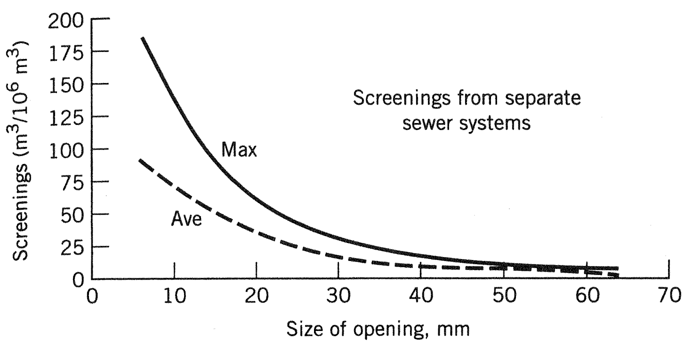

Figure 11.3 Screenings volume variation with bar opening for separate sewer systems. From WEF and ASCE (1992), *Design of Municipal Wastewater Treatment Plants,* vol. 1, WEF, © WEF 1992.

debris. For a submerged intake from a reservoir or lake, smaller coarse screens can be used. Screens at these intakes are not usually mechanically cleaned. These coarse screens are usually followed by screens with smaller openings at the treatment plant. The screens at the treatment plant may be mechanically or manually cleaned depending on the size of the operation.

Quantities of screenings collected at water treatment installations are highly variable depending on the opening of the bars and screens and the raw water source. Screenings may be washed back into the water source. The primary water treatment installation for the city of Ottawa, Ontario, which draws water from the Ottawa River, has mechanically cleaned traveling screens with openings of 1.3 mm and the screenings are washed back into the Ottawa River. The secondary water treatment facility has manually cleaned 1.3-mm (0.05-in.) opening screens that collect an average volume of 0.29 L/1 000 m^3 (0.039 ft^3/Mgal) on an annual basis. Quantities collected during the spring freshet are much higher than at other times during the year.

Screens at Wastewater Treatment Plants

Coarse screens with openings from 50 to 150 mm (2 to 6 in.) are used ahead of raw wastewater pumps (ASCE and WPCF, 1977). Screens with smaller openings (25 mm) are suitable for most other devices or processes (ASCE and WPCF, 1977). A screen with smaller openings would be installed at the beginning of the treatment plant after the water is pumped from the trunk sewer or influent wet well, which are protected by coarse bar racks. At medium to large installations, mechanically cleaned screens are used to reduce labor costs, provide better flow conditions, and improve capture.

Figure 11.3 is a design chart for estimating the maximum and average quantities of coarse screenings collected from separate sewer systems as a function of the bar opening size. It is based on data collected from 133 installations in the United States. Table 11.1 provides other information on screenings collected from separate and combined sewer systems. Combined sewer areas can produce several times the amount of screenings collected from separate sewered areas during storm flows. The peak daily collection can vary as high as 20:1 on an hourly basis.

Microstrainers

Microstrainers (Fig. 11.4) have been used to reduce suspended solids for raw waters that contain high concentrations of algae or to further reduce suspended solids in

TABLE 11.1 Coarse Screenings Characteristics[a]

Item	Range
Quantities	
Separate sewer system	
Average	3.5–35 L/1 000 m^3 (0.47–4.7 ft^3/Mgal/d)
Peaking factor (hourly flows)	1:1–5:1
Combined sewer system	
Average	3.5–84 L/1 000 m^3 (0.47–11 ft^3/Mgal/d)
Peaking factor (hourly flows)	2:1–>20
Solids content	10–20%
Bulk density	640–1 100 kg/m^3 (40–70 lb/ft^3)
Volatile content of solids	70–95%
Fuel value	12 600 kJ/kg (5 400 Btu/lb)

[a]From WEF and ASCE (1992), *Design of Municipal Wastewater Treatment Plants,* vol. 1, WEF, © WEF 1992.

effluent from secondary clarifiers following biological wastewater treatment. The microstrainer is made of a very fine fabric or screen wound around a drum. The drum is typically 75% submerged and rotated with water commonly flowing from the inside to the outside of the drum. In some strainers flow moves from the perimeter to the center. The solids deposit is removed by water jets that can be activated by a pressure differential across the screen. The water jets are directed at the exposed drum surface and collected in a channel under the top of the drum.

Openings in microstrainers vary from 20 to 60 μm. Suspended solids will be removed but bacteria will not be removed to any significant extent. To minimize slime growth that will cause high headloss, ultraviolet light may be applied to the strainer.

The headloss performance of a microstrainer is evaluated by a semi-empirical equation. It is observed that the headloss is directly proportional to the flow rate,

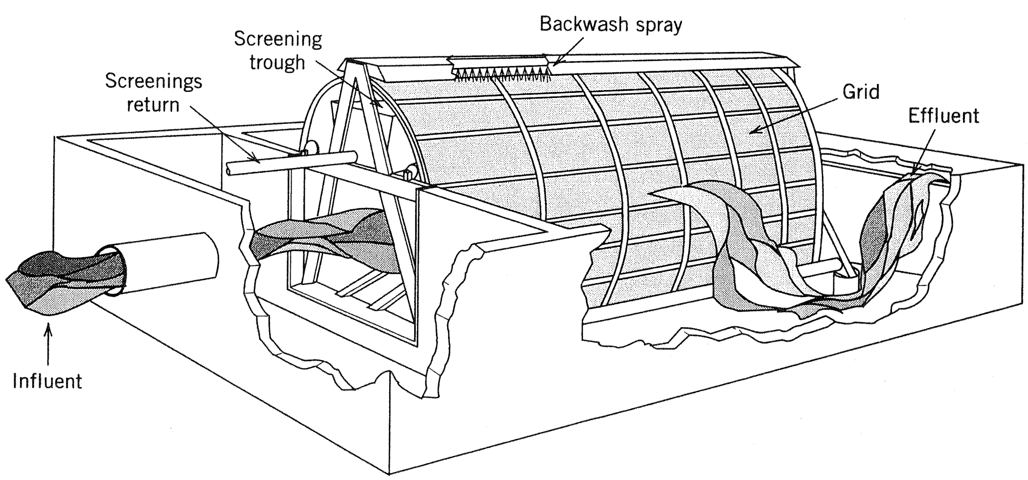

Figure 11.4 Microstrainer. Courtesy of Envirex.

TABLE 11.2 Microstrainer Design Parameters[a]

Item	Typical value
Screen mesh	20–25 μm
Submergence	75% of height (66% of area)
Hydraulic loading	12–24 $m^3/m^2/h$ (300–600 $gal/ft^2/d$) of submerged drum surface area
Headloss through screen (h_L)	7.5–15 cm (3–6 in.)
Max. h_L	30–45 cm[b] (12–18 in.)
Peripheral drum speed	4.5 m/min at 7.5-cm h_L (15 ft/min at 3-in. h_L) 40–45 m/min at 15-cm h_L (130–150 ft/min at 6-in. h_L)
Typical drum diameter	3m (10 ft)
Washwater flow	2% of throughput at 345 kN/m^2 (50 psi) 5% of throughput at 100 kN/m^2 (14.5 psi)

[a]After USEPA (1975).
[b]Typical designs provide an overflow to bypass part of the flow when h_L exceeds 15–20 cm (6–8 in.).

degree of clogging, and time, and inversely proportional to the surface area (A) of the strainer. These parameters are incorporated into a first-order relation:

$$\frac{dh_L}{dt} = k\frac{Q}{A}h_L \tag{11.4}$$

where
k is a characteristic loss coefficient

The above equation integrates to:

$$h_L = h_0 e^{k\frac{Q}{A}t} \tag{11.5}$$

where
h_0 is the headloss of the clean strainer

The loss coefficient for the strainer should be experimentally determined.

Typical design parameters for solids removal from secondary effluents are given in Table 11.2. The USEPA (1975) surveyed a number of microstrainers treating secondary effluent with solids concentrations in the range of 6–65 mg/L and found average removals from 43 to 85%. Microstrainers also find application in the treatment of stormwater runoff and the polishing of effluents (removal of algae) from stabilization pond systems.