Substituting these data into the EPA model, Equation 5.226, gives

$$M_t = 0.0442aPfs = 0.0442(0.0336)(147)(0.72)(0.7) = 0.11 \text{ kg/ha}$$

For a 100-ha development, the annual phosphate load is 100 ha × 0.11 kg/ha = 11 kg. The average concentration in the runoff is obtained by dividing the annual load (= 11 kg) by the annual runoff. Equation 5.230 gives the depression storage, d, as

$$\begin{aligned} d &= 0.64 - 0.476\left(\frac{I}{100}\right) \\ &= 0.64 - 0.476\left(\frac{40}{100}\right) \\ &= 0.45 \text{ cm} \end{aligned}$$

and Equation 5.229 gives the runoff, R, as

$$\begin{aligned} R &= \left[0.15 + 0.75\left(\frac{I}{100}\right)\right]P - 3.004d^{0.5957} \\ &= \left[0.15 + 0.75\left(\frac{40}{100}\right)\right](147) - 3.004(0.45)^{0.5957} \\ &= 64 \text{ cm} \end{aligned}$$

Since the catchment area is 100 ha = 10^6 m^2, then the volume, V, of annual runoff is 0.64×10^6 m^3 and the average concentration, c, of PO_4 is given by

$$c = \frac{11 \text{ kg}}{0.64 \times 10^6 \text{ m}^3} = 1.7 \times 10^{-5} \text{ kg/m}^3 = 0.017 \text{ mg/L}$$

The USGS and EPA regression equations are useful in estimating the annual pollutant loads on receiving waterbodies. In contrast to estimating annual pollutant loads, individual storm-event pollutant loads are typically estimated using either regression equations or process-based water-quality models. The regression equations empirically relate the event loads to the storm and catchment characteristics (e.g., Jewell and Adrian, 1981, 1982; Driver and Lystrom, 1986; Driver and Tasker, 1988; Driver, 1990). The process-based water-quality models typically simulate the dry-weather accumulation of pollutants on the catchment surface and the subsequent washoff caused by surface runoff (e.g., Heany et al., 1976; Geiger and Dorsch, 1980; Johanson et al., 1984; Hemain, 1986; Huber and Dickinson, 1988). A comparative evaluation of urban stormwater-quality models concluded that, once calibrated, both regression equations and process-based models can estimate event pollutant loads satisfactorily (Vaze and Chiew, 2003). Therefore, if only estimates of event loads are required, versus contaminant concentrations during the runoff events, regression models should be used because they are simpler and require less data compared to process-based models. A detailed review of approaches for controlling the quality of surface runoff from urban catchments can be found in Chin (2006).

5.7 Design of Stormwater-Management Systems

Stormwater-management systems are designed to control the quantity, quality, timing, and distribution of runoff resulting from storm events. Other objectives in the design of stormwater-management systems include: erosion control, reuse storage, and ground-water recharge. A typical urban stormwater-management system has two distinct subsystems: a minor and a major one. The minor system consists of storm sewers that route the design runoff to receiving waters, and it is typically designed to handle runoff events with return periods of 2 to 10 years. Typical return periods for various types of service areas are given in Table 5.4. The major system consists of the above-ground conveyance routes that transport stormwater from larger runoff events with return periods from 25 to 100 years. Major urban conveyance systems that are covered by the National Flood Insurance Program (in the United States) are typically designed for a runoff with a 100-year return period.

5.7.1 Minor System

Most minor stormwater-management systems are designed for urban environments, and the principal hydraulic elements of the minor system are shown in Figure 5.45. The (minor) stormwater-management system collects surface runoff via inlets, and the surface runoff is routed to a treatment unit and/or receiving waterbody, usually through underground pipes called *storm sewers*. In some cases, the surface runoff is discharged directly into a receiving body of water such as a drainage canal. In some older U.S. and European cities, storm and sanitary sewers are combined into a single system; these are called *combined-sewer systems*.

5.7.1.1 Storm sewers

Storm sewers are typically located a short distance behind the curb, or in the roadway near the curb. These sewers should be straight between manholes (where possible); where curves are necessary to conform to street layout, the radius of curvature should not be less than 30 m. There should be at least 0.9 m (3 ft) of cover over the crowns of the sewer pipes to prevent excessive loading on the pipe, and crossings with underground utilities should be avoided whenever possible, but, if necessary, should be at an angle greater than 45°. Manholes, also called *clean-out structures* (ASCE, 1992) or *access holes* (USFHWA, 1996), are placed along the sewer pipeline to provide convenient access for inspection, maintenance, and repair of storm-drainage systems; they are normally located at the junctions of sewers, changes in grade or alignment, and where there are changes in pipe size. Manhole spacings depend on the pipe sizes, and typical maximum spacings are given in Table 5.33. Drop manholes (see Figure 3.53(b)) are provided for sewers entering a manhole at an elevation of 0.6 m or more above the manhole invert.

The rational method is commonly used to determine the peak flows to be handled by storm sewers. The flow calculations proceed from the most upstream pipe in the system and, with each new inlet (inflow), the pipe immediately downstream of the inlet is expected to carry the runoff from a storm of duration equal to the time of concentration of the contributing area. Two separate contributing areas must

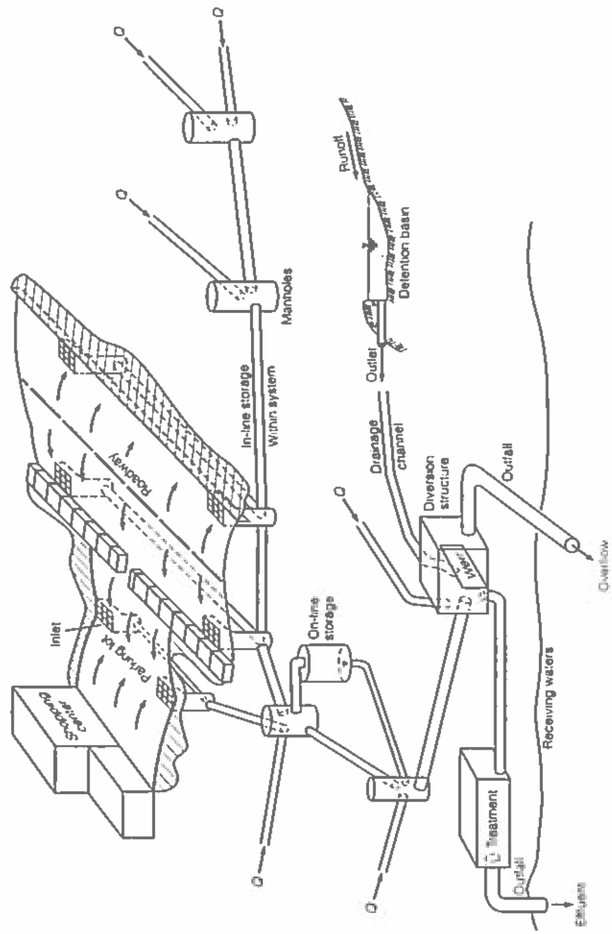


FIGURE 5.45: Principal hydraulic elements in (minor) stormwater management system.
Source: ASCE (1992).

TABLE 5.33: Typical Manhole Spacings

Pipe size	Maximum spacing
38 cm or less	122 m
46 cm to 91 cm	152 m
107 cm or greater	183 m

Source: Boulder County (1984).

be considered: (1) the entire contributing upstream area, and (2) the impervious upstream area directly connected to the inlets. Directly connected impervious area (DCIA) must be considered separately, since it typically has a considerably shorter time of concentration than the entire upstream contributing area, resulting in a higher design-rainfall intensity and possibly a higher peak runoff rate than the entire catchment. A minimum time of concentration such as 5 min is generally adopted to preclude unrealistically high design rainfall intensities. Most impervious areas are transportation related, mainly roads, driveways, and parking lots.

The minimization of directly connected impervious area is by far the most effective method of controlling the quality of surface runoff (ASCE, 1992) and, in many cases, DCIA is a key indicator of urbanization's effect on the quantity and quality of surface runoff (Lee and Heany, 2003; Jones et al., 2005). Typically, runoff from non-DCIA areas occurs only for larger storms, and the accurate estimation of DCIA is an important component of the cost-effective design and adequacy of roadway drainage systems (Aronica and Lanza, 2005a). Portions of roadways contributing to DCIA generally have curbs and gutters, while portions drained by roadside swales are usually not associated with DCIA. The procedure for calculating the design flows in storm sewers is illustrated in the following example.

EXAMPLE 5.44

Consider the two inlets and two pipes shown in Figure 5.46. Catchment A has an area of 1 ha and is 50% impervious; catchment B has an area of 2 ha and is 10% impervious.

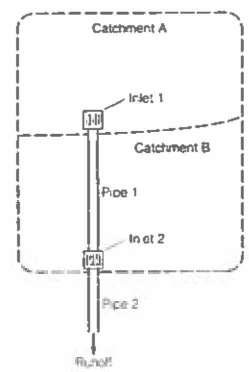


FIGURE 5.46: Computation of peak inlet and pipe flows
Adapted from ASCE (1992)

TABLE 5.34: Catchment Characteristics

Catchment	Surface	C	L (m)	n	S_0
A	Pervious	0.2	80	0.2	0.01
	Impervious	0.9	60	0.1	0.01
B	Pervious	0.2	140	0.2	0.01
	Impervious	0.9	65	0.1	0.01

All impervious areas are directly connected to the sewer inlets. The runoff coefficient, C; length of overland flow, L; roughness coefficient, n; and average slope, S_0, of the pervious and impervious surfaces in both catchments are given in Table 5.34. The design storm has a return period of 10 years, and the 10-year IDF curve can be approximated by

$$i = \frac{7620}{t + 36}$$

where i is the average rainfall intensity in mm/h and t is the duration of the storm in minutes. Calculate the peak flows to be handled by the inlets and pipes.

Solution Using the given IDF curve, the effective rainfall rate, i_e, is given by the rational formula as

$$i_e = Ci = C\frac{7620}{t_c + 36} \tag{5.231}$$

where C is the runoff coefficient. The duration, t, of the design storm is taken to be equal to the time of concentration, t_c, given by Equation 5.94 as

$$t_c = 6.99\frac{(nL)^{0.6}}{i_e^{0.4}S_0^{0.3}} \tag{5.232}$$

Simultaneous solution of Equations 5.231 and 5.232 using the catchment characteristics in Table 5.34 leads to the following times of concentration, t_c:

Catchment	Surface	t_c (min)
A	Pervious area	46
	Impervious area	11
B	Pervious	71
	Impervious	12

Consider now the flows at specific locations.

Inlet 1 and Pipe 1: When the entire catchment A is contributing, the time of concentration is 46 min (this is the time for both pervious and impervious areas to be fully contributing), the average rainfall rate, i, from the IDF curve is 92.9 mm/h ($= 2.58 \times 10^{-5}$ m/s), and the weighted-average runoff coefficient, $\overline{C}$, is given by

$$\overline{C} = 0.5(0.9) + 0.5(0.2) = 0.55$$

Since the area of the catchment is 1 ha ($= 10{,}000$ m^2), the peak runoff rate, Q_p, from the catchment is given by the rational formula as

$$Q_p = \overline{C}iA = (0.55)(2.58 \times 10^{-5})(10{,}000) = 0.142 \text{ m}^3\text{/s}$$

Considering only the impervious portion of the catchment, the time of concentration is 11 min, the average rainfall rate, i, from the IDF curve is 162 mm/h ($= 4.50 \times 10^{-5}$ m/s), the runoff coefficient, C, is 0.9, the contributing area is 0.5 ha ($= 5000$ m^2), and the peak runoff rate, Q_p, is given by

$$Q_p = CiA = (0.9)(4.50 \times 10^{-5})(5000) = 0.203 \text{ m}^3\text{/s}$$

The calculated peak runoff from the directly connected impervious area is greater than the calculated runoff from the entire area, and therefore the design flow to be handled by Inlet 1 and Pipe 1 is controlled by the directly connected impervious area and is equal to 0.203 m^3/s.

Inlet 2: When the entire catchment B is contributing, the time of concentration is 71 min, the average rainfall rate, i, from the IDF curve is 71.2 mm/h ($= 1.98 \times 10^{-5}$ m/s), and the weighted average runoff coefficient, $\overline{C}$, is given by

$$\overline{C} = 0.1(0.9) + 0.9(0.2) = 0.27$$

Since the area of the catchment is 2 ha ($= 20{,}000$ m^2), then the peak runoff rate, Q_p, from the catchment is given by the rational formula as

$$Q_p = \overline{C}iA = (0.27)(1.98 \times 10^{-5})(20{,}000) = 0.107 \text{ m}^3\text{/s}$$

Considering only the impervious portion of the catchment, the time of concentration is 12 min, the average rainfall rate, i, from the IDF curve is 159 mm/h ($= 4.41 \times 10^{-5}$ m/s), the runoff coefficient, C, is 0.9, the contributing area is 0.2 ha ($= 2000$ m^2), and the peak runoff rate, Q_p, is given by

$$Q_p = CiA = (0.9)(4.41 \times 10^{-5})(2000) = 0.079 \text{ m}^3\text{/s}$$

The calculated peak runoff from the directly connected impervious area is less than the calculated runoff from the entire area, and therefore the design flow to be handled by Inlet 2 is controlled by the entire catchment and is equal to 0.107 m^3/s.

Pipe 2: First consider the case where the entire tributary area of 3 ha ($= 30{,}000$ m^2) is contributing runoff to pipe 2. The time of concentration of catchment A is equal to 46 min plus the time of flow in pipe 1, which, in lieu of hydraulic calculations, can be taken as 2 min. Therefore, the time of concentration of catchment A is 48 min. The time of concentration of catchment B is 71 min, and therefore the time of concentration of the entire tributary area to pipe 2 (including both catchments A and B) is equal to 71 min. The average rainfall intensity corresponding to this duration (from the IDF curve) is 71.2 mm/h ($= 1.98 \times 10^{-5}$ m/s); the area-weighted runoff coefficient, $\overline{C}$, is given by

$$\overline{C} = \frac{1}{3}[(0.5 + 0.2)(0.9) + (0.5 + 1.8)(0.2)] = 0.36$$

and the rational formula gives the peak runoff rate, Q_p, as

$$Q_p = \overline{C}iA = (0.36)(1.98 \times 10^{-5})(30{,}000) = 0.214 \text{ m}^3\text{/s}$$

Considering only the impervious portions of catchments A and B, the contributing area is 0.7 ha ($= 7000$ m^2), the time of concentration is 13 min (equal to the time of concentration for Inlet 1 plus travel time of 2 min in pipe), the corresponding average rainfall intensity from the IDF curve is 156 mm/h ($= 4.32 \times 10^{-5}$), the runoff coefficient is 0.9, and the rational formula gives a peak runoff, Q_p, of

$$Q_p = CiA = (0.9)(4.32 \times 10^{-5})(7000) = 0.272 \text{ m}^3/\text{s}$$

Therefore, the peak runoff rate calculated by using the entire catchment is less than the peak runoff rate calculated by considering only the directly connected impervious portion of the tributary area. The design flow to be handled by pipe 2 is therefore controlled by the directly connected impervious area and is equal to 0.272 m³/s.

Most storm sewers are sized to flow full at the design discharge, although where ground elevations are sufficient, a limited surcharge above the pipe crown may be permitted (ASCE, 1992). To prevent deposition of suspended materials, the minimum slope of the sewer should produce a velocity of at least 60 to 90 cm/s (2 to 3 ft/s) when the sewer is flowing full; to prevent scouring, the velocity should be less than 3 to 4.5 m/s (10 to 15 ft/s). The appropriate flow equation for sizing storm sewers is the Darcy–Weisbach equation, but it is also common practice to use the Manning equation. Caution should be exercised when using the Manning equation, which is valid only for hydraulically rough (fully turbulent) flow and is appropriate only when the following condition is satisfied (French, 1985):

$$n^6\sqrt{RS_0} \geq 9.6 \times 10^{-14} \tag{5.233}$$

where n is the Manning roughness coefficient, R is the hydraulic radius of the pipe (in meters), and S_0 is the slope of the pipe. Manning roughness coefficients recommended for closed-conduit flow are given in Table 5.35. In cases where the Darcy–Weisbach equation is used, the pipe roughness is commonly assumed to be independent of the pipe material (due to the accumulation of slime) and a value of 0.6 mm is typically

TABLE 5.35: Manning Coefficient in Closed Conduits

Material	n
Asbestos-cement pipe	0.011–0.015
Brick	0.013–0.017
Cast-iron pipe (cement lined and seal coated)	0.011–0.015
Concrete (monolithic):	
Smooth forms	0.012–0.014
Rough forms	0.015–0.017
Concrete pipe	0.011–0.015
Corrugated metal pipe (1.3-cm × 6.4-cm corrugations):	
Plain	0.022–0.026
Paved invert	0.018–0.022
Spun asphalt lined	0.011–0.015
Plastic pipe (smooth)	0.011–0.015
Vitrified clay:	
Pipes	0.011–0.015
Liner plates	0.013–0.017

Source: ASCE (1982).

recommended (Butler and Davies, 2000). In cases where the pipe material has a larger roughness height than 0.6 mm, the roughness height of the pipe material should be used.

In the hydraulic design of sewer pipes, the basic objective is to calculate the size and slope of the pipes that will carry the design flows at velocities that are within a specified range and with flow depths that are less than or equal to the diameter of the pipes. In most situations, it can be assumed that the flow is uniform and any losses other than pipe friction can be accounted for by assuming point losses at each manhole. In calculating the diameter, D, of storm sewers, the Manning equation can be put in the convenient form

$$\boxed{D = \left(\frac{3.21Qn}{\sqrt{S_0}}\right)^{\frac{3}{8}}} \tag{5.234}$$

where Q is the design flowrate in m³/s and D is in meters. If the Darcy–Weisbach equation is used, the convenient form is

$$\boxed{D = \left(\frac{0.811fQ^2}{gS_0}\right)^{\frac{1}{5}}} \tag{5.235}$$

where f is the friction factor. The actual size of the pipe to be used should be the next larger commercial size than calculated using either Equation 5.234 or 5.235. Concrete, asbestos-cement, and clay pipes are commonly used for diameters between 10 cm (4 in.) and 60 cm (25 in.), with reinforced or prestressed concrete pipes commonly used for diameters larger than 60 cm (Novotny et al., 1989). It is generally recommended to choose a pipe diameter larger than 30 cm (12 in.) to prevent clogging and facilitate maintenance (Gribbin, 1997).

Junction and manhole losses usually have a significant effect on flows in sewers. Junctions are locations where two or more pipes join together and enter another pipe or channel, and these transitions need to be smooth to avoid high head losses. Conditions that promote turbulent flow and associated high head losses include a large angle between the incoming pipes (>60°), a large vertical distance between the pipes (>15 cm between the two inverts), and the absence of a semicircular channel at the bottom of the manhole (ASCE, 1992). Manholes are generally placed at sewer junctions, as well as at other locations, to permit access to the entire pipeline system. The head loss, h_m, in manholes can be estimated using the equation

$$\boxed{h_m = K_c \frac{V^2}{2g}} \tag{5.236}$$

where K_c is a head-loss coefficient and V is the average velocity in the inflow pipe. Head-loss coefficients for manholes with single inflow and outflow pipes aligned opposite to each other vary between 0.12 and 0.32, while the loss coefficients vary between 1.0 and 1.8 for inflow and outflow pipes at 90° to each other (ASCE, 1992). To maintain the specific energy of the flow, a minimum of 3 to 6 cm drop in the sewer invert at manholes is advisable. Under no conditions should the crown of the upstream pipe be lower than the crown of the downstream pipe. In cases where the pipe diameter increases, it is recommended that the crowns of the upstream and downstream pipes be aligned.

EXAMPLE 5.45

A concrete sewer pipe is to be laid parallel to the ground surface on a slope of 0.5% and is to be designed to carry 0.43 m^3/s of stormwater runoff. Estimate the required pipe diameter using (a) the Manning equation, and (b) the Darcy–Weisbach equation. If service manholes are placed along the pipeline, estimate the head loss at each manhole.

Solution From the given data: $S_0 = 0.005$, $Q = 0.43$ m^3/s, and $n = 0.013$ (Table 5.35, average for concrete pipe).

(a) The Manning equation (Equation 5.234) gives

$$D = \left[\frac{3.21Qn}{\sqrt{S_0}}\right]^{\frac{3}{8}} = \left[\frac{3.21(0.43)(0.013)}{\sqrt{0.005}}\right]^{\frac{3}{8}} = 0.60\ \text{m}$$

Use the limitation given by Equation 5.233 to check whether the Manning equation is valid:

$$n^6\sqrt{RS_0} = (0.013)^6\sqrt{(0.6/4)(0.005)} = 1.3 \times 10^{-13} \geq 9.6 \times 10^{-14}$$

Therefore, the Manning equation is valid. Using a 60-cm pipe, the flow velocity, V, is given by

$$V = \frac{Q}{A} = \frac{0.43}{\frac{\pi}{4}(0.6)^2} = 1.52\ \text{m/s}$$

This velocity exceeds the minimum velocity to prevent sedimentation (0.60 to 0.90 m/s) and is less than the maximum velocity to prevent excess scour (3 to 4.5 m/s). According to the Manning equation, the pipe should have a diameter of 60 cm.

(b) The Darcy–Weisbach equation (Equation 5.235) gives

$$D = \left[\frac{0.811fQ^2}{gS_0}\right]^{\frac{1}{5}} = \left[\frac{0.811f(0.43)^2}{(9.81)(0.005)}\right]^{\frac{1}{5}} = 1.250f^{\frac{1}{5}}$$

which can be put in the form

$$f = 0.328D^5 \tag{5.237}$$

The friction factor, f, also depends on D via the Colebrook equation (Equation 2.35) which is given by

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{k_s/D}{3.7} + \frac{2.51}{\text{Re}\sqrt{f}}\right) \tag{5.238}$$

The equivalent sand roughness, k_s, of concrete is in the range 0.3–3.0 mm (Table 2.1) and can be taken as $k_s = 1.7$ mm. Assuming that the temperature of the water is 20°C, the kinematic viscosity, ν, is equal to 1.00×10^{-6} m/s^2, and the Reynolds number, Re, is given by

$$\text{Re} = \frac{VD}{\nu} = \frac{4Q}{\pi D\nu} = \frac{4(0.43)}{\pi D(1.00 \times 10^{-6})} = \frac{5.48 \times 10^5}{D} \tag{5.239}$$

Combining Equations 5.237 to 5.239 gives

$$\frac{1}{\sqrt{0.328D^5}} = -2\log\left(\frac{0.0017/D}{3.7} + \frac{2.51}{\frac{5.48\times10^5}{D}\sqrt{0.328D^5}}\right)$$

which simplifies to

$$1.75D^{-5/2} = -2\log(4.59 \times 10^{-4}D^{-1} + 8.00 \times 10^{-6}D^{-3/2})$$

and yields

$$D = 0.60\ \text{m} = 60\ \text{cm}$$

Therefore, the Darcy–Weisbach equation requires that the sewer pipe be at least 60 cm in diameter.

The service manholes placed along the pipe will each cause a head loss, h_m, where

$$h_m = K_c\frac{V^2}{2g}$$

For inflow and outflow pipes aligned opposite to each other, K_c is between 0.12 and 0.32 and can be assigned an average value of $K_c = 0.22$. Since $V = 1.52$ m/s, the head loss, h_m, is therefore given by

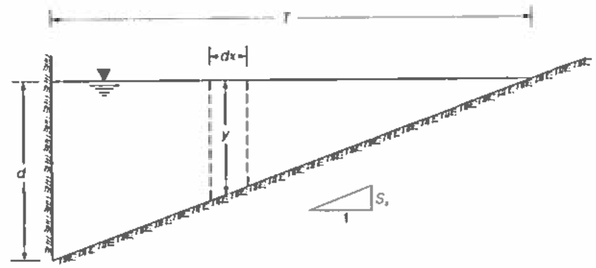
$$h_m = 0.22\frac{(1.52)^2}{2(9.81)} = 0.026\ \text{m} = 0.26\ \text{cm}$$

This head loss must be accounted for in computing the energy grade line in the sewer system.

5.7.1.2 Street gutters and inlets

Surface runoff from urban streets are typically routed to sewer pipes through street gutters and inlets. To facilitate drainage, urban roadways are designed with both cross-slopes and longitudinal slopes. The cross slope directs the surface runoff to the sides of the roadway, where the pavement intersects the curb and forms an open channel called a *gutter*. Longitudinal slopes direct the flow in the gutters to stormwater *inlets* that direct the flow into sewer pipes. Typical cross slopes on urban roadways are in the range of 1.5% to 6%, depending on the type of pavement surface (Easa, 1995), and typical longitudinal slopes are in the range of 0.5% to 5%, depending on the topography. The spacing between stormwater inlets depends on several criteria, but it is usually controlled by the allowable water spread toward the crown of the street.

FIGURE 5.47: Triangular curb gutter



Flow in a triangular curb gutter is illustrated in Figure 5.47. The incremental flow, dQ, through any gutter width dx can be estimated using the relation

$$dQ = Vy\,dx \tag{5.240}$$

where V is the average velocity and y is the depth of flow within an elemental flow area of width dx. Using the Manning equation, the average velocity within the elemental flow area can be estimated by

$$V = \frac{1}{n} y^{2/3} S_0^{1/2} \tag{5.241}$$

where n is the Manning roughness coefficient, and S_0 is the longitudinal slope of the gutter. If S_x is the cross slope of the gutter, then

$$\frac{dy}{dx} = S_x \tag{5.242}$$

Combining Equations 5.240 to 5.242 gives

$$dQ = \frac{1}{n} y^{5/3} \frac{S_0^{1/2}}{S_x}\,dy \tag{5.243}$$

Since the flow depth, y, varies from 0 to d across the gutter, the total flow, Q, in the gutter is given by

$$Q = \int_0^d \frac{1}{n} y^{5/3} \frac{S_0^{1/2}}{S_x}\,dy \tag{5.244}$$

which yields (ASCE, 1992; USFHWA, 1996)

$$\boxed{Q = 0.375\left(\frac{1}{nS_x}\right) d^{8/3} S_0^{1/2}} \tag{5.245}$$

Equation 5.245 is viewed as preferable to the direct application of the Manning equation to the gutter, since the hydraulic radius does not adequately describe the gutter cross section, particularly when the top-width T exceeds 40 times the depth at the curb (ASCE, 1992). A limitation of Equation 5.245 is that it neglects the resistance

TABLE 5.36: Typical Manning n Values for Street and Pavement Gutters

Type of gutter or pavement	Manning n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016

Source: USFHWA (1984b).

of the curb face, which is negligible if the cross slope is 10% or less (Johnson and Chang, 1984). The depth and top-width of gutter flow are related by

$$d = TS_x \tag{5.246}$$

Typical Manning n values for street and pavement gutters are given in Table 5.36 (USFHWA, 1984b), and a Manning n of 0.016 is recommended for most applications (Guo, 2000). To facilitate proper drainage, it is recommended that the gutter grade exceed 0.4% and the street cross slope exceed 2% (ASCE, 1992). A gutter cross slope may be the same as that of the pavement or may be designed to be steeper. The gutter, together with a curb, should be at least 15 cm (6 in.) deep and 60 cm (2 ft) wide, with the deepest portion adjacent to the curb. The maximum allowable width of street flooding depends on the type of street and is usually specified separately for minor and major design-runoff events. Typical regulatory requirements for allowable pavement encroachment are given in Table 5.37, and these typically correspond to rainfall events with a return period of 10 years. Allowable pavement encroachment and design return period are the bases for computing the street drainage capacity using the modified Manning equation (Equation 5.245). The flowrate corresponding to the allowable pavement encroachment is commonly called the *street hydraulic conveyance capacity*.

EXAMPLE 5.46

A four-lane collector roadway is to be constructed with 3.66-m (12-ft) lanes, a cross slope of 2%, a longitudinal slope of 0.5%, and pavement made of rough asphalt. The (minor) roadway-drainage system consists of curbs and gutters and is to be designed for a rainfall intensity of 150 mm/h. Determine the spacing of the inlets.

Solution For a collector street, Table 5.37 indicates that at least one lane must be free of water. However, since the roadway has four lanes, the drainage system must necessarily be designed to leave two lanes free of water (one on each side of the crown). Since each lane is 3.66 m wide, the allowable top-width, T, is 3.66 m, with n = 0.016 (rough asphalt), S_x = 0.02, and S_0 = 0.005. The maximum allowable depth of flow, d, at the curb is given by

$$d = TS_x = (3.66)(0.02) = 0.0732 \text{ m} = 7.32 \text{ cm}$$

TABLE 5.37: Typical Regulatory Requirements for Pavement Encroachment

Street type	Minor storm runoff	Major storm runoff
Local*	No curb overtopping;† flow may spread to crown of street	Residential dwellings, public, commercial and industrial buildings shall not be inundated at the ground line, unless buildings are floodproofed. The depth of water over the gutter flow line shall not exceed an amount specified by local regulation, often 30 cm (12 in.).
Collector‡	No curb overtopping;† flow spread must leave at least one lane free of water	Same as for local streets
Arterial§	No curb overtopping;† flow spread must leave at least one lane free of water in each direction	Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line, unless buildings are floodproofed. Depth of water at the street crown shall not exceed 15 cm (6 in.) to allow operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed a locally prescribed amount.
Freeway**	No encroachment allowed on any traffic lanes	Same as for arterial streets

Source: Denver Regional Urban Storm Drainage Criteria Manual (1984).
*A local street is a minor traffic carrier within a neighborhood characterized by one or two moving lanes and parking along curbs. Traffic control may be by stop or yield signs.
†Where no curb exists, encroachment onto adjacent property should not be permitted.
‡A collector street collects and distributes traffic between arterial and local streets. There may be two or four moving traffic lanes and parking may be allowed adjacent to curbs.
§An arterial street permits rapid and relatively unimpeded traffic movement. There may be four to six lanes of traffic, and parking adjacent to curbs may be prohibited. The arterial traffic normally has the right-of-way over collector streets. An arterial street will often include a median strip with traffic channelization and signals at numerous intersections.
**Freeways permit rapid and unimpeded movement of traffic through and around a city. Access is normally controlled by interchanges at major arterial streets. There may be eight or more traffic lanes, frequently separated by a median strip.

and the maximum allowable flowrate, Q, in the gutter is given by the Manning equation (Equation 5.245) as

$$Q = 0.375\left[\frac{1}{nS_x}\right]d^{8/3}S_0^{1/2} = 0.375\left[\frac{1}{(0.016)(0.02)}\right](0.0732)^{8/3}(0.005)^{1/2}$$
$$= 0.0777\ \text{m}^3/\text{s}$$

Since the design-rainfall intensity is 150 mm/h = 4.17×10^{-5} m/s, the contributing area, A, required to produce a runoff of 0.0777 m^3/s is given by

$$A = \frac{0.0777}{4.17 \times 10^{-5}} = 1863\ \text{m}^2$$

The roadway has two lanes contributing runoff to each gutter. Therefore, the width of the contributing area is $2 \times 3.66 = 7.32$ m, and the length, L, of roadway required for a contributing area of 1863 m^2 is given by

$$L = \frac{1863}{7.32} = 255\ \text{m}$$

The required spacing of inlets is therefore 255 m. This is a rather large spacing, and assumes that 100% of the gutter flow is intercepted by each inlet. In reality, the inlets will probably not be designed to intercept all the gutter flow, resulting in some carryover and required spacings less than 255 m. The requirement that inlets be placed at immediately upgrade of intersections, at pedestrian cross walks, upstream of bridges, and at vertical sag locations may also affect the actual spacing of inlets in the gutter.

There are a number of locations where inlets are required, regardless of the contributing drainage area. These locations are called *geometric controls*, since they are determined by the geometry of the roadway system, and must generally be marked on drainage plans before any computations of runoff, roadway encroachment, and inlet capacity. Examples of geometric-control locations are (Young and Stein, 1999):

- At all sag points in the gutter grade
- Immediately upstream of median breaks, entrance/exit ramps, crosswalks, and street intersections (i.e., at any location where water could flow onto the roadway)
- Immediately upgrade of bridges (to prevent pavement drainage from flowing onto bridge decks)
- Immediately downstream of bridges (to intercept bridge deck drainage)
- Immediately upgrade of cross-slope reversals
- At the end of channels in cut sections
- On side streets immediately upgrade from intersections
- Behind curbs, shoulders, or sidewalks to drain low areas

In addition to these areas, stormwater from cut slopes and adjacent regions draining toward the roadway should be intercepted before it reaches the pavement. In practice, inlet locations are frequently dictated by street-geometrical conditions rather than spread-of-water computations.

Stormwater inlets can take many forms but are usually classified as either curb inlets, grate inlets, combination inlets, or slotted drains. The various inlet types are illustrated in Figure 5.48. Municipalities sometimes specify the manufacturer and specific inlet types that are acceptable within their jurisdiction. Since it is usually uneconomical to make stormwater inlets as wide as the design spread on roadways, inlets on continuous grades typically intercept only a portion of the gutter flow, and the fraction of flow intercepted under design conditions is called the *inlet efficiency*, E, and is given by

$$E = \frac{Q_i}{Q} \tag{5.247}$$

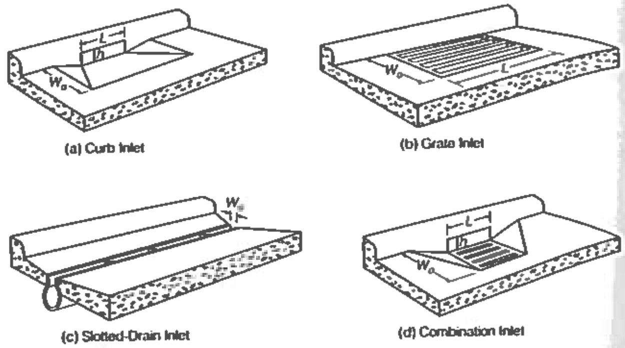


FIGURE 5.48: Stormwater inlets. Source: USFHWA (1984a).

where Q_i is the intercepted flow, and Q is the total gutter flow. The flow that is not intercepted by an inlet is called the *bypass flow* or *carryover flow*. The efficiency of inlets in passing debris is critical in sag locations, since all runoff entering the sag must be passed through the inlet. For this reason, grate inlets are not recommended in sag locations, because of their tendency to become clogged, and curb-opening or combination inlets are recommended for use in these locations.

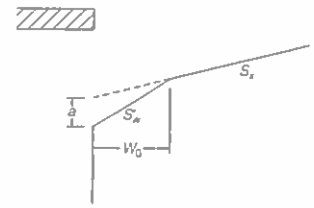
Curb inlets. Curb inlets are vertical openings in the curb covered by a top slab. The capacity of a curb inlet depends on the size of the inlet, the amount of debris blockage, whether the inlet is depressed, and whether deflectors are used. Details on the performance of curb inlets can be found in regulatory manuals such as the Federal Highway Administration Urban Drainage Design Manual (USFHWA, 1996) and the Denver Regional Urban Storm Drainage Criteria Manual (1984). Curb-opening heights vary in dimension, with typical opening heights in the range of 10 cm to 15 cm (4 in. to 6 in.). Opening heights should not exceed 15 cm (6 in.) to reduce the risk of children entering the inlet. Curb inlets are most effective on flatter slopes (less than 3%), in sags, and with gutter flows that carry significant amounts of floating debris (USFHWA, 1996).

On continuous grades, the length, L_T, of a curb inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is given by (USFHWA, 1996)

$$\boxed{L_T = 0.817Q^{0.42}S_0^{0.3}\left(\frac{1}{nS_x}\right)^{0.6}} \tag{5.248}$$

where L_T is in meters, Q is the gutter flow in m³/s, S_0 is the longitudinal slope of the gutter, n is the Manning roughness coefficient, and S_x is the cross slope. For curb inlets shorter than L_T, the ratio, R, of the intercepted flow to the gutter flow is given

FIGURE 5.49: Depressed curb inlet



by (USFHWA, 1996)

$$\boxed{R = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}} \tag{5.249}$$

where L is the length of the curb inlet in the same units as L_T. In cases of a depressed curb inlet, shown in Figure 5.49, Equations 5.248 and 5.249 can still be used to calculate the inlet capacity, with the cross slope, S_x, replaced by the *equivalent cross slope*, S_e, given by

$$S_e = S_x + S_w' R_w \tag{5.250}$$

where S_x is the cross slope of the pavement shown in Figure 5.49, and S_w' is the cross slope of the gutter measured from the cross slope of the pavement, given by

$$S_w' = \frac{a}{W_0} \tag{5.251}$$

where a is the gutter depression, and W_0 is the width of the depressed gutter, and R_w is the ratio of the frontal flow to the depressed section to the total gutter flow given by (USFHWA, 1996)

$$R_w = 1 - \left(1 - \frac{W_0}{T}\right)^{8/3} \tag{5.252}$$

EXAMPLE 5.47

A smooth-asphalt roadway has a cross slope of 3%, a longitudinal slope of 2%, a curb height of 15 cm, and a 90-cm wide concrete gutter. If the gutter flow is 0.08 m³/s, determine the length of a 12-cm high curb inlet that is required to remove all the water from the gutter. Consider the cases where: (a) there is an inlet depression of 25 mm; and (b) there is no inlet depression. What inlet length would remove 80% of the gutter flow?

Solution From the given data: Q = 0.08 m³/s, S_x = 3% = 0.03, and S_0 = 2% = 0.02. Assuming that a significant portion of the gutter flow extends onto the smooth-asphalt pavement, Table 5.36 gives n = 0.013. Using the Manning equation

(Equation 5.245) to find the depth, d, at the curb gives

$$Q = 0.375\left(\frac{1}{nS_x}\right)d^{8/3}S_0^{1/2}$$

$$0.08 = 0.375\left(\frac{1}{0.013 \times 0.03}\right)d^{8/3}(0.02)^{1/2}$$

and solving for d leads to

$$d = 0.061 \text{ m} = 6.1 \text{ cm}$$

This flow depth is less than the curb height of 15 cm, and therefore the flow is constrained within the roadway. The top-width, T, of the gutter flow, is given by

$$T = \frac{d}{S_x} = \frac{0.061}{0.03} = 2.03 \text{ m}$$

The spread of 2.03 m is much larger than the gutter width of 90 cm, verifying the assumption that a significant portion of the gutter flow extends onto the smooth-asphalt pavement, and hence justifying the assumed n value of 0.013. For an inlet width, W_0, of 0.90 m, the ratio, R_w, of frontal flow to the total gutter flow is given by Equation 5.252 as

$$R_w = 1 - \left(1 - \frac{W_0}{T}\right)^{8/3}$$

$$= 1 - \left(1 - \frac{0.90}{2.03}\right)^{8/3} = 0.79$$

(a) For a gutter depression, a, of 25 mm (= 0.025 m), the cross slope, S'_w, of the gutter relative to the pavement slope is given by Equation 5.251 as

$$S'_w = \frac{a}{W_0} = \frac{0.025}{0.90} = 0.028$$

Hence the equivalent cross slope, S_e, of the depressed inlet is given by Equation 5.250 as

$$S_e = S_x + S'_w R_w$$

$$= 0.03 + (0.028)(0.79) = 0.052$$

The length, L_T, of the curb inlet required to intercept all of the gutter flow is given by Equation 5.248, with S_e replacing S_x, as

$$L_T = 0.817Q^{0.42}S_0^{0.3}\left(\frac{1}{nS_e}\right)^{0.6}$$

$$= 0.817(0.08)^{0.42}(0.02)^{0.3}\left(\frac{1}{0.013 \times 0.052}\right)^{0.6}$$

$$= 6.98 \text{ m}$$

(b) If there is no inlet depression, then $S_e = S_x = 0.03$ and the length, L_T, of the curb inlet required to intercept all of the gutter flow is given by Equation 5.248 as

$$L_T = 0.817Q^{0.42}S_0^{0.3}\left(\frac{1}{nS_x}\right)^{0.6}$$

$$= 0.817(0.08)^{0.42}(0.02)^{0.3}\left(\frac{1}{0.013 \times 0.03}\right)^{0.6}$$

$$= 9.71 \text{ m}$$

It is interesting to note that the required length of the curb inlet without a depression (= 9.71 m) is 39% longer than the required length with a depressed inlet (= 6.98 m).

The fraction, R, of gutter flow removed by an inlet of length L is given by Equation 5.249. When $R = 80\% = 0.80$, Equation 5.249 gives

$$0.80 = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$

Solving for L/L_T gives

$$\frac{L}{L_T} = 0.59$$

Therefore, the length of the curb inlet required to intercept 80% of the flow is 0.59(6.98 m) = 4.12 m for a depressed inlet, and 0.59(9.71 m) = 5.73 m for an undepressed inlet.

In sag vertical-curve locations, curb inlets act as weirs up to a depth equal to the opening height, and the inlet operates as an orifice when the water depth is greater than 1.4 times the opening height. Between these depths, transition between weir and orifice flow occurs. The weir flow equation gives the flowrate, Q_i (m^3/s), into the curb inlet as (USFHWA, 1996)

$$\boxed{Q_i = 1.25(L + 1.8W_0)d^{1.5}} \tag{5.253}$$

where L is the length of the curb opening (m), W_0 is the width of the inlet depression (m), and d is the depth of flow at the curb upstream of the gutter depression. The weir equation, Equation 5.253, is valid when

$$d \leq h + a \tag{5.254}$$

where h is the height of the curb opening, and a is the depth of the gutter depression. Without a depressed gutter, the inflow to a curb inlet is given by

$$\boxed{Q_i = 1.60Ld^{1.5}, \quad d \leq h} \tag{5.255}$$

For curb-opening lengths greater than 3.6 m, Equation 5.255 for nondepressed inlets gives inlet capacities greater than those calculated using Equation 5.253 for depressed inlets. Since depressed inlets will perform at least as well as nondepressed inlets of the same length, Equation 5.255 should be used for all curb-opening inlets having lengths greater than 3.6 m (USFHWA, 1996). When the flow depth, d, exceeds 1.4 times the opening height, h, the inflow to the curb inlet is given by the orifice equation

$$\boxed{Q_i = 0.67A\left[2g\left(d - \frac{h}{2}\right)\right]^{\frac{1}{2}}} \tag{5.256}$$

where A is the area of the curb opening ($= hL$) and d is the flow depth in the depressed gutter.

EXAMPLE 5.48

A roadway has a flow depth of 8 cm in a 60-cm wide gutter, and a corresponding flowrate of 0.08 m³/s. If the gutter flow drains into a curb inlet in a sag location, determine the length of a 15-cm (6-in.) high inlet that is required to remove all the water from the gutter. Consider the cases where (a) the inlet is depressed, and (b) the inlet is not depressed.

Solution

(a) Since the flow depth (8 cm) is less than the height of the inlet (15 cm), the curb inlet acts as a weir. In this case, $Q_i = 0.08$ m³/s, $W_0 = 0.6$ m, $d = 0.08$ m, and the weir equation (Equation 5.253) can be put in the form

$$L = \frac{Q_i}{1.25d^{1.5}} - 1.8W_0$$

The required weir length, L, is therefore given by

$$L = \frac{0.08}{1.25(0.08)^{1.5}} - 1.8(0.6) = 1.75 \text{ m}$$

With an inlet depression, the length of the curb opening should be at least 1.75 m.

(b) In the case of no inlet depression, the inlet equation (Equation 5.255) can be put in the form

$$L = \frac{Q_i}{1.60d^{1.5}}$$

and the required weir length is given by

$$L = \frac{0.08}{1.60(0.08)^{1.5}} = 2.21 \text{ m}$$

The presence of an inlet depression reduces the required curb length from 2.21 m to 1.75 m, a reduction of 21%.

Grate inlets. Grate inlets consist of an opening in the gutter covered by one or more grates. The main advantage of grate inlets are that they are installed along the roadway where water is flowing, and their main disadvantage is interference with bicycles and the tendency for debris blockage. If clogging due to debris is not expected, then a grate or grate/curb combination type inlet will provide more capacity than a curb

TABLE 5.38: Grate Inlets

Name	Description
P-50	Parallel-bar grate with bar spacing 48 mm on center.
P-50 × 100	Parallel-bar grate with bar spacing 48 mm on center and 10-mm diameter lateral rods spaced at 102 mm on center.
P-30	Parallel-bar grate with 29-mm-on-center bar spacing.
Curved vane	Curved-vane grate with 83-mm longitudinal bar and 108-mm transverse-bar spacing on center.
45°–60 Tilt bar	45° tilt-bar grate with 57-mm longitudinal bar and 102-mm transverse-bar spacing on center.
45°–85 Tilt bar	45° tilt-bar grate with 83-mm longitudinal bar and 102-mm transverse-bar spacing on center.
30°–85 Tilt bar	30° tilt-bar grate with 83-mm longitudinal bar and 102-mm transverse-bar spacing on center.
Reticuline	"Honeycomb" pattern of lateral bars and longitudinal bearing bars.

inlet. Grates typically consist of longitudinal and/or transverse bars oriented parallel and perpendicular to the gutter flow, respectively, and design procedures have been developed for the grates listed in Table 5.38 (USFHWA, 1996). The P-50 grate has been found to be unsafe for bicycle traffic (Burgi, 1978; Nicklow, 2001). Grates are typically available with longitudinal dimensions in the range 610 mm to 1220 mm (2 ft to 4 ft) and transverse dimensions in the range 381 mm to 914 mm (1 ft to 3 ft). Typical P-50×100 and reticuline grates are shown in Figure 5.50.

The type of grate to be used in any area is usually specified by local municipal codes. In cases where there is some flexibility in grate selection, it is instructive to note that the top five grates for debris-handling efficiency are, in rank order (USFHWA, 1996): curved vane, 30°–85 tilt bar, 45°–85 tilt bar, P-50, and P-50×100.

Grated inlets on continuous grades intercept a portion of the frontal flow and a portion of the side flow. *Splash-over* occurs when a portion of the frontal flow passes directly over the inlet, and the ratio, R_f, of the frontal flow intercepted by the inlet to the total frontal flow is called the *frontal-flow interception efficiency*, which can be estimated using the relation (USFHWA, 1996)

$$R_f = 1 - 0.295(V - V_0) \tag{5.257}$$

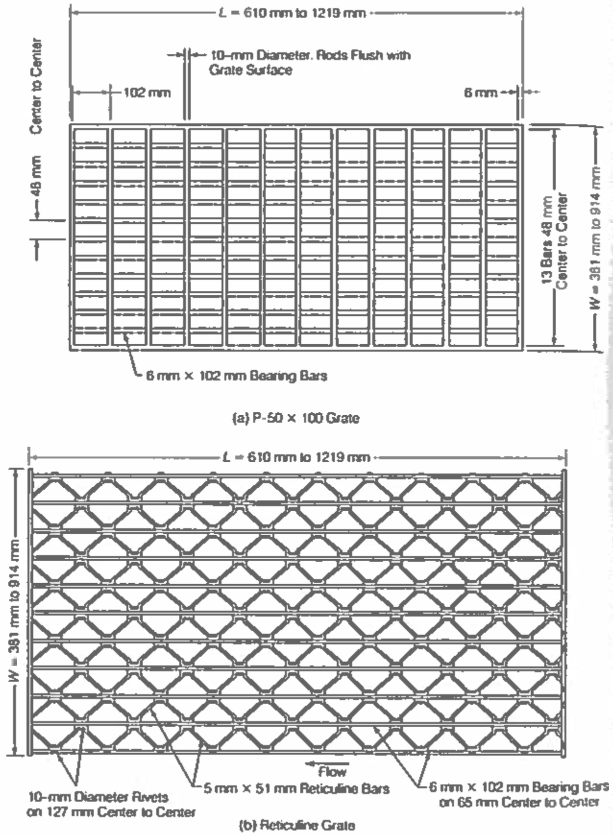
where V is the gutter flow velocity, and V_0 is the critical gutter velocity where splash-over first occurs. The *splash-over velocity*, V_0, depends on the length and type of inlet in accordance with the experimental relations given in Figure 5.51. The ratio, R_s, of the side flow intercepted to the total side flow is called the *side-flow interception efficiency*, and can be estimated using

$$R_s = \left[1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right]^{-1} \tag{5.258}$$

where S_x is the cross slope of the gutter, and L is the length of the grate inlet. The Manning equation (Equation 5.245) gives the ratio of frontal flow to total gutter flow, R_w, by

$$R_w = 1 - \left(1 - \frac{W_0}{T}\right)^{8/3} \tag{5.259}$$

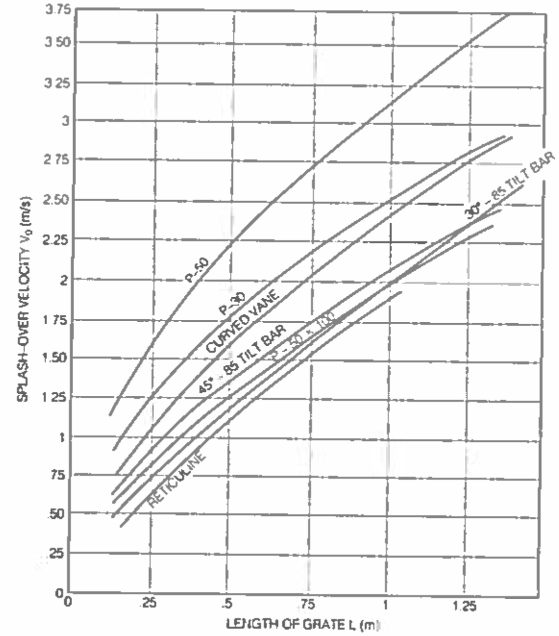
FIGURE 5.50: Types of grates



where W_0 is the frontal width of the grate inlet and T is the top-width of flow in the gutter. This is the same as Equation 5.252 that was used to calculate the frontal flow over a width W_0. Combining Equations 5.257 to 5.259 gives the ratio, R, of the flow intercepted by the grate inlet to the total gutter flow as

$$\boxed{R = R_f R_w + R_s(1 - R_w)} \tag{5.260}$$

FIGURE 5.51: Splash-over velocities at grate inlets



Therefore, if the flow in the gutter is Q, the amount of flow intercepted by the grate inlet is RQ.

EXAMPLE 5.49

A smooth-asphalt roadway has a cross slope of 3%, a longitudinal slope of 2%, a curb height of 8 cm, and a 90-cm wide concrete gutter. If the gutter flow is estimated to be 0.08 m³/s, determine the size and interception capacity of a P-50 × 100 grate that should be used to intercept as much of the flow as possible.

Solution From the given data: Q = 0.08 m³/s, S_x = 3% = 0.03, and S_0 = 2% = 0.02. Assuming that a significant portion of the gutter flow extends onto the smooth-asphalt pavement, Table 5.36 gives n = 0.013. Using the Manning equation (Equation 5.245) to find the depth, d, at the curb gives

$$Q = 0.375\left(\frac{1}{nS_x}\right)d^{8/3}S_0^{1/2}$$

$$0.08 = 0.375\left(\frac{1}{0.013 \times 0.03}\right)d^{8/3}(0.02)^{1/2}$$

and solving for d leads to

$$d = 0.061 \text{ m} = 6.1 \text{ cm}$$

This flow depth is less than the curb height of 8 cm, and therefore the flow is constrained within the roadway. The top-width, T, of the gutter flow, the flow area, A, and the flow velocity, V, are given by

$$T = \frac{d}{S_x} = \frac{0.061}{0.03} = 2.03 \text{ m}$$

$$A = \frac{1}{2}dT = \frac{1}{2}(0.061)(2.03) = 0.0619 \text{ m}^2$$

$$V = \frac{Q}{A} = \frac{0.08}{0.0619} = 1.29 \text{ m/s}$$

The spread of 2.03 m is much wider than the gutter width of 0.90 m, verifying the assumption that a significant portion of the gutter flow extends onto the smooth-asphalt pavement, and hence justifying the assumed n value of 0.013. Since the spread (T = 2.03 m) exceeds the maximum transverse dimension of typical grates (= 914 mm), use a grate with a transverse dimension, W_0, of 914 mm. According to Figure 5.51, the length of a P–50×100 grate corresponding to a splash-over velocity of 1.29 m/s is approximately 50 cm. Therefore, any grate longer than 50 cm will intercept 100% of the frontal flow, and have a frontal-flow efficiency, R_f, equal to 1.0. To maximize side-flow interception, use a (typical) maximum available grate length of 1220 mm. The selected grate therefore has dimensions of 914 mm × 1220 mm (= 3 ft × 4 ft).

The ratio, R_w, of frontal flow to total gutter flow is given by Equation 5.259 as

$$R_w = 1 - \left(1 - \frac{W_0}{T}\right)^{8/3}$$

$$= 1 - \left(1 - \frac{0.914}{2.03}\right)^{8/3} = 0.80$$

The ratio, R_s, of the side flow intercepted to the total side flow is given by Equation 5.258 as

$$R_s = \left[1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right]^{-1}$$

$$= \left[1 + \frac{0.0828(1.29)^{1.8}}{(0.03)(1.22)^{2.3}}\right]^{-1} = 0.27$$

Therefore, the ratio, R, of the intercepted flow to the total gutter flow is given by Equation 5.260 as

$$R = R_f R_w + R_s(1 - R_w)$$

$$= (1.0)(0.80) + (0.27)(1 - 0.80) = 0.85$$

Based on this result, the grate intercepts $0.85(0.08 \text{ m}^3/\text{s}) = 0.068 \text{ m}^3/\text{s}$, and $0.08 \text{ m}^3/\text{s} - 0.068 \text{ m}^3/\text{s} = 0.012 \text{ m}^3/\text{s}$ bypasses the grate inlet.

Grate inlets in sag vertical curves operate as weirs for shallow ponding depths and as orifices at greater depths. The depths at which grates operate as weirs or orifices depend on the bar configuration and size of the grate, and grates of larger dimension will operate as weirs to greater depths than smaller grates or grates with less opening area (USFHWA, 1996). Typically, for depths of water not exceeding 12 cm, the following weir equation can be used to calculate the capacity, Q_i (m³/s), of a grate inlet

$$\boxed{Q_i = 1.66Pd^{1.5}} \tag{5.261}$$

where P is the perimeter of the grate opening (m) and d is the depth of flow above the grate (m). If the grate is adjacent to a curb, then that side of the grate is not counted in the perimeter. Typically, if the flow depth over a grate exceeds 43 cm, then the following orifice equation is used to compute the capacity, Q_i, of the grate inlet

$$\boxed{Q_i = 0.67A\sqrt{2gd}} \tag{5.262}$$

where A is the open area in the grate and d is the depth of flow above the grate. For depths of flow typically between 12 cm and 43 cm, the capacity of the grate is somewhere between that calculated by Equations 5.261 and 5.262. Grates alone are not typically recommended for installation in sags because of their tendency to clog and cause flooding during severe weather. A combination inlet (see next section) is usually a better choice in sag locations.

EXAMPLE 5.50

A roadway has a cross slope of 2%, a flow depth at the curb of 8 cm, and a corresponding flowrate in the gutter of 0.08 m³/s. The gutter flow is to be removed in a vertical sag by a grate inlet that is mounted flush with the curb. Calculate the minimum dimensions of the grate inlet. Assume that the grate opening is 50% clogged with debris, which covers 25% of the perimeter of the grate.

Solution Since the depth of flow is less than 12 cm, the inflow to the inlet is probably given by the weir equation (Equation 5.261), which can be put in the form

$$P = \frac{Q_i}{1.66d^{1.5}}$$

where P is the grate-inlet perimeter not including the side adjacent to the curb, $Q_i = 0.08$ m³/s, $d = 0.08$ m, and

$$P = \frac{0.08}{1.66(0.08)^{1.5}} = 2.13 \text{ m}$$

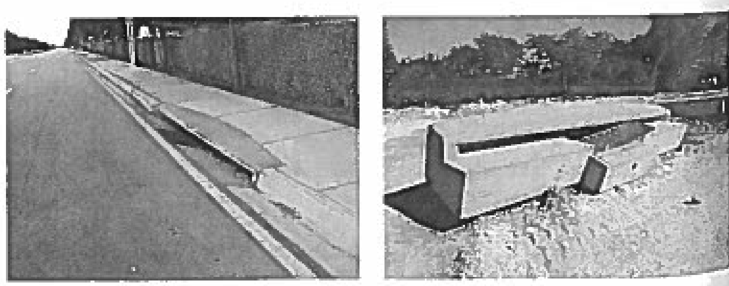
Since the grate is flush-mounted with the curb, the length plus twice the width must equal 213 cm. Since 25% of the grate perimeter is blocked by debris, the grate must have a perimeter that exceeds 213/0.75 = 284 cm. Hence a 914 mm × 1219 mm grate

Grate inlet in
nd uncurbed

with perimeter of 305 cm would have some reserve capacity to accommodate debris blockage.

In addition to draining roadways with curbs and gutters, grate inlets are also used to drain parking lots and roadways without curbs and gutters. Examples of these grate inlets are shown in Figure 5.52. In draining uncurbed roadways, the grate inlet is typically placed in a rectangular paved area that is directly connected to the roadway pavement, as illustrated in the righthand picture. The capacities of the grate inlets shown in Figure 5.52 are calculated using the weir equation, Equation 5.261, for depths less than 12 cm; the orifice equation, Equation 5.262, for depths greater than 43 cm; and an interpolated weir/orifice capacity for intermediate depths.

Combination inlets. Combination inlets consist of combined curb and grate inlets. Typically, the curb opening is upstream of the grate, intercepting debris, reducing the spread on the roadway pavement, and increasing the efficiency of the grate inlet. A combination inlet in which the curb opening is upstream of the grate is called a *sweeper configuration*, and the upstream curb inlet is called a *sweeper inlet*. In sag locations the grate is best placed at the center of the curb opening. A combination inlet with the sweeper configuration is shown in Figure 5.53, where the precast unit, prior to installation, is also shown. It should be clear that this combination inlet is positioned in a sag location. In most combination inlets the grate is located in a depressed gutter. The interception capacity of a combination inlet is equal to the sum of the capacity



: Combination inlet
unit

of the curb opening upstream of the grate plus the grate capacity. The three-step procedure to calculate the capacity of a combination inlet is:

1. Calculate the capacity of the curb inlet upstream of the grate inlet using the procedure described previously for curb inlets. Subtract the curb-inlet capacity from the gutter flow to obtain the gutter flow approaching the grate inlet.
2. Calculate the capacity of the grate inlet. The gutter cross section is typically depressed over a width, W_0, equal to the width of the grate. In this case, the ratio, R_w, of flow over width W_0 to flow over the top width T is given by

$$R_w = \left\{ 1 + \frac{S_w/S_x}{\left[1 + \frac{S_w/S_x}{\frac{T}{W_0}-1}\right]^{8/3} - 1} \right\}^{-1} \tag{5.263}$$

where S_w is the slope of the depressed gutter, and S_x is the slope of the undepressed gutter. The side-flow interception capacity, R_s, is calculated using Equation 5.258, the frontal-flow interception, R_f, is calculated using Equation 5.257, and the fraction of gutter flow intercepted by the grate is given by Equation 5.260.
3. Add the capacity of the curb opening calculated in step 1 to the capacity of the grate inlet calculated in step 2 to obtain the capacity of the combination inlet.

EXAMPLE 5.51

A combination inlet consists of a 2-m curb inlet and a 0.6-m by 0.6-m P-30 grate inlet adjacent to the downstream 0.6 m of the curb opening. The gutter section has a width of 0.6 m, a longitudinal slope of 2%, a cross slope of 3%, and a gutter depression of 25 mm in front of the curb opening. The roadway pavement consists of smooth asphalt. For a gutter flow of 0.07 m^3/s, calculate the interception capacity of the combination inlet.

Solution

Step 1. Calculate the interception capacity of the curb opening upstream of the grate inlet. From the given data, $Q = 0.07$ m^3/s, $S_0 = 2\% = 0.02$, $n = 0.013$ (Table 5.36), $S_x = 3\% = 0.03$, $a = 25$ mm $= 0.025$ m, and $W_0 = 0.6$ m. The fraction, R_w, of gutter flow over the depressed section in front of the curb opening is given by Equation 5.263, where

$$R_w = \left\{ 1 + \frac{S_w/S_x}{\left[1 + \frac{S_w/S_x}{\frac{T}{W_0}-1}\right]^{8/3} - 1} \right\}^{-1} \tag{5.264}$$

In this case,

$$S_w = S_x + \frac{a}{W_0} = 0.03 + \frac{0.025}{0.6} = 0.072$$

and Equation 5.264 can be written as

$$\frac{Q_w}{Q} = \left\{1 + \frac{0.072/0.03}{\left[1 + \frac{0.072/0.03}{\frac{T}{0.6}-1}\right]^{8/3} - 1}\right\}^{-1}$$

$$= \left\{1 + \frac{2.4}{\left[1 + \frac{2.4}{1.67T-1}\right]^{8/3} - 1}\right\}^{-1} \tag{5.265}$$

where Q_w is the flow over width W_0, and Q is the total gutter flow given as 0.07 m³/s. Hence, Equation 5.265 can be written as

$$Q_w = 0.07\left\{1 + \frac{2.4}{\left[1 + \frac{2.4}{1.67T-1}\right]^{8/3} - 1}\right\}^{-1} \tag{5.266}$$

It is convenient for subsequent analyses to work with the flow, Q_s, over the section outside of the depressed section, in which case

$$Q_s = Q - Q_w = 0.07 - 0.07\left\{1 + \frac{2.4}{\left[1 + \frac{2.4}{1.67T-1}\right]^{8/3} - 1}\right\}^{-1} \tag{5.267}$$

This equation must be solved simultaneously with the Manning equation, Equation 5.245, which can be written as

$$Q_s = \frac{0.375}{n} S_x^{5/3} S_0^{1/2} (T - W_0)^{8/3} \tag{5.268}$$

which, in this case gives

$$Q_s = \frac{0.375}{0.013}(0.03)^{5/3}(0.02)^{1/2}(T - 0.6)^{8/3}$$

which simplifies to

$$Q_s = 0.0118(T - 0.6)^{8/3} \tag{5.269}$$

Simultaneous solution of Equations 5.267 and 5.269 gives Q_s = 0.0162 m³/s and T = 1.72 m. The flow ratio, R_w, over width W_0 is therefore given by

$$R_w = \frac{Q - Q_s}{Q} = \frac{0.07 - 0.0162}{0.07} = 0.77$$

The equivalent cross slope, S_e, in the gutter depression is given by Equation 5.250 as

$$S_e = S_x + S_w' R_w = S_x + \left(\frac{a}{W_0}\right) R_w = 0.03 + \left(\frac{0.025}{0.6}\right) 0.77 = 0.062$$

Equation 5.248 gives the length, L_T, of curb opening for 100% interception as

$$L_T = 0.817 Q^{0.42} S_0^{0.3}\left(\frac{1}{nS_e}\right)^{0.6}$$

$$= 0.817(0.07)^{0.42}(0.02)^{0.3}\left(\frac{1}{0.013 \times 0.062}\right)^{0.6} = 5.94 \text{ m}$$

Since the length, L, of curb opening upstream of the grate is 2 m − 0.6 m = 1.4 m, the efficiency of the curb opening is given by Equation 5.249 as

$$R = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{1.4}{5.94}\right)^{1.8} = 0.384$$

The flow, Q_{ic}, intercepted by the curb opening is therefore given by

$$Q_{ic} = 0.384(0.07 \text{ m}^3/\text{s}) = 0.027 \text{ m}^3/\text{s}$$

Step 2. **Calculate the interception capacity of the grate inlet.** The gutter flow immediately upstream of the grate, Q_g, is given by

$$Q_g = Q - Q_{ic} = 0.07 - 0.027 = 0.043 \text{ m}^3/\text{s}$$

The ratio of flow over the grate, Q_w, to the gutter flow upstream of the grate, Q_g, is given by Equation 5.265 (with Q_g replacing Q) and, similar to Equation 5.267, the side flow, Q_s, is given by

$$Q_s = 0.043 - 0.043\left\{1 + \frac{2.4}{\left[1 + \frac{2.4}{1.67T-1}\right]^{8/3} - 1}\right\}^{-1} \tag{5.270}$$

The Manning equation is given by Equation 5.268, which leads to Equation 5.269. Solving Equations 5.269 and 5.270 simultaneously gives Q_s = 0.00556 m³/s and T = 1.35 m. The ratio, R_w, of flow over W_0 to the total gutter flow is therefore given by

$$R_w = \frac{Q_g - Q_s}{Q_g} = \frac{0.043 - 0.00556}{0.043} = 0.87$$

Next, calculate the frontal-flow interception efficiency, R_f. The total flow area, A, in the gutter is given by

$$A = \frac{1}{2}[T^2 S_x + aW_0] = \frac{1}{2}[(1.35)^2(0.03) + (0.025)(0.6)] = 0.0348 \text{ m}^2$$

and hence the average velocity, V, in the gutter is given by

$$V = \frac{Q_g}{A} = \frac{0.043}{0.0348} = 1.24 \text{ m/s}$$

For a 0.6-m long P-30 grate, the splash-over velocity is approximately 2 m/s (see Figure 5.51), and since the average gutter velocity is 1.24 m/s, the frontal-flow interception efficiency is 100% and hence

$$R_f = 1.0$$

The side-flow interception efficiency, R_s, is given by Equation 5.258 as

$$R_s = \left[1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right]^{-1}$$

$$= \left[1 + \frac{0.0828(1.24)^{1.8}}{(0.03)(0.6)^{2.3}}\right]^{-1} = 0.071$$

The flow intercepted by the grate, Q_{ig}, is given by Equation 5.260 as

$$Q_{ig} = Q_g[R_f R_w + R_s(1 - R_w)] = 0.043[(1.0)(0.87) + (0.071)(1 - 0.87)] = 0.038 \text{ m}^3/\text{s} \tag{5.271}$$

Step 3. Calculate the total interception capacity of the combination inlet. The interception capacity of the combination inlet, Q_i, is the sum of the curb opening capacity, Q_{ic}, and the grate capacity, Q_{ig}, hence

$$Q_i = Q_{ic} + Q_{ig} = 0.027 + 0.038 = 0.065 \text{ m}^3/\text{s}$$

Since the gutter flow upstream of the combination inlet is 0.07 m^3/s, then 0.07 m^3/s − 0.065 m^3/s = 0.005 m^3/s bypasses the combination inlet.

In sag vertical curve locations, the capacity of a combination inlet is equal to the capacity of the sweeper portion of the curb inlet plus the grate capacity. For flow depths less than the opening height of the sweeper inlet, the capacity of the sweeper inlet is given by the weir equation, Equation 5.253 (depressed gutter) or Equation 5.255 (undepressed gutter), and for flow depths greater than 1.4 times the opening height the capacity of the sweeper inlet is given by the orifice equation, Equation 5.256. For flow depths less than about 12 cm, the capacity of a grate inlet is given by the weir equation, Equation 5.261, and for depths greater than about 43 cm, the capacity of a grate inlet is given by Equation 5.262. For flow depths in between weir and orifice conditions, inlet capacities can usually be interpolated. Combination inlets in sag locations are frequently designed assuming complete clogging of the grate (Nicklow, 2001).

EXAMPLE 5.52

A combination inlet is to be placed at a sag location of a roadway to remove a gutter flow of 0.15 m^3/s. The combination inlet consists of a 15-cm high by 2.5-m long curb opening with a 0.6-m by 1.2-m P-50 grate centered in front of the curb opening. If the roadway has a cross slope of 3% and the perimeter of the grate is 30% clogged, determine the spread of water on the roadway.

Solution Assuming that the flow depth, d, in the gutter is less than the height of the curb opening (15 cm), the curb opening acts like a weir, and the inflow to the curb opening, Q_c, is given by Equation 5.255 as

$$Q_c = 1.60Ld^{1.5} \tag{5.272}$$

where L = 2.5 m − 1.2 m = 1.3 m. Assuming that the flow depth over the grate is less than 12 cm, then the grate inlet acts like a weir, and the inflow to the grate inlet, Q_g, is given by Equation 5.261 as

$$Q_g = 1.66Pd^{1.5} \tag{5.273}$$

where, for 30% clogging, $P = 0.7(L + 2W_0) = 0.7(1.2 + 2 \times 0.6) = 1.68$ m. Combining Equations 5.272 and 5.273 gives the capacity of the combination inlet, Q, as

$$Q = Q_c + Q_g \tag{5.274}$$

Taking Q = 0.15 m^3/s, combining Equations 5.272 to 5.274, and substituting the given data yields

$$0.15 = 1.60(1.30)d^{1.5} + 1.66(1.68)d^{1.5}$$

which gives

$$d = 0.098 \text{ m} = 9.8 \text{ cm}$$

This depth of flow at the curb verifies weir-flow assumptions for the curb opening and the grate inlet. The spread, T, on the roadway is given by

$$T = \frac{d}{S_x} = \frac{0.098}{0.03} = 3.27 \text{ m}$$

Slotted-drain inlets. Slotted inlets are used in areas where it is desirable to intercept sheet flow before it crosses onto a section of roadway. Typical isometric and elevation views of a slotted drain are illustrated in Figure 5.54, and an operational slotted drain and typical corrugated metal pipe (CMP) used in constructing slotted drains are shown in Figure 5.55. The main advantage of slotted drains is their ability to intercept flow over a wide section of roadway, and their main disadvantage is that they are very susceptible to clogging from sediments and debris. Sediment deposition in the pipe

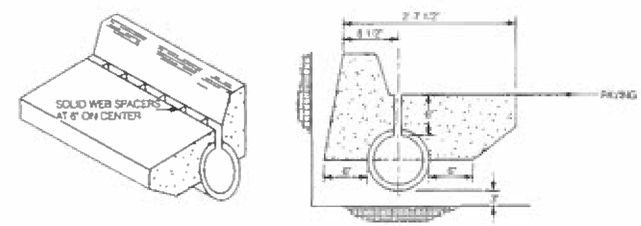


FIGURE 5.54: Slotted-drain inlet

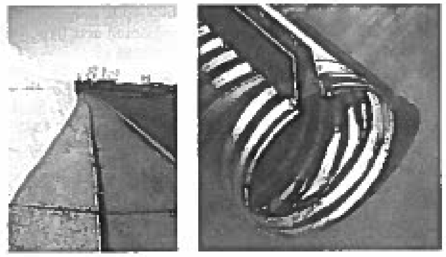


FIGURE 5.55: Operational slotted drain and typical slotted-drain pipe

is the most frequently encountered problem (USFHWA, 1996). Slotted-drain inlets function like curb inlets when they are placed parallel to the flow, and like grate inlets when placed perpendicular to the flow.

For slotted-drain inlets installed parallel to the gutter flow on continuous grades, and having slot widths greater than 4.45 cm (1.75 in.), the length, L_T, of drain required to intercept a flow Q_i is given by Equation 5.248, which is also used to calculate the length of curb inlets for 100% flow interception, and is repeated here for convenience as

$$\boxed{L_T = 0.817Q^{0.42}S_0^{0.3}\left(\frac{1}{nS_x}\right)^{0.6}} \qquad (5.275)$$

where S_0 is the longitudinal slope of the drain, n is the Manning roughness coefficient of the gutter, and S_x is the cross slope of the gutter. In cases where the slotted-drain inlet is shorter than L_T, the ratio, R, of the intercepted flow to the gutter flow is given by

$$R = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \qquad (5.276)$$

It is common practice to use slotted-drain lengths that are much longer than lengths used for curb inlets (Loganathan et al., 1996).

Slotted-drain inlets installed perpendicular to the flow direction perform like short grate inlets. Assuming a splash-over velocity of 0.3 m/s and no side flow, the grate-inlet efficiency equations can be used to calculate the interception capacity of slotted-drain inlets (Young and Stein, 1999).

In sag locations, slotted drains operate as weirs for depths below approximately 5 cm (2 in.) and as orifices in locations where the depth at the upstream edge of the slot is greater than about 12 cm (5 in.). Between these depths transition flow occurs (USFHWA, 1996). The capacity of a slotted inlet operating as a weir can be computed using the relation (Young and Stein, 1999)

$$Q_i = 1.4Ld^{0.5} \qquad (5.277)$$

where L is the length of the slot (m), and d is the depth of flow adjacent to the inlet (m). The capacity of a slotted inlet operating as an orifice can be computed using the

relation (Young and Stein, 1999)

$$Q_i = 0.8Lw\sqrt{2gd} \qquad (5.278)$$

where w is the width of the slot. Slotted drains are usually not recommended in sag locations because of potential problems with clogging from debris.

EXAMPLE 5.53

A roadway has a cross slope of 2.5% and a longitudinal slope of 1.5%, and the flowrate in the gutter is 0.1 m³/s. The flow is to be removed by a slotted drain with a slot width of 5 cm, and the Manning n of the roadway is 0.015. Estimate the minimum length of slotted drain that can be used.

Solution From the given data, $Q_i = 0.1$ m³/s, $S_0 = 0.015$, $n = 0.015$, $S_x = 0.025$, and Equation 5.275 gives the length of the drain as

$$L_T = 0.817Q_i^{0.42}S_0^{0.3}\left[\frac{1}{nS_x}\right]^{0.6} = 0.817(0.1)^{0.42}(0.015)^{0.3}\left[\frac{1}{(0.015)(0.025)}\right]^{0.6} = 10.0 \text{ m}$$

Hence the slotted drain must be at least 10.0 m long to remove the gutter flow.

5.7.1.3 Roadside and median channels

Roadside channels are commonly used with uncurbed roadway sections to convey runoff from the roadway pavement and from areas which drain toward the roadway. Right-of-way constraints limit the use of roadside channels on most urban roadways. These channels are normally trapezoidal or triangular in cross section, lined with grass or other protective lining, and outlet to a storm-drain piping system via a drop inlet, to a detention or retention basin, or to an outfall channel. A typical drop inlet and median channel are illustrated in Figure 5.56. Drop inlets are similar to grate inlets used in pavement drainage and should be placed flush with the channel bottom and constructed of traffic-safe bar grates. In addition, paving around the inlet perimeter can help prevent erosion and might slightly increase the interception capacity of the inlet by accelerating the flow. Small dikes are often placed downstream of drop inlets to impede bypass flow. The height of dike required for 100% interception is not large and can be determined by calculating the ponding height required for interception of the flow by a sag grate. Roadside channels are typically designed as grass-lined channels following the design protocol described in Section 3.5.3.2. Design flows typically have return periods of 5 to 10 years. It is recommended that the channel side slopes not exceed 3:1 and, in areas where traffic safety may be of concern, channel side slopes should be 4:1 or flatter (USFHWA, 1996). Longitudinal slopes are generally dictated by the road profile; however, channels with gradients greater than 2% tend to flow in a supercritical state (USFHWA, 1988) and may require the use of flexible linings to maintain stability. Most flexible lining materials are suitable for protecting channel gradients up to 10% (USFHWA, 1988). In roadside channels a freeboard of 15 cm under design flow conditions is generally considered adequate.

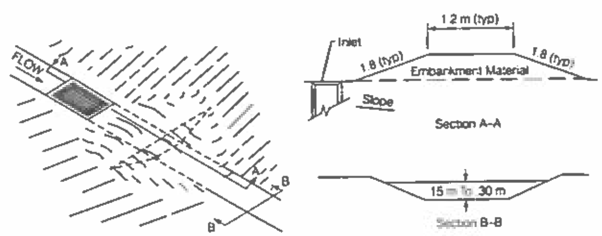


FIGURE 5.56: Typical median inlet
Source: USFHWA (1988).

5.7.2 Runoff Controls

Urban stormwater-management systems are designed to control both the quantity and quality of stormwater runoff. Quantity control usually requires that peak postdevelopment runoff rates do not exceed peak predevelopment runoff rates (for a design rainfall), and quality control usually requires a defined level of treatment, such as a specified detention time in a sedimentation basin or the retention of a specified volume of initial runoff. The runoff events used to design flood-control and water-quality control systems are generally different. Flood-control systems are designed for large, infrequent runoff events with return periods of 10 to 100 years, while quality-control systems are designed for small frequent events with return periods of less than one year. On a long-term basis, most of the pollutant load in stormwater runoff is contained in the smaller, more frequent storms.

Runoff controls can be either on-site or regional controls. *On-site controls* handle runoff from individual developments, while *regional controls* handle runoff from several developments. The main advantage of on-site facilities is that developers can be required to build them, while the major disadvantage is the larger overall land area that is required compared with regional controls. The main advantage of regional facilities is that they provide more storage and can be designed for longer release periods, while their major disadvantages are the complex arrangements that are necessary to collect funds from developers and to use those funds efficiently for the intended stormwater-management facilities. The minimization of directly connected impervious areas remains one of the most effective source controls that can be implemented to reduce the quantity and improve the quality of runoff at the source, and it can significantly reduce the capacity requirements in other runoff controls. Under ideal conditions, the minimization of directly connected impervious areas can virtually eliminate surface runoff from storms with less than 13 mm (0.5 in.) of precipitation (Urbonas and Stahre, 1993).

5.7.2.1 Stormwater impoundments

Stormwater impoundments are facilities (basins) that collect surface runoff and release it either at a reduced rate through an outlet or by infiltration into the ground. These impoundments are frequently used for both flood-control and water-quality control purposes. The two major types of impoundments are detention and retention impoundments (basins). *Detention basins* are water-storage areas where the stored water is released gradually through an uncontrolled outlet, and *retention basins* are water-storage areas where there is either no outlet or the impounded water is stored for a prolonged period. Infiltration basins and ponds that maintain water permanently, with freeboard provided for flood storage, are the most common types of retention basins (ASCE, 1992). Storage impoundments where a permanent water body forms the base of the storage area are called *wet* basins; impoundments where the ground surface forms the base of the storage area are called *dry* basins. Wet-detention basins are commonly referred to as *detention ponds*.

Most detention basins are constructed by a combination of cut and fill and must have at least one service outlet and an emergency spillway. In some cases, multiple outlets at different elevations are used to facilitate the discharge from the basin under multiple design storms with return periods between 20 and 50 years (ASCE, 1992). An emergency spillway provides for controlled overflow during large storm events, typically the 100-year storm (Yen and Akan, 1999). A schematic diagram of a (dry) detention basin is given in Figure 5.57, and a schematic diagram of a detention pond is illustrated in Figure 5.58. Dry-detention basins are typically open areas characterized by low-level outlets that can discharge any accumulated basin inflows, while wet-detention basins (detention ponds) have high-level outlets that discharge when the water level in the pond rises above the permanent pool. Dry-detention basins generally empty after a storm, while detention ponds retain the water much longer above a permanent pool of water. In modern practice, detention ponds are sometimes referred to as retention ponds (because they retain pollutants); however, the latter term should be reserved for ponds without outlets.

The most common types of outlets from detention ponds are orifice-type and weir-type outlets, and these outlets are typically part of a *single-stage riser* as shown in

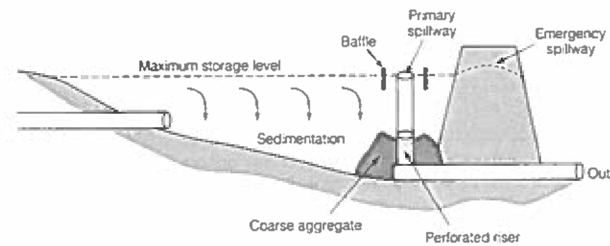


FIGURE 5.57: Dry detention basin
Source: Urbonas and Roesner (1993).

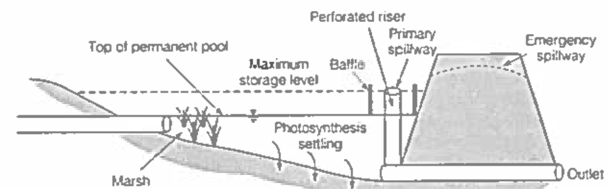


FIGURE 5.58: Detention pond
Source: Urbonas and Roesner, 1993.

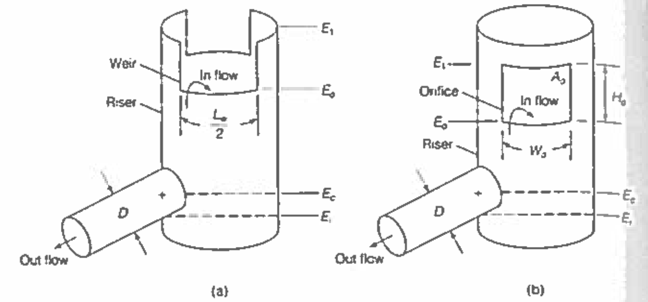


FIGURE 5.59: Single-stage riser
Source: McCuen (2005).

Figure 5.59. The water stored in the detention pond above the permanent pool flows through the orifice or over the weir into the riser and out of the pipe leading from the riser. For weir flow, the discharge over the weir, Q_w, is given by

$$Q_w = C_w L_w h^{3/2} \qquad (5.279)$$

where C_w is the weir coefficient, L_w is the length of the weir, and h is the elevation of the water surface above the crest of the weir. A typical value of C_w is 1.83 when Q_w is in m^3/s, and L_w and h are in meters. For orifice flow, the discharge through the orifice, Q_0, is given by

$$Q_0 = C_d A_0 \sqrt{2gh} \qquad (5.280)$$

where C_d is a discharge coefficient, A_0 is the cross-sectional area of the orifice, and h is the elevation of the water surface above the center of the orifice (for free discharges) or h is the difference between the headwater and tailwater elevations (for submerged discharges). Typical values of C_d are 0.6 for square-edge uniform-entrance conditions, and 0.4 for ragged-edge orifices (USFHWA, 1996).

Where drainage policies require control of flow rates of two exceedance frequencies, the *two-stage riser* is an alternative for control. Its structure is similar to that of the single-stage riser, except that it includes either two weirs or a weir and an orifice as shown in Figure 5.60. For the commonly used weir/orifice structure, the orifice is used to control the more frequent event, and the larger event is controlled using the weir. The runoff from the smaller and larger events are also referred to as the low-stage and high-stage events. According to McCuen (2005), there have been few theoretical or empirical studies on the hydraulics of two-stage risers; and a number of procedures have been proposed for routing flows through these structures. Some proposed procedures have assumed that the flow through the orifice ceases when weir flow begins, while other proposed procedures have assumed that the flow through the orifice is independent of the flow over the weir. It appears to be more realistic to assume that the two flows are not independent and that the interdependence between weir flow and orifice flow increases as the elevation of the water surface above the weir increases. The general form of the stage-discharge relationship is typically

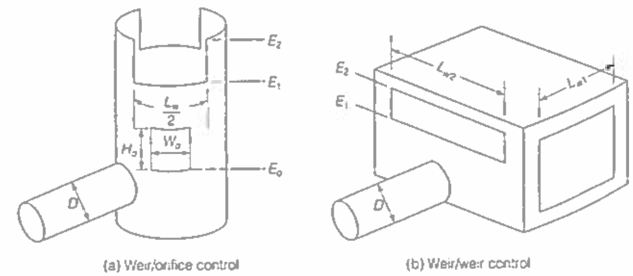


FIGURE 5.60: Two-stage riser
Source: McCuen (2005).

given by

$$Q = \begin{cases} 0, & E \leq E_0 \\ C_w L_w (E - E_0)^{1.5}, & E_0 \leq E \leq E_0 + H_0 \\ C_d A_0 \sqrt{2g(E - E_0)}, & E_0 + H_0 \leq E \leq E_1 \\ C_w L_w (E - E_1)^{1.5} + C_d A_0 \sqrt{2g(E - E_1)}, & E_1 < E \end{cases} \qquad (5.281)$$

where E is the water-surface elevation in the pond surrounding the riser, E_0 is the elevation of the bottom of the orifice, H_0 is the maximum height of the orifice, and E_1 is the elevation of the weir. If the barrel diameter of the riser is large enough to assume that the orifice and weir flows are independent, then the orifice part of Equation 5.281 would be $C_d A_0 \sqrt{2g(E - E_0)}$ rather than $C_d A_0 \sqrt{2g(E - E_1)}$.

EXAMPLE 5.54

An outlet structure from a detention pond is to be designed for both water-quality and water-quantity control purposes. For water-quality control, the outlet structure is to be sized such that the first 2.5 cm of runoff from the catchment is discharged in not less than 24 hours. For water-quantity control, the outlet structure is to be sized such that the peak runoff is less than or equal to the predevelopment peak of 0.92 m^3/s. The catchment area is 46 ha, and the average wet-season pool elevation in the detention pond is 102.50 m. Storage of the water-quality volume causes the water-surface elevation in the detention pond to rise to 102.85 m, and under design-flood conditions the maximum allowable pool elevation in the detention pond is 104.00 m. The outlet structure is to consist of a 1.21-m diameter vertical circular pipe (riser), with the water-quality volume discharged through an orifice in the riser, and the flood flow discharged over a high-level weir at the top of the riser. The high-level discharge weir must have a crest length not greater than 50% of the riser diameter. Determine the required dimensions and elevations of the orifice and weir components of the two-stage riser outlet.

Solution The water-quality volume will be discharged through an unsubmerged orifice (which acts like a weir), where the bottom side of the orifice is at the average wet-season pool elevation of 102.50 m and the top of the orifice is at the pool elevation when the water-quality volume is stored, which is 102.85 m. Since the catchment area is 46 ha = 4.6×10^5 m^2 and the water-quality depth is 2.5 cm = 0.025 m, then

$$\text{water-quality volume} = 4.6 \times 10^5\ \text{m}^2 \times 0.025\ \text{m} = 11,500\ \text{m}^3$$

If this water-quality volume is to be detained in the pond for one day (= 86,400 seconds), then the average discharge rate through the orifice/weir, Q_0, is given by

$$Q_0 = \frac{\text{water-quality volume}}{1\ \text{day}} = \frac{11,500\ \text{m}^3}{86,400\ \text{s}} = 0.133\ \text{m}^3/\text{s}$$

Specifying the maximum orifice discharge rate as equal to the average discharge rate for 24-h detention (which guarantees a detention time greater than 24 h) yields, according to Equation 5.281,

$$Q_0 = C_w L_w (E - E_0)^{1.5}$$

where C_w can be taken as 1.83, E = 102.85 m, and E_0 = 102.50 m. Substituting into the above equation yields

$$0.133 = 1.83 L_w (102.85 - 102.50)^{1.5}$$

which gives

$$L_w = 0.35\ \text{m}$$

The required height of the orifice, H_0, is 102.85 − 102.50 m = 0.35 m, and therefore the dimensions of the orifice required to detain the water-quality volume for at least 24 hours ($L_w \times H_0$) is 0.35 m × 0.35 m.

The riser diameter, D, is equal to 1.21 m and the maximum length of the flood-discharge weir is $0.5D$ = 0.5(1.21 m) = 0.605 m. The discharge from the outlet structure when the pool elevation exceeds the crest elevation of the flood-discharge weir is given by Equation 5.281 as

$$Q = C_w L_w (E - E_1)^{1.5} + C_d A_0 \sqrt{2g(E - E_1)}$$

When the pool elevation is at its maximum allowable level, Q = 0.92 m^3/s, C_w = 1.83, L_w = 0.605 m, E = 104.00 m, C_d = 0.6, and A_0 = 0.35 m × 0.35 m = 0.1225 m^2. Substituting into the above equation yields

$$0.92 = 1.83(0.605)(104.00 - E_1)^{1.5} + 0.6(0.1225)\sqrt{2(9.81)(104.00 - E_1)}$$

which gives

$$E_1 = 103.30\ \text{m}$$

Therefore, the bottom of the flood discharge weir should be at elevation 103.30 m to give a design-flood discharge equal to the predevelopment value of 0.92 m^3/s.

In summary, the required two-stage riser should have a 0.35-m × 0.35-m orifice with top and bottom elevations of 102.85 m and 102.50 m, and a 0.605-m long flood-discharge weir with a crest elevation of 103.30 m. The riser will meet both the water-quality and water-quantity (flood-discharge) objectives of the detention pond.

By their very nature, flood-control basins tend to be wet, soggy, and soft. It should never be assumed that a concrete outlet structure can be adequately supported in this environment without piles or other geotechnical treatment. Without proper support, the outlet structure can settle and separate from the outfall pipe, creating a guaranteed failure scenario, or at least an expensive maintenance situation (Paine and Akan, 2001). As the height of the outlet riser increases, conditions become more unstable, and, in order to increase stability, the outlet structure should be located in the embankment rather than in the open bottom of the basin.

5.7.2.2 Flood control

Both detention and retention basins are used for flood control. The design of retention basins for flood control consists first of providing sufficient freeboard in an impoundment area to store the runoff volume resulting from the design runoff event and then of verifying that the infiltration capacity of the impoundment is sufficient to remove the stored water in a reasonable amount of time, usually a typical interstorm period. The design runoff event is usually equal to the runoff resulting from a storm of specified duration and frequency, such as a 72-hour storm with a 25-year return period. The design of detention basins for flood control consists of the following steps: (1) select a drainage-basin configuration; (2) select an outlet structure; (3) route the design-runoff hydrograph through the detention basin and determine the peak discharge from the detention basin and the maximum water elevation in the basin; and (4) repeat steps 1 to 3 until the peak discharge and maximum water-surface elevation are acceptable. An acceptable peak discharge is usually one that is less than or equal to the predevelopment-peak discharge, and an acceptable maximum water-surface elevation maintains the stored runoff within the confines of the detention basin. Most drainage ordinances require that postdevelopment discharges not exceed predevelopment discharges for multiple events, such as the 2-year, 10-year, 25-year, and 50-year storms (ASCE, 1996a). Detention facilities should be designed to drain within the typical interstorm period, usually on the order of 72 hours (Debo and Reese, 1995). Some guidelines for determining the required size of a detention basin for flood control are given below.

- **Make a preliminary selection of a drainage basin.** A preliminary estimate of the required drainage-basin volume can be obtained by subtracting the predevelopment runoff volume from the postdevelopment runoff volume. This volume represents the approximate storage requirement to ensure that the postdevelopment peak discharge rate is less than or equal to the predevelopment peak runoff rate.
- **Make a preliminary selection of an outlet structure.** Use the required volume of the detention basin estimated in step 1 with the storage-elevation function for the site to estimate the maximum headwater elevation at the outlet location.

Select an outlet structure that will pass the maximum allowable outflow at this headwater elevation.

- **Route the runoff hydrograph through the detention basin.** The runoff hydrograph is routed through the detention basin using the modified Puls method described in Section 5.5.1.1. This procedure yields the discharge hydrograph from the detention basin.
- **Assess the performance of the detention basin.** Determine whether the peak discharge from the detention basin is less than or equal to the predevelopment peak discharge. If it is, the required detention volume corresponds to the maximum stage in the detention basin. If the peak discharge from the detention basin exceeds the predevelopment peak discharge, then adjust the outlet structure to discharge the predevelopment peak at the maximum stage in the detention basin, and repeat the previous two steps.

EXAMPLE 5.55

The estimated runoff hydrographs from a site before and after development are as follows:

Time (min)	0	30	60	90	120	150	180	210	240	270	300	330	360	390
Before (m^3/s)	0	1.2	1.7	2.8	1.4	1.2	1.1	0.91	0.74	0.61	0.50	0.28	0.17	0
After (m^3/s)	0	2.2	7.7	1.9	1.1	0.80	0.70	0.58	0.38	0.22	0.11	0	0	0

The postdevelopment detention basin is to be a detention pond drained by an outflow weir. The elevation versus storage in the detention pond is

Elevation (m)	Storage (m^3)
0	0
0.5	5,544
1.0	12,200
1.5	20,056

where the weir crest is at elevation 0 m, which is also the initial elevation of the water in the detention pond prior to runoff. The performance of the weir is given by

$$Q = 1.83bh^{\frac{3}{2}}$$

where Q is the overflow rate (m^3/s), b is the crest length (m), and h is the head on the weir (m). Determine the required crest length of the weir for the detention pond to perform its desired function. What is the maximum water-surface elevation expected in the detention pond?

Solution The required detention-pond volume is first estimated by subtracting the predevelopment runoff volume, V_1, from the postdevelopment runoff volume, V_2. From the given hydrographs:

$$V_1 = (30)(60)[1.2 + 1.7 + 2.8 + 1.4 + 1.2 + 1.1 + 0.91 + 0.74 + 0.61 + 0.50 + 0.28 + 0.17] = 22{,}698\ \text{m}^3$$

and

$$V_2 = (30)(60)[2.2 + 7.7 + 1.9 + 1.1 + 0.80 + 0.70 + 0.58 + 0.38 + 0.22 + 0.11] = 28{,}242\ \text{m}^3$$

A preliminary estimate of the required volume, V, of the detention pond above the normal pool elevation is

$$V = V_2 - V_1 = 28{,}242 - 22{,}698 = 5544\ \text{m}^3$$

From the storage-elevation function, the head h corresponding to a storage volume of 5544 m^3 is 0.50 m. The maximum predevelopment runoff, Q, is 2.8 m^3/s, and the weir equation gives

$$Q = 1.83bh^{\frac{3}{2}}$$

or

$$b = \frac{Q}{1.83h^{3/2}} = \frac{2.8}{1.83(0.50)^{3/2}} = 4.33\ \text{m}$$

Based on this preliminary estimate of the crest length, use a trial length of 4.25 m. The corresponding weir-discharge equation is

$$Q = 1.83bh^{\frac{3}{2}} = 1.83(4.25)h^{\frac{3}{2}} = 7.78h^{\frac{3}{2}}$$

The postdevelopment-runoff hydrograph can be routed through the detention pond using $\Delta t = 30$ min. The storage and outflow characteristics of the detention pond can be put in the following form

Elevation (m)	Storage, S (m^3)	Outflow, O (m^3/s)	$2S/\Delta t + O$ (m^3/s)
0	0	0	0
0.5	5,544	2.75	8.91
1.0	12,200	7.78	21.34
1.5	20,056	14.29	36.58

The routing computations (using the modified Puls method) are summarized in the following table:

Time (min)	Inflow, I (m^3/s)	$2S/\Delta t - O$ (m^3/s)	$2S/\Delta t + O$ (m^3/s)	O (m^3/s)
0	0	0	0	0
30	2.2	0.84	2.2	0.68
60	7.7	3.76	10.74	3.49
90	1.9	4.26	13.36	4.55
120	1.1	2.78	7.26	2.24

From these results, it is already clear that the maximum outflow from the detention pond is 4.55 m^3/s, which is higher than the predevelopment peak of 2.8 m^3/s and is therefore unacceptable. Decreasing the crest length (=4.25 m) of the weir by the factor 2.8/4.55 = 0.62 gives a new crest length, b, of 0.62(4.25 m) = 2.64 m. Using a rounded number of 2.50 m, the revised weir equation is

$$Q = 1.83bh^{\frac{3}{2}} = 1.83(2.5)h^{\frac{3}{2}} = 4.58h^{\frac{3}{2}}$$

The revised storage-outflow characteristics of the detention basin are:

Elevation (m)	Storage, S (m^3)	Outflow, O (m^3/s)	$2S/\Delta t + O$ (m^3/s)
0	0	0	0
0.5	5,544	1.62	7.78
1.0	12,200	4.58	18.14
1.5	20,056	8.41	30.70

The routing computations are summarized in the following table:

Time (min)	Inflow, I (m^3/s)	$2S/\Delta t - O$ (m^3/s)	$2S/\Delta t + O$ (m^3/s)	O (m^3/s)
0	0	0	0	0
30	2.2	1.28	2.2	0.46
60	7.7	6.00	11.18	2.59
90	1.9	7.90	15.60	3.85
120	1.1	5.88	10.90	2.51

From these results, it is already clear that the maximum outflow from the detention basin is 3.85 m^3/s, which is higher than the predevelopment peak of 2.8 m^3/s and is therefore unacceptable. The crest length must be further decreased until the maximum weir discharge is less than or equal to 2.8 m^3/s. This occurs when the crest length, b, is decreased to 1.30 m. The revised weir-discharge equation is then given by

$$Q = 1.83bh^{\frac{3}{2}} = 1.83(1.30)h^{\frac{3}{2}} = 2.38h^{\frac{3}{2}}$$

and the revised storage-outflow characteristics of the reservoir are:

Elevation (m)	Storage, S (m^3/s)	Outflow, O (m^3/s)	$2S/\Delta t + O$ (m^3/s)
0	0	0	0
0.5	5,544	0.84	7.00
1.0	12,200	2.38	15.94
1.5	20,056	4.37	26.66

The routing computations are summarized in the following table:

Time (min)	Inflow, I (m^3/s)	$2S/\Delta t - O$ (m^3/s)	$2S/\Delta t + O$ (m^3/s)	O (m^3/s)
0	0	0	0	0
30	2.2	1.68	2.2	0.26
60	7.7	8.32	11.58	1.63
90	1.9	12.42	17.92	2.75
120	1.1	10.84	15.42	2.29
150	0.80	9.08	12.74	1.83
180	0.70	7.66	10.58	1.46
210	0.58	6.60	8.94	1.17
240	0.38	5.68	7.56	0.94
270	0.22	4.78	6.28	0.75
300	0.11	3.89	5.11	0.61
330	0	3.04	4.00	0.48

Time (min)	Inflow, I (m^3/s)	$2S/\Delta t - O$ (m^3/s)	$2S/\Delta t + O$ (m^3/s)	O (m^3/s)
360	0	2.32	3.04	0.36
390	0	1.76	2.32	0.28
420	0	1.34	1.76	0.21
450	0	1.02	1.34	0.16
480	0	0.78	1.02	0.12
510	0	0.60	0.78	0.09
540	0	0.46	0.60	0.07
570	0	0.34	0.46	0.06
600	0	0.26	0.34	0.04
630	0	0.20	0.26	0.03
660	0	0.16	0.20	0.02
690	0	0.12	0.16	0.02
720	0	0.10	0.12	0.01

For a crest length of 1.30 m, the maximum postdevelopment discharge is 2.75 m^3/s and is therefore acceptable. The maximum water level in the detention pond corresponds to a weir overflow rate of 2.75 m^3/s; from the weir-discharge equation, this corresponds to h = 1.10 m. If this water elevation is excessive, the engineer could consider expanding the proposed detention basin.

It is interesting to note that for any pond with an uncontrolled outlet the peak water-surface elevation in the pond will always occur at the time when the outflow hydrograph intersects the receding limb of the inflow hydrograph. Prior to this intersection, inflow exceeds outflow and the water level is rising, and beyond this intersection outflow exceeds inflow and the water level is falling.

Flood-control systems are primarily designed to ensure that postdevelopment peak-discharge rates do not exceed predevelopment peak-discharge rates. Using detention basins to accomplish this goal generally results in a postdevelopment-discharge hydrograph that is shifted in time and has an overall greater volume compared to the predevelopment-discharge hydrograph. Consequently, the postdevelopment-discharge hydrograph generally has higher off-peak discharge rates than the predevelopment hydrograph. Effects of increased runoff volumes from developed areas include (1) prolonged rise in the water surface downstream of the development, which might affect the slope and stability of channels; (2) the increase in runoff volume represents the amount of ground-water recharge that is no longer being absorbed on-site; and (3) an increased volume of water is released into downstream detention ponds (Haestad Methods, Inc., 1997a).

In cases where there is not ample surface area available to meet storage requirements, *underground detention* might be necessary. Underground detention may consist of a series of large pipes or prefabricated custom chambers manufactured specifically for underground detention.

5.7.2.3 Water-quality control

The quality of urban runoff is determined principally by nonstructural controls and structural controls. *Nonstructural controls* are practices that reduce the accumulation and generation of potential pollutants at or near their source, while *structural controls* are practices that involve an engineered facility to control the quality of runoff. Examples of nonstructural controls include land-use planning and management, floodplain protection, public education, fertilizer and pesticide application control, street sweeping, household hazardous waste recycling programs, and erosion control at construction sites. Examples of structural controls include stormwater-detention basins and infiltration basins.

Water-quality control regulations usually require a defined level of treatment, such as a specified detention time in a sedimentation basin and/or the retention of a specified volume of initial runoff called the *water-quality volume*. The most effective stormwater-management systems are designed to satisfy both detention and retention criteria. Detention basins are commonly used for sedimentation purposes, and infiltration basins or underground exfiltration trenches are used for retention purposes. The specified retention volume (i.e., the water-quality volume) is usually less than or equal to the runoff volume in at least 90% of the annual runoff events, and typically corresponds to a runoff depth on the order of 1.3 cm (0.5 in.).

Detention Systems

The design of detention basins for water-quality control is fundamentally different than for flood control because flood-control detention basins are designed to attenuate peak runoff rates from large storms (with long return periods), whereas water-quality detention basins are designed to provide sufficient detention time for the sedimentation of pollutant loads in smaller, more frequent storms that usually contain much higher concentrations of pollutants than runoff from large-rainfall events. Processes such as natural die-off of bacteria and plant uptake of soluble nitrogen and phosphorus also occur in detention basins, but sedimentation is the principal process of pollutant removal. A common practice is to use *dual-purpose basins* designed to control both peak discharges and pollution from stormwater runoff.

The main design criteria for water-quality detention basins are (Akan, 1993): (1) detain the design runoff long enough to provide the targeted level of treatment, and (2) evacuate the design runoff soon enough to provide available storage for the next runoff event. The required detention time is determined by the settling velocities of the pollutants in the runoff. A mean detention time of about 18 hours is usually sufficient to settle out 60% of total suspended solids, lead, and hydrocarbons, and 45% of total BOD, copper, and phosphates from urban storm runoff (Whipple and Randall, 1983). The required evacuation time for a water-quality detention basin is based on the average time between design runoff events and is usually specified by local regulatory agencies. According to the Environmental Protection Agency (USEPA, 1986a), the average interval between storms in most parts of the United States is between 73 and 108 hours, while the average time between storms in the southwestern part of the United States is much higher, on the order of 277 hours (USEPA, 1986a). Florida requires that detention basins empty within 72 hours after a storm event, and Delaware requires that 90% of the runoff be evacuated in 36 hours or in 18 hours for residential areas (Akan, 1993).

The design of a detention basin for given detention and evacuation times is a reservoir routing problem. *Wet-detention basins* (detention ponds) contain a permanent pool of water, and the detention time, t_d, is estimated from the outflow

hydrograph by the relation

$$\int_0^{t_d} O(t)\, dt = V \tag{5.282}$$

where $O(t)$ is the outflow from the detention basin as a function of time and V is the average volume of the detention basin. The evacuation time in wet-detention basins is typically taken as the time from the peak of the outflow hydrograph (when the storage is a maximum) to the time that 95% of the surcharge storage has been evacuated (Wurbs and James, 2002). Detention basins without a permanent pool of water are called *dry-detention basins*. The detention time in dry-detention basins is typically taken as the time difference between the centroids of the inflow and outflow hydrographs (Haan et al., 1994; Wurbs and James, 2002). The evacuation time is taken as the interval between when inflow first enters the detention basin and when outflow from the basin ceases.

EXAMPLE 5.56

A runoff hydrograph is routed through a detention pond, and the results are given in the following table:

Time (min)	Inflow (m^3/s)	Storage (m^3)	Outflow (m^3/s)
0	0	10,000	0
30	2.2	11,746	0.26
60	7.7	18,955	1.63
90	1.9	23,653	2.75
120	1.1	21,817	2.29
150	0.8	19,819	1.83
180	0.7	18,208	1.46
210	0.58	16,993	1.17
240	0.38	15,958	0.94
270	0.22	14,977	0.75
300	0.11	14,050	0.61
330	0	13,168	0.48
360	0	12,412	0.36
390	0	11,836	0.28
420	0	11,395	0.21
450	0	11,062	0.16
480	0	10,810	0.12
510	0	10,621	0.09
540	0	10,477	0.07
570	0	10,360	0.06
600	0	10,270	0.04
630	0	10,207	0.03
660	0	10,162	0.02
690	0	10,126	0.02
720	0	10,099	0.01

Estimate the detention time and evacuation time of the detention pond.

Solution The average volume, V, of the detention pond is determined by averaging the storage over the duration of the discharge (0 to 720 min), which yields

V = 13,567 m^3. The detention time, t_d, is defined by Equation 5.282, and t_d is the time when the cumulative outflow is equal to V. The cumulative outflow as a function of time is tabulated as follows:

Time (min)	Outflow (m^3/s)	Cumulative outflow (m^3)	Time (min)	Outflow (m^3/s)	Cumulative outflow (m^3/s)
0	0	0	390	0.28	26,406
30	0.26	234	420	0.21	26,847
60	1.63	1,935	450	0.16	27,180
90	2.75	5,877	480	0.12	27,432
120	2.29	10,413	510	0.09	27,621
150	1.83	14,121	540	0.07	27,765
180	1.46	17,082	570	0.06	27,882
210	1.17	19,449	600	0.04	27,972
240	0.94	21,348	630	0.03	28,035
270	0.75	22,869	660	0.02	28,080
300	0.61	24,093	690	0.02	28,116
330	0.48	25,074	720	0.01	28,143
360	0.36	25,830			

From these results, t_d = 146 min = 2.4 h. The peak of the outflow hydrograph (2.75 m^3/s) occurs at t = 90 min and corresponds to a surcharge of 23,653 m^3 − 10,000 m^3 = 13,653 m^3. When 95% of the surcharge has been evacuated, the remaining storage is 10,000 m^3 + 0.05(13,653 m^3) = 10,682 m^3, and from the given data this storage occurs at t = 500 min. The evacuation time is therefore equal to 500 min − 90 min = 410 min = 6.8 h.

EXAMPLE 5.57

If the inflow and outflow hydrographs given in the previous example were from a dry detention basin, estimate the detention time and evacuation time.

Solution The detention time can be approximated by the difference between the centroids of the inflow and outflow hydrographs. Denoting the points on the inflow hydrograph by $I_i = I(t_i)$, and the outflow hydrograph by $O_i = O(t_i)$, the centroids of the inflow and outflow hydrographs, t_I and t_O, are defined as

$$t_I = \frac{\sum_{i=1}^{N_I} t_i I_i}{\sum_{i=1}^{N_I} I_i}$$

and

$$t_O = \frac{\sum_{i=1}^{N_O} t_i O_i}{\sum_{i=1}^{N_O} O_i}$$

where N_I and N_O are the number of points on the inflow and outflow hydrographs, respectively. The computations of t_I and t_O are summarized in the following table:

t_i (min)	I_i (m^3/s)	t_iI_i (m^3)	O_i (m^3/s)	t_iO_i (m^3)
0	0	0	0	0
30	2.2	3,960	0.26	468
60	7.7	27,720	1.63	5,868
90	1.9	10,260	2.75	14,850
120	1.1	7,920	2.29	16,488
150	0.80	7,200	1.83	16,470
180	0.70	7,560	1.46	15,768
210	0.58	7,308	1.17	14,742
240	0.38	5,472	0.94	13,536
270	0.22	3,564	0.75	12,150
300	0.11	1,980	0.61	10,980
330	0	0	0.48	9,504
360	0	0	0.36	7,776
390	0	0	0.28	6,552
420	0	0	0.21	5,292
450	0	0	0.16	4,320
480	0	0	0.12	3,456
510	0	0	0.09	2,754
540	0	0	0.07	2,268
570	0	0	0.06	2,052
600	0	0	0.04	1,440
630	0	0	0.03	1,134
660	0	0	0.02	792
690	0	0	0.02	828
720	0	0	0.01	432
Total	15.69	82,944	15.64	169,920

Based on these results:

$$t_I = \frac{82,944}{15.69} = 88 \text{ min}$$

$$t_O = \frac{169,920}{15.64} = 181 \text{ min}$$

The detention time, t_d, is therefore given by

$$t_d = t_O - t_I = 181 - 88 = 93 \text{ min} = 1.6 \text{ h}$$

The evacuation time is equal to the time from when inflow first enters the detention basin to when outflow from the basin ceases. Inflow begins at t = 0 min and ceases at about t = 720 min. Therefore, the evacuation time is equal to 720 min or 12 h.

Wet-detention basins. Wet-detention basins are the most common type of detention basin used in stormwater management. *Wet-detention basins*, also called *wet-detention ponds* or simply *detention ponds*, are designed to maintain a permanent pool of water and temporarily store runoff until it is released at a controlled rate. Detention ponds remove pollutants by physical, chemical, and biological processes. In addition to sedimentation, chemical flocculation occurs when heavier sediment particles overtake and coalesce with smaller (lighter) particles; biological removal is accomplished by uptake of pollutants by aquatic plants and metabolism by phytoplankton and microorganisms. Detention ponds are typically used for drainage areas of 4 ha or more, and the runoff volume to be treated, called the *water-quality volume*, typically corresponds to 1.3 cm of runoff from the drainage area. The removal of dissolved pollutants primarily occurs between storms. In cases where there is no pond outlet, detention ponds are called *retention ponds*, although the term "retention pond" is frequently used (incorrectly) to describe wet-detention ponds in general. Detention and retention ponds are sometimes called *amenity lakes*, and can be included in the design of golf courses and the landscaping of parks and open spaces.

The layout of a typical detention pond is illustrated in Figure 5.61. The three most important factors in determining the removal efficiency of detention ponds are: (1) the volume of the permanent pool; (2) the depth of the permanent pool; and (3) the presence of a shallow littoral zone. The volume of the permanent pool should be sufficient to provide two to four weeks of detention time so that algae can grow, and the ratio of the volume of the detention pond to the detained volume for water-quality treatment should be at least 4 to achieve total suspended-sediment removal rates of 80% to 90% (ASCE, 1998). The depth of the permanent pool should be greater than 1 to 2 meters, to prevent wind-generated waves from resuspending accumulated bottom sediments and to reduce bottom-weed growth by minimizing sunlight penetration to the bottom of the pond. The depth of the pond should be less than 3 to 5 meters so that the water remains well mixed and the bottom sediment remains aerobic. An anaerobic condition in the bottom of the pond will mobilize nutrients and metals into the water column and significantly reduce the effectiveness of the detention pond. In Florida, detention ponds up to 9 meters deep have been used successfully when excavated in high-ground-water areas, probably because of the improved circulation at the bottom of the pond as a result of ground water moving through the pond (ASCE, 1998). The presence of a littoral zone is essential to the proper performance of a detention

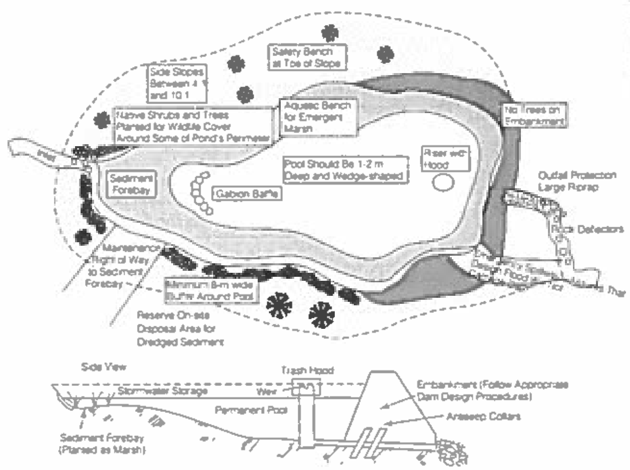


FIGURE 5.61: Layout of detention pond.
Source: Schueler (1987).

pond, since the aquatic plants in the littoral zone provide much of the biological assimilation of the dissolved stormwater pollutants. The littoral zone should cover 25% to 50% of the surface area of the detention pond (ASCE, 1998) and have a slope of 6:1 (H:V) or less to a depth of 60 cm (2 ft) below the permanent-pond elevation. This is sometimes called an *aquatic bench* (Paine and Akan, 2001). Small side slopes provide a measure of public safety, especially for children. It is recommended that the flow length in the detention pond be extended as much as possible between the inlet and outlet structures and that the outlet structure from the detention pond be designed such that an average-annual runoff event, captured as a surcharge above the permanent pool, be drained in approximately 72 hours, or whatever is the local interstorm duration (Urbonas and Stahre, 1993). The length-to-width ratio of detention ponds is sometimes recommended to be greater than 4:1 (Yousef and Wanielista, 1989), but ASCE (1998) suggests that a length-to-width ratio of 3:1 is preferable. This length-to-width requirement is intended to minimize short circuiting, enhance sedimentation, and prevent vertical stratification within the permanent pool (Hartigan, 1989). Overflow from detention ponds is allowed for larger-rainfall events, with appropriate restrictions for flood control.

Dry-detention basins. *Dry-detention basins* are areas that are normally dry, but function as detention reservoirs during runoff events. Dry-detention basins remove pollutants primarily by sedimentation. The removal efficiency of these basins is regarded as poor for detention times shorter than 12 hours, good for detention times longer than 24 hours, and excellent for a detention times of around 48 hours (USFHWA, 1996). The design of dry-detention basins for pollutant removal is much less scientific than for detention ponds. Although sedimentation is still the primary pollutant-removal process, the estimation of basin performance is derived mostly from empirical results. Dry-detention basins should have a volume at least equal to the average runoff event during the year (Grizzard et al., 1986), and this volume be drained in no less than 40 hours (Urbonas and Stahre, 1993). However, dry-detention basins with long drain times tend to be breeding grounds for mosquitoes, have "boggy" bottoms with wetland vegetation, and are usually difficult to maintain and clean. Typical removal rates for properly designed dry-detention basins are given in Table 5.39.

Dry-detention basins are typically used where the drainage area exceeds 4 ha; however, because they take up large areas, dry-detention basins are generally not well suited for high-density residential developments. In many cases, dry-detention

TABLE 5.39: Typical Removal Rates in Dry-Detention Basins

Pollutant	Removal Rate
TSS	50%–70%
TP	10%–20%
Nitrogen	10%–20%
Organic matter	20%–40%
Pb	75%–90%
Zn	30%–60%
Hydrocarbons	50%–70%
Bacteria	50%–90%

Source: Urbonas and Stahre (1993)

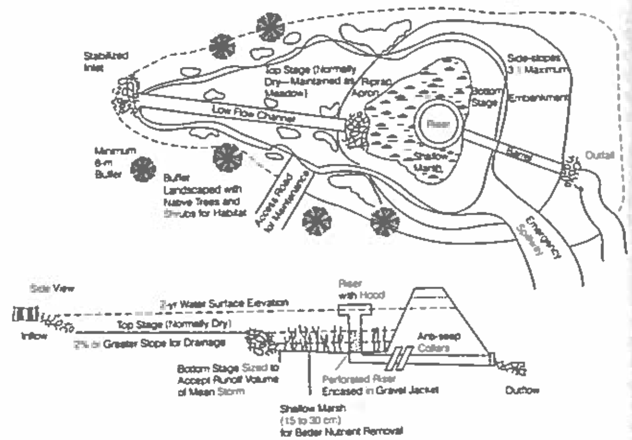


FIGURE 5.62: Layout of extended (dry) detention basin.
Source: Schueler (1987).

basins have a dual purpose in both quality and quantity (peak-flow) control. For dual-purpose dry-detention basins, the bottom portion of the basin is designed with a its own low-level outlet structure for water-quality control, while the basin as a whole is designed to control the peak discharge. The bottom portion of the basin used for water-quality control typically has a detention time much longer than the basin as a whole and this (bottom) portion of the basin is sometimes called an *extended dry-detention basin* or simply *extended detention basin* and is illustrated in Figure 5.62. The water-quality volume in the dry-detention basin typically corresponds to 1.3 cm of runoff from the drainage area. In designing dry-detention basins, the volume of the basin can be taken as equal to the runoff volume to be treated, provided the catchment area is less than 100 ha. If the catchment area is larger than 100 ha, reservoir routing is necessary to determine the volume of the basin. The calculated volume should be increased by 20% to account for sediment accumulation (ASCE, 1998). The outlet structure, such as a V-notch weir or perforated riser, should be designed to drain the basin in the specified design period. To ensure that small-runoff events will be adequately detained, ASCE (1998) recommends that the outlet empty less than 50% of the design volume in the first one-third of the design emptying period. The shape of dry-detention basins should be such that they gradually expand from the inlet and contract toward the outlet to reduce short circuiting. Riprap or other methods of stabilization should be provided within a low-flow channel and at the outflow channel to resist erodible velocities. A length-to-width ratio of 2 : 1 or greater, preferably up to a ratio of 4 : 1, is recommended (ASCE, 1998). Basin side slopes of 4 : 1 (H : V) or greater provide for facility maintenance and safety concerns, and a forebay with a volume equal to approximately 10% of the total design volume can help with the maintenance of the basin by facilitating sediment deposition near the inflow, thereby extending the service life of the remainder of the basin. Embankments for small on-site

basins should be protected from at least the 100-year flood; whenever possible, dry-detention basins should be incorporated within larger flood-control facilities. Some dry-detention basins are hardly noticed by the public, since they may be implemented as multiple-use facilities that function as parks and recreation areas, and are filled only during exceptional storms. Maintenance of dry-detention basins is both essential and costly, with the general objectives being to prevent clogging, prevent standing water, and prevent the growth of weeds and wetland plants.

EXAMPLE 5.58

A dry-detention basin is to be designed for a 60-ha (150-ac) residential development. The runoff coefficient of the area is estimated as 0.3, and the average rainfall depth during the year is 3.3 cm (1.3 in.). Design the detention basin and estimate the suspended-solids removal efficiency.

Solution The average runoff volume, V, is given by

$$V = CAD$$

where C is the runoff coefficient (= 0.3), A is the catchment area (=60 ha), and D is the average rainfall depth (=3.3 cm). Hence,

$$V = (0.30)(60 \times 10^4)(3.3 \times 10^{-2}) = 5940 \text{ m}^3$$

The required volume of the dry-detention basin should be increased by 20% to account for sediment accumulation and is therefore equal to $1.2(5940 \text{ m}^3) = 7130 \text{ m}^3$. The outlet between the bottom of the detention basin and the elevation corresponding to a basin storage of 7130 m^3 should be designed to discharge 7130 m^3 in approximately 40 hours. An appropriate outlet structure could be a perforated riser or a V-notch weir, and overflow from the detention basin should be accommodated when the storage volume exceeds 7130 m^3. In accordance with Table 5.39, the suspended-solids removal in the basin is expected to be in the range of 50% to 70%.

Wet- vs. dry-detention basins. In deciding whether to use a wet- or dry-detention basin as a water-quality control, an important consideration is whether nutrient removal (nitrogen and phosphorus) is an important requirement. This is particularly the case when the quality of the receiving water is sensitive to nutrient loadings. Properly designed wet-detention basins (detention ponds) generally provide much better nutrient removal than dry-detention basins, since many of the nutrients in surface runoff are in dissolved form and are not significantly affected by the sedimentation process in dry-detention basins (Hartigan, 1989). This functional advantage of wet-detention basins must be balanced against their greater land requirements. For example, the permanent pool of a wet-detention basin can require anywhere from two to seven times more storage than the alternative dry-detention basin.

Retention systems

Stormwater retention is the most effective quality-control method, but it can be used only in situations where the captured volume of water can infiltrate into the ground before the next storm. The most common retention systems are infiltration basins, swales, and below-ground exfiltration trenches. In many cases, these systems closely reproduce the predevelopment water balance (Dodson, 1998).

Infiltration basins. *Infiltration basins* are excavated areas that impound stormwater runoff, which then infiltrates into the ground. Infiltration basins, also called *dry-retention basins*, are similar in appearance and construction to dry-detention basins, except that the detained stormwater runoff is exfiltrated through permeable soils beneath the basin. The layout of a typical infiltration basin is illustrated in Figure 5.63. Infiltration basins typically serve areas ranging from front yards to 20 ha (ASCE, 1992). Infiltration basins are classified as either on-line or off-line. *On-line infiltration basins* retain a specified water-quality volume; when a larger runoff occurs, it overflows the basin, which then acts as a detention pond for the larger event. Some drainage systems divert the water-quality volume out of the normal drainage path and into *off-line infiltration basins* that hold it for later treatment.

Infiltration basins must be located in soils that allow the runoff to infiltrate within 72 hours, or within 24 to 36 hours for infiltration areas that are planted with grass. The seasonal high-water table should be at least 1.2 m (4 ft) below the ground surface in the infiltration basin to assure that the pollutants in the runoff are removed by the vegetation, soil, and microbes before reaching the water table (Urbonas and Stahre, 1993). Infiltration rates can be calculated using the Green–Ampt or similar models (see Section 5.3.3.3), but soils with saturated infiltration rates less than 8 mm/h are not suitable for infiltration basins (ASCE, 1998). Design guidelines suggested by ASCE (1998) are that water ponding in infiltration basins be less than 0.3 m during the design storm and that the design infiltration rate be limited to a maximum of 50 mm/h to account for clogging. The bottom of an infiltration basin should be graded as flat as

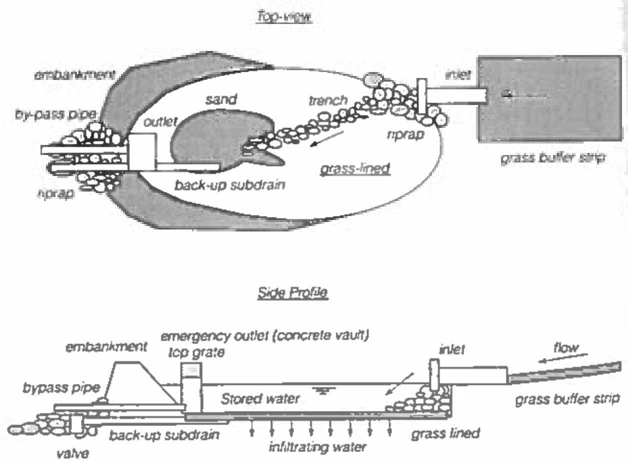


FIGURE 5.63: Layout of infiltration basin
Source: Guo (2001a).

possible to allow for uniform ponding and infiltration, and the side slope of the basin should be less than 3 : 1 (H : V) to allow for easier mowing and better bank stabilization. Water-tolerant turf such as reed canary grass or tall fescue should be used in the basin to promote better infiltration and pollutant filtering. In urban settings, infiltration basins are commonly integrated into recreational areas, greenbelts, neighborhood parks, and open spaces, while in highway drainage they may be located in rights-of-way or in open space within freeway interchange loops. Infiltration basins are susceptible to clogging and sedimentation, and can require large land areas. Although the two main problems typically associated with infiltration basins are clogging and contamination of underlying soil and ground water, data collected at several infiltration basins have indicated minimal clogging and soil-contamination depths less than 50 cm after about 20 years of operation (Dechesne et al., 2005). During routine operation, standing water in infiltration basins can create problems of security and insect breeding.

EXAMPLE 5.59

An infiltration basin is to be designed to retain the first 1.3 cm of runoff from a 10-ha catchment. The area to be used for the infiltration basin is turfed, and field measurements indicate that the native soil has a minimum infiltration rate of 150 mm/h. If the retained runoff is to infiltrate within 24 hours, determine the surface area that must be set aside for the basin.

Solution The volume, V, of the runoff corresponding to a depth of 1.3 cm = 0.013 m on an area of 10 ha = 10^5 m^2 is given by

$$V = (0.013)(10^5) = 1300 \text{ m}^3$$

The native soil has a minimum infiltration rate of 150 mm/h. To account for clogging, however, the design infiltration rate will be taken as 50 mm/h = 0.05 m/h. The area, A, of infiltration basin required to infiltrate 1300 m^3 in 24 h is given by

$$A = \frac{1300}{(0.05)(24)} = 1080 \text{ m}^2 = 0.11 \text{ ha}$$

If runoff is not to pond to more than 0.3 m, then the maximum volume that can be handled by a 0.11-ha infiltration basin is 0.3 m × 0.11 ha = 330 m^3. Since the runoff volume to be handled is 1300 m^3, the area of the infiltration basin should be at least 1300/330 × 0.11 ha = 0.43 ha.

In areas where the water table is shallow, particular care should be taken to assess the mounding of the water table below the basin. In cases where the infiltrated water causes the water table to rise to the ground elevation, ponding will occur and the infiltration basin will not function properly. A methodology for assessing the mounding beneath circular infiltration basins can be found in Guo (2001).

Swales. Swales are shallow vegetated (grass) open channels with small longitudinal and side slopes that transport and infiltrate runoff from adjacent land areas. Swales are commonly used in highway medians and for roadside drainage on rural roads. Typically, swales are designed as free-flowing open channels with inclined slopes, but they can also be designed to be nonflowing, with all the surface runoff retained and

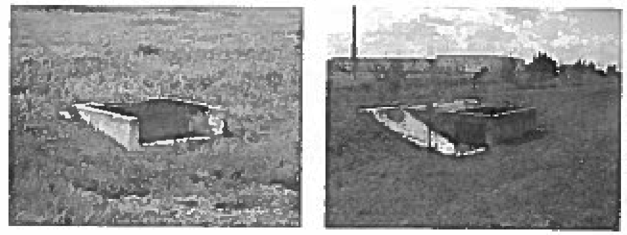


FIGURE 5.64: Overflow weirs from retention areas

infiltrated into the ground. Nonflowing swales are sometimes called *retention swales* (Field et al., 2000).

Retention swales can be designed either to retain a fixed volume of runoff or to retain the entire runoff from a design storm. Retention swales used for water-quality control are typically designed to retain a fixed depth of runoff and are sized based on their stage-storage relation; elevated outlets, such as those shown in Figure 5.64, are used for excess runoff above the water-quality volume. Retention swales used for water-quantity control are designed to retain the entire runoff from a design storm, and are sized such that the infiltration rate is equal to the peak runoff rate. In this case, if the peak runoff rate is Q_p, then the length, L, of swale required for the infiltration rate to be equal to the runoff rate is given by

$$L = \frac{Q_p}{fP} \tag{5.283}$$

where f is equal to the infiltration capacity of the soil and P is the wetted perimeter of the swale. For any cross-sectional shape, the wetted perimeter can be expressed in terms of the runoff rate via the Manning equation. For triangular-shaped swales, Equation 5.283 can be combined with the Manning equation to yield (Wanielista and colleagues, 1997)

$$\boxed{L = \frac{151,400 Q_p^{5/8} m^{5/8} S^{3/16}}{n^{3/8}(1 + m^2)^{5/8} f}} \tag{5.284}$$

where L is the swale length in meters, Q_p is the peak runoff rate in m^3/s, m is the side slope, S is the longitudinal slope, n is the Manning roughness coefficient, and f is the infiltration capacity in cm/h. Typical values of the Manning roughness coefficient are $n = 0.20$ for routinely mowed swales and $n = 0.24$ for infrequently mowed swales (ASCE, 1998). In the case of trapezoidal sections

$$\boxed{L = \frac{360,000 Q}{\left\{b + 2.38\left[\frac{Qn}{(2\sqrt{1+m^2}-m)S^{1/2}}\right]^{3/8}\sqrt{1+m^2}\right\} f}} \tag{5.285}$$

where b is the bottom width of the swale in meters. Equation 5.285 applies to the *best trapezoidal section*, where the perimeter, P, is related to the flow depth, y, and the side slope m by

$$P = 4y\sqrt{1 + m^2} - 2my \tag{5.286}$$

For swales to be effective, the infiltration rate of the underlying soil should be greater than 13 mm/h, the longitudinal grade should be set as flat as possible to promote infiltration, never steeper than 3%–5% (Urbonas and Stahre, 1993; Yu et al., 1994), and side slopes should be flatter than 4 : 1 (H : V) to maximize the contact area. It is sometimes recommended that swales be designed for 2-year storms and that the runoff volume in nonflowing or slow-moving swales be infiltrated within 36 hours (Urbonas and Stahre, 1993). In cases where the length of the swale required to infiltrate the runoff is excessive, then a *check dam* can be used to store the runoff within the swale, in which case the depth in the swale should not exceed 0.5 m. Check dams usually consist of crushed rock or pretreated timber up to 0.6 m in height. Grassed swales should be mowed to stimulate vegetative growth, control weeds, and maintain the capacity of the system.

EXAMPLE 5.60

A triangular-shaped swale is to retain the runoff from a catchment with design peak-runoff rate of 0.02 m^3/s. The longitudinal slope of the swale is to be 3% with side slopes of 4 : 1 (H : V). If the grassed swale has a Manning n of 0.24 (infrequently mowed grass) and a minimum infiltration rate of 150 mm/h, determine the length of swale required.

Solution From the given data: $Q = 0.02$ m^3/s, $m = 4$, $S = 0.03$, $n = 0.24$, and $f = 150$ mm/h $= 15$ cm/h. Equation 5.284 gives the required swale length, L, as

$$L = 151,400 \frac{Q^{5/8} m^{5/8} S^{3/16}}{n^{3/8}(1 + m^2)^{5/8} f}$$

$$= 151,400 \frac{(0.02)^{5/8}(4)^{5/8}(0.03)^{3/16}}{(0.24)^{3/8}(1 + 4^2)^{5/8} 15}$$

$$= 314 \text{ m}$$

This length of swale is quite long. A ponded infiltration basin would require less area and would probably be more cost effective.

In the previous example, the swale was designed to retain and infiltrate the entire surface runoff. In cases where the swale is to provide treatment of the surface runoff, primarily through trapping a portion of the sediment and organic biosolids in the vegetative cover, the swale is designed as a *biofilter* and is called a *biofiltration swale*. For the swale to perform adequately as a biofilter, the following design criteria are recommended (ASCE, 1998):

- Minimum hydraulic residence time of 5 minutes
- Maximum flow velocity of 0.3 m/s

- Maximum bottom width of 2.4 m
- Minimum bottom width of 0.6 m
- Maximum depth of flow no greater than one-third of the gross or emergent vegetation height for infrequently mowed swales, or no greater than one-half of the vegetation height for regularly mowed swales, up to a maximum height of approximately 75 mm for grass and approximately 50 mm below the normal height of the shortest plant species
- Minimum length of 30 m

For biofiltration swales, ASCE (1998) recommends longitudinal slopes of 1%–2%, with a minimum of 0.5% and a maximum of 6%. When the longitudinal slope is less than 1%–2%, perforated underdrains should be installed or, if there is adequate moisture, wetland species should be established. If the slope is greater than 2%, check dams should be used to reduce the effective slope to approximately 2%. Using these guidelines, the following design procedure is proposed for biofiltration swales (updated from ASCE, 1998):

1. Estimate the runoff rate for the design event and limit the discharge to approximately 0.03 m^3/s by dividing the flow among several swales, installing upstream detention to control release rates, or reducing the developed surface area to reduce the runoff coefficient and gain space for biofiltration.
2. Establish the slope of the swale.
3. Select a vegetation cover suitable for the site.
4. Estimate the height of vegetation that is expected to occur during the storm runoff season. The design flow depth should be at least 50 mm less than this vegetation height and a maximum of approximately 75 mm in biofiltration swales and 25 mm in filter strips.
5. Typically, biofiltration swales are designed as trapezoidal channels (skip this step for filter-strip design). When using a rectangular section, provide reinforced vertical walls.
6. For a trapezoidal cross section, select a side slope that is no steeper than 3:1 (H:V), with 4:1 (H:V) or flatter preferred.
7. Compute the bottom-width and flow velocity. Limit the design velocity to less than 0.3 m/s.
8. Compute the swale length using the design velocity from step 7 and an assumed hydraulic detention time, preferably greater than 5 minutes. If the computed swale length is less than 30 m, increase the swale length to 30 m and adjust the bottom-width.

Biofiltration swales are low-cost stormwater best-management practices (BMP) that have proven effective for controlling runoff pollution from land surfaces, especially highways and agricultural lands. Biofiltration swales are attractive options for agencies such as departments of transportation, since they are easily incorporated into the landscape, such as highway medians. The primary mechanisms for pollutant removal in swales are filtration by vegetation, settling of particulates, and infiltration into the subsurface zone. In general, biofiltration swales show good performance for removal of suspended solids, but are not considered efficient for removal of nutrients (Yu et al., 2001). Recent studies indicate that the most effective biofiltration swales have a minimum length of 75 m and a maximum longitudinal slope of 3% (Yu et al., 2001).

EXAMPLE 5.61

A biofiltration swale is to be constructed on a 1% slope to handle a design runoff of 0.03 m^3/s. During the wet season, the swale is expected to be covered with grass having an average height of 130 mm with Class E retardance. Design the biofiltration swale.

Solution From the given data: $Q = 0.03$ m^3/s, $S_0 = 0.01$, and the average height of the vegetation is 130 mm. This given data covers the specifications in steps 1 to 3 of the design procedure.

Step 4. The design depth in the biofiltration swale should be at least 50 mm below the height of the vegetation (130 mm − 50 mm = 80 mm), with a maximum height of 75 mm. Therefore, in this case, the design flow depth is taken as 75 mm.

Step 5. Use a trapezoidal section for the swale.

Step 6. Use side slopes of 4:1 ($m = 4$), a bottom-width b, and a depth $y = 75$ mm ($=0.075$ m). The flow area, A, wetted perimeter, P, and hydraulic radius, R, are given by

$$A = by + my^2 = b(0.075) + (4)(0.075)^2 = 0.075b + 0.0225$$

$$P = b + 2\sqrt{1+m^2}\,y = b + 2\sqrt{1+4^2}(0.075) = b + 0.618$$

$$R = \frac{A}{P} = \frac{0.075b + 0.0225}{b + 0.618} \tag{5.287}$$

where 0.6 m $< b <$ 2.4 m.

Step 7. The Manning equation requires that

$$Q = \frac{1}{n}AR^{2/3}S_0^{1/2} = \frac{1}{n}\frac{A^{5/3}}{P^{2/3}}S_0^{1/2}$$

In this case,

$$0.03 = \frac{1}{n}\frac{(0.075b + 0.0225)^{5/3}}{(b + 0.618)^{2/3}}(0.01)^{1/2}$$

or

$$\frac{1}{n^3}\frac{(0.075b + 0.0225)^5}{(b + 0.618)^2} = 0.027 \tag{5.288}$$

For Class E retardance, Manning's n is given by Table 3.23 as

$$n = \frac{1.22R^{1/6}}{52.1 + 19.97\log(R^{1.4}S_0^{0.4})} \tag{5.289}$$

Simultaneous solution of Equations 5.287, 5.288, and 5.289 gives

$$b = 4.44 \text{ m}$$

which exceeds the maximum bottom-width (for a uniform flow distribution) of 2.4 m. Repeated solution of Equations 5.287, 5.288, and 5.289

shows that a flow of 0.014 m^3/s will require a bottom-width, b of 2.4 m, with a flow velocity ($= Q/A$) of 0.067 m/s. Since the flow velocity is less than the maximum velocity of 0.3 m/s, two swales each having a bottom-width of 2.4 m and handling half the flow should be used.

Step 8. Using a detention time of 5 minutes with the design velocity of 0.067 m/s gives the length, L, of the swale as

$$L = Vt = (0.067)(5 \times 60) = 20.1 \text{ m}$$

which is shorter than the minimum length of 30 m. Therefore use a length of 30 m.

In summary, two biofiltration swales are required, each with a trapezoidal cross-section with a bottom width of 2.4 m, side slopes of 4 : 1, and 30 m long.

Vegetated filter strips. *Vegetated filter strips* (also called *buffer strips*) are mildly sloping vegetated surfaces, usually grass, that are located between impervious surfaces (pollutant sources) and water-quality control areas. Vegetated filter strips are designed to slow the runoff velocity from the impervious area, reducing the peak-runoff rate and increasing the opportunities for infiltration, sedimentation, and trapping of the pollutants. Vegetated filter strips are often used as pretreatment (to remove sediments) for other structural practices such as dry-detention basins and exfiltration trenches. These areas are designed to receive overland sheet flow, and they provide little treatment for concentrated flows. The design procedure for filter strips is the same as that for biofiltration swales, with the additional constraints that the average depth of flow be no more than 25 mm and the hydraulic radius be taken equal to the flow depth (ASCE, 1998). The width of the filter strip should be sufficiently limited to achieve a uniform-flow distribution. Grassed filter strips may develop a berm of sediment at the upper edge that must be periodically removed. Mowing will maintain a thicker vegetative cover, providing better sediment retention. Recommended areas for use are in agriculture and low-density developments. Although studies indicate highly varying effectiveness, trees in strips can be more effective than grass strips alone because of their greater uptake and long-term retention of plant nutrients. Properly constructed forested and grass fiter strips can be expected to remove more than 60% of the particulates and perhaps as much as 40% of plant nutrients in urban runoff (Dodson, 1998). In arid and semiarid climates, grass buffer strips need to be irrigated (Field et al., 2000).

Exfiltration trenches. Exfiltration trenches, also called *infiltration trenches*, *percolation trenches*, and *french drains*, are common in urban areas with large impervious areas and high land costs. An exfiltration trench typically consists of a long narrow excavation, ranging from 1 to 4 m in depth, backfilled with gravel aggregate (2.5 to 7.6 cm) and surrounded by a filter fabric to prevent the migration of fine soil particles into the trench, which can cause clogging of the gravel aggregate (Harrington, 1989). The maximum trench depth is limited by trench-wall stability, seasonal high ground-water levels, and the depth to any impervious soil layer. Exfiltration trenches 1 m wide and 1 to 2 m deep seem to be most efficient (ASCE, 1998). Exfiltration trenches can have their top elevation either at the ground surface or below ground.

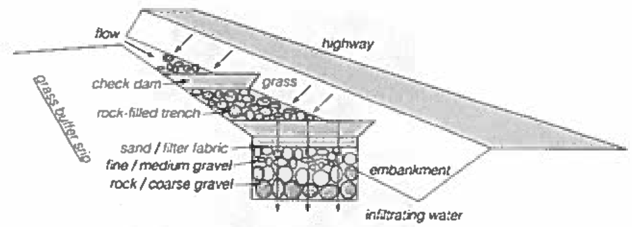


FIGURE 5.65: Layout of surface exfiltration trench
Source: Guo (2001a).

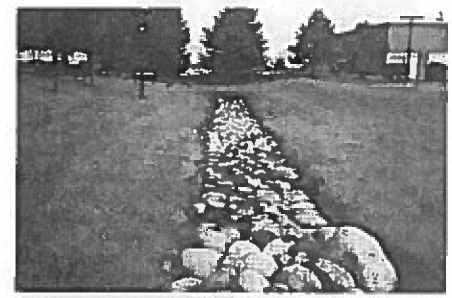


FIGURE 5.66: Picture of surface exfiltration trench
Source: Guo (2001a).

Surface trenches receive sheet flow runoff directly from adjacent areas after it has been filtered by a grass buffer. The layout of a typical surface exfiltration trench is shown in Figure 5.65 and pictured in Figure 5.66. Surface exfiltration trenches are typically used in residential areas where smaller loads of sediment and oil can be trapped by grassed filter strips that are at least 6 m wide (Harrington, 1989). For below-ground exfiltration trenches, runoff is collected by a stormwater inlet on top of a catch basin, and water collected in the catch basin is delivered to the below-ground exfiltration trench by a perforated pipe. A below-ground exfiltration trench under construction is shown in Figure 5.67, and a close-up view of the perforated pipe used in the trench is shown in Figure 5.68. The exfiltration trench shown in Figure 5.67 consists of a catch basin in the center, perforated pipe extending from both sides into the trench, and a partial fill of rockfill aggregate which is surrounded by a filter fabric. For below-ground exfiltration trenches, a minimum of 0.3 m of soil cover should be provided for the establishment of vegetation. Adequate ground-water (quality) protection is generally obtained by providing at least 1.2 m separation between the bottom of the trench and the seasonal high water table.

Exfiltration trenches are frequently used for highway median strips and parking lot "islands" (depressions between two lots or adjacent sides of one lot). To prevent clogging, inlets to underground exfiltration trenches should include trash racks, catch basins, and baffles to reduce sediment, leaves, other debris, and oils and greases. Exfiltration trenches are especially vulnerable to clogging during construction. Once

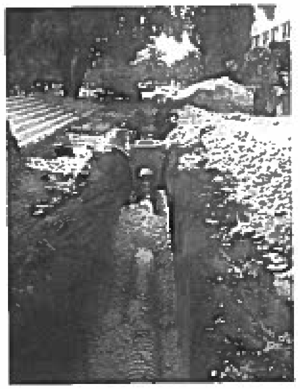


FIGURE 5.67: Below-ground exfiltration trench under construction

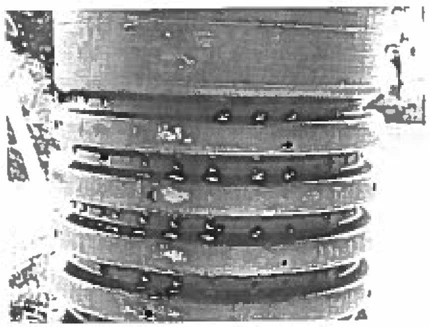


FIGURE 5.68: Perforated pipe used in below-ground exfiltration trench

operational, routine maintenance consists of vacuuming debris from the catch basin inlets and, if needed, using high-pressure hoses to wash clogging materials out of the pipe. However, once trenches are clogged, rehabilitation is difficult. Surface trenches are more susceptible to sediment accumulations than underground trenches, but their accessibility makes them easier to maintain. About 50% of exfiltration trenches constructed in the eastern United States have failed (Schueler et al., 1991). Although the nature and reason for these failures are not typically reported, clogging within the trench and of its infiltrating surfaces is suspected (Field et al., 2000).

Exfiltration trenches are considered off-line systems, and their purpose is to store and exfiltrate the runoff from frequent storms into the ground. Pollutant-removal mechanisms in exfiltration trenches include adsorption, filtering, and microbial decomposition below the trench. Exfiltration trenches are typically designed to serve

single-family residential areas up to 4 ha (10 ac) and commercial areas up to 2 ha (5 ac) in size, and are particularly suited for rights-of-way, parking lots, easements, and other areas with limited space. Due to concerns about soil and ground-water contamination, exfiltration trenches should not be used at industrial and commercial sites that may be susceptible to spillage of soluble pollutants, such as gasoline, oils, and solvents.

Exfiltration trenches can be used only at sites with porous soils, favorable site geology, and proper ground-water conditions. Site conditions that are favorable to exfiltration trenches are (Stahre and Urbonas, 1989; Harrington, 1989; ASCE, 1998):

1. The hydraulic conductivity surrounding the trench exceeds 2 m/d.
2. The distance between the bottom of the trench and the seasonal high water table or bedrock exceeds 1.2 m.
3. Water-supply wells are more than 30 m from the trench (to prevent possible contamination).
4. The trench is located at least 6 m from building foundations (to avoid possible hydrostatic pressures on foundations or basements).
5. The ground slope downstream of the trench does not exceed 20%, which would increase the chance of downstream seepage and slope failure.

Exfiltration trenches are generally designed to retain a volume equal to the difference between the runoff volume and the volume of water exfiltrated during a storm (ASCE, 1998). Assuming that the water in the trench percolates through one-half of the trench height, saturated-flow conditions exist between the trench and the water table, and there is negligible outflow from the bottom of the trench (due to clogging), the total outflow rate, Q_{out}, from the two (long) sides of the trench is given by Darcy's law (see Chapter 6, Section 6.2.1) as

$$Q_{out} = 2\left(K_t \frac{H}{2} L\right) = K_t HL \tag{5.290}$$

where K_t is the trench hydraulic conductivity, H is the height of the trench, L is the length of the trench, and the hydraulic gradient is assumed equal to unity. In cases where saturated-flow conditions do not exist between the trench and the water table, the assumption of a unit hydraulic gradient gives a conservative estimate of the trench outflow (Duchene et al., 1994). Taking t as the duration of the storm, the volume exfiltrated in time t, V_{out}, is given by

$$V_{out} = Q_{out} t = K_t HLt \tag{5.291}$$

The runoff volume into the trench during time t, V_{in}, is given by the rational formula (see Section 5.4.2.1) as

$$V_{in} = CiAt \tag{5.292}$$

where C is the runoff coefficient, i is the average intensity of a storm with duration t, and A is the area of the catchment contributing flow to the exfiltration trench. The storage capacity of the trench, V_{stor}, is given by

$$V_{stor} = nWHL \tag{5.293}$$

where n is the porosity in the trench, typically taken as 40% (ASCE, 1998). The trench dimensions must be such that

$$V_{\text{in}} = V_{\text{stor}} + V_{\text{out}} \tag{5.294}$$

Combining Equations 5.291 to 5.294 yields

$$CiAt = nWHL + K_t HLt \tag{5.295}$$

Solving for the trench length, L, gives

$$\boxed{L = \frac{CiAt}{(nW + K_t t)H}} \tag{5.296}$$

The rainfall intensity, i, is related to the storm duration, t, by an intensity-duration-frequency (IDF) curve that typically has the form

$$i = \frac{a}{(t + b_1)^{c_1}} \tag{5.297}$$

where a, b_1, and c_1 are constants (see Section 5.2.1). The combination of Equations 5.296 and 5.297 indicates that the required trench length, L, varies as a function of the storm duration, t, and the design length should be chosen as the maximum length required for any storm duration (with a given return period). In some cases, the runoff from a specified rainfall depth is required to be handled by the trench, and in these cases ASCE (1998) recommends that the trench be designed for an IDF curve with a return period in which the specified rainfall depth falls in 1 hour. The contributing area to an exfiltration trench is usually less than 4 ha due to storage requirements for peak-runoff control.

EXAMPLE 5.62

An exfiltration trench is to be designed to handle the runoff from a 1-ha commercial area with an average runoff coefficient of 0.7. The IDF curve for the design rainfall is

$$i = \frac{548}{(t + 7.24)^{0.73}}$$

where i is the average rainfall intensity in mm/h and t is the storm duration in minutes. The trench hydraulic conductivity estimated from field tests is 15 m/d, the seasonal high water table is 5 m below the ground surface, and local regulations require that a safety factor of 2 be applied to the trench hydraulic conductivity to account for clogging. Design the exfiltration trench.

Solution From the given data: $C = 0.7$, $A = 1$ ha $= 10{,}000$ m^2, and $K_t = 15/2 = 7.5$ m/d $= 0.31$ m/h (using a safety factor of 2). According to ASCE (1998) guidelines, the porosity, n, of the gravel pack can be taken as 40% ($n = 0.4$), and a trench width, W, of 1 m and height, H, of 2 m can be expected to perform efficiently. Substituting these values into Equation 5.296 gives

$$L = \frac{CiAt}{(nW + K_t t)H} = \frac{(0.7)i(10{,}000)t}{(0.4 \times 1 + 0.31t)(2)} = \frac{7000it}{0.8 + 0.62t} \tag{5.298}$$

where i in m/h, and t in hours are related by

$$i = \frac{0.548}{(60t + 7.24)^{0.73}} \text{ m/h} \tag{5.299}$$

Combining Equations 5.298 and 5.299 gives the required trench length, L, as a function of the storm duration, t, as

$$L = \frac{3836t}{(0.8 + 0.62t)(60t + 7.24)^{0.73}} \tag{5.300}$$

Taking the derivative with respect to t gives

$$\frac{dL}{dt} = \frac{[(0.8 + 0.62t)(60t + 7.24)^{0.73}]3836 - 3836t[(0.8 + 0.62t)0.73(60t + 7.24)^{-0.27}60 + (60t + 7.24)^{0.73}0.62]}{[(0.8 + 0.62t)(60t + 7.24)^{0.73}]^2}$$

$$= \frac{3836(0.8 + 0.62t)(60t + 7.24)^{0.73} - 168{,}000t(0.8 + 0.62t)(60t + 7.24)^{-0.27} - 2378t(60t + 7.24)^{0.73}}{[(0.8 + 0.62t)(60t + 7.24)^{0.73}]^2}$$

and the maximum-value criterion, $dL/dt = 0$, yields

$$3836(0.8 + 0.62t)(60t + 7.24)^{0.73} - 168{,}000t(0.8 + 0.62t)(60t + 7.24)^{-0.27} - 2378t(60t + 7.24)^{0.73} = 0$$

which gives $t = 0.759$ h, and substituting into Equation 5.300 yields $L = 127$ m. On the basis of this result, a trench length of 127 m gives the trench volume required to handle the design storm without causing surface ponding. Since the seasonal high-water table is 5 m below the ground surface and the minimum allowable spacing between the bottom of the trench and the water table is 1.2 m, a (maximum) trench height of 5 m $-$ 1.2 m $=$ 3.8 m would still be adequate and would yield the shortest possible trench length (for the specified trench width of 1 m). Since the trench length is inversely proportional to the trench height (see Equation 5.296), using a trench height, H, of 3.8 m would give a required trench length, L, of

$$L = 127 \text{ m} \times \frac{2}{3.8} = 67 \text{ m}$$

Hence, a trench capable of handling the design storm is 1 m wide, 3.8 m high, and 67 m long. If the (vertical) side slopes are not stable for a trench of this depth, then the trench height and length can be adjusted according to their inverse proportionality.

In designing an exfiltration trench there must be reasonable assurance that the aquifer beneath it can conduct water away from the trench as fast as water exfiltrates from the trench to the water table. Under normal circumstances, the water table beneath the trench will mound until there is sufficient induced gradient to conduct the exfiltrated water away. The trench will fail if the water-table mound rises above the trench bottom to interfere with the operation of the trench. A method to predict the mounding depth under exfiltration trenches is presented in Chapter 6, Section 6.8.8.

5.7.2.4 Best-management practices

A stormwater *best-management practice* (BMP) is a method or combination of methods found to be the most effective and feasible means of preventing or reducing the amount of pollution generated by nonpoint sources to a level compatible with water-quality goals (South Florida Water Management District, 2002). Methods of controlling pollutants in stormwater runoff are categorized as nonstructural or structural practices. Nonstructural BMPs are practices that improve water quality by reducing the accumulation and generation of potential pollutants at or near their source. Structural BMPs involve building an engineered facility for controlling quantity and quality of urban runoff.

Nonstructural BMPs generally fall into the following four categories: planning and regulatory tools; source controls; maintenance and operational procedures; and educational and outreach programs. Planning and regulatory tools include hazardous materials codes, zoning, land-development and land-use regulations, water-shortage and conservation policies, and controls on types of flow allowed to drain into municipal storm-sewer systems. Source controls include erosion and sediment control during construction, collection and proper disposal of waste materials, and modified use of chemicals such as fertilizer, pesticides, and herbicides. Maintenance and operational procedures include turf and landscape management, street cleaning, catch-basin cleaning, road maintenance, and canal/ditch maintenance. Educational and outreach programs include distributing toxics checklists for meeting household hazardous-waste regulations, producing displays and exhibits for school programs, distributing free seedlings for erosion control, and creating volunteer opportunities such as water-quality monitoring. Prevention practices such as planning and zoning tools to ensure setback, buffers, and open-space requirements can be implemented with ease at the planning stage of any development with a high degree of success

Structural BMPs for controlling stormwater runoff generally fall into two main categories: retention systems and detention systems. Retention BMP systems include exfiltration trenches, infiltration basins, grassed swales, vegetated filter strips, and retention ponds. Detention BMP systems include dry- and wet-detention basins. Important factors to be considered in selecting a BMP are the amount of runoff to be handled by the system and the soil type. Guidance for selecting best-management practices is given in Table 5.40. Generally, well-designed BMPs listed in Table 5.40 can provide high pollutant removal for nonsoluble particulate pollutants, such as suspended sediment and trace metals. Much lower removal rates are achieved for soluble pollutants such as phosphorus and nitrogen. Each site has its natural attributes that influence the type and configuration of the stormwater-management system. For

TABLE 5.40: Water-Quality Control System Selection Criteria (USFHWA, 1996)

Best management practices	Contributing Area (ha)					Acceptable NRCS soil types
	0–2	2–4	4–12	12–20	>20	
Exfiltration trenches	•	•				A, B
Infiltration basins		•	•	•		A, B
Grassed swales	•	•	•			A, B, C
Filter strips	•					A, B, C
Retention ponds			•	•	•	B, C, D
Detention ponds			•	•	•	A, B, C, D

example, a site with sandy soils would suggest the use of infiltration practices such as retention areas integrated into a development's open space and landscaping, while natural low areas offer opportunities for detention systems.

A comprehensive stormwater-management program should include a combination of structural and nonstructural components that are properly selected, designed, implemented, inspected, and regularly maintained.

5.7.3 Major System

The major drainage system includes features such as natural and constructed open channels, streets, and drainage easements such as floodplains. The major drainage system handles runoff events that exceed the capacity of the minor drainage system, typically the 100-year storm, and must be planned concurrently with the design of the minor drainage system. The design of open channels has been discussed extensively in Chapter 4, Section 3.5. In major urban drainage systems, concrete- and grass-lined channels are the most common, with grass lining usually preferred for aesthetic reasons. Concrete-lined channels have smaller roughness coefficients, require smaller flow areas, and are used when hydraulic, topographic, and right-of-way needs are important considerations.

5.8 Evapotranspiration

Evaporation is the process by which water is transformed from the liquid phase to the vapor phase, and *transpiration* is the process by which water moves through plants and evaporates through leaf stomatae, which are small openings in the leaves. In cases where the ground surface is covered by vegetation, it is usually not easy to differentiate between evaporation from the ground surface and transpiration through plants; the combined process of evaporation and transpiration is called *evapotranspiration*. The relative contribution of evaporation and transpiration to evapotranspiration varies. For example, when crops are in their initial stages of development and cover less than 10% of the ground area, water is predominantly lost by soil evaporation, but once the crop is well developed and completely covers the soil, transpiration becomes the dominant process. Evaporation from soil is affected by such factors as soil water content, type, the presence or absence of surface mulches, and environmental conditions imposed on the soil (Burt et al., 2005); transpiration is affected by such factors as the type of vegetation and the stage of development.

Evapotranspiration does not contribute significantly to the water budget over time scales of individual storms, but over longer time periods (weeks and months) it is a major component in the terrestrial water budget. On an annual basis, approximately 70% of the rainfall in the United States is returned to the atmosphere via evapotranspiration, and predicting evapotranspiration is of primary interest in the design of irrigation and surface-water storage systems.

Three standard evapotranspiration rates are commonly used in practice: (1) potential evapotranspiration, (2) reference-crop evapotranspiration, and (3) actual evapotranspiration. *Potential evapotranspiration* is used synonymously with the term *potential evaporation*, which is defined as the quantity of water evaporated per unit area, per unit time, from an idealized, extensive free water surface under existing atmospheric conditions. Potential evaporation is typically used as a measure of the meteorological control on evaporation from an open water surface (e.g., lake,