10 Urban Stormwater Management

The purpose of urban stormwater management is to enhance the quality of life in urban areas by (1) protecting human life and reducing flood safety risks, (2) preventing damage to private and public property, (3) minimizing the disruption of normal urban activities caused by storm runoff, and (4) protecting water quality. Urban stormwater management includes the development of structural and non-structural measures necessary to achieve these objectives, and requires a basic knowledge of hydrology and open channel hydraulics.

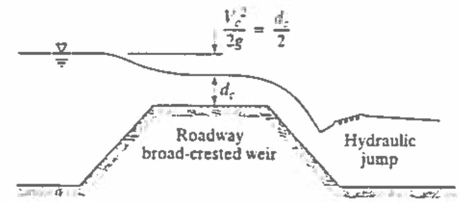
The National Flood Insurance Program was established in 1968 to restrict development in flood-prone areas and provide federally-backed flood insurance. The program is administered through the Federal Insurance Administration and Mitigation Directorate of the Federal Emergency Management Agency. Over 19,000 communities, representing most of the urban areas in the U.S., participate in the program. Participating communities must enforce mandatory land use regulations for flood prone areas. A flood insurance study is required to delineate the 10-, 50-, 100-, and 500-year recurrence interval flood-prone areas. The flood insurance study is used to establish flood insurance premiums and land use restrictions for the 100-year floodplain. The 100-year return period flood has become widely accepted as the standard for flood damage mitigation.

Because of the flood safety risks, low water crossing structures on major streams should be avoided in urban areas. A low water crossing structure (sometimes referred to as dips) is when the road cross drainage structure (such as a culvert) is undersized, and during a major flood event, water flows over the

roadway. A vehicle might be stalled in the roadway during a flood, and if the water gets high enough, the vehicle can be washed off the roadway into the downstream channel. The roadway becomes a broad-crested weir, and with a low tailwater elevation, the velocity of the water on the roadway will be near critical velocity.

Example 10.1 Low Water Crossing Structure

Assume a vehicle was stalled at a low water crossing structure, and the water depth on the roadway was 2.0 ft and rising. The person in the vehicle can either stay in the vehicle and hope that vehicle remains on the roadway or try and walk to safety. Compute the force on the vehicle for a water depth of 2.0 ft and a drag coefficient of 2.0. The vehicle is 16 ft long, with 1.0 ft of clearance under the vehicle and at a water depth of 2.0 ft has an area of 19.0 ft^2 exposed to the flow.



Velocity

$$V_c = \sqrt{2gd_c/2} = \sqrt{2g(2.0)/2} = 8.0 \text{ fps}$$

Force

$$F_D = \frac{C_D \rho A V_c^2}{2} = 2.0 \times 1.94 \times 19.0 \times \frac{8.0^2}{2.0} = 2{,}360 \text{ lbs}$$

Compute the drag force on a person who is standing sideways to the flow and 0.75 wide, with a drag coefficient of 1.2.

$$F_D = 1.2 \times 1.94 \times (0.75 \times 2.0) \times \frac{8.0^2}{2.0} = 112 \text{ lbs}$$

Because of buoyancy, the vehicle will probably not remain on the roadway, and it will be extremely difficult to walk to safety.

10.1 STORMWATER COLLECTION SYSTEMS

In urban areas, stormwater runoff generally occurs as sheet flow to the streets, conveyed as gutter flow along the streets to drain inlets; collected by storm sewers in buried conduits; and discharged into streams, lakes, or ponds. Because of the adverse effect that urbanization has on water quality and the peak discharge rate, a detention and/or sedimentation basin may be required before the effluent from the storm sewer is discharged into the receiving stream or lake. A schematic of a stormwater collection system is shown in Fig. 10.1. The stormwater collection system is a combination of streets and storm sewers. The lots are graded so that the runoff flows toward the street, and the street conveys the runoff to the storm sewer drain inlets. The storm sewers are generally designed for a 5-, 10-, or 25-year return period storm. Some street flooding can be expected during a major flood event. Because the stormwater collection system consists of both the streets and storm sewers, the system is designed for at least a 100-year return period storm. The streets must be able to carry flood flow that is in excess of the storm sewer capacity.

The system of streets and storm sewers used to collect stormwater runoff is often referred to as the minor urban drainage system, and the rational method is commonly used to estimate the peak discharge rates. The rational equation is expressed as

$$Q = CIA \tag{8.11}$$

where Q is the peak discharge rate in cfs (cms) at the point of design in the collection system, C is the coefficient of runoff representing the ratio of runoff rate to rainfall rate, I is the average rainfall intensity in in./hr (m/s) during a time period equal to the time of concentration, and A is the drainage area in acres (m^2) contributing to the flow at the point of design. The time of concentration (t_c) is the time required for water to flow from the most hydraulically remote point in the drainage area to the point of design. t_c can also be defined as the time for the runoff from the drainage area to reach equilibrium under a steady rainfall rate.

Although the total storm duration may be considerably longer than t_c, the rainfall intensity (I) is the maximum average rainfall intensity that occurs in the storm for a period of time equal to t_c. The rainfall intensity for the design of the stormwater collection system can be given by the intensity-duration frequency (IDF) equation such as

$$I = \frac{a}{(t_c + b)^c} \tag{7.50}$$

where I is rainfall intensity in in./hr (mm/hr); t_c is time of concentration in minutes; and a, b, and c are constants for the location and storm frequency.

The area contributing to the flow at design point 1 in Fig. 10.2 is ABGF, whereas the area contributing to the flow at design point 5 is AELK. As the drainage area becomes larger, the time of concentration increases and the design rainfall intensity decreases. The time of concentration for the storm sewer system consists of overland flow time plus gutter flow time plus conduit flow time. The gutter flow time and the conduit flow time can be estimated by dividing the channel length by the average water velocity.

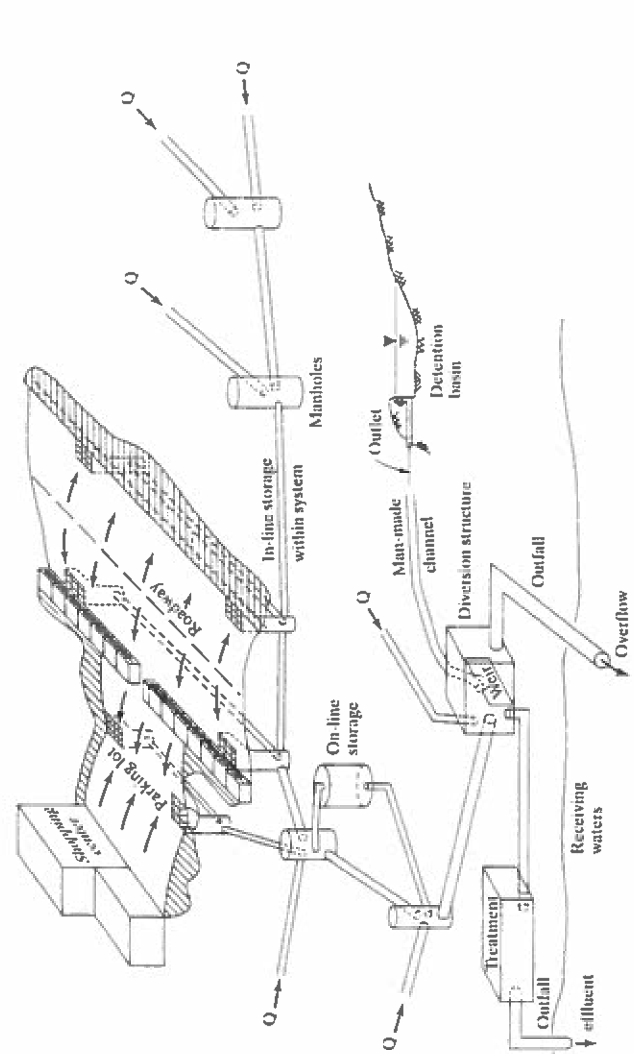
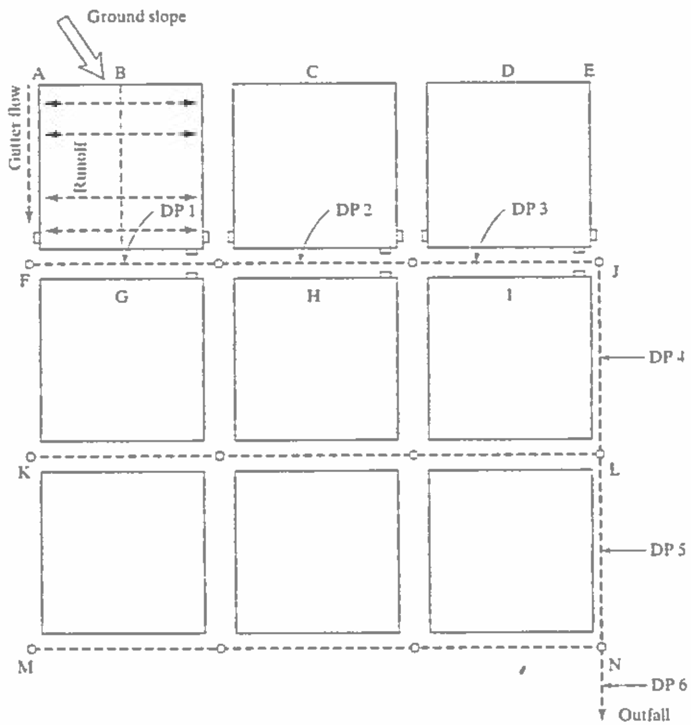


Figure 10.1 Principal elements in urban stormwater collection system. (American Society of Civil Engineers and Copyright Water Environment Federation, Alexandria, Va. 1992; reprinted with permission.)



Legend
DP - Design point
○ - Manhole
- Drain inlet located upstream PD cross walk

DP	Area
1	ABGF
2	ACHF
3	ADIF
4	AEJF
5	AELK
6	AENM

Figure 10.2 Typical stormwater collection system.

The overland flow time can be estimated based on the kinematic wave equation

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = CI \tag{6.13}$$

Using Manning's equation

$$q = \alpha y^m$$

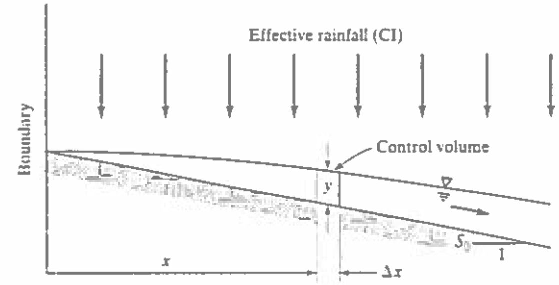


Figure 10.3 Kinematic wave overland flow travel time.

where

$$\alpha = \frac{C_m S_o^{1/2}}{n}$$

and

$$m = 5/3$$

Combining the kinematic wave and the Manning's equations gives

$$\frac{\partial y}{\partial t} + \alpha m y^{m-1} \frac{\partial y}{\partial x} = CI \tag{10.1}$$

The control volume in Fig. 10.3 is moving at a velocity of dx/dt. If t is the time for the control volume to travel from the boundary to a distance x, it follows that

$$\frac{dy}{dt} = CI = \frac{\partial y}{\partial t} + \frac{dx}{dt}\frac{\partial y}{\partial x} \tag{10.2}$$

Comparing Eqs. 10.1 and 10.2 gives

$$\frac{dx}{dt} = \alpha m y^{m-1} \tag{10.3}$$

Integrating Eq. 10.2 gives

$$y = CIt \tag{10.4}$$

Combining Eqs. 10.3 and 10.4 and integrating gives

$$x = \alpha (CI)^{m-1} t^m \tag{10.5}$$

The travel time (t_c) for $x = L$ is

$$t_c = \left(\frac{L}{\alpha (CI)^{m-1}}\right)^{1/m} \tag{10.6}$$

where t_c = time of concentration in seconds.

$$\alpha = \frac{C_m S_o^{1/2}}{n}$$

m = 5/3 for turbulent flow
S_o = slope of the ground
C_m = 1.49 (1.0)
n = Manning's roughness coefficient for overland flow (Table 6.1)
L = length of overland flow, ft (m)
C = runoff coefficient and
I = average rainfall intensity, fps (mps).

Table 10.1 can be used to estimate the runoff coefficient (C) based on land use, soil type, ground slope, and rainfall intensity. Two sets of runoff coefficients

TABLE 10.1 RUNOFF COEFFICIENTS FOR USE IN THE RATIONAL FORMULA

	NRCS Hydrologic soil group and ground surface slope range											
Land use	A			B			C			D		
	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+
Impervious areas	0.90[*]	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
	0.95[†]	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Freeways and	0.57	0.59	0.60	0.58	0.60	0.61	0.59	0.61	0.63	0.60	0.62	0.64
expressways	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
High density[‡]	0.47	0.49	0.50	0.48	0.50	0.52	0.49	0.51	0.54	0.51	0.53	0.56
residential	0.58	0.60	0.61	0.59	0.61	0.64	0.60	0.62	0.66	0.62	0.64	0.69
Medium density[§]	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
residential	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Low density[¶]	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.25	0.28	0.35
residential	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Agricultural	0.08	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Open space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39

[*]Smaller runoff coefficients for use with storm recurrence intervals less than 25 years.
[†]Larger runoff coefficients for use with storm recurrence intervals of 25 years or more.
[‡]High density residential = greater than 15 dwelling units per acre.
[§]Medium density residential = 4–15 dwelling units per acre.
[¶]Low density residential = 1–4 dwelling units per acre.

are listed in Table 10.1 for each land use category. The smaller values of runoff coefficients are to be used with storm recurrence intervals of less than 25 years, and the larger values are to be used for storm recurrence intervals of 25 years or more.

Example 10.2 Time of Concentration

Determine the time of concentration and peak discharge rate for design point 1 in Fig. 10.2. The stormwater collection system is for a high-density residential area with NRCS (Natural Resource Conservation Service) hydrologic soil group C. The lots have been graded to drain toward the street at a slope of 0.01 and have a Manning "n" value for overland flow of 0.2. The street gutter (A–F) has a longitudinal slope of 0.015, a Manning "n" value of 0.017, and a transverse side slope of 30:1.

The blocks in Fig. 10.2 are 350 feet square, which includes a 50-ft wide street. The storm sewers are to be designed for a 25-year return period storm. The design IDF (intensity-duration-frequency) equation for the system is

$$I = \frac{89}{(t_c - 8.5)^{0.754}}$$

where I is rainfall intensity in in./hr.

Solution The residential area contributing to the flow is

$$A_1 = \frac{300 \times 150}{43{,}560} = 1.03 \text{ ac}$$

with a runoff coefficient of 0.60 (Table 10.1). The area of the street contributing to the flow is

$$A_2 = \frac{325 \times 25}{43{,}560} = 0.19 \text{ ac}$$

with a runoff coefficient of 0.95.

Equation 10.6 is used to compute the overland flow travel time. The time of concentration (t_c) is

$$t_c = t_{\text{overland}} + t_{\text{gutter}}$$

For BG units, Eq. 10.6 can be written as

$$t_c = \frac{1}{60}\left(\frac{L}{\alpha(CI)^{m-1}}\right)^{1/m}$$

where

$$\alpha = \frac{C_m S_o^{1/2}}{n}$$

$C_m = 1.49$
$S_o = 0.01$
$n = 0.2$
$\alpha = 0.74$
$L = 150$ ft
$C = 0.60$
$m = 5/3$

and

$$I = \frac{89}{(t_c + 8.5)^{0.754}} \text{ in./hr} = \frac{0.00206}{(t_c + 8.5)^{0.754}} \text{ fps}$$

Using an assumed value of 1.5 minutes for gutter flow time, the above equation becomes

$$t_c = \frac{1}{60}\frac{150^{0.5}(t_c + 8.5)^{0.30}}{0.74^{0.6}(0.6 \times 0.00206)^{0.4}} + 1.5 = 5.88(t_c + 8.5)^{0.30} - 1.5$$

$$t_c = 17.2 \text{ min}$$

$$I = \frac{89}{(17.2 + 8.5)^{0.754}} = 7.7 \text{ in./hr}$$

Q_{peak} at design point 1 = CIA = $(0.60 \times 1.03 + 0.95 \times 0.19)7.7 = 6.1$ cfs
Check gutter travel time based on an average discharge rate of 3.1 cfs.

$$Q = \frac{1.49}{n}\frac{SS}{3.2}d^{8/3}S_o^{1/2}$$

$$3.1 = \frac{1.49}{0.017}\frac{30}{3.2}d^{8/3}(0.015)^{1/2} = 101\,d^{8/3}$$

$$d = 0.27 \text{ ft}$$

$$A_g = \frac{SS}{2}d^2 = 15(0.27)^2 = 1.09 \text{ ft}^2$$

$$V_g = \frac{Q}{A_g} = \frac{3.1}{1.09} = 2.8 \text{ fps}$$

$$t_{gutter} = \frac{300}{2.8 \times 60} = 1.8 \text{ min}$$

Correcting for gutter travel time

$$t_c = 17.3 \text{ min}$$

$$I = 7.7 \text{ in./hr}$$

$$Q = 6.1 \text{ cfs}$$

10.1.1 Design Criteria

The design capacity of the combined street and storm sewer collection system must equal or exceed the runoff rate for the 100-year return period storm. During a major flood event, street flooding may occur, but residential, public, commercial, and industrial buildings should not flood (Fig. 10.4). The capacity of the storm sewer collection system is generally selected to handle a 5-, 10-, or 25-year return period storm. Streets and open channels are required to carry the flood flows that are in excess of the storm sewer capacity.

The design of the stormwater collection system is based on the principles of steady flow in open channels, and Manning's equation is typically used to compute the water velocity and depth in gutters and the velocity and pipe diameter in the storm sewers.

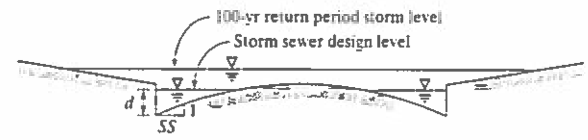


Figure 10.4 Water levels in street.

10.1.2 Street Gutters

The discharge rate in a triangular curb gutter based on Manning's equation is

$$Q = \frac{C_m}{n}\frac{SS}{3.2}d^{8/3}S_o^{1/2} \tag{5.9}$$

where Q is the discharge rate in cfs (cms), C_m is 1.49 for British Gravitational (BG) units and 1.0 for the International System (SI) of units, n is Manning roughness coefficient, SS is the street cross slope (ranges from 10 to 50), d is the normal depth in ft (m), and S_o is the longitudinal slope of the gutter.

A Manning "n" value of 0.017 is recommended for gutter flow. Because of the shallow depth of flow, the recommended "n" value is greater than that normally used for smooth concrete or asphalt pavement. The curb height should be at least 0.5 ft (0.15 m). No overtopping of the curb is allowed for the design storm. For the safety of children, the drain inlets should be spaced so that the maximum water depth in the gutter does not exceed 0.5 ft (0.15 m), and the water velocity does not exceed 10 fps (3.0 mps). A minimum longitudinal slope of 0.004 will generally ensure a low flow cleansing velocity in the gutter.

The top width of flow (T) is the spread of water on the street and is equal to

$$T = SS \times d \tag{10.7}$$

For local streets, the water can spread to the crown of the street during the design storm. Collector streets are usually designed so that one traffic lane is free of water, and arterial streets are usually designed so that one traffic lane in each direction is free of water during the design storm. Because of the potential for hydroplaning, no encroachment of water is allowed on any traffic lane for high speed roads.

10.1.3 Drain Inlets

Inlets are used in the drainage system to intercept the surface runoff (primarily in the street gutter) and direct the flow into the storm sewer. They are often located at street intersections upstream of the crosswalks. As shown in Fig. 10.5, inlets can be curb, grate, combination, or slotted. At design capacity, the flow into the inlet is by weir flow. During a major flood event, ponding over the inlet may occur and flow into the inlet will be by orifice flow.

Curb inlets are the most common as they provide the least interference with bicycle traffic and have the least tendency for blockage with debris. The height of the

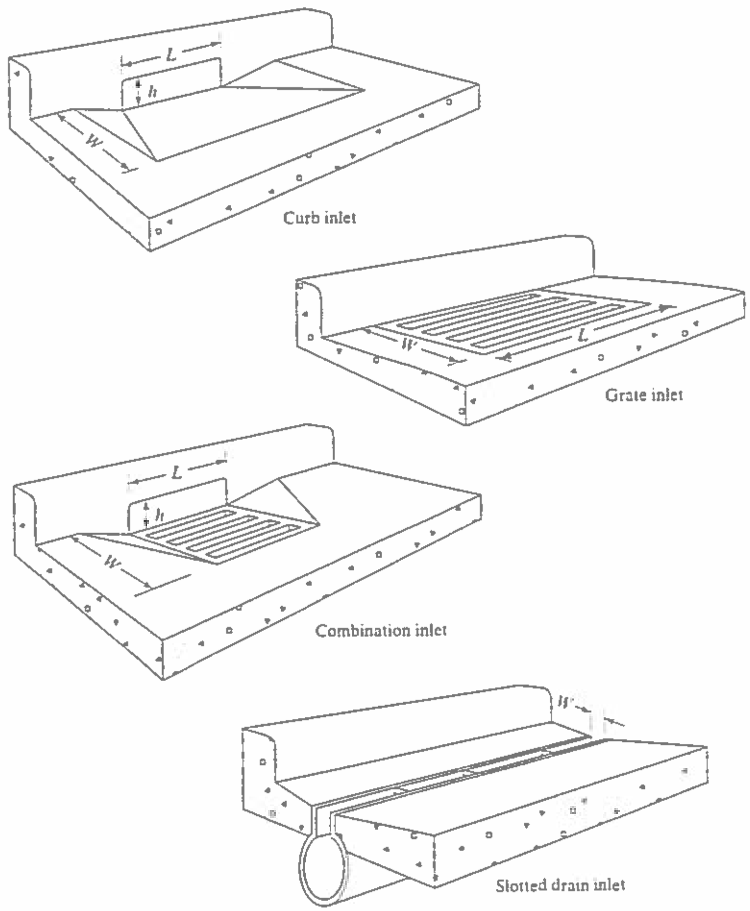


Figure 10.5 Perspective views of curb, grate, combination, and slotted drain inlets.

curb opening should not exceed 0.5 ft (0.15 m) because of the risk to small children. To increase the capacity of the inlet, the invert of the inlet is depressed below the invert of the street gutter. Typically the inlet is depressed 0.25–0.50 ft (0.07–0.15 m) with W (Fig. 10.5) ranging from 1.0–2.0 ft (0.3–0.6 m). The weir flow equation gives the discharge into the curb inlet on a mild slope (Q_I)

$$Q_I = C_w(L + 1.8W)d^{3/2} \tag{10.8}$$

where C_w is 2.3 for BG units and 1.25 for SI units, L and W are defined in Fig. 10.5, and d is the normal depth of the water in the approach gutter. Because some flow may bypass the curb inlet located on a gutter with a steep slope, drainage manuals should be consulted for sizing drain inlets located on a gutter with a steep slope.

Inlets in sumps are located at the low point in a roadway (such as the sag in a vertical curve) and are located to prevent ponding of water on the roadway. If clogging by debris is not anticipated, grate inlets work well when located in a sump. A grate inlet will operate as a weir at shallow water depths (d) less than 0.5 ft (0.12 m) and the grate inlet capacity (Q_I) is

$$Q_I = C_w P d^{3/2} \tag{10.9}$$

where C_w is 3.0 for BG units and 1.7 for SI units, P is the perimeter of the grate opening in ft (m), not including the side against the curb, and d is the depth of flow in ft (m).

A slotted drain inlet will function as a weir at shallow depths $d < 0.2$ ft (0.06 m) and operate as an orifice when submerged with a depth > 0.4 ft (0.12 m). The capacity (Q_I) of a slotted drain inlet with a width (W) > 0.15 ft (0.04 m) is

$$Q_I = C_w L d^{3/2} \tag{10.10}$$

for weir flow and

$$Q_I = 0.6A\sqrt{2gd} \tag{10.11}$$

for orifice flow. C_w is equal to 2.3 for BG units and 1.25 for SI units, and A is the area of the open space in the grate.

Example 10.3 Gutter Flow and Drain Inlet

Determine the maximum water depth and velocity in gutter and the length of a curb inlet at point F in Fig. 10.2 ($W = 1.5$ ft).

From Ex. 10.2

$$Q = 6.1 \text{ cfs} = 101d^{5/3}$$

Maximum depth in gutter

$$d = 0.35 \text{ ft}$$

$$A_g = 15d^2 = 1.84 \text{ ft}^2$$

Maximum velocity in gutter

$$V_g = \frac{Q}{A_g} = \frac{6.1}{1.84} = 3.3 \text{ fps}$$

Length of curb opening

$$Q_i = C_w(L - 1.8W)d^{3/2}$$

$$6.1 = 2.3(L + 1.8 \times 1.5)0.35^{3/2}$$

$$L = 12.8 - 2.7 = 10.1 \text{ ft}$$

10.1.4 Storm Sewers

The design of storm sewers is based on flow in open channels under steady flow conditions. Some general guidelines are as follows:

(1) The storm sewer pipe is assumed to flow full, and the pipe size is computed using the Manning equation.
(2) The minimum pipe size considered for storm sewers ranges from 12 to 18 inches (0.30–0.46 m) in diameter.
(3) To prevent sedimentation, the minimum water velocity for the pipe flowing full is 2.5 fps (0.75 mps).
(4) To avoid clogging with debris in the storm sewer, the pipe size should not decrease downstream.
(5) Pipe grades are typically specified in terms of elevation of the invert at manholes.
(6) At changes in pipe size, the crown of the two pipes are at the same elevation.
(7) Pipe slopes should conform with the ground slope where possible, and there should be at least 3.0 ft (0.9 m) of cover over the pipe to prevent excessive wheel loading on the pipe.
(8) Manholes are used for maintenance access and are located where two or more pipes join, where there is a change in pipe size or grade, and at sharp bends. The maximum spacing between manholes depends on the pipe size and may range from 500 ft (150 m) to over 1,000 ft (300 m).

Headlosses in manholes are computed as minor losses and are included in the design by lowering the invert elevation of outlet pipe at the manhole. The typical form of the minor loss equation is

$$H_M = K\frac{V^2}{2g} \tag{4.11}$$

The minor loss includes junction loss, bend loss, and transition loss. For inflow and outflow pipes aligned with each other, K is between 0.1 and 0.3. As the amount of turbulence in the manhole increases, the value for K will also increase.

Example 10.4 Pipe Size

Determine the discharge rates and pipe sizes for design points 1, 2, and 3 in lateral F–J and design points 4, 5, and 6 in main J–N. The lateral has a length of 1,050 ft and a slope of 0.01. The main has a length of 700 ft and slope of 0.02. Use a concrete pipe with a Manning's "n" of 0.015. The minimum pipe size is 18 inches in diameter. Use an overland flow time of concentration of 16 minutes and a runoff coefficient of 0.65.

Design point 1 (pipe 1)

From Example 10.2, $Q = 6.1$ cfs

$$Q = \frac{C_m}{n}\frac{\pi D^2}{4}\left(\frac{D}{4}\right)^{2/3} S_o^{1/2}$$

$$D = \left[\frac{3.21\ Qn}{1.49\ S_o^{1/2}}\right]^{3/8} = \left[\frac{3.21}{1.49} \times \frac{Q}{(0.01)^{1/2}} \times 0.015\right]^{3/8}$$

$$D_1 = 0.65\ Q^{3/8} = 1.28 \text{ ft}$$

Compute travel time in pipe 1 (T_1) using a minimum pipe size of 1.5 ft.

$$V_1 = \frac{1.49}{n}R^{2/3}S_o^{1/2} = \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.01)^{1/2} = 5.2 \text{ fps}$$

$$T_1 = \frac{350}{5.2 \times 60} = 1.1 \text{ min}$$

Design point 2 (pipe 2)

$$t_c = 16 + 1.8 + 1.1 = 18.9 \text{ min}$$

$$I = \frac{89}{(18.9 + 8.5)^{0.754}} = 7.3 \text{ in./hr}$$

Area contributing to flow

$$A = \frac{350 \times 525}{43{,}560} = 4.2 \text{ ac}$$

$$Q_2 = CIA = 0.65 \times 7.3 \times 4.2 = 19.9 \text{ cfs}$$

$$D_2 = 0.65(19.9)^{0.375} = 1.99 \text{ ft}$$

Use $D_2 = 2.0$ ft.

Travel time, pipe 2 (T_2)

$$V_2 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.01)^{1/2} = 6.3 \text{ fps}$$

$$T_2 = \frac{350}{6.3 \times 60} = 0.9 \text{ min}$$

Design point 3 (pipe 3)

$$t_c = 18.9 + 0.9 = 19.8 \text{ min}$$

$$I = \frac{89}{(19.8 + 8.5)^{0.754}} = 7.2 \text{ in./hr}$$

$$A = \frac{350 \times 875}{43{,}560} = 7.0 \text{ ac}$$

$$Q_3 = CIA = 0.65 \times 7.2 \times 7.0 = 32.8 \text{ cfs}$$

$$D_3 = 0.65(32.8)^{0.375} = 2.4 \text{ ft}$$

Use $D_3 = 2.5$ ft.

Travel time in pipe 3 (T_3)

$$V_3 = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.01)^{1/2} = 7.3 \text{ fps}$$

$$T_3 = \frac{350}{7.3 \times 60} = 0.8 \text{ min}$$

Design point 4 (pipe 4)

$$t_c = 19.8 + 0.8 = 20.6 \text{ min}$$

$$I = \frac{89}{(20.6 + 8.5)^{0.754}} = 7.0 \text{ in./hr}$$

$$A = \frac{350 \times 1{,}050}{43{,}560} = 8.4 \text{ ac}$$

$$Q_4 = 0.65 \times 7.0 \times 8.4 = 38.2 \text{ cfs}$$

$$D_4 = \left[\frac{3.21 \times 0.015}{1.49 \times (0.02)^{1/2}}\right]^{3/8} Q^{3/8} = 0.57 \times Q^{3/8} = 2.25 \text{ ft}$$

Pipe diameter cannot be smaller than upstream pipe. Use $D_4 = 2.5$ ft.

Travel time in pipe 4 (T_4)

$$V_4 = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.02)^{1/2} = 10.3 \text{ fps}$$

$$T_4 = \frac{350}{10.3 \times 60} = 0.6 \text{ min}$$

Design point 5 (pipe 5)

$$t_c = 20.6 + 0.6 = 21.2 \text{ min}$$

$$I = \frac{89}{(21.2 + 8.5)^{0.754}} = 6.9 \text{ in./hr}$$

$$A = \frac{700 \times 1{,}050}{43{,}560} = 16.9 \text{ ac}$$

$$Q_5 = CIA = 0.65 \times 6.9 \times 16.9 = 75.8 \text{ cfs}$$

$$D_5 = 0.57(75.8)^{3/8} = 2.9 \text{ ft}$$

Use $D_5 = 3.0$ ft.

$$V_5 = \frac{1.49}{0.015}\left(\frac{3.0}{4}\right)^{2/3}(0.02)^{1/2} = 11.6 \text{ fps}$$

$$T_5 = \frac{350}{11.6 \times 60} = 0.5 \text{ min}$$

Design point 6 (pipe 6)

$$t_c = 21.2 + 0.5 = 21.7 \text{ min}$$

$$I = \frac{89}{(21.7 + 8.5)^{0.754}} = 6.8 \text{ in./hr}$$

$$A = \frac{1{,}050 \times 1{,}050}{43{,}560} = 25.3 \text{ ac}$$

$$Q_6 = CIA = 0.65 \times 6.8 \times 25.3 = 111.8 \text{ cfs}$$

$$D_6 = 0.57(111.8)^{3/8} = 3.35$$

Use $D_6 = 3.5$ ft.

Example 10.5 Pipe Invert Elevations

If the upstream invert elevation of pipe 1 of Example 10.4 is 98.35 ft, determine the remaining invert elevations of pipes 1, 2, and 3. The manholes are 4.0 ft in diameter and have headloss coefficients (Eq. 4.11) of 0.3. Plot profile of lateral 1.

Length of line 1

$$L_1 = 350 - 4.0 = 346 \text{ ft}$$

Downstream invert elevation pipe 1

$$El = 98.35 - L_1 \times S_1 = 98.35 - 346 \times 0.01 = 94.89 \text{ ft}$$

$$H_L = 0.3\frac{V^2}{2g} = 0.3 \times \left(\frac{6.3^2}{64.4}\right) = 0.18 \text{ ft}$$

Upstream invert elevation pipe 2

$$El = 94.89 - H_L - \Delta D = 94.89 - 0.18 - 0.50 = 94.21 \text{ ft}$$

Downstream invert elevation pipe 2

$$El = 94.21 - 346 \times 0.01 = 90.75 \text{ ft}$$

Headloss in manhole

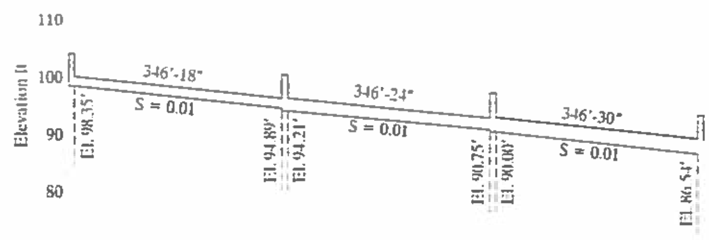
$$H_L = 0.3 \times \frac{7.3^2}{64.4} = 0.25 \text{ ft}$$

Upstream invert elevation pipe 3

$$El = 90.75 - H_L - \Delta D = 90.75 - 0.25 - 0.50 = 90.00 \text{ ft}$$

Downstream invert elevation pipe 3

$$El = 90.00 - 346 \times 0.01 = 86.54 \text{ ft}$$



Example 10.5 Profile of lateral 1.

10.2 ON-SITE DETENTION BASINS

As the drainage area is urbanized, the peak discharge rate increases. Because the runoff channels have been hydraulically improved, the runoff occurs faster after urbanization. Because of increased impervious areas caused by pavements, sidewalks, and roofs, the amount of runoff increases. In addition, grading and compacting the landscape and brush and forest litter removal can result in less interception and depression storage of precipitation after urbanization.

Most stormwater management plans require that the peak runoff discharge rate after development be no greater than the peak discharge rate before development. Detention basins can be used to reduce the peak discharge rate after urbanization. Normally, the detention basin is designed to reduce peak discharge rate to predevelopment levels for flood events ranging from a 2-year to a 100-year return period flood level. Generally, roof-top storage should be avoided; particularly for large buildings where wind set-up might cause design depth exceedance and failure of the structure. Because detention basin design requires knowledge of both discharge rates and runoff volumes, the unit hydrograph methodology will be used in the following discussion.

10.2.1 Unit Hydrograph

The Natural Resource Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), developed a dimensionless unit hydrograph based on instrumentation of a large number of natural watersheds representing a wide range of sizes and geographical areas. Although it was developed from natural watersheds, it is frequently applied to urban watersheds. The NRCS dimensionless unit hydrograph is tabulated in Table 8.6 and plotted in Fig. 8.11. The normalized runoff (q/q_p)

is a function of the normalized time (t/t_p), where q_p is the unit hydrograph peak discharge and t_p is the time to peak. The unit hydrograph is computed by multiplying t/t_p in Table 8.6 by t_p and q/q_p in Table 8.6 by q_p. Because the unit hydrograph has a fixed shape as specified in Table 8.6 and because the area under the unit hydrograph in Fig. 8.11 represents one unit of runoff from the watershed, there is a direct relation between q_p and t_p that is

$$q_p = K_u \frac{A}{t_p} \tag{10.12}$$

where q_p is the peak discharge rate in cfs/in. (cms/mm), K_u equals 484 for BG units and 0.208 for SI units, A is the watershed area in mi.^{2} (sq km), and t_p is the time to peak in hours. The time to peak is related to the time of concentration of the watershed (t_c) by

$$t_p = \frac{dt}{2} + 0.6 \times t_c \tag{10.13}$$

where dt is the duration of the rainfall excess and is equal to the computational time step of the runoff model. Because of variation in the rainfall intensity during a storm, the storm is divided into a number of short duration storms, each with a duration dt. To define the rising limb of the unit hydrograph in the runoff model, dt is typically taken as t_p divided by 5 and Eq. 10.13 becomes

$$t_p = 2/3\, t_c \tag{10.14}$$

For hand computations, the curvilinear unit hydrograph is often approximated with a triangular unit hydrograph with a height q_p and a base $2.67t_p$ (Fig. 8.11). Equations 10.12 and 10.14 can also be used with the triangular unit hydrograph to compute the peak discharge rate and time to peak.

Example 10.6 Unit Hydrograph

Plot the before and after development triangular unit hydrograph for a 30-acre watershed if the time of concentration before development was 60 minutes and after development is 40 minutes.

Before development unit hydrograph

$$Q_p = 484 \times \frac{A}{t_p}$$

$$A = \frac{30}{640} = 0.0469 \text{ mi.}^2$$

$$t_p = 0.667 \times t_c = 0.667 \times \frac{60}{60} = 0.667 \text{ hr}$$

$$q_p = 484 \times \frac{0.0469}{0.667} = 34.0 \text{ cfs}$$

Base of unit hydrograph $= 2.67 \times 0.667 = 1.78$ hrs