from: Mays, L. W. (2001). Water-Resources Engineering. 1st EditionWiley.

Chapter 15

Stormwater Control: Storm Sewers and Detention

5.1 STORMWATER MANAGEMENT

Stormwater management is knowledge used to understand, control, and utilize waters in their different forms within the hydrologic cycle (Wanielista and Yousef, 1993). The goal of this chapter is to provide an introduction to the various concepts and design procedures involved in stormwater management. The overall key component of stormwater management is the drainage system. Urbonas and Roesner (1993) point out the following vital functions of a drainage system:

1. It removes stormwater from the streets and permits the transportation arteries to function during bad weather; when this is done efficiently, the life expectancy of street pavement is extended.
2. The drainage system controls the rate and velocity of runoff along gutters and other surfaces in a manner that reduces the hazard to local residents and the potential for damage to pavement.
3. The drainage system conveys runoff to natural or manmade major drainage ways.
4. The system can be designed to control the mass of pollutants arriving at receiving waters.
5. Major open drainage ways and detention facilities offer opportunities for multiple use such as recreation, parks, and wildlife preserves.

Storm drainage criteria are the foundation for developing stormwater control. Table 15.1.1 provides a checklist for developing storm drainage criteria. These criteria should set limits on development, provide guidance and methods of design, provide details of key components of drainage and flood control systems, and ensure longevity, safety, aesthetics, and maintainability of the system served (Urbonas and Roesner, 1993).

Table 15.1.1 Checklist for Developing Local Storm Drainage Criteria

Governing legislation and statements of policy and procedure
Legal basis for criteria
Define what constitutes the drainage system
Benefits of the drainage system
Policy for dedication of right-of-way
Compatible multipurpose uses
Review and approval procedures
Procedures for obtaining variances or waivers of criteria

Table 15.1.1 Checklist for Developing Local Storm Drainage Criteria (*continued*)

- Initial and major drainage system provisions
  - Definitions of initial system and major system
  - Where should a separate formal major drainageway begin?
  - Allowable flow capacities in streets for initial and major storms
  - Maximum and minimum velocities in pipes and channels
  - Maximum flow depths in channels and freeboard requirements
- Data required for design, such as:
  - Watershed boundaries
  - Local rainfall and runoff data
  - History of flooding in the area
  - Defined regulatory flood plains and floodways
  - Existing and projected land use for project site
  - Existing and planned future land uses upstream
  - Existing and planned drainage systems off-site
  - Tabulation of previous studies affecting site
  - Conflicts with existing utilities
  - Design storms, intensity-duration-frequency data
  - Hydrologic methods and/or models
  - Storm sewer design criteria, including materials
  - Manhole details and spacing, inlet details and spacing, types of inlets, trenching, bedding, backfill, etc.
  - Street flow calculations and limitations
- Details of major system components such as channels, drop structures, erosion checks, transitions, major culverts and pipes, bridges, bends, energy dissipators, riprap, sediment transport
- Detention requirements
  - When and where to use detention
  - Design storms
  - Hydrologic sizing criteria and/or procedures
  - Safety, aesthetics, and maintainability criteria
  - Multipurpose uses and design details for each
- Water quality criteria
  - Goals and objectives
  - Minimum capture volumes
  - Required or acceptable best management practices (BMP)
  - Technical design criteria for each BMP
- Special considerations
  - Right-of-way dedication requirements
  - Use of irrigation ditches
  - Flood proofing and when it is accepted
  - Any other items reflecting local needs
  - List of technical references

Source: Urbonas and Roesner (1993).

15.2 STORM SYSTEMS

15.2.1 Information Needs and Design Criteria

To begin the design process of a storm sewer system, one must collect a considerable amount information. A condensed checklist of information needs for storm sewer design is presented Table 15.2.1. There are many sources for this information, ranging from various local governme agencies to federal agencies to pipe and pump manufacturers. The designer must also obtain futu development information for areas surrounding the site of interest.

Table 15.2.1 Condensed Checklist of Information Needs for Storm Sewer Design

- Local storm drainage criteria and design standards
- Maps, preferably topographic, of the subbasin in which the new system is to be located
- Detailed topographic map of the design area
- Locations, sizes, and types of existing storm sewers and channels located upstream and downstream of design area
- Locations, depths, and types of all existing and proposed utilities
- Layout of design area including existing and planned street patterns and profiles, types of street cross-sections, street intersection elevations, grades of any irrigation and drainage ditches, and elevations of all other items that may post physical constraints to the new system
- Soil borings, soil mechanical properties, and soil chemistry to help select appropriate pipe materials and strength classes
- Seasonal water table levels
- Intensity-duration-frequency and design storm data for the locally required design return periods
- Pipe vendor information for the types of storm sewer pipe materials accepted by local jurisdiction

Source: Urbonas and Roesner (1993).

Design criteria vary from one city to another, but for the most part the following are a fairly standard set of assumptions and constraints used in the design of storm sewers (American Society of Civil Engineers, 1969, 1992):

(a) For small systems, free-surface flow exists for the design discharges; that is, the sewer system is designed for "gravity flow" so that pumping stations and pressurized sewers are not considered.

(b) The sewers are commercially available circular pipes.

(c) The design diameter is the smallest commercially available pipe that has flow capacity equal to or greater than the design discharge and satisfies all the appropriate constraints.

(d) Storm sewers must be placed at a depth that will not be susceptible to frost, will drain basements, and will allow sufficient cushioning to prevent breakage due to ground surface loading. Therefore, minimum cover depths must be specified.

(e) The sewers are joined at junctions such that the crown elevation of the upstream sewer is no lower than that of the downstream sewer.

(f) To prevent or reduce excessive deposition of solid material in the sewers, a minimum permissible flow velocity at design discharge or at barely full-pipe gravity flow is specified.

(g) To prevent the occurrence of scour and other undesirable effects of high-velocity flow, a maximum permissible flow velocity is also specified. Maximum velocities in sewers are important mainly because of the possibilities of excessive erosion on the sewer inverts.

(h) At any junction or manhole, the downstream sewer cannot be smaller than any of the upstream sewers at that junction.

(i) The sewer system is a dendritic network converging towards downstream without closed loops.

Table 15.2.2 lists the more important typical technical items and limitations to consider.

Table 15.2.2 Technical Items and Limitations to Consider in Storm Sewer Design

Velocity:	
Minimum design velocity	2–3 ft/s (0.6–0.9 m/s)
Maximum design velocity	
Rigid pipe	15–21 ft/s (4.6–6.4 m/s)
Flexible pipe	10–15 ft/s (3.0–4.6 m/s)
Maximum manhole spacing:	
(function of pipe size)	400–600 ft (122–183 m)

Table 15.2.2 Technical Items and Limitations to Consider in Storm Sewer Design (*continued*)

Minimum size of pipe	12–24 in (0.3–0.6 m)
Vertical alignment at manholes:	
Different size pipe	Match crown of pipe or 80 to 85% depth lines
Same size pipe	Minimum of 0.1–0.2 ft (0.03–0.06 m) in invert drop
Minimum depth of soil cover	12–24 in (0.3–0.6 m)
Final hydraulic design	Check design for surcharge and junction losses by using backwater analysis
Location of inlets	In street where the allowable gutter flow capacity is exceeded

Source: Urbonas and Roesner (1993).

15.2.2 Rational Method Design

From an engineering viewpoint the design can be divided into two main aspects: runoff predictio and pipe sizing. The rational method, which can be traced back to the mid-nineteenth century, i still probably the most popular method used for the design of storm sewers (Yen and Akan, 1999' Although criticisms have been raised of its adequacy, and several other more advanced method have been proposed, the rational method, because of its simplicity, is still in continued use fc sewer design when high accuracy of runoff rate is not essential.

Using the rational method, the storm runoff peak is estimated by the rational formula

$$Q = KCiA \quad (15.2.1$$

where the peak runoff rate Q is in ft^3/s (m^3/s), K is 1.0 in U.S. customary units (0.28 for SI units' C is the runoff coefficient (Table 15.2.3), i is the average rainfall intensity in in/hr (mm/hr) fror intensity-duration frequency relationships for a specific return period and duration t_c in min, an A is the area of the tributary drainage area in acres (km^2). The duration is taken as the time of cor centration t_c of the drainage area.

Table 15.2.3 Runoff Coefficients for Use in the Rational Method

	Return Period (years)						
Character of Surface	2	5	10	25	50	100	500
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass areas (lawns, parks, etc.)							
Poor condition (grass cover less than 50% of the area)							
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair condition (grass cover 50% to 75% of the area)							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good condition (grass cover larger than 75% of the area)							
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58

Table 15.2.3 Runoff Coefficients for Use in the Rational Method (*continued*)

	Return Period (years)						
haracter of Surface	2	5	10	25	50	100	500
ndeveloped							
Cultivated land							
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/range							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/woodlands							
Flat, 0–2%	0.20	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7%	0.31	0.34	0.26	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

›te: The values in the table are the standards used by the City of Austin, Texas.

urce: Chow, Maidment, and Mays (1988).

In urban areas, the drainage area usually consists of subareas or subcatchments of substantially different surface characteristics. As a result, a composite analysis is required that must take into account the various surface characteristics. The areas of the subcatchments are denoted by A_j and the runoff coefficients for each subcatchment are denoted by C_j. Then the peak runoff is computed using the following form of the rational formula:

$$Q = Ki\sum_{j=1}^{m} C_j A_j \tag{15.2.2}$$

where m is the number of subcatchments drained by a sewer.

The *rainfall intensity i* is the average rainfall rate considered for a particular drainage basin or subbasin. The intensity is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The frequency of occurrence is a statistical variable that is established by design standards or chosen by the engineer as a design parameter.

The *time of concentration* t_c used in the rational method is the time associated with the peak runoff from the watershed to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire watershed is contributing; in this case, the time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. Runoff may reach a peak prior to the time the entire watershed is contributing. A trial-and-error procedure can be used to determine the critical time of concentration. The time of concentration to any point in a storm drainage system is the sum of the inlet time t_0 and the flow time t_f in the upstream sewers connected to the catchment, that is,

$$t_c = t_0 + t_f \tag{15.2.3}$$

where the flow time is

$$t_f = \sum \frac{L_j}{V_j} \tag{15.2.4}$$

where L_j is the length of the jth pipe along the flow path in ft (m) and V_j is the average flow velocity in the pipe in ft/s (m/s). The inlet time t_0 is the longest time of overland flow of water in a catchment to reach the storm sewer inlet draining the catchment.

In the rational method each sewer is designed individually and independently (except for tl computation of sewer flow time) and the corresponding rainfall intensity i is computed repeated for the area drained by the sewer. For a given sewer, all the different areas drained by this sew have the same i. Thus, as the design progresses towards the downstream sewers, the drainage ar increases and usually the time of concentration increases accordingly. This increasing t_c in tu gives a decreasing i that should be applied to the entire area drained by the sewer.

Inlet times, or times of concentration for the case of no upstream sewers, can be computed usi a number of methods, some of which are presented in Table 15.2.4. The longest time of conce tration among the times for the various flow routes in the drainage area is the critical time of co centration used.

Table 15.2.4 Summary of Time of Concentration Formulas

Method and Date	Formula for t_c (min)	Remarks
Kirpich (1940)	$t_c = 0.0078L^{0.77}S - 0.385$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply t_c by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.
California Culverts Practice (1942)	$t_c = 60(11.9L^3/H)^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft	Essentially the Kirpich formula; developed from small mountaino basins in California (U.S. Bureau of Reclamation, 1973, 1987).
Izzard (1946)	$t_c = \dfrac{41.025(0.0007i + c)L^{0.33}}{S^{0.333}i^{0.667}}$ i = rainfall intensity, in/h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product i times L should be < 500.
Federal Aviation Administration (1970)	$t_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from airfield drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.
Kinematic wave formulas (Morgali and Linsley (1965); Aron and Erborge (1973))	$t_c = \dfrac{0.94L^{0.6}n^{0.6}}{(i^{0.4}S^{0.3})}$ L = length of overland flow, ft n = Manning roughness coefficient i = rainfall intensity in/h S = average overland slope ft/ft	Overland flow equation developed from kinematic wave analysi of surface runoff from developed surfaces; method requires iteration since both i (rainfall intensity) and t_c are unknown; superposition of intensity–duration–frequency curve gives direct graphical solution for t_c.
SCS lag equation (U.S. Soil Conservation Service (1975))	$t_c = \dfrac{100L^{0.8}[(1000/\mathrm{CN}) - 9]^{0.7}}{1900S^{0.5}}$ L = hydraulic length of watershed (longest flow path), ft CN = SCS runoff curve number S = average watershed slope, %	Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paves; for mixed areas it tens to overestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that $t_c = 1.67 \times$ basin lag.

Table 15.2.4 Summary of Time of Concentration Formulas (*continued*)

lethod and Date	Formula for t_c (min)	Remarks
CS average elocity charts J.S. Soil onservation ervice 1975, 1986)	$t_c = \frac{1}{60}\sum \frac{L}{V}$ L = length of flow path, ft V = average velocity in feet per second for various surfaces found using Figure 8.8.2	Overland flow charts in U.S. Soil Conservation Service (1986) show average velocity as function of watercourse slope and surface cover.

urce: Kibler (1982).

EXAMPLE 15.2.1

The computational procedure in the rational method is illustrated through an example design of sewers to drain a 20-ac area along Goodwin Avenue in Urbana, Illinois, as shown in Figure 15.2.1. The physical characteristics of the drainage basin are given in Table 15.2.5. The catchments are identified by the manholes they drain directly into. The sewer pipes are identified by the number of the upstream manhole of each pipe. The Manning's roughness factor n is 0.014 for all the sewers in the example (adapted from Yen, 1978).

Table 15.2.5 Characteristics of Catchments of Goodwin Avenue Drainage Basin

(1) Catchment	(2) Ground Elevation at Manhole (ft)	(3) Area A (ac)	(4) Runoff Coefficient C	(5) Inlet Time t_o (min)
1.1	731.08	2.20	0.65	11.0
1.2	725.48	1.20	0.80	9.2
2.1	724.27	3.90	0.70	13.7
2.2	723.10	0.45	0.80	5.2
3.1	722.48	0.70	0.70	8.7
3.2	723.45	0.60	0.85	5.9
3.3	721.89	1.70	0.65	11.8
4.1	720.86	2.00	0.75	9.5
4.2	719.85	0.65	0.85	6.2
5.1	721.19	1.25	0.70	10.3
5.2	719.10	0.70	0.65	11.8
5.3	722.00	1.70	0.55	17.6
6.1	718.14	0.60	0.75	7.3
7.1	715.39	2.30	0.70	14.5

Source: Yen (1978).

LUTION

Table 15.2.6 shows the computations for the design of 12 sewer pipes, namely, all the pipes upstream of sewer 6.1. The rainfall intensity-duration relationship is developed using National Weather Service report HYDRO-35 (see Chapter 7 or Frederick et al., 1977) and plotted in Figure 15.2.2 for the design return period of two years. The entries in Table 15.2.6 are explained as follows:

Columns 1, 2, and 3: The sewer number and its length and slope are predetermined quantities.

Column 4: Total area drained by a sewer is equal to the sum of the areas of the subcatchments drained by the sewer, e.g., for sewer 3.1, the area 8.45 acres is equal to the area drained by sewer 2.1 (7.30 ac in column 4) plus the area drained by sewer 2.2 (0.45 ac) plus the incremental area given in column 6 (0.70 ac for subcatchment 3.1).

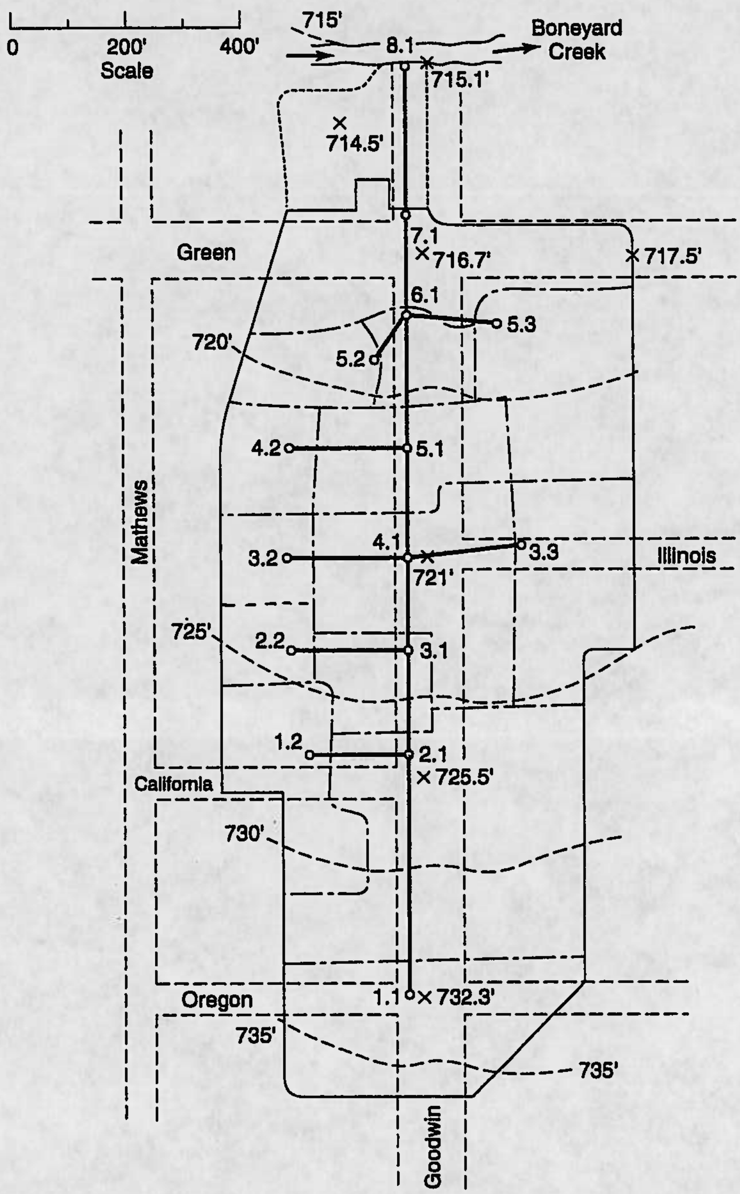


Figure 15.2.1 Goodwin Avenue drainage basin at Urbana, Illinois (from Yen (1978)).

Column 5: The identification number of the incremental subcatchments that drain directly through ma hole or junction into the sewer being considered.

Column 6: Size of the incremental subcatchment identified in column 5 (Table 15.2.5).

Column 7: Value of runoff coefficient for each subcatchment (Table 15.2.5).

Column 8: Product of C and the corresponding subcatchment area.

Column 9: Summation of CA for all the areas drained by the sewer which is equal to the sum of co tributing values in column 9 and the values in column 8 for that sewer, e. g., for sewer 3.1, 5.97 5.12 + 0.36 + (0.49).

Column 10: Values of inlet time (Table 15.2.5) for the subcatchment drained (computed using meth in Table 15.2.4), i.e., the overland flow inlet time if the upstream subcatchment is no more than

Table 15.2.6 Design of Sewers by the Rational Method

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
				Increment														
Sewer	Length L	Slope S (ft)	Total Area Drained (ac)	Catchment	Area (ac)	C	CA	ΣCA	Inlet Time (min)	Upstream Sewer Flow Time (min)	t_c (min)	t_d (min)	i (in/hr)	Design Discharge Q_p (cfs)	Computed Diameter D_r (ft)	Pipe Size Used D_n (ft)	Flow Velocity (fps)	Sewer Flow Time min
1.1	390	0.0200	2.20	1.1	2.20	0.65	1.43	1.43	11.0	–	11.0	11.0	4.00	5.72	1.08	1.25	4.6	1.42
1.2	183	0.0041	1.20	1.2	1.20	0.80	0.96	0.96	9.2	–	9.2	9.2	4.30	4.13	1.28	1.50	2.3	1.31
2.1	177	0.0245		2.1	3.90	0.70	2.73		13.7	–	13.7							
				1.1					11.0	1.4	12.4							
				1.2					9.2	1.3	10.5							
			7.30					5.12				13.7	3.68	18.8	1.62	1.75	7.8	0.38
2.2	200	0.0180	0.45	2.2	0.45	0.80	0.36	0.36	5.2	–	5.2	5.2	5.30	1.91	0.73	0.83	3.5	0.95
3.1	156	0.0104		3.1	0.70	0.70	0.49		8.7	–	8.7							
									13.7	0.4	14.1							
				2.2					5.2	1.0	6.2							
			8.45					5.97				14.1	3.63	21.6	2.00	2.00	6.9	0.39
3.2	210	0.0175	0.60	3.2	0.60	0.85	0.51	0.51	5.9	–	5.9	5.9	5.07	2.59	0.82	0.83	4.7	0.74
3.3	130	0.0300	1.70	3.3	1.70	0.65	1.11	1.11	11.8	–	11.8	11.8	3.90	4.32	0.90	1.00	5.5	0.39
4.1	181	0.0041		4.1	2.00	0.75	1.50		9.5	–	9.5							
									14.1	0.4	14.5							
				3.3					11.8	0.4	12.2							
			12.75					9.09				14.5	3.60	32.7	2.79	3.00	4.6	0.65
4.2	200	0.0026	0.65	4.2	0.65	0.85	0.55	0.55	6.2	–	6.2	6.2	4.98	2.75	1.20	1.25	2.2	1.49
5.1	230	0.0028		5.1	1.25	0.70	0.88		10.3	–	10.3							
									14.5	0.7	15.2							
			14.65					10.52				15.2	3.50	36.8	3.13	3.50	3.8	1.00
5.2	70	0.0250	0.70	5.2	0.70	0.65	0.46	0.46	11.8	–	11.8	11.8	3.90	1.79	0.67	0.67	5.1	0.23
5.3	130	0.0060	1.70	5.3	1.70	0.55	0.94	0.94	17.6	–	17.6	17.6	3.30	3.10	1.07	1.25	2.5	0.86

Source: Yen (1978).

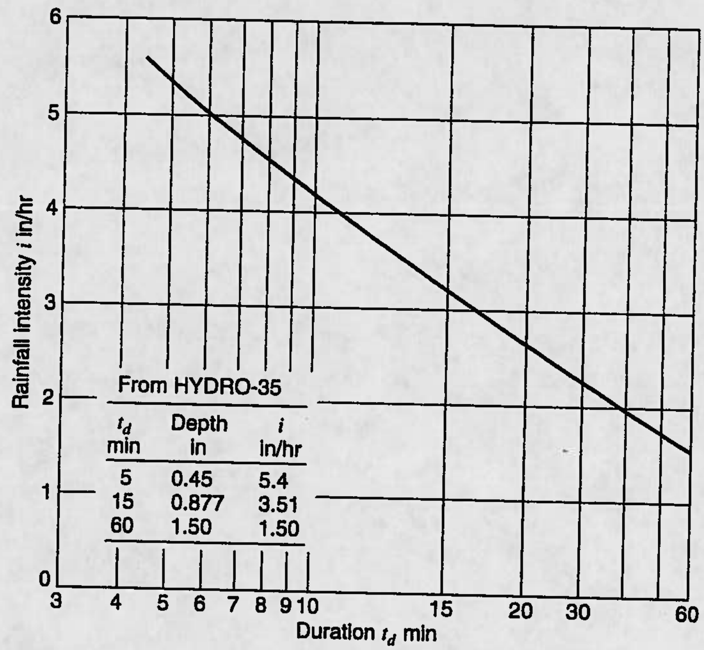


Figure 15.2.2 Variation of rainfall intensity with duration at Urbana, Illinois (from Yen (1978)).

sewer away from the sewer being designed (e.g., in designing sewer 3.1, 5.2 min for subcatchment 2 and 8.7 min for subcatchment 3.1); otherwise it is the total flow time to the entrance of the immedi upstream sewer (e.g., in designing sewer 3.1, 13.7 min for sewer 2.1).

Column 11: The sewer flow time of the immediate upstream sewer as given in column 19.

Column 12: The time of concentration t_c for each of the possible critical flow paths; t_c = inlet time (co umn 10) + sewer flow time (column 11) for each flow path.

Column 13: The rainfall duration t_d is assumed equal to the longest of the different times of concentr tion of different flow paths to arrive at the entrance of the sewer being considered; e.g., for sewer 3. t_d is equal to 14.1 min for sewer 2.1, which is longer than from sewer 2.2 (6.2 min) or directly from su catchment 3.1 (8.7 min).

Column 14: The rainfall intensity i for the duration given in column 13 is based on HYDRO-35 for t two-year design return period (see Figure 15.2.2).

Column 15: Design discharge is computed by using equation (15.2.2), i.e., the product of columns and 14.

Column 16: Required sewer diameter in feet, as computed using Manning's formula (equation 15.2.7 with $n = 0.014$, Q is given in column 15 and S_0 in column 3.

Column 17: The nearest commercial nominal pipe size that is not smaller than the computed size adopted.

Column 18: Flow velocity computed by using $V = 4Q_p/(\pi D^2)$, i.e., column 13 multiplied by $4/\pi$ ar divided by the square of column 17.

Column 19: Sewer flow time is computed as equal to L/V, i.e., column 2 divided by column 18 and co verted into minutes.

This example demonstrates that in the rational method each sewer is designed individually and ind pendently (except for the computation of sewer flow time) and the corresponding rainfall intensity i computed repeatedly for the area drained by the sewer. For a given sewer, all the different areas draine by this sewer have the same i. Thus, as the design progresses towards downstream sewers, the drainage area increases and usually the time of concentration increases accordingly. This increasing t_c in turn gives a decreasing i, which should be applied to the entire area drained by the sewer. Failure to realize this variation of i is the most common mistake made in using the rational method for sewer design.

The size of a particular pipe is based upon computing the smallest available commercial pipe that can handle the peak flow rate determined using the rational formula (15.2.2). Manning's equation (equation (5.1.23) or (5.1.25)) has been popular in the United States for sizing pipes:

$$Q = \frac{m}{n} S_f^{1/2} A R^{2/3} \tag{15.2.5}$$

where m is 1.486 for U.S. customary units (1 for SI units), S_f is the friction slope, A is the inside cross-sectional area of the pipe $\pi D^2/4$ in ft^2 (m^2), R is the hydraulic radius, $R = A/P = D/4$ in ft (m), P is the wetted perimeter (πD) in ft (m), and K is the inside pipe diameter in ft (m). By substituting in the bed slope S_0 for the friction slope (assuming uniform flow) and $A = \pi D^2/4$ and $R = D/4$ (assuming that the pipe is flowing full under gravity, not pressurized), Manning's equation becomes

$$Q = \frac{m}{n} S_0 \left(\frac{\pi D^2}{4}\right)\left(\frac{D}{4}\right)^{2/3} = m\left(\frac{0.311}{n}\right) S_0^{1/2} D^{8/3} \tag{15.2.6}$$

Equation (15.2.6) can be solved for the diameter

$$D = \left(\frac{m_D Q n}{\sqrt{S_0}}\right)^{3/8} \tag{15.2.7}$$

where m_D is 2.16 for U.S. customary units (3.21 for SI units). Q is determined using the rational formula, and D is rounded up to the next commercial size pipe. The Darcy–Weisbach equation (4.3.13) can also be used to size pipes,

$$Q = A\left(\frac{8g}{f} R S_f\right)^{1/2} \tag{15.2.8a}$$

Equation (15.2.8a) can be solved for D using $S_f = S_0$ as

$$D = \left(\frac{0.811 f Q^2}{g S_0}\right)^{1/5} \tag{15.2.8b}$$

which is valid for any dimensionally consistent set of units.

15.2.3 Hydraulic Analysis of Designs

To analyze the hydraulic effectiveness of storm sewer design, it is necessary to analyze the hydraulic gradient. The hydraulic gradient can be used to determine if design flows can be accommodated without causing flooding at various locations or causing flows to exit the system at locations where this is not acceptable. Such analysis can be done manually or by computer. This section first discusses the form losses, then the hydraulic gradient calculations, and finally hydrograph routing.

15.2.3.1 Form Losses

During the propagation of flows through storm sewers, both open-channel flow and pressurized pipe flow can occur, depending upon the magnitude of the flows. Consequently the form loss equations for both types of flow are presented here.

Transition Losses (open-channel flow)

Contraction losses for open-channel flow are expressed as

$$H_c = 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \text{ for } V_2 > V_1 \tag{15.2.9}$$

where V_1 is the upstream velocity and V_2 is the downstream velocity. Expansion losses ar expressed as

$$H_e = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \text{ for } V_1 > V_2 \tag{15.2.10}$$

Simple size transitions through manholes with straight-through flow can be analyzed with th above two equations.

Transition losses (pressurized flow)

Contraction losses for pressurized flow are expressed as

$$H_c = K\left(\frac{V_2^2}{2g}\right)\left[1 - \frac{A_2}{A_1}\right]^2 \tag{15.2.1}$$

where $K = 0.5$ for a sudden contraction, $K = 0.1$ for a well-designed transition, A_1 is the cros sectional area of flow at the beginning of the transition, and A_2 is the cross-sectional area of flo at the end of the transition. Expansion losses for pressurized flow are expressed as

$$H_e = K\left[\frac{(V_1 - V_2)^2}{2g}\right] \tag{15.2.1}$$

where $K = 1.0$ for a sudden expansion and $K = 0.2$ for a well-designed transition. These K va ues for the contractions and expansions are for approximation. For detailed analysis, Tabl 15.2.7–15.2.10 can be used in conjunction with the following form of the headloss equation:

$$H = K\left(\frac{V^2}{2g}\right) \tag{15.2.1}$$

Exit losses can be computed with the following equation:

$$H_{ext} = K_e\left(\frac{V^2}{2g}\right) \tag{15.2.1}$$

Table 15.2.7 Values of K_2 for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Formula $H_2 = K_2(V_1^2/2g$

$\frac{D_2}{D_1}$	Velocity V_1 in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

D_2/D_1 = ratio of larger pipe to smaller pipe; V_1 = velocity in smaller pipe

Source: American Iron and Steel Institute (1995).

Table 15.2.8 Values of K_2 for Determining Loss of Head Due to Gradual Enlargement in Pipes from the Formula $H_2 = K_2(V_1^2/2g)$

$\frac{D_2}{D_1}$	Angle of Cone													
	2°	4°	6°	8°	10°	15°	20°	25°	30°	35°	40°	45°	50°	60°
1.1	.01	.01	.01	.02	.03	.05	.10	.13	.16	.18	.19	.20	.21	.23
1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
∞	.03	.04	.05	.06	.08	.16	.31	.40	.49	.56	.60	.64	.67	.72

$_2/D_1$ = ratio of diameter of larger pipe to diameter of smaller pipe. Angle of cone is twice the angle between the axis of the cone and its side.

urce: American Iron and Steel Institute (1995).

Table 15.2.9 Values of K_3 for Determining Loss of Head Due to Sudden Contraction from the Formula $H_2 = K_2(V_1^2/2g)$

$\frac{D_2}{D_1}$	Velocity V_2 in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
.0.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45.	44	.41	.38

$_2/D_1$ = ratio of larger pipe to smaller diameter; V_2 = velocity in smaller pipe.

urce: American Iron and Steel Institute (1995).

Table 15.2.10 Entrance Loss Coefficients for Corrugated Steel Pipe or Pipe-Arch

Inlet End of Culvert	Coefficient K_2
Projecting from fill (no headwall)	0.9
Headwall, or headwall and wingwalls square-edge	0.5
Mitered (beveled) to conform to fill slope	0.7
End-section conforming to fill slope	0.5
Headwall, rounded edge	0.2
Beveled ring	0.25

Source: American Iron and Steel Institute (1995).

Manhole losses

In many cases manhole losses can comprise a significant percentage of the overall losses i storm sewer system. The losses that occur at storm sewer junctions are dependent upon the fl characteristics, junction geometry, and relative sewer diameters. For a *straight-through manh* with no change in pipe sizes, the losses can be expressed as

$$H_m = 0.05\frac{V^2}{2g} \tag{15.2.1}$$

Losses at *terminal manholes* can be estimated using

$$H_m = \frac{V^2}{2g} \tag{15.2.1}$$

For *junction manholes* with one or more incoming laterals, the total manhole loss (pressu change) can be estimated using the following equation form:

$$H_m = K\frac{V^2}{2g} \tag{15.2.1}$$

where Figure 15.2.3 shows manhole junction types and nomenclature. Values of K for vario types of manhole configurations can be found in Figures 15.2.4–15.2.8.

Bend losses

Bend losses in storm sewers can be estimated using

$$H_b = K_b\frac{V^2}{2g} \tag{15.2.1}$$

where

$$K_b = 0.25\sqrt{\frac{\Phi}{90}} \tag{15.2.1}$$

for curved sewer segments where the angle of deflection Φ is less than 40°. For greater angles deflection and for bends in manholes, the loss coefficient can be obtained from Figure 15.2.9.

EXAMPLE 15.2.2

Approximate the sudden expansion loss for a 400-mm sewer pipe connecting to a 450-mm sewer pi for a design discharge of 0.3 m³/s assuming full-pipe flow.

SOLUTION

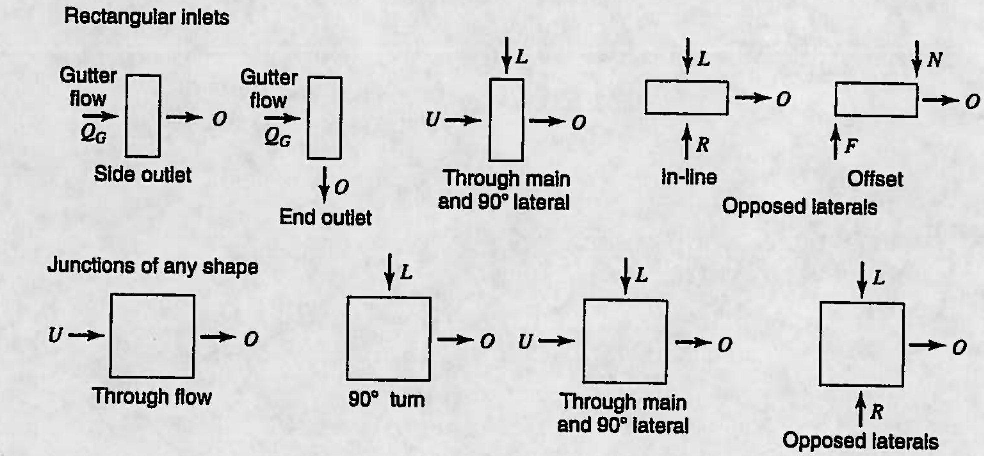
First compute the velocity of flow in each sewer pipe:

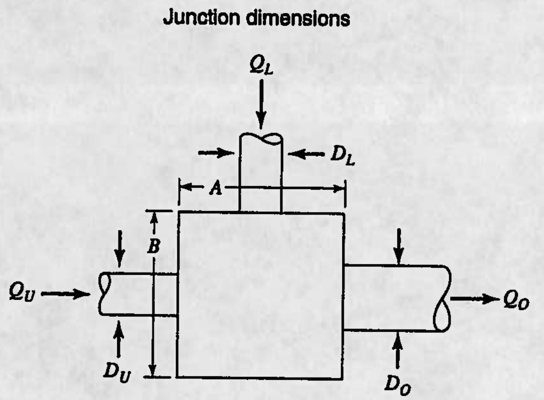
$$V_1 = \frac{Q}{A_1} = \frac{0.3\ \text{m}^3/\text{s}}{\left[\pi\,(400/1000)^2\right]/4} = 2.39\ \text{m/s}$$

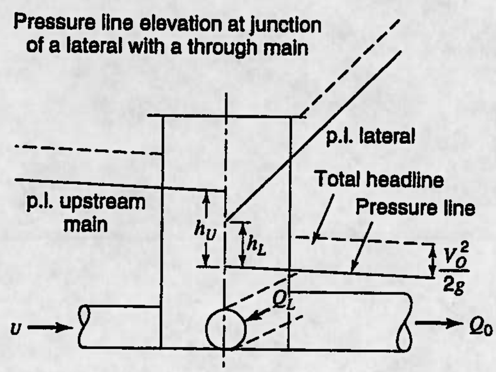
$$V_2 = \frac{Q}{A_2} = \frac{0.3\ \text{m}^3/\text{s}}{\left[\pi\,(450/1000)^2\right]/4} = 1.89\ \text{m/s}$$

The expansion loss is then determined using equation (15.2.12) with $K = 1.0$:

$$H_e = (1)\left[\frac{(2.39-1.89)^2}{2(9.81)}\right] = 0.0127\ \text{m}$$







Nomenclature

Q	rate of flow
D	diameter of pipe
A	dimension of junction in direction of outfall pipe
B	dimension of junction at right angles to outfall pipe
d	depth of water in inlet
S	slope of pipe
S_f	friction slope
Q_G	flow into inlet through top grate
$D_O\,Q_O$	diameter and flow in outfall
$D_U\,Q_U$	diameter and flow in upstream main
$D_L\,Q_L$	diameter and flow in left lateral
$D_R\,Q_R$	diameter and flow in right lateral
$D_N\,Q_N$	diameter and flow in near lateral(1)
$D_F\,Q_F$	diameter and flow in far lateral(1)
$D_{hv}\,Q_{hv}$	diameter and flow in lateral w/higher-velocity flow(1)
$D_{lv}\,Q_{lv}$	diameter and flow in lateral w/lower-velocity flow(1)
	Pressure change coefficients for inlet water depth and an upstream pipe pressure relative to the outfall pipe pressure
K_S	water depth with all flow through grate
K_U	upstream main pressure
K_R or K_L	lateral pipe pressure
K_N	near lateral pipe pressure(3)
K_F	far lateral pipe pressure(3)
$\overline{K}_U$, $\overline{K}_L$	pressure coefficient at $Q_L = Q_O$
M_U, M_L	multiplier for $\overline{K}_U$ or $\overline{K}_L$ to obtain K_U or K_L

(1) Of oppposed laterals; designation with reference to position relative to outfall end of junction.
(2) For in-line opposed laterals only.
(3) Offset opposed laterals.

$$h_U = K_U \frac{V_O^2}{2g} \qquad h_L = K_L \frac{V_O^2}{2g}$$

igure 15.2.3 Manhole junction types and nomenclature (from Sangster et al. (1958)).

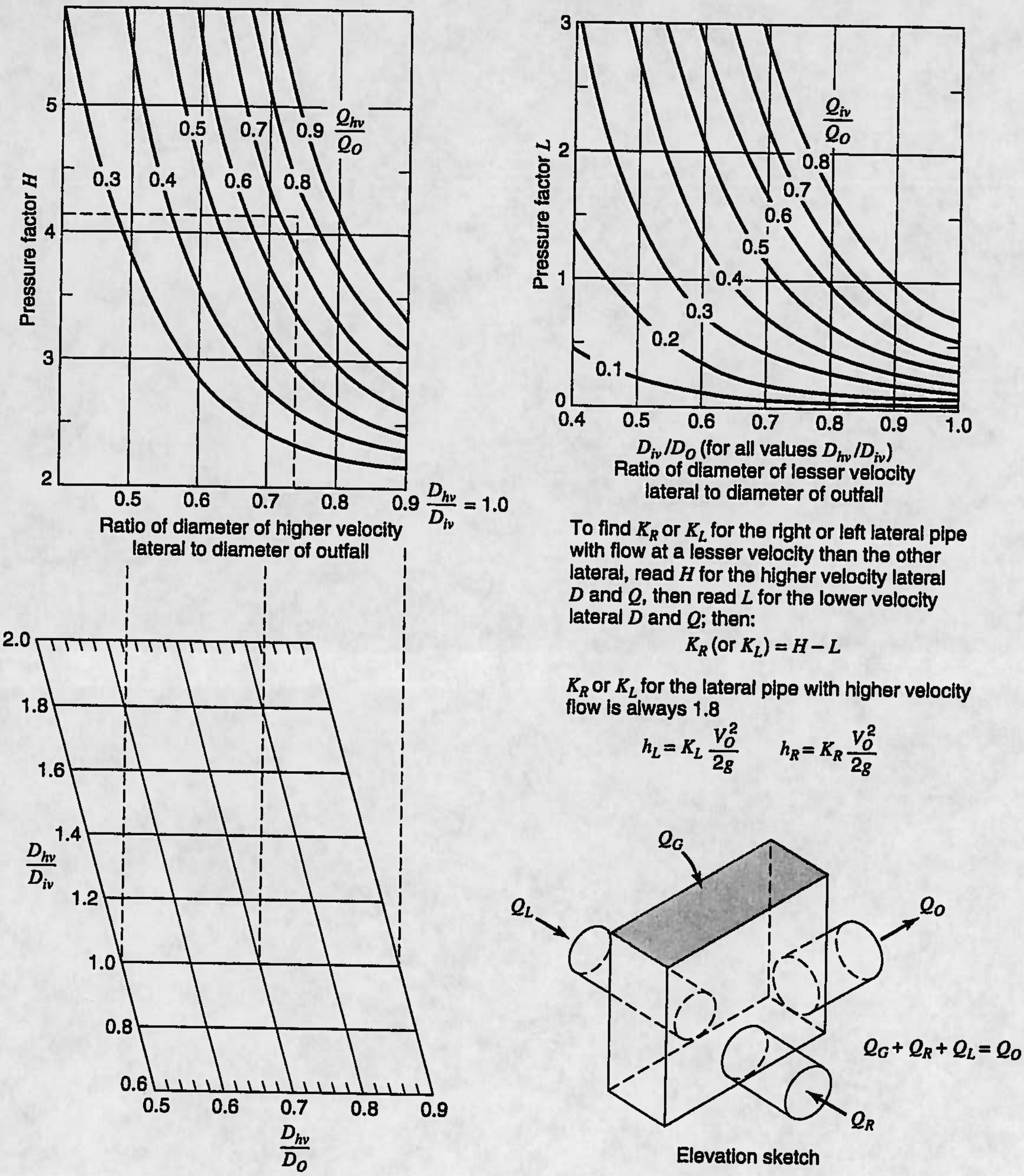
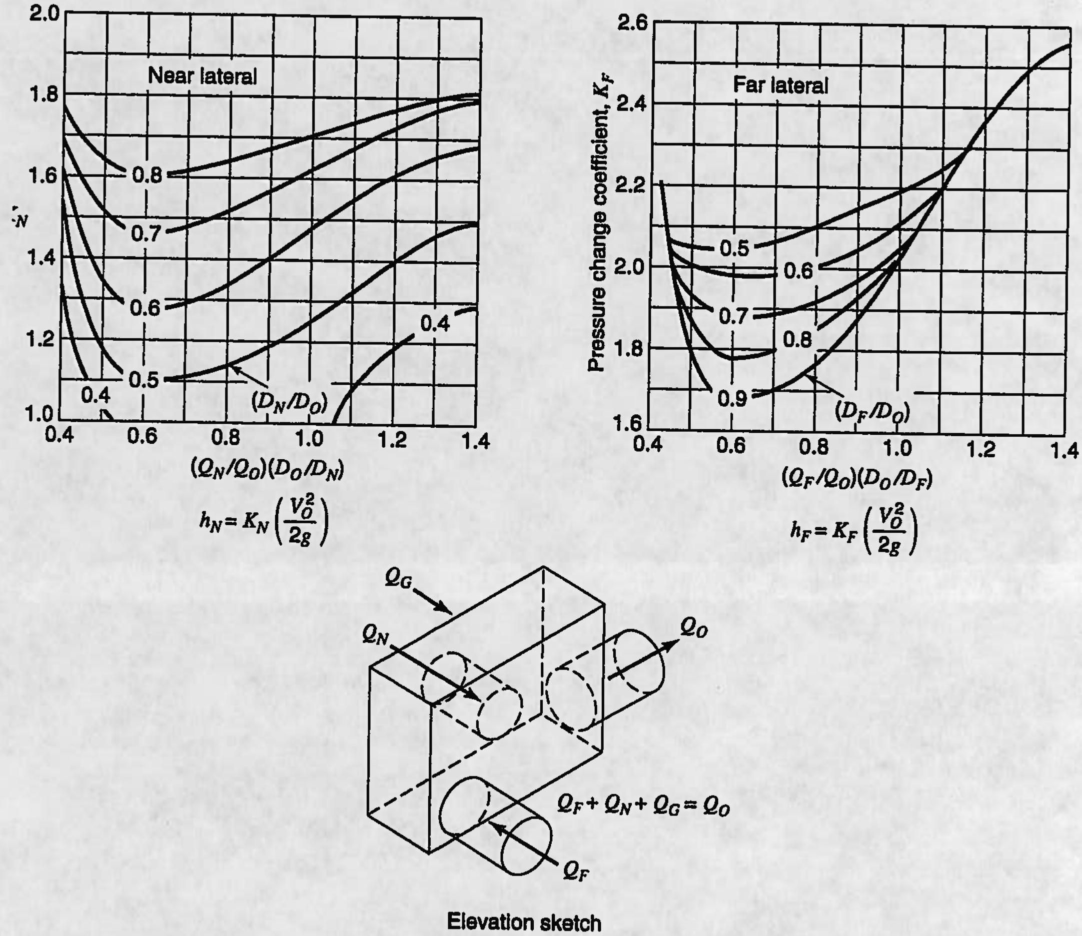
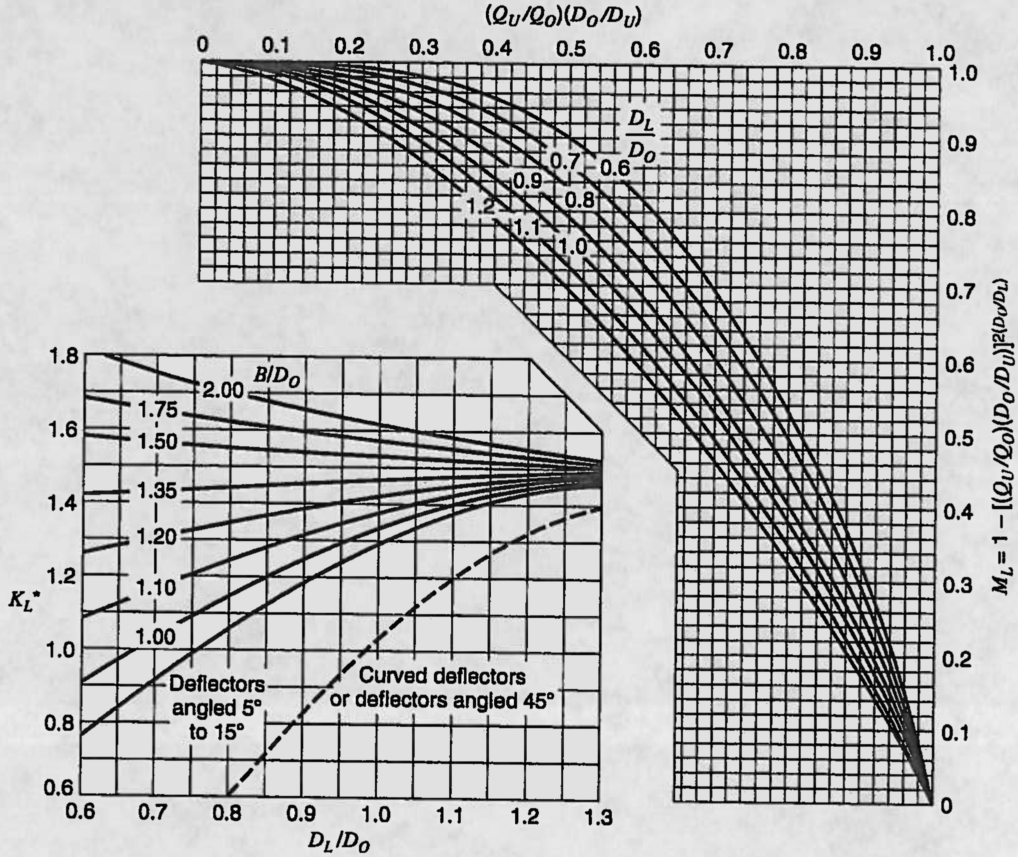
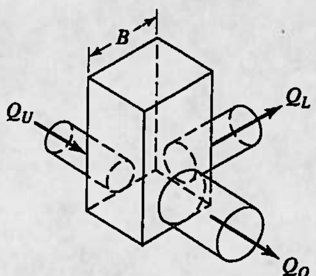


Figure 15.2.4 Rectangular manhole with in-line opposed lateral pipes each at 90° to outfall (with or without grate flow) (from Sangster et al. (1958)).



igure 15.2.5 Rectangular manhole with offset opposed lateral pipes—each at 90° to outfall (with or without let flow) (from Sangster et al. (1958)).





To find K_L for the for the lateral pipe, first read $\bar{K}_L^*$ from the lower graph. Next determine M_L. Then

$$K_L = K_L^* \times M_L$$

Dashed curve for curved or 45° angled deflectors applies only to manholes without upstream in-line pipe.

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of K_L^* by 0.2 for combining flow.

For $(Q_U/Q_O)^* \times (D_O/D_U) > 1$ use

$$h_L = K_L \frac{V_O^2}{2g} \text{ from Figure 15.2.8}$$

For $D_L = D_O < 0.6$ use

$$h_L = K_L \frac{V_O^2}{2g} \text{ from Figure 15.2.8}$$

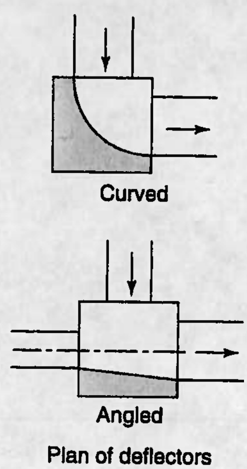
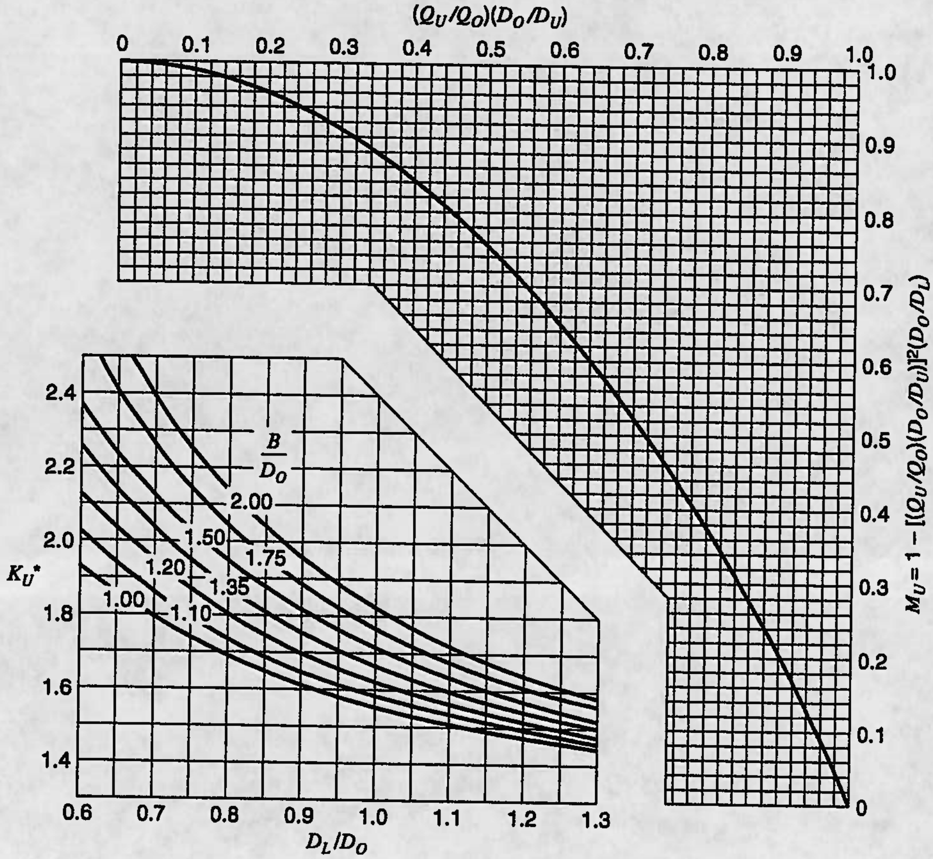
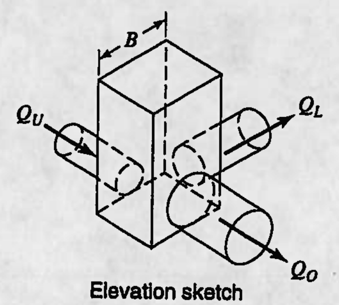


Figure 15.2.6 Manhole at 90° deflection or on through pipeline at junction of 90° lateral pipe (lateral coefficient) (from Sangster et al. (1958)).





To find K_U for the upstream main, read K_U^* from the lower graph. Next determine M_U. Than

$$K_U = K_U^* \times M_U$$

For manholes with deflectors at 0° to 15°, read K_U^* on curve for $B/D_O = 1.0$.

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of K_U^* by 0.2 for combining flow.

For deflectors refer to sketches on

$(Q_U/Q_O) \times (D_O/D_U) > 1$ use

$$h_U = K_U \frac{V_O^2}{2g} \text{ from Figure 4.15}$$

For $D_L/D_O < 0.6$ use

$$h_U = K_U \frac{V_O^2}{2g} \text{ from Figure 4.15}$$

Figure 15.2.7 Manhole on through pipeline at junction of a 90° lateral pipe (in-line pipe coefficient) (from Sangster et al. (1958)).

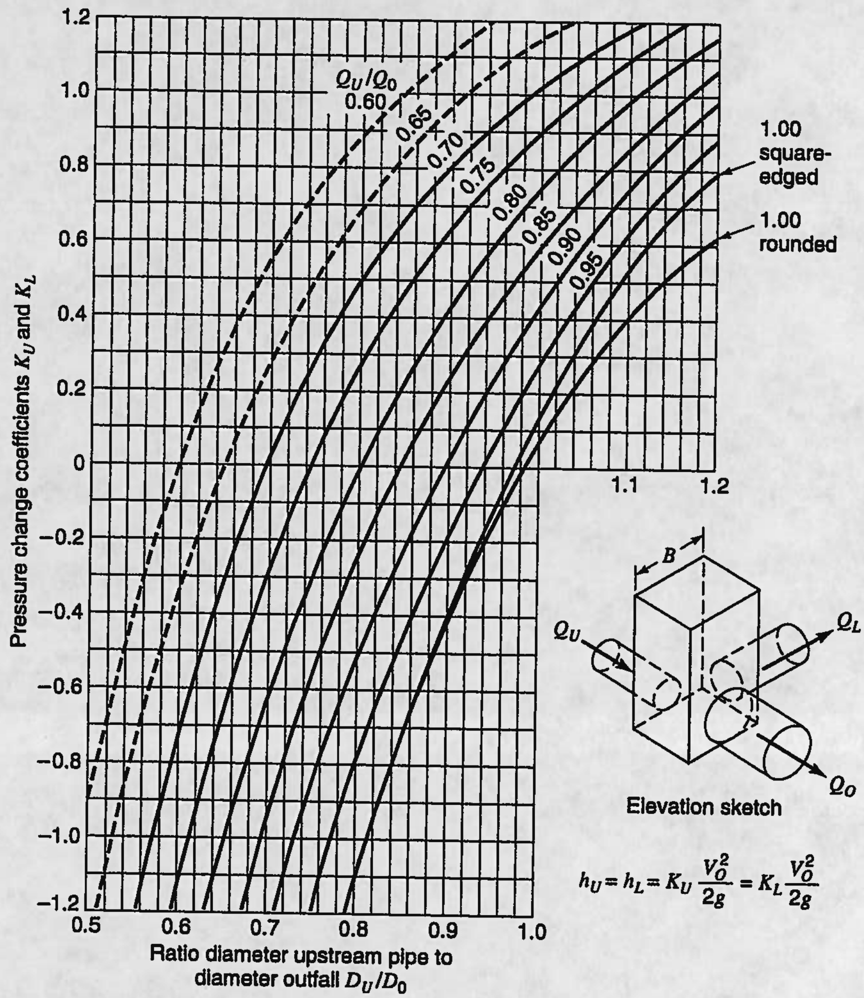


Figure 15.2.8 Manhole on through pipeline at junction of a 90° lateral pipe (for conditions outside the range of Figures 15.2.6 and 15.2.7 (from Sangster et al. (1958)).

EXAMPLE 15.2.3

Compute the bend loss for a 30° bend in a 400-mm sewer pipe with a discharge of 0.3 m³/s assumin full-pipe flow.

SOLUTION

First compute the flow velocity in the sewer pipe:

$$V = \frac{Q}{A} = \frac{0.3}{\left[\pi\,(400/1000)^2\right]/4} = 2.39 \text{ m/s}$$

Next compute the bend loss coefficient using equation (15.2.19):

$$K_b = 0.25\sqrt{\frac{\Phi}{90}} = 0.25\sqrt{\frac{30}{90}} = 0.144$$

Use equation (15.2.18) to compute the bend loss:

$$H_b = K_b \frac{V^2}{2g} = 0.144 \frac{(2.39)^2}{2(9.81)} = 0.0419 \text{ m}$$

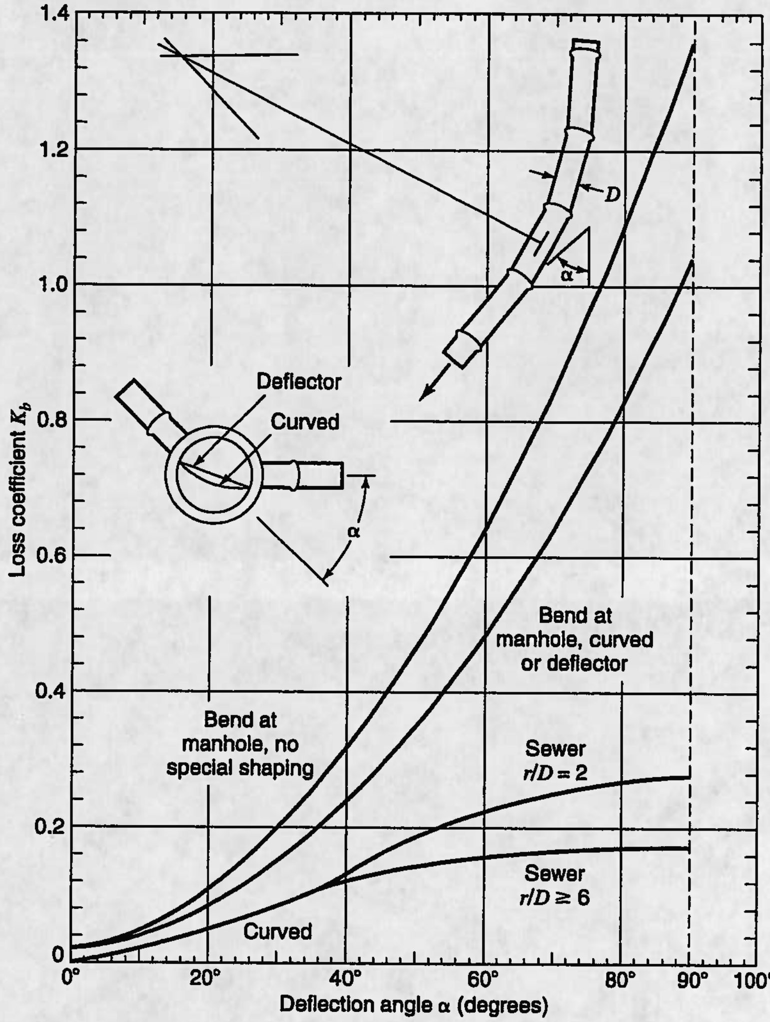


Figure 15.2.9 Sewer bend loss coefficient (from Sangster et al. (1958)).

EXAMPLE 15.2.4

The hypothetical storm sewer layout shown in Figure 15.2.10 includes an existing portion and an extension of the existing system. The objective for this example is to analyze the hydraulics of manhole number 4 (MH-4). Refer to Figure 15.2.11 for details of the manhole. We have

Top of manhole elevation	476.00 ft
Bottom of manhole elevation	470.15 ft
Manhole diameter	48.0 in
Lateral flow, Q_L	25.0 cfs
Upstream in-line flow, Q_u	46.0 cfs
Outfall flow, Q_0	71.0 cfs
Diameter of lateral line, D_L	30.0 in
Diameter of upstream in-line, D_u	42.0 in
Diameter of outfall line, D_0	48.0 in
Elevation of outfall pipe pressure line at MH = 6	475.08 ft

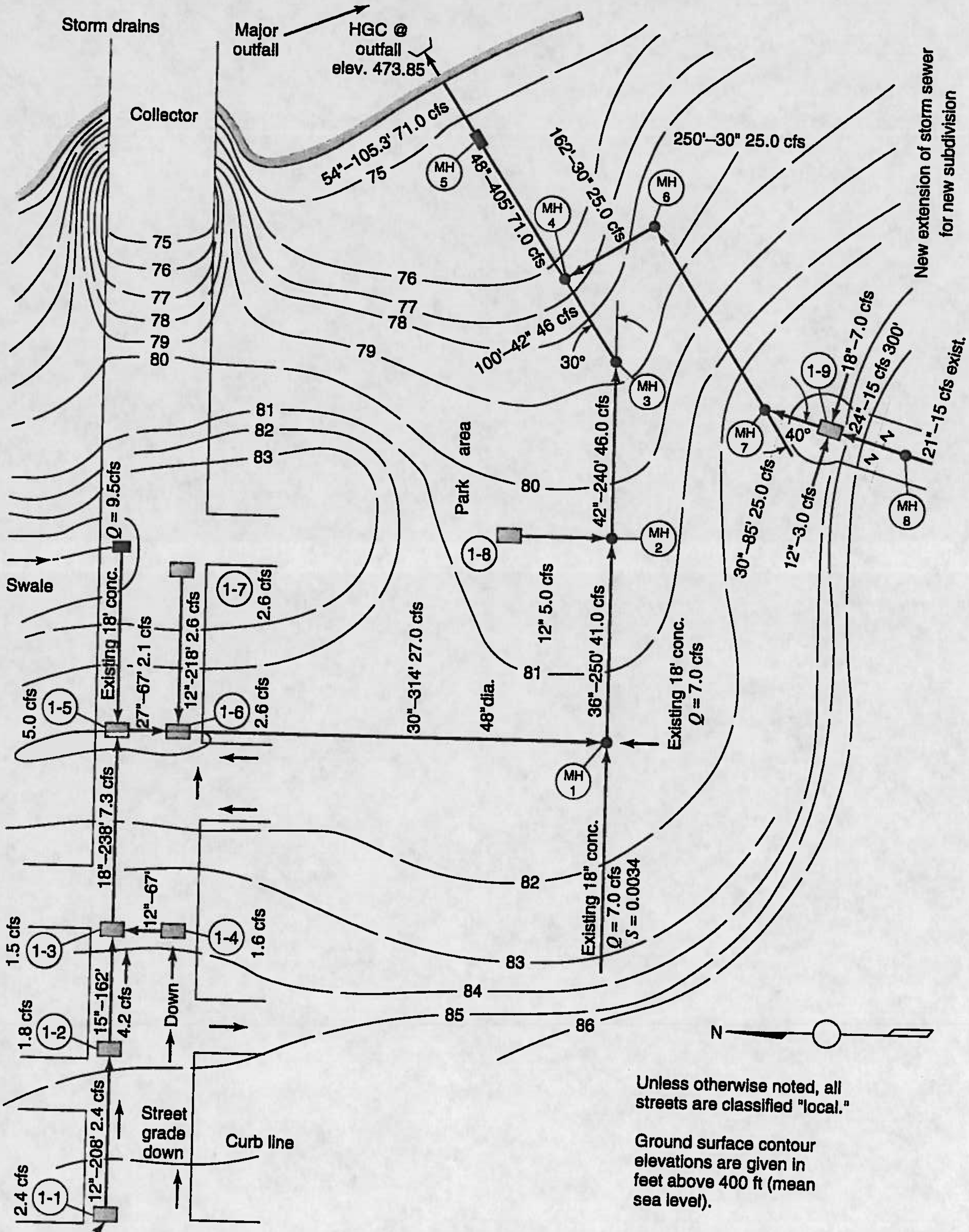


Figure 15.2.10 Storm drain design example (from Flood Control District of Maricopa County, 1992).

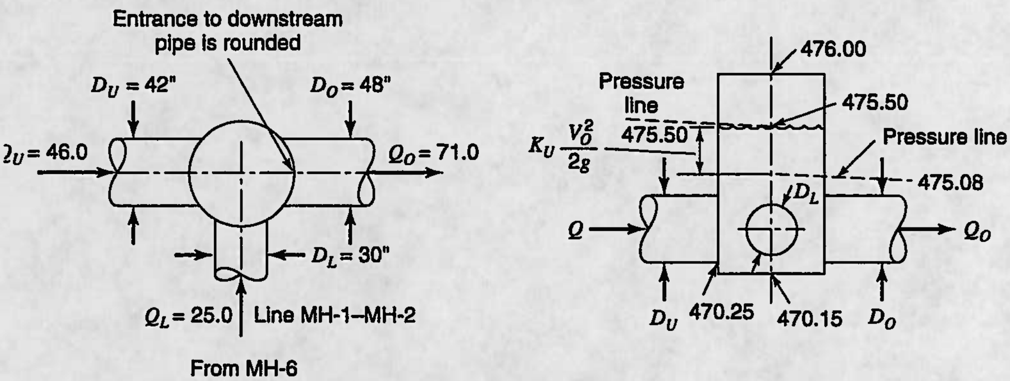


Figure 15.2.11 Storm drain design example for manhole no. 4 (from Flood Control District of Maricopa County, vol. II, 1992).

SOLUTION

1. The outfall pressure line elevation at manhole is given as 475.08 ft.
2. The velocity head at the outfall is

$$\frac{V_0^2}{2g} = \frac{1}{2g}\left(\frac{Q}{A}\right)^2 = \frac{1}{2g}\left(\frac{71}{\pi\ 4^2/4}\right)^2 = 0.50 \text{ ft} \quad (\text{Note } D = 48 \text{ in} = 4 \text{ ft})$$

3. Compute the ratios

$$\frac{Q_u}{Q_o} = \frac{46}{71} = 0.65, \quad \frac{D_u}{D_0} = \frac{42}{48} = 0.88, \quad \frac{D_L}{D_0} = \frac{30}{48} = 0.63$$

4. Compute $B/D_0 = 48/48 = 1.0$ (where B is the manhole diameter).
5. $$\left(\frac{Q_u}{Q_o}\right) \times \left(\frac{D_0}{D_u}\right) = 0.65 \times \frac{1}{0.88} = 0.74$$

Consider the lateral pipe:

6. Using Figure 15.2.6, $K_L^* = 0.95$ for $D_L/D_0 = 0.63$ and $B/D_0 = 1.0$. For a round-edged manhole $K_L^* = K_L^* - 0.2 = 0.95 - 0.20 = 0.75$, where 0.2 is obtained from the table of reductions for K_L^* for manholes with a rounded entrance (see Table 15.2.11). When $V_0^2/2g < 1.0$ it is usually not economical to use a rounded entrance from the manhole to the outlet pipe; therefore, keep $K_L^* = 0.95$ for a square-edged entrance.

Table 15.2.11 Reductions for K_L^* for a Manhole with Rounded Entrance Reductions for K_L

	D_L/D_0			
B/D_0	0.6	0.8	1.0	1.2
1.75	0.4	0.3	0.2	0.0
1.33	0.3	0.2	0.1	0.0
1.10	0.2	0.1	0.0	0/.0

Source: Flood Control District of Maricopa County, vol. II, (1992).

7. Determine m_L using $(Q_u/Q_0) \times (D_u/D_0) = 0.74$; $m_L = 0.61$ from Figure 15.2.6.
8. $K_L = m_L \times K_L^* = 0.61 \times 0.95 = 0.58$ (for square-edged entrance).
9. Lateral pipe pressure change $= K_L(V_0^2/2g) = 0.58 \times 0.50 = 0.29$ ft.
10. Lateral pipe pressure $= 475.08 + 0.29 = 475.37$ ft.

Now consider the upstream in-line pipe:

11. From Figure 15.2.7, $K_u^* = 1.86$.
12. Because the velocity head is less than 1.0 ft/s, a rounded entrance to the outfall pipe will not l appropriate and a square-edged entrance will be used.
13. From Figure 15.2.7, $m_u = 0.45$.
14. $K_u = m_u \times K_u^* = 0.45 \times 1.86 = 0.84$.
15. $h_u = K_u \times (V_0^2/2g) = 0.84 \times 0.50 = 0.42$.
16. The in-line upstream pressure elevation is 475.08 + 0.42 = 475.50, which is also the water surfac elevation, as shown in Figure 15.2.11.

15.2.3.2 Hydraulic Gradient Calculations

Any storm sewer design must be analyzed to determine if the design flows can be accommodate without causing flows to exit the system and creating flooding conditions. Figure 15.2.12 illus trates the difference between an improper design and a proper design. Note the energy an hydraulic grade lines for the improper design as opposed to the proper design.

If the hydraulic grade line is above the pipe crown at the next upstream manhole, pressure flov calculations are indicated; if it is below the pipe crown, then open-channel flow calculation should be used at the upstream manhole. The process is repeated throughout the storm drain sys tem. If all HGL elevations are acceptable, then the hydraulic design is adequate. If the HGl exceeds an inlet elevation, then adjustments to the trial design must be made to lower the wate surface elevation. Computer programs such as HYDRA (FHWA, 1993) are recommended for th design of storm drains and include a hydraulic grade-line analysis and a pressure flow simulation

15.2.3.3 Hydrograph Routing for Design

Hydrograph design methods consider design hydrographs as input to the upstream end of sewers and use some form of routing to propagate the inflow hydrograph to the downstream end of the sewer. The routed hydrograph is added to the surface runoff hydrograph to the manhole at the downstream junction, and the routed hydrograph for each sewer is added also. The combined hydrographs for all upstream connecting pipes plus the hydrograph for the surface runoff represents the design inflow hydrograph to the next (adjacent) downstream sewer pipe. The pipe size and sewer slope are selected by solving for the commercial size pipe that can handle the peak discharge of the inflow hydrograph and maintain a gravity flow.

A simple and rather effective hydrograph design method rather effective method is the *hydrograph time lag method* (Yen, 1978), which is a hydrologic (lumped) routing method. The inflow hydrograph of a sewer is shifted without distortion by the sewer flow time t_f to produce the sewer outflow hydrograph. The outflow hydrographs of the upstream sewers at a manhole are added, at the corresponding times, to the direct manhole inflow hydrograph to produce the inflow hydrograph for the downstream sewer in accordance with the continuity relationship

$$\sum Q_{ij} + Q_j - Q_o = \frac{ds}{dt} \tag{15.2.20}$$

in which Q_{ij} is the inflow from the *i*th upstream sewer into the junction *j*, Q_0 is the outflow from the junction into the downstream sewer, Q_j is the direct inflow into the manhole or junction, and S is the water stored in the junction structure or manhole. For point type junctions where there is no storage, dS/dt is 0.

The sewer flow time t_f that is used to shift the hydrograph is estimated by

$$t_f = \frac{L}{V} \tag{15.2.21}$$

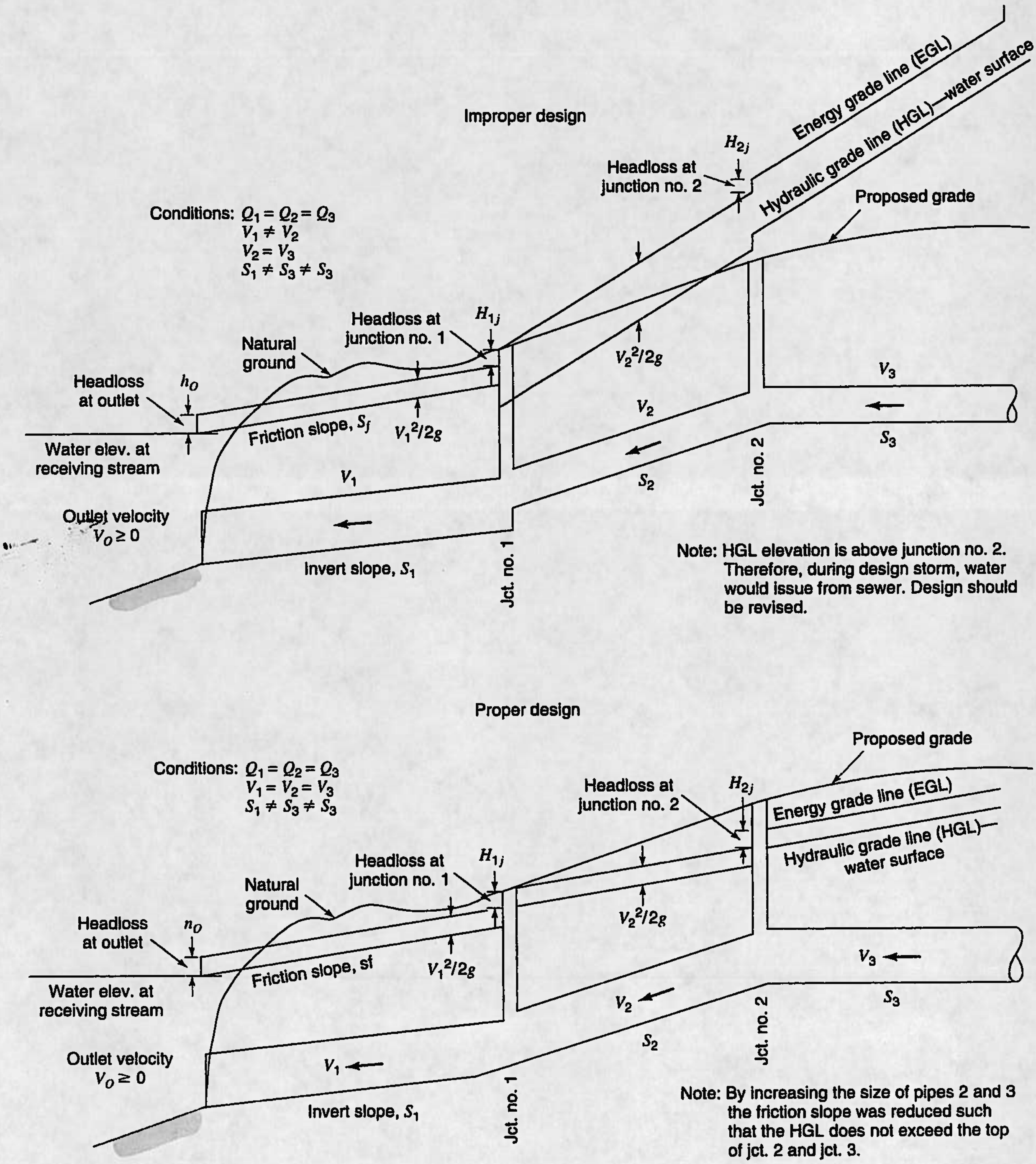


Figure 15.2.12 Use of energy losses in developing a storm drain system: energy and hydraulic grade lines for storm sewer under constant discharge (from AASHTO (1991)).

in which L is the length of the sewer and V is a sewer flow velocity. The velocity could be cor puted, assuming a full-pipe flow, using

$$V = \frac{4Q_p}{\pi D^2} \tag{15.2.2:}$$

where Q_p is the peak discharge and D is the pipe diameter. Also steady uniform flow equatioı such as Manning's equation or the Darcy-Weisbach equation can be used to compute the velocit

In this method, the continuity relationship of the flow within the sewer is not directly consic ered. The routing of the sewer flow is done by shifting the inflow hydrograph by t_f and no coı sideration is given to the unsteady and nonuniform nature of the sewer flow. Shifting of hydrc graphs approximately accounts for the sewer flow translation time but offers no wave attenuatioı However, the computational procedure through interpolation introduces numerical attenuation.

15.2.4 Storm Sewer Appurtenances

Certain appurtenances are essential to the functioning of storm sewers. These include manhole: bends, inlets, and catch basins. The discussion here briefly explains some of the common appuı tenances; for additional details refer to American Society of Civil Engineers (1992).

Manholes are located at the junctions of sewer pipes and at changes of grade, size, and align ment, with street intersections being typical locations. Typical manholes for small sewers (sewe diameters less than 2 ft) are shown in Figure 15.2.13 and for intermediate sized sewers (sewe diameters greater than 2 ft) are shown in Figure 15.2.14. Manholes provide convenient access tı the sewer for observation and maintenance operation. They should be designed to cause a mini mum of interference with the hydraulics of the sewer. Manholes should be spaced 400 ft (120 m or less for sewers 15 in (375 mm) or less and 500 ft (150 m) for sewers 18 to 30 in (460 tı 760 mm) (Great Lakes–Upper Mississippi Board of State Sanitary Engineers, 1978). For large sewers, spacing of up to a maximum of 600 ft (180 m) can be used.

Drop manholes (see Figure 15.2.15) are provided for sewers entering a manhole at an elevatioı of 24 in (0.6 m) or more above the manhole invert. These manholes are constructed with either aı internal or external drop connection. For structural reasons, external connections are preferred.

Inlets are structures where stormwater enters the sewer system. Section 16.1 provides a detailec description of and design procedures for inlets.

15.2.5 Risk-Based Design of Storm Sewers

Proposed water-excess management solutions are subject—like are most solutions to engineering problems—to an element of uncertainty. The uncertainty inherent in storm sewer design derives from both hydraulic and hydrologic aspects of the problem. Recommended references on risk-based design include Ang and Tang (1975, 1984), Chow et al. (1988), Harr (1987), Kapur and Lamberson (1977), Kececioglu (1991), Mays and Tung (1992), and Yen (1986). Also see Sections 10.6 and 10.7.

The key question is the ability of the proposed sewer design to accommodate the surface runoff generated by a storm. Although a factor of safety SF is inherent in the choice of a design frequency, the relationship between the sewer capacity Q_c and the storm runoff Q_L can also be explicitly considered: that is, $Q_c = SF \times Q_L$. Using risk/reliability analysis, a probability of failure $P(Q_L > Q_c)$ can be calculated for selected frequencies and safety factors. The corresponding risks and safety factors for each return period (recurrence interval) can be plotted to derive the risk-safety factor relationship for each return period. The procedure is as follows:

1. Select the return period T.

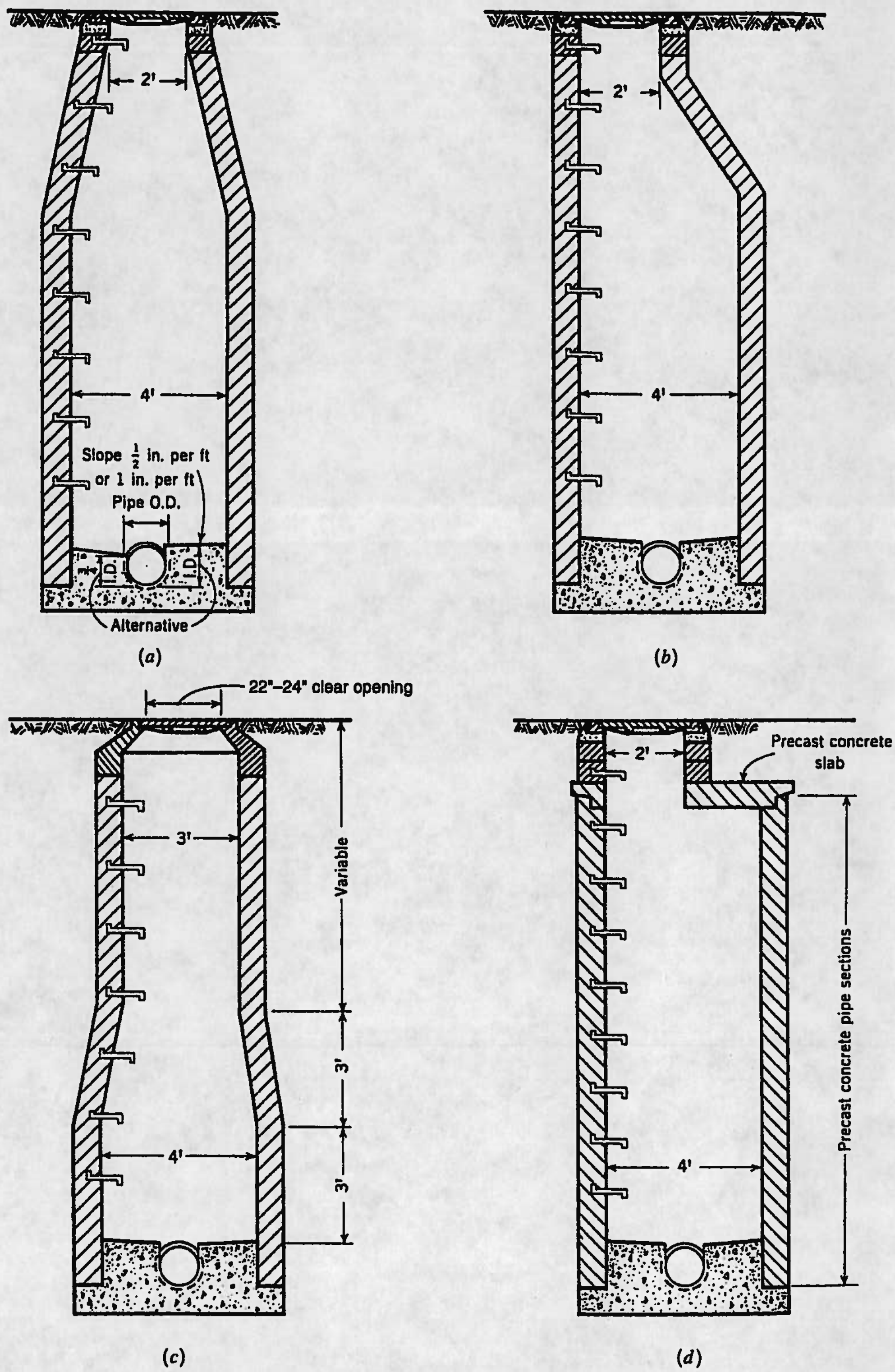


Figure 15.2.13 Typical manholes for small sewers (from ASCE (1992)).

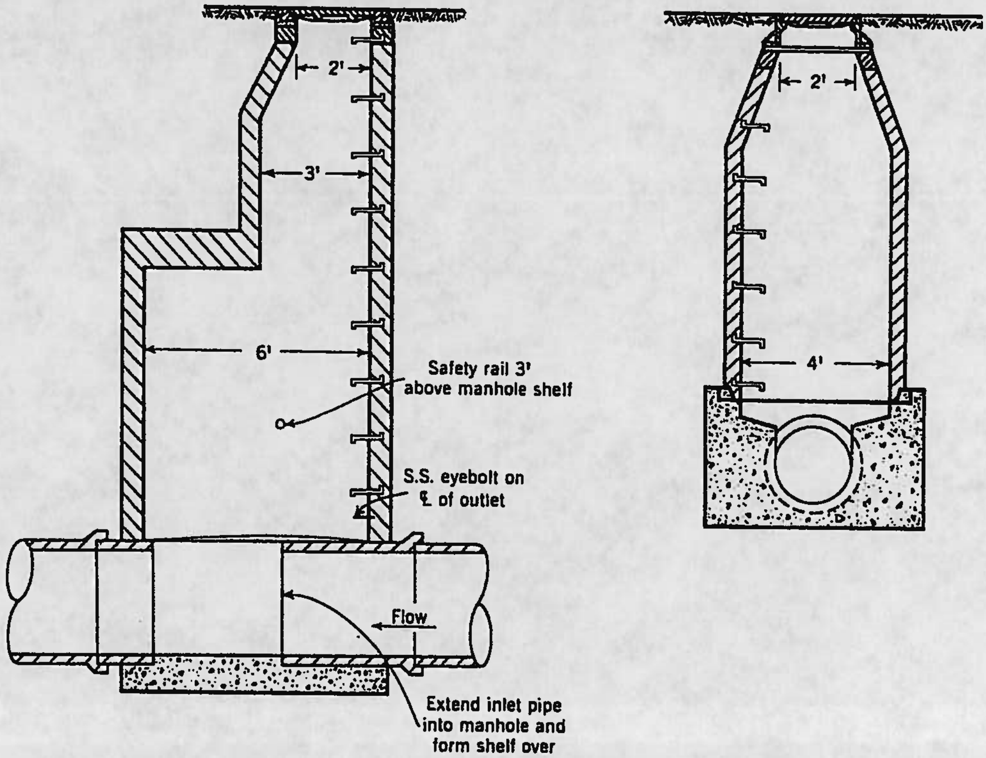


Figure 15.2.14 Two manholes for intermediate-sized sewers (from ASCE (1992)).

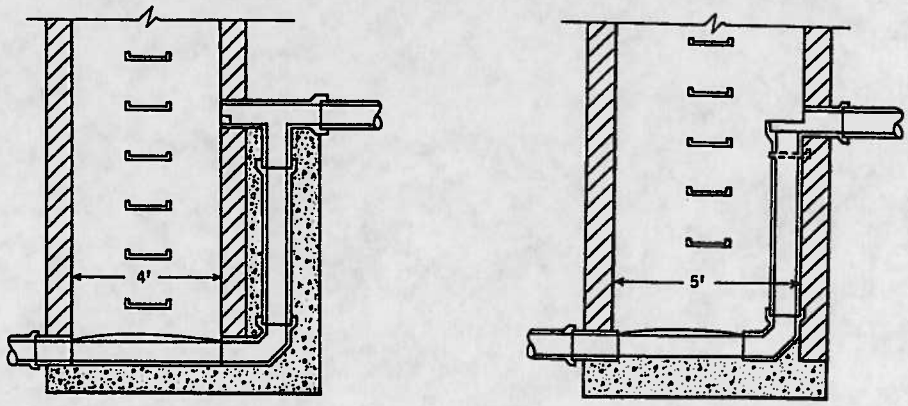


Figure 15.2.15 Drop manholes (from ASCE (1992)).

2. Use a rainfall-runoff model (such as the rational method) and perform an uncertainty analysis to compute the mean loading on the sewer $\overline{Q}_L$ (the mean surface runoff) and its coefficient of variation Ω_{QL}, where

$$\overline{Q}_L = \overline{C}\,\overline{i}\,\overline{A} \tag{15.2.23}$$

where $\overline{C}$ is the mean runoff coefficient, $\overline{A}$ is the mean basin area in acres, $\overline{i}$ is the mean rainfall intensity in in/hr, and (see derivation in Section 10.6)

$$\Omega_{QL}^2 = \Omega_C^2 + \Omega_i^2 + \Omega_A^2 \tag{15.2.24}$$

where Ω_c, Ω_i, and Ω_A are the coefficients of variation of C, i, A, respectively.

3. Select a pipe diameter D and compute $\bar{Q}_c$ and Ω_{Qc}, where $\bar{Q}_c$ is the mean capacity of the sewer and Ω_{Qc} is the coefficient of variation of Ω_c. The mean capacity can be obtained from a modified form of the Manning equation:

$$\bar{Q}_c = \frac{0.463}{\bar{n}} D^{8/3} \bar{S}_0^{1/2} \tag{15.2.25}$$

in which $\bar{n}$ is the mean Manning roughness coefficient, $\bar{S}_0$ is the mean sewer slope, and D is the sewer diameter. The coefficient of variation Ω_{Qc} is computed using (see derivation in Section 10.6)

$$\Omega_{Qc}^2 = \Omega_n^2 + \frac{1}{4}\Omega_{S_0}^2 + \left(\frac{8}{3}\right)^2 \Omega_D^2 \tag{15.2.26}$$

in which Ω_n, Ω_D, and Ω_{S_0} are the coefficients of variation of n, d, and S_0, respectively.

4. Compute the risk and safety factor (see Section 10.7).
5. Repeat Step 3 for other diameters.
6. Repeat Step 2 for other rainfall duration.
7. Return to Step 1 for each return period to be considered.

EXAMPLE 15.2.5

Determine the coefficient of variation of the loading and the capacity for the following parameters. Assume a uniform distribution to define the uncertainty of each parameter.

Parameter	Mode	Range
C	0.75	0.70–0.80
i	7.5 in/hr	7.2–7.8 in/hr
A	12 acres	11.9–12.1 acres
n	0.015	0.0145–0.0155
D	5 ft	4.96–5.04 ft
S_0	0.001 ft/ft	0.0009–0.0011 ft/ft

SOLUTION

The loading Q_L is estimated using the rational equation (15.2.23), the capacity Q_c is estimated using Manning's equation (15.2.25), and the coefficients of variation of Q_L and Q_C are determined using equations (15.2.24) and (15.2.26), respectively. All the random parameters are assumed to follow a uniform distribution, so the mean and variance of each parameter are calculated using mean = $(a + b)/2$ and variance = $(b - a)^2/12$, in which a and b are the lower and upper bounds, respectively. Hence,

$$\Omega = \frac{\sqrt{\dfrac{(b-a)^2}{12}}}{(b+a)/2} = \left(\frac{b-a}{b+a}\right)\frac{1}{\sqrt{3}}$$

Based on the above formula, the means, variance, and coefficients of each parameter are calculated in the following table:

Parameter	Range	Mean	Variance	Ω
C	0.70–0.80	0.75	8.33×10^{-4}	3.85×10^{-2}
i (in/hr)	7.2–7.8	7.5	3.00×10^{-2}	2.31×10^{-2}
A (ac)	11.9–12.1	12.0	3.33×10^{-3}	4.81×10^{-3}
n	0.0145–0.0155	0.0150	8.33×10^{-8}	1.92×10^{-2}
D (ft)	4.96–5.04	5.00	5.33×10^{-4}	4.62×10^{-3}
S_0	0.0009–0.0011	0.0010	3.33×10^{-9}	5.77×10^{-2}

Now, the coefficients of Q_L and Q_C can be calculated as

$$\Omega_{Q_L} = \sqrt{(3.85\times10^{-2})^2 + (2.31\times10^{-2})^2 + (4.81\times10^{-3})^2} = 4.52\times10^{-2}$$

$$\Omega_{Q_C} = \sqrt{(1.92\times10^{-2})^2 + \left(\frac{8}{3}\times4.62\times10^{-3}\right)^{-2} + (\frac{1}{2}\times5.77\times10^{-2})^{-2}} = 3.68\times10^{-2}$$

EXAMPLE 15.2.6

Using the results of Example 15.2.5, determine the risk of the loading exceeding the capacity of th sewer pipe. Assume the use of a safety margin ($SM = Q_C - Q_L$) that is normally distributed.

SOLUTION

The risk is defined (see Section 10.7) as Risk = $P[Q_L > Q_C] = P[SM < 0]$, in which $SM = Q_C - Q_l$ The mean and variance of SM are, respectively, $\mu_{SM} = \mu_{Qc} - \mu_{QL}$ and $\sigma^2_{SM} = \sigma^2_{Q_C}$. From exampl 15.2.5 we have

$$\mu_{Q_L} = \overline{C}\overline{i}\overline{A} = (0.75)(7.5)(12) = 67.50 \text{ cfs}$$

$$\sigma_{Q_L} = \mu_{Q_L}\Omega_{Q_L} = (67.50)(4.52\times10^{-2}) = 3.05 \text{ cfs}$$

$$\mu_{Q_C} = \frac{0.463}{\overline{n}}\left(\overline{D}\right)^{8/3}\left(\overline{S}_0\right)^{1/2} = \frac{0.464}{0.015}(5)^{8/3}(0.001)^{1/2} = 71.51 \text{ cfs}$$

$$\sigma_{Q_C} = \mu_{Q_C}\Omega_{Q_C} = (71.51)(3.68\times10^{-2}) = 2.63 \text{ cfs}$$

Therefore, $\mu_{SM} = 71.51 - 67.50 = 4.01$ and $\sigma^2_{SM} = 3.05^2 + 2.63^2 = 16.22$. The risk then is calculate as

$$\text{Risk} = P[SM < 0] = P\left(Z < \frac{0 - \mu_{SM}}{\sigma_{SM}}\right)$$

$$= P\left(Z < \frac{-4.01}{\sqrt{16.22}}\right) = P(Z < -1.00) = 0.159$$

15.3 STORMWATER DRAINAGE CHANNELS

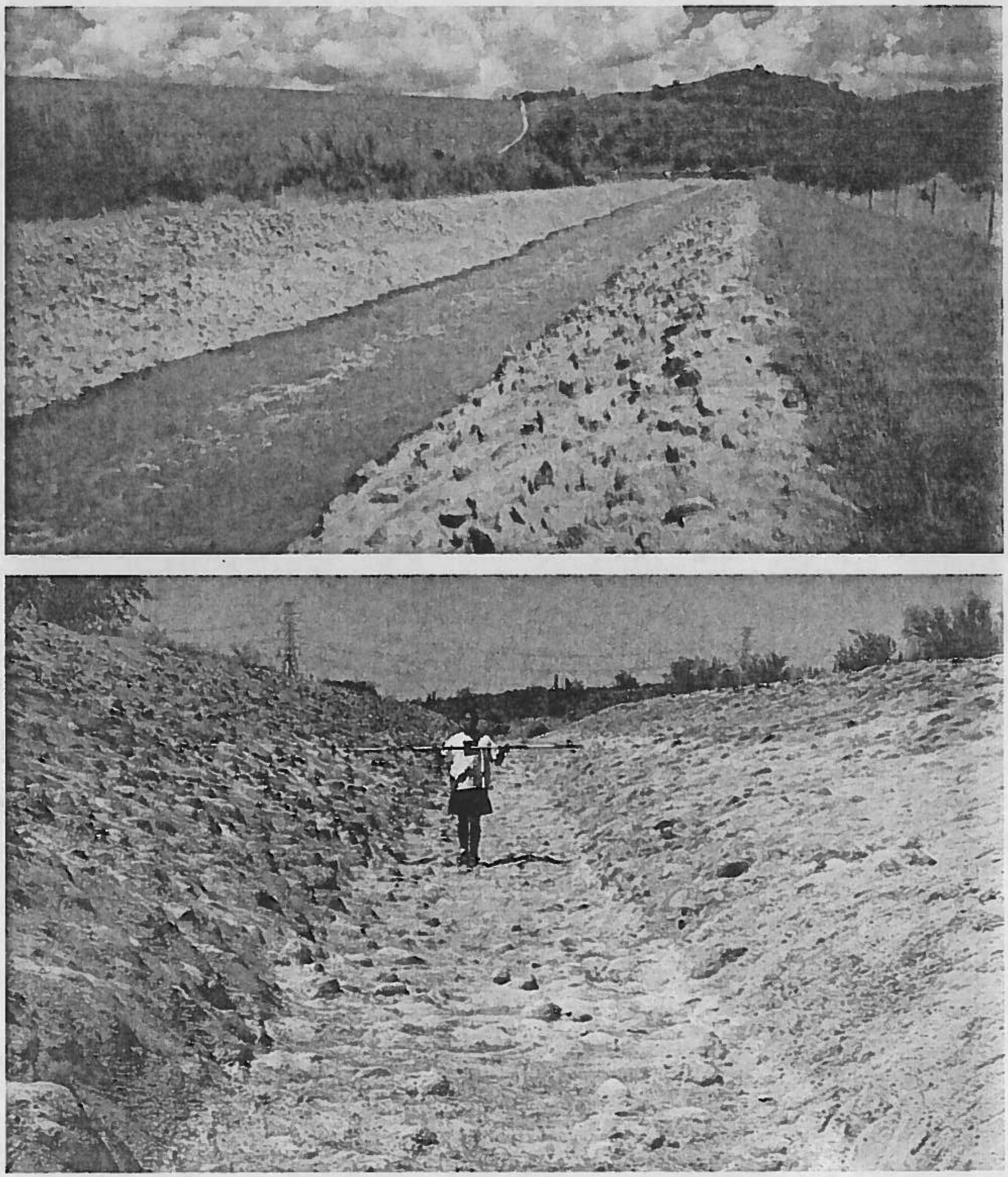
Stormwater-drainage channels (or flood-control channels) must behave in a stable, predictabl manner to ensure that a known flow capacity will be available for a design storm event. In mos cases, the design goal is a noneroding channel boundary, although, in certain cases, a dynami channel is desired (Cotton, 1999).

Because most soils erode under a concentrated flow, either temporary or permanent channel linings are needed to achieve channel stability.

Channel linings can be classified in two broad categories: rigid or flexible. ***Rigid linings*** include channel pavements of concrete or asphaltic concrete and a variety of precast interlocking block and articulated mats. ***Flexible linings*** include such materials as loose stone (riprap), vegetation manufactured mats of lightweight materials fabrics, or combinations of these materials. Rigid linings are capable of high conveyance and high-velocity flow. Rigid-lined channels are used tc reduce the amount of land required for a surface drainage system. The selection of lining is a function of the design context related to the consequences of flooding, the availability of land and environmental needs (Cotton, 1999). Figures 15.3.1 and 15.3.2 show two constructed channels ir Arizona, each with and without flow.

15.3.1 Rigid-Lined Channels

Rigid-lined channels are nonerodible channel sides typically lined with concrete grouted riprap, stone masonry, or asphalt. The steps in the design of such a channel are as follows:



'igure 15.3.1 Views of constructed reach of Cave Creek below Cave Buttes Dam, Phoenix, Arizona. The constructed reach is traight, and cross-sections are trapezoidal in shape. The bottom and sides of the channel are composed of rounded cobbles imbed-.ed in a matrix of cement (approximate mean diameter of the rock projections was 80 mm, about half of which seemed to be xposed to flow). Roughness elements are constant throughout the reach. The channel gradient increases from about 0.002 ft/ft at ross-section 1 to about 0.010 ft/ft at cross-section 8. The stream is ephemeral, and flow is regulated by Cave Buttes Dam (Phillips nd Ingersoll (1998)).

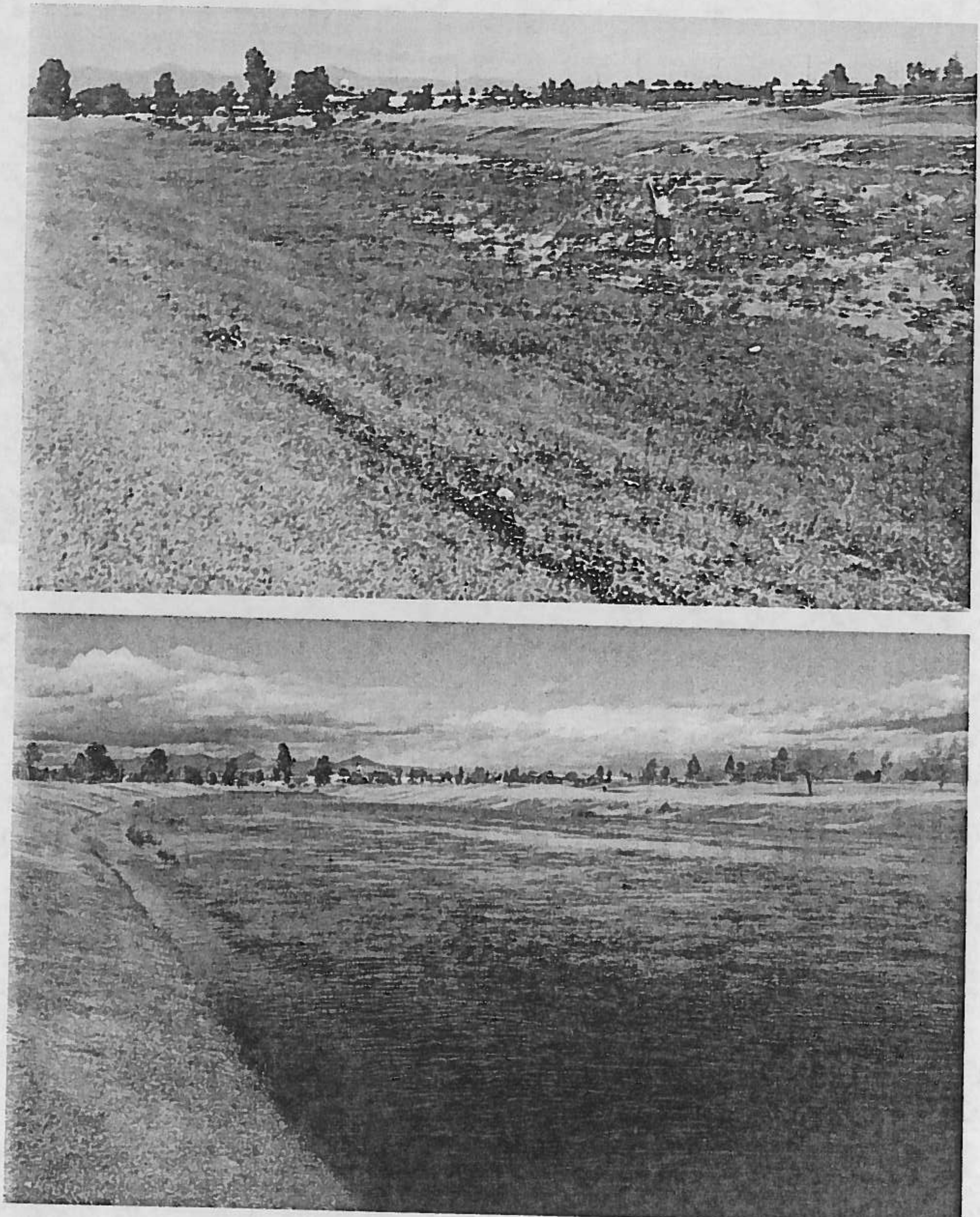


Figure 15.3.2 Views of Indian Bend Wash in Scottsdale, Arizona. The constructed channel is uniform and cross-sections are trapezoidal in shape (channel bottom is firm earth with seasonal growth of grasses and small brush) (Phillips and Ingersoll (1998)).

1. Select the Manning roughness coefficient, n (see Table 15.3.1), the channel side slope z, and the channel bottom slope S_0. The bottom slope is based upon the topography and other considerations such as alignment.

Table 15.3.1 Manning's Roughness Coefficients

		Manning's n		
		Depth Ranges		
Lining Category	Lining Type	0–0.5 ft (0–15 cm)	0.5–2.0 ft (15–60 cm)	>2.0 ft (>60 cm)
Rigid	Concrete	0.015	0.013	0.013
	Grouted riprap	0.040	0.030	0.028
	Stone masonry	0.042	0.032	0.030
	Soil cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare soil	0.023	0.020	0.020
	Rock cut	0.045	0.035	0.025
Temporary	Woven paper net	0.016	0.015	0.015
	Jute net	0.028	0.022	0.019
	Fiberglass roving	0.028	0.021	0.019
	Straw with net	0.065	0.033	0.025
	Curled wood mat	0.066	0.035	0.028
	Synthetic mat	0.036	0.025	0.021
Gravel riprap	1 in (2.5 cm) D_{50}	0.044	0.033	0.030
	2 in (5 cm) D_{50}	0.066	0.041	0.034
Rock riprap	6 in (15 cm) D_{50}	0.104	0.069	0.035
	12 in (30 cm) D_{50}	—	0.078	0.040

Source: Chen and Cotton (1988).

2. Compute the uniform flow section factor (see Chapter 5)

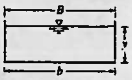
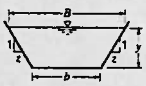
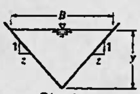
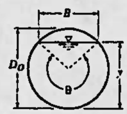
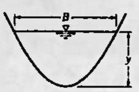
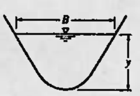
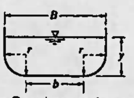
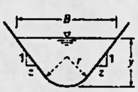
$$AR^{2/3} = \frac{Qn}{K_a S^{1/2}} \tag{15.3.1}$$

in which A is the cross-sectional area of flow, ft^2 (m^2), R is the hydraulic radius in ft (m), Q is the design discharge in ft^3/s (m^3/s), and $K_a = 1.49$ for U.S. customary units ($K_a = 1.0$ for SI units).

3. Determine the channel dimensions and flow depth for the uniform flow section factor computed in step 2. Choose the expression for the uniform flow section factor $AR^{2/3}$, as a function of depth, in Table 15.3.2 and solve for the depth using the value of $AR^{2/3}$ from equation (15.3.1). For a trapezoidal channel,

$$\left[\frac{(B_w + zy)^5 y^5}{\left(B_w + 2y\sqrt{1+z^2}\right)^2}\right]^{1/3} = AR^{2/3} \tag{15.3.2}$$

Table 15.3.2 Geometric Elements of Channel Cross-Sections

Cross-Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width B
 Rectangle	by	$b + 2y$	$\frac{by}{b+2y}$	b
 Trapezoid	$(b + zy)y$	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b + 2zy$
 Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$
 Circle	$\frac{1}{8}(\theta - \sin\theta)D_0^2$	$\frac{1}{2}\theta D_0$	$\frac{1}{4}\left(1-\frac{\sin\theta}{\theta}\right)D_0$	$(\sin \tfrac{1}{2}\theta)D_0$ or $2\sqrt{y(D_0 - y)}$
 Parabola $y = cx$	$\frac{2}{3}By$	$B+\frac{8}{3}\frac{y^2}{B}$ †	$\frac{2B^2y}{3B^2+8y^2}$ †	$\frac{3}{2}\frac{A}{y}$
 Power $y = cx^{1/m}$	$\frac{By}{(m+1)}$	$\frac{2y}{m}\sqrt{\frac{m^2}{c^{2m}}y^{2m-2}+1}+\sum$ §	$\frac{A}{P}$	$2\left(\frac{y}{c}\right)^m$
 Round-cornered rectangle ($y > r$)	$\left(\frac{\pi}{2}-2\right)r^2+(b+2r)y$	$(\pi-2)r + b + 2y$	$\frac{(\pi/2-2)r^2+(b+2r)y}{(\pi-2)r+b+2y}$	$b + 2r$
 Round-bottomed triangle	$\frac{B^2}{4z}-\frac{r^2}{z}(1-z\cot^{-1}z)$	$\frac{B}{z}\sqrt{1+z^2}-\frac{2r}{z}(1-z\cot^{-1}z)$	$\frac{A}{P}$	$2\left[z(y-r)+r\sqrt{1+z^2}\right]$

*For the section factor Z_c, the energy correction factor α or momentum correction factor β are assumed equal to unity. Otherwise, $Z_c = Q/\sqrt{g/\alpha}$ c $Z_c = Q/\sqrt{g/\beta}$.

†Satisfactory approximation for the interval $0 < x < 1$, where $x = 4y/b$. When $x > 1$, use the exact expression

$$P=(B/2)\left[\sqrt{1+x^2}+1/x\ \ln\left(x+\sqrt{1+x^2}\right)\right].$$

‡For trapezoid, approximate Y_c valid for $0.1 < Q/b^{2.5} < 0.4$; when $Q/b^{2.5} < 0.1$, use rectangular formula.

$$^{\S}\sum=\left(1-\frac{1}{m}\right)y\sum_{k=0}^{\infty}\frac{\left(\frac{1}{2}\right)_k\left(\frac{1}{2m-2}\right)\left[-\frac{m}{c^m}y^{m-1}\right]^{2k}}{\left(1+\frac{1}{2m-2}\right)_k k!},\ \text{where } (w)_k = w(w+1)\ldots(w+k-1),\ k=1,2,\ldots,(w)_{k=0}=1.$$

Source: Yen (1996).

Hydraulic Mean Depth D	Uniform Flow Section Factor $AR^{2/3} = \frac{Qn}{K_n S^{0.5}}$	Critical Flow Section Factor* $Z_c = Q/\sqrt{g} = A_c\sqrt{D_c}$	Critical Depth y_c
y	$\left[\frac{b^5 y^5}{(b+2y)^2}\right]^{1/3}$	$by^{1.5}$	$\left(\frac{Z_c}{b}\right)^{2/3}$
$\frac{(b+zy)y}{b+2zy}$	$\left[\frac{(b+zy)^5 y^5}{(b+2y\sqrt{1+z^2})^2}\right]^{1/3}$	$\frac{[(b+zy)y]^{1.5}}{\sqrt{b+2zy}}$	$0.81\left(\frac{Z_c^4}{z^{1.5} b^{2.5}}\right)^{0.135} - \frac{b}{30z}$ ‡
$\frac{y}{2}$	$\left[\frac{z^5 y^8}{4(1+z^2)}\right]^{1/3}$	$\frac{\sqrt{2}}{2} zy^{2.5}$	$\left(\frac{\sqrt{2} Z_c}{z}\right)^{0.4}$
$\frac{1}{8}\left(\frac{\theta - \sin\theta}{\sin \frac{1}{2}\theta}\right) D_0$	$\frac{1}{16}\left[\frac{(\theta - \sin\theta)^5 D_0^3}{2\theta^2}\right]^{1/3}$	$\frac{\sqrt{2}}{32}\left(\frac{(\theta - \sin\theta)^{1.5}}{(\sin \frac{1}{2}\theta)^{0.5}}\right) D_0^{2.5}$	$\frac{1.01}{D_0^{0.26}} Z_c^{0.5}$ † for $0.02 \le y_c/D_0 \le 0.85$
$\frac{2}{3} y$	$\frac{2}{3}\left[\frac{4B^7 y^5}{(3B^2 + 8y^2)^2}\right]^{1/3}$ †	$\frac{2\sqrt{6}}{9} By^{1.5}$	$0.958(c^{0.5} Z_c)^{0.5}$
$\frac{y}{(m+1)}$	$\left[\frac{A^5}{P^2}\right]^{1/3}$	$\frac{By^{1.5}}{(m+1)^{1.5}}$	$\left[\frac{c^m}{2}(m+1)^{1.5} Z_c\right]^{1/(m+1.5)}$
$\frac{(\pi/2 - 2)r^2}{b+2r} + y$	$\left[\frac{A^5}{P^2}\right]^{1/3}$	$\frac{\left[(\pi/2 - 2)r^2 + (b+2r)y\right]^{1.5}}{\sqrt{b+2r}}$	
$\frac{A}{B}$	$\left[\frac{A^5}{P^2}\right]^{1/3}$	$A\sqrt{\frac{A}{B}}$	

where B_w is the bottom width. By assuming several values of B_w and z, a number of combinations of section dimensions can be obtained. Final dimensions should be based upon hydraulic efficiency and practicability. If the best hydraulically efficient section is required, select the expression for $AR^{2/3}$ from Table 15.3.3 and solve for the depth using the value of $AR^{2/3}$ from equation (15.3.1). For a trapezoidal channel,

$$\sqrt{3}\left(\frac{y^8}{4}\right)^{1/3} = AR^{2/3} \tag{15.3.3}$$

4. Check the minimum velocity to see if the water carries the silt.
5. Add an appropriate freeboard to the depth of the channel section.

Table 15.3.3 Best Hydraulically Efficient Sections Without Freeboard

Cross-Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width B	Hydraulic Depth D	$AR^{2/3}$
Trapezoid, half of a hexagon	$\sqrt{3}y^2$	$2\sqrt{3}y$	$\frac{1}{2}y$	$\frac{4}{3}\sqrt{3}y$	$\frac{3}{4}y$	$\sqrt{3}\left(\frac{y^8}{4}\right)^{1/3}$
Rectangle, half of a square	$2y^2$	$4y$	$\frac{1}{2}y$	$2y$	y	$(2y^8)^{1/3}$
Triangle, half of a square	y^2	$2\sqrt{2}y$	$\frac{1}{4}\sqrt{2}y$	$2y$	$\frac{1}{2}y$	$\frac{1}{2}y^{8/3}$
Semicircle	$\frac{\pi}{2}y^2$	πy	$\frac{1}{2}y$	$2y$	$\frac{\pi}{4}y$	$\frac{\pi}{2}(2y^8)^{1/3}$
Parabola, $B = 2\sqrt{2y}$	$\frac{4}{3}\sqrt{2}y^2$	$\frac{8}{3}\sqrt{2}y$	$\frac{1}{2}y$	$2\sqrt{2}y$	$\frac{2}{3}y$	$\frac{2\sqrt{2}}{3}(2y^8)^{1/3}$
Hydrostatic catenary	$1.39586y^2$	$2.9836y$	$0.46784y$	$1.917532y$	$0.72795y$	$0.84122y^{8/3}$

Source: Yen (1996).

EXAMPLE 15.3.1 Design a nonerodible trapezoidal channel to carry a discharge of 11.33 m³/s.

SOLUTION

1. The Manning's $n = 0.025$ slope, $S_0 = 0.0016$.
2. The uniform flow section factor is computed using equation (15.3.1):

$$AR^{2/3} = \frac{Qn}{K_a S^{1/2}} = \frac{11.33 \times 0.025}{(0.0016)^{1/2}} = 7.08$$

3. Equation (15.3.3) is used for the best hydraulic section:

$$\sqrt{3}\left(\frac{y^8}{4}\right)^{1/3} = AR^{2/3} = 7.08, \text{ solving } y = 2.02 \text{ m.}$$

   Because the best hydraulic section is half of a hexagon, the side slopes are $z = \tan 30° = 0.577$ z horizontal to 1 vertical. The area for a best hydraulic section is $A = \sqrt{3}y^2 = \sqrt{3}(2.02) = 7.07\text{m}^2 = (B_w + zy)y$, so $[B_w + (0.577)(2.02)]2.02 = 7.07$. Solving, the bottom width is 2.33 m.
4. The velocity is $Q/A = 11.33/7.07 = 1.60$ m/s, which is greater than the minimum permissible velocity for inducing silt deposition (if any exists).
5. A required freeboard, e.g., 1 m, can be added, so the total channel depth would be 3.02 m.

15.3.2 Flexible-Lined Channels

Flexible-lined channels include rock riprap and vegetable linings and are considered flexible because they can conform to change in channel slope. Flexible linings have the following advantages for stormwater conveyance: they (1) permit infiltration and exfiltration; (2) filter out contaminants; (3) provide greater energy dissipation; (4) allow flow conditions that provide better habitat opportunities for local flora and fauna, and (5) are less expensive. For a given design flow, channel geometry, and slope, the following design procedure can be used to select the appropriate flexible lining (Kouwen et al., 1969; Bathurst et al., 1981; Cotton, 1999; Wang and Shen, 1985; Chen and Cotton, 1988):

1. Choose a flexible lining from Table 15.3.4 and note its permissible shear stress τ_p.

Table 15.3.4 Permissible Shear Stresses for Lining Materials

Lining Category	Lining Type	Permissible Shear Stress, τ_p lb/ft^2	kg/m^2
Temporary	Woven paper net	0.15	0.73
	Jute net	0.45	2.20
	Fiberglass roving		
	Single	0.60	2.93
	Double	0.85	4.15
	Straw with net	1.45	7.08
	Curled wood mat	1.55	7.57
	Synthetic mat	2.00	9.76
Vegetative	Class A	3.70	18.06
	Class B	2.10	10.25
	Class C	1.00	4.88
	Class D	0.60	2.93
	Class E	0.35	1.71
Gravel riprap	1-in	0.33	1.61
	2-in	0.67	3.22
Rock riprap	6-in	2.00	9.76
	12-in	4.00	19.52

Source: Chen and Cotton (1988).

2. Assume an appropriate flow depth y (for vegetative lining only; for nonvegetative lining, assume the range of flow depth and go to step 3).
3. Use Table 15.3.1 for nonvegetative lining to find the Manning n. For vegetative lining use Table 15.3.5 to determine the appropriate retardant class. The Manning n for vegetative lining is given by the general equation

$$n = k_1/(a_c + k_2) \tag{15.3.4}$$

in which $k_1 = R^{1/6}$, where R is the hydraulic radius, ft; $k_2 = 19.97 \log(R^{1.4}S_0^{0.4})$ where S_0 is the channel longitudinal slope, ft/ft; $a_c = 15.8$, 23.0, 30.2, 34.6, and 37.7 for retardance classes A, B, C, D, E respectively.

Table 15.3.5 Classification of Vegetal Covers by Degree of Retardance

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30 in [76 cm])
	Yellow bluestem *Ischaemum*	Excellent stand, tall (average 36 in [91 cm])
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall [average 12 in (30 cm)]
	Native grass mixture (little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrasses	Good stand, (average 24 in [61 cm])
	Lespedeza sericea	Good stand, not woody, tall (average 19 in [48 cm])

Table 15.3.5 Classification of Vegetal Covers by Degree of Retardance (*continued*)

	Alfalfa	Good stand, uncut (average 11 in [28 cm])
	Weeping lovegrass	Good stand, unmowed (average 13 in [28 cm])
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13 in [28 cm])
C	Crabgrass	Fair stand, uncut (10 to 48 in [25 to 120 cm])
	Bermuda grass	Good stand, mowed (average 6 in [15 cm])
	Common lespedeza	Good stand, uncut (average 11 in [28 cm])
	Grass-legume mixture—summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8 in [15 to 20 cm])
	Centipedegrass	Very dense cover (average 6 in [15 cm])
	Kentucky bluegrass	Good stand, headed (6 to12 in [15 to 30 cm])
D	Bermuda grass	Good stand, cut 2.5-in height (6 cm)
	Common lespedeza	Excellent stand, uncut (average 4.5 in [11 cm])
	Buffalo grass	Good stand, uncut (3 to 6 in [8 to 15 cm])
	Grass-legume mixture—fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 in [10 to 13 cm])
	Lespedeza sericea	After cutting to 2-in height (5 cm) Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5-in height (4 cm)
	Bermuda grass	Burned stubble

Source: U.S. Soil Conservation Service (1954).

4. Calculate the computed flow depth y_{comp} from the Manning equation using the value of *n* from step 3.
5. For vegetative lining, compare y and y_{comp}; if they are not close enough replace y based on y_{comp} for a new y and go to step 3. For nonvegetative lining, go to step 6.
6. Compute shear stress for the design condition by

$$\tau_{des} = \gamma R S_0 \tag{15.3.5}$$

If $\tau_{des} < \tau_p$ (Table 15.3.4) the lining is acceptable. Otherwise go to step 1 and choose a different lining.

EXAMPLE 15.3.2

Design a flexible-lined trapezoidal channel for a slope of $S_0 = 0.0016$ (same flow rate of 11.33 m^3/s and slope as example 15.3.1). Use a nonvegetative lining.

SOLUTION

1. A gravel riprap (2.5 cm) D_{50} is chosen.
2. A flow depth of greater than 60 cm is assumed.
3. From Table 15.3.1, the Manning's roughness factor is $n = 0.03$.
4. Compute the flow depth using Manning's equation:

$$AR^{2/3} = \frac{Qn}{K_a S^{1/2}} = \frac{11.33 \times 0.03}{(0.0016)^{1/2}} = 8.50$$

Assume a bottom width of 6 m and $z = 2$. So

$$AR^{2/3} = 8.50 = \left[\frac{\left(B_w + zy)^5 y^5\right)}{\left(B_w + 2y\sqrt{1+z^2}\right)^2}\right]^{1/3}$$

Solving yields $y_{comp} = 1.14$ m. Skip step 5 for nonvegetative lining and go to step 6.

6. The shear stress is computed using equation (15.3.5) with $R \approx y$,
$\tau_{des} = \gamma y S_0 = (9810)(1.14 \text{ m})(0.0016) = 17.89 \text{ N/m}^2 \ (1.73 \text{ kg/m}^2)$
The allowable shear stress is $\tau = 1.61 \text{ kg/m}^2$ (from Table 15.3.4).
Because $\tau_{des} > \tau_{allowable}$, a 5 cm gravel riprap with approximately double the allowable shear stress only increases Manning's n from 0.03 to 0.034. So, returning to step 4, we find

$$AR^{2/3} = \frac{Qn}{K_a S^{1/2}} = \frac{11.33 \times 0.034}{(0.0016)^{1/2}} = 9.63$$

$$= \left[\frac{\left(B_w + zy)^5 y^5\right)}{\left(B_w + 2y\sqrt{1+z^2}\right)^2} \right]^{1/3} = 9.63$$

Solving yields $y \approx 1.22$ m. The shear stress is $\tau_{des} = 1.95 \text{ kg/m}^2$ and the permissible shear stress is 3.22 kg/m^2 (Table 15.3.4).

7. Freeboard can be added.

EXAMPLE 15.3.3

Design a gravel riprap triangle-shaped channel to carry a discharge of 11.33 m^3/s at a slope of 0.0016.

OLUTION

1. Select a $D_{50} = 2.5$ cm gravel riprap.
2. Assume $y > 60$ cm.
3. From Table 15.3.1, $n = 0.034$.
4. Compute flow depth:

$$AR^{2/3} = \frac{Qn}{S^{1/2}} = \frac{11.33 \times 0.034}{(0.0016)^{1/2}} = 9.63$$

Using the best hydraulic section $AR^{2/3} = 1/2y^{8/3} = 9.63$; then $y = 3.03$ m.

5. The assumed and computed depths for the selected Manning's n are OK.
6. The design shear stress using $R = (\sqrt{2}/4)y$ is $\tau_{des} = \gamma R S_0 = 9810(\sqrt{2}/4)(3.03)(0.0016) = 16.81$ $\text{N/m}^2 = 1.71 \text{ Kg/m}^2 < \tau_{allowable} = 3.22$ (Table 15.3.4).
7. A freeboard can be added.

5.4 STORMWATER DETENTION

5.4.1 Why Detention? Effects of Urbanization

Urban stormwater management systems typically include detention and retention facilities to mitigate the negative impacts of urbanization on stormwater drainage. The effects of urbanization on stormwater runoff include increased total volumes of runoff and peak flow rates, as depicted in Figure 15.4.1. In general, major changes in flow rates in urban watershed are the result of (Chow et al., 1988):

1. The increase in the volume of water available for runoff because of the increased impervious cover provided by parking lots, streets, and roofs, which reduce the amount of infiltration;
2. Changes in hydraulic efficiency associated with artificial channels, curbing, gutters, and storm drainage collection systems, which increase the velocity of flow and the magnitude of flood peaks.

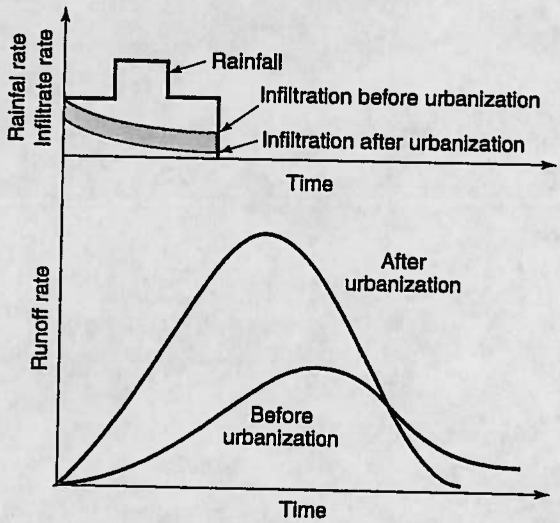


Figure 15.4.1 Effect of urbanization on stormwater runoff.

Stahre and Urbonas (1990) present the classification of storage facilities shown in Figu 15.4.2. The major classification is *source control* or *downstream control*. Source control involve the use of smaller facilities located near the source, allowing better use of the downstream co veyance system. Downstream control uses storage facilities that are larger and consequently fewer locations, such as at watershed outlets. As Figure 15.4.2 shows, source control consists local disposal, inlet control, and on-site detention. *Local disposal* is the use of infiltration

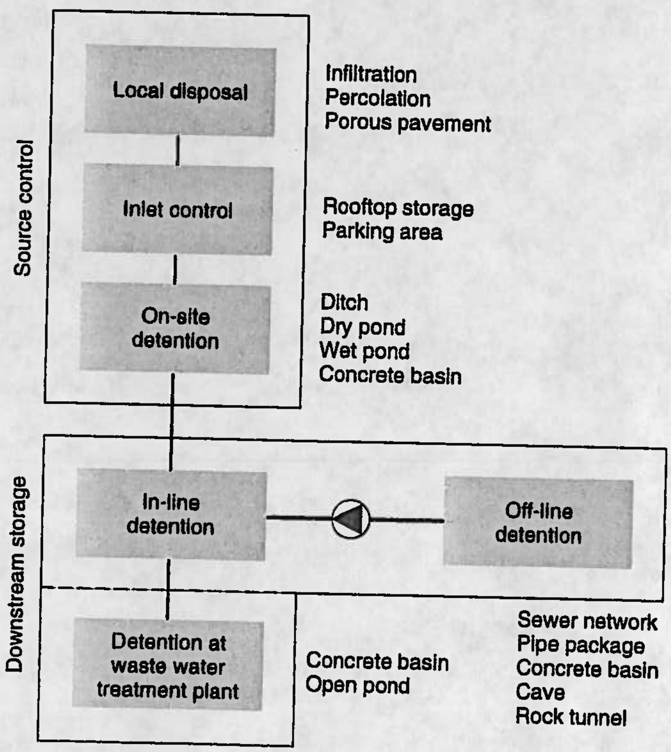


Figure 15.4.2 Classification of storage facilities (from Stahre and Urbonas (1990)).

percolation. *Inlet control* entails detaining stormwater where the precipitation occurs (such as rooftops and parking lots). *On-site detention* typically refers to detaining stormwater from larger areas than the previous two and includes swales, ditches, dry ponds, wet ponds, concrete basins (which are typically underground), and underground piping. Wet ponds have a permanent water pool as opposed to dry ponds.

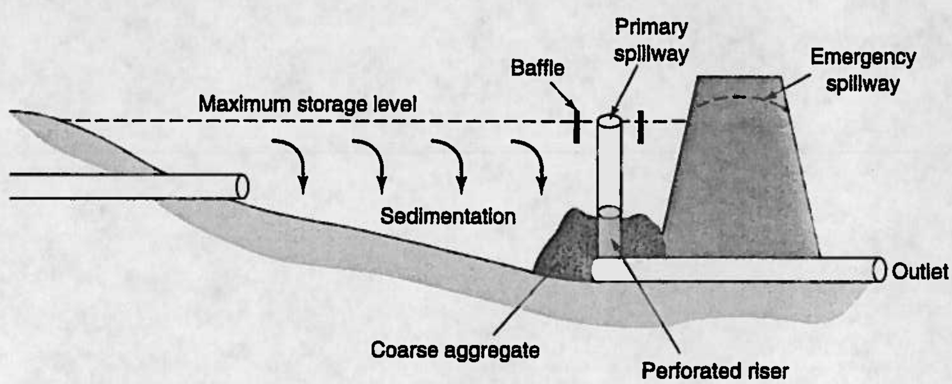
Downstream storage includes *in-line detention*, *off-line detention*, and *detention at wastewater treatment plants*. In-line detention refers to detention storage in sewer lines, tunnels, storage vaults, pipes, surface ponds, or other facilities that are connected in-line with a stormwater conveyance network. Off-line storage facilities are not in line with the stormwater conveyance system.

Detention as described in this chapter is of two major types: (1) underground or subsurface systems, and (2) surface systems. Most of this section discusses surface detention: section 15.4.3 discusses various methods for sizing detention ponds, section 15.4.4 discusses various types of detention, section 15.4.5 discusses infiltration methods, and section 15.4.6 discusses water quality aspects.

Additional references on stormwater management include Overton and Meadows (1974), Meadows (1976), Stahre and Urbonas (1990), Loganathan et al. (1996), and Whipple et al. (1983).

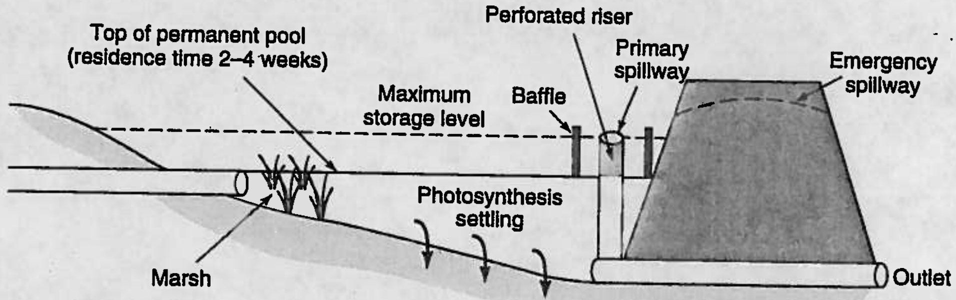
.5.4.2 Types of Surface Detention

Surface detention, for purposes of this discussion, refers to *extended detention basins* (or *dry detention basins*) and *retention ponds* (or *wet detention ponds*). Dry detention ponds empty after a storm, whereas retention ponds retain the water much longer above a permanent pool of water. Dry detention is the most widely used technique in the United States and many other countries. Figure 15.4.3 illustrates an extended detention basin. Water enters the basin, is impounded behind the embankment, and is slowly discharged through a perforated riser outlet. The coarse aggregate around the perforated riser minimizes clogging by debris. Typically once a required water-quality volume is filled, the remaining inflow is diverted around the basin or the pond overflows through a primary spillway. A large part of the sediment from the stormwater settles in the basin. Refer to Loganathan et al. (1996), Loganathan et al. (1985; 1989), Segaua and Loganathan (1992), and Wanielista and Yousef (1993).



- Efficiency: Poor for detention times under 12 hours
  Good for detention times greater than 24 hours
- Function: Settle pollutants out; soluble pollutants pass through
- Maintenance is moderate if properly designed
- Improper design can make facilities an eyesore and a mosquito-breeding mudhole
- Newer designs are incorporating a shallow marsh around outlet
  Result: Better removal efficiency and no mosquito nuisance
- Regional detention facilities serving 100–200 acres can be aesthetically developed
  Result: Lower maintenance costs

Figure 15.4.3 Design of an extended detention basin (from Urbonas and Roesner (1993)).



- Efficiency: Excellent if properly designed.
  Can be poor if bottom goes anoxic.
- Function: Removes pollutants by settling dissolved pollutants biochemically.
- Maintenance: Relatively free after first year except for major cleanout at about 10 years.
- Aesthetic design can make pond an asset to community.
  Excellent as a regional facility.

Figure 15.4.4 Design of a retention pond (from Urbonas and Roesner (1993)).

Figure 15.4.4 illustrates a retention pond, which is basically a lake that can be designed t remove pollutants. The figure illustrates the basic treatment processes that occur in the retentio pond. Pollutants are removed by settling. Nutrients are removed by phytoplankton growth in th water column and by shallow marsh plants around the pond perimeter.

A multipurpose detention basin for quantity and quality is illustrated in Figure 15.4.5. The ou let works are staged so that the water-quality design volume is released very slowly. The othe stages (see figure) provide storage and outlet peak discharges for erosion and flood contro Whipple et al. (1987) discuss the implementation of dual purpose stormwater detention program

Outlet works for extended detention and retention ponds differ because of the different operat ing functions of each. Figures 15.4.6 and 15.4.7 illustrate outlet works for dry (extended) deten tion ponds, which allow the entire storage volume to drain. The structure in Figure 15.4.6 has fixed gate control and the structure in Figure 15.4.7 has a fixed orifice control. Figure 15.4.8 illus trates an outlet for a retention pond in which the water level drops to a permanent pool level tha is controlled by positioning the openings.

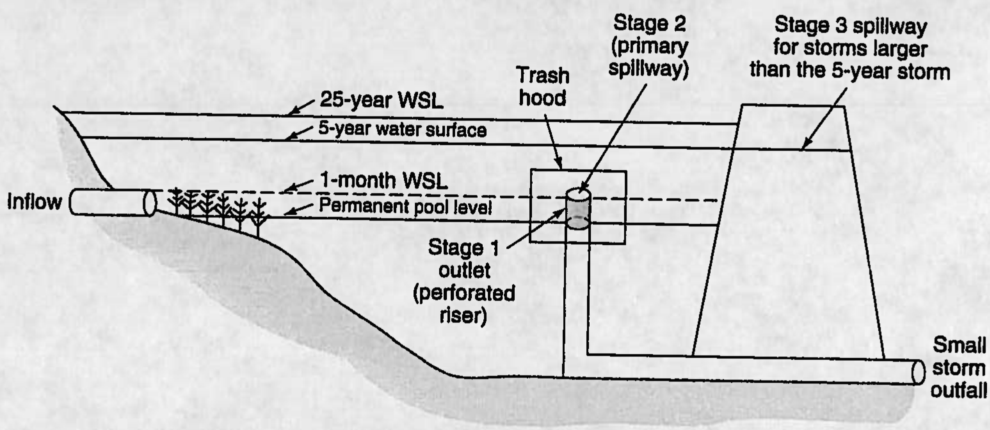


Figure 15.4.5 Conceptual design of a multipurpose pond (from Urbonas and Roesner (1993)).

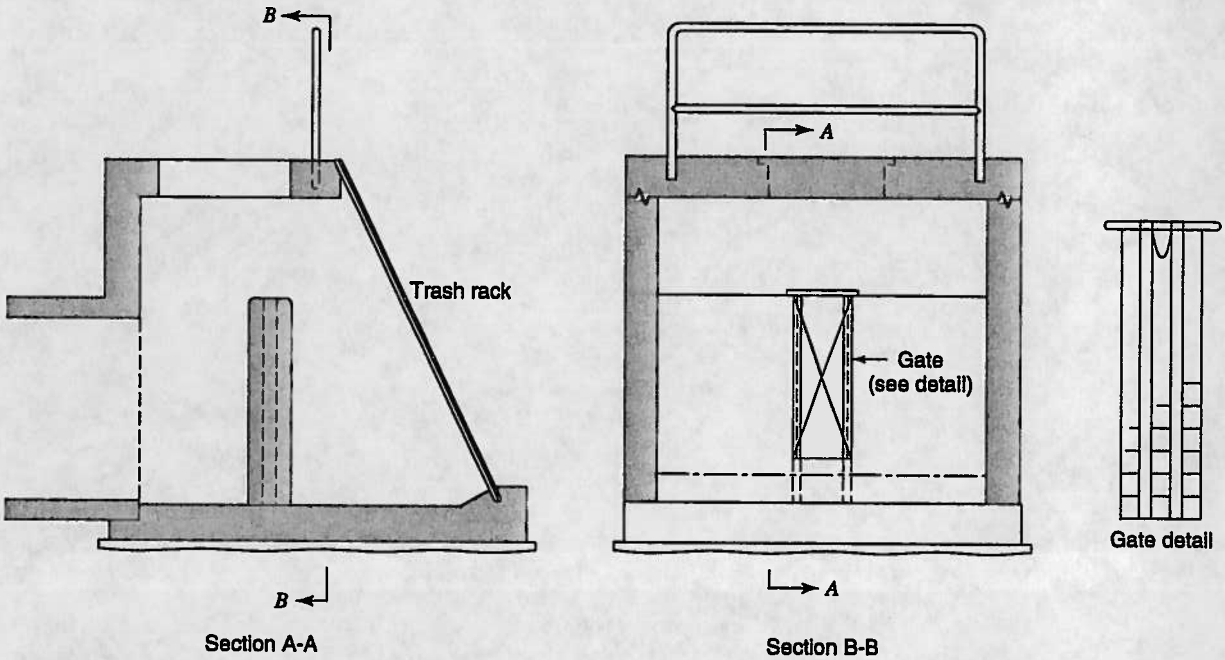


Figure 15.4.6 Outlet structure with a fixed gate control (from Stahre and Urbonas (1990)).

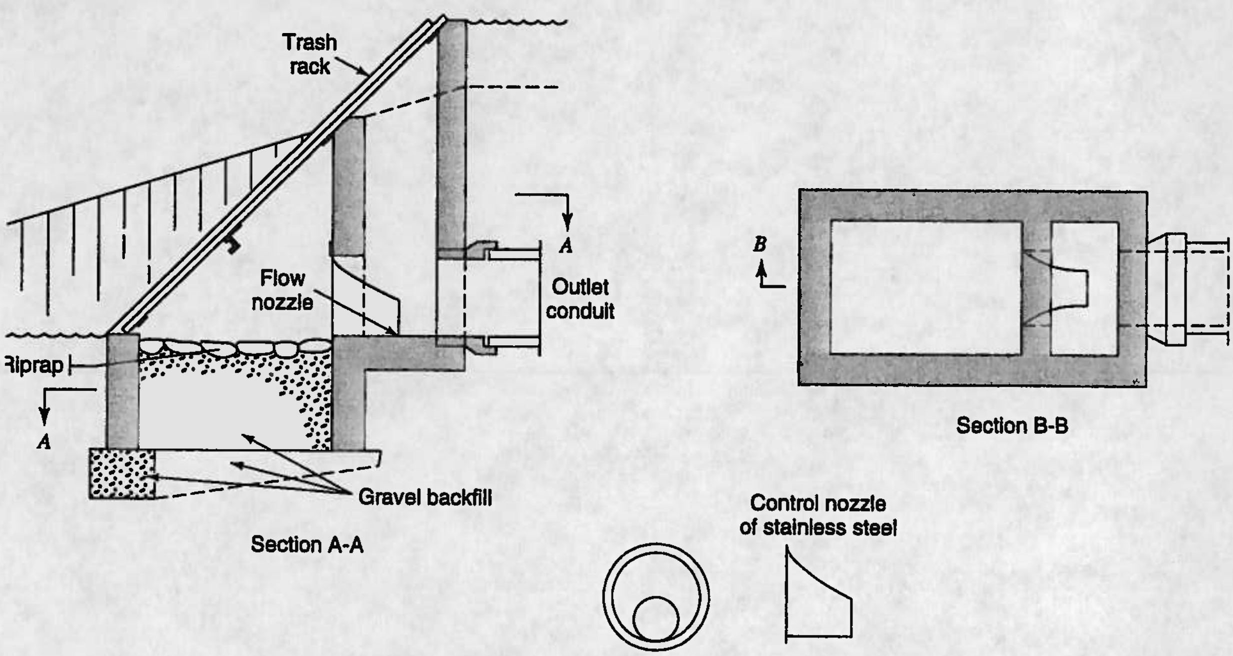


Figure 15.4.7 Outlet structure with a fixed orifice control (from Stahre and Urbonas (1990)).

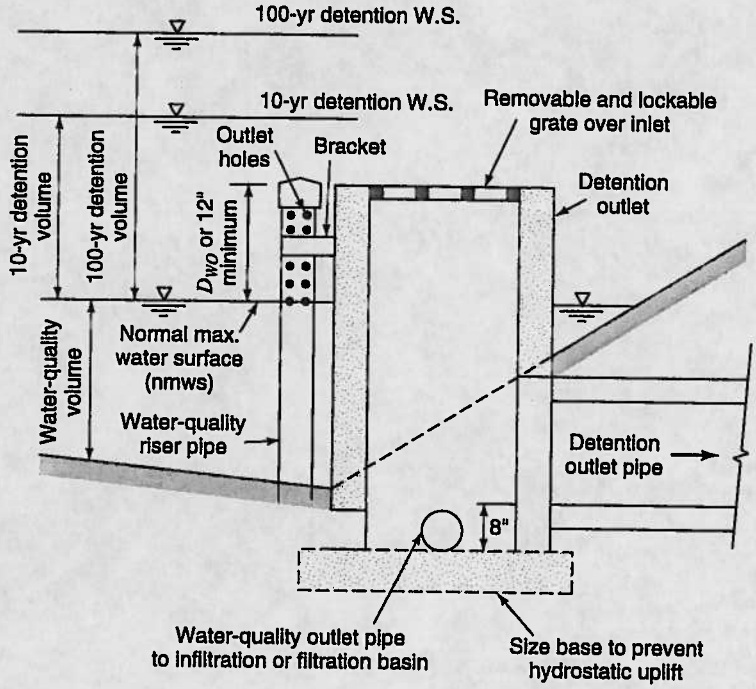


Figure 15.4.8 Outlet riser for a pond with a permanent pool (from Stahre and Urbonas (1990)).

15.4.3 Sizing Detention

Several simplified methods for sizing detention have been proposed in the literature (Abt an Grigg, 1978; Akan, 1989, 1990, 1993; Aron and Kibler, 1990; Donahue et al., 1981; Kessler an Diskin, 1991; Mays and Tung, 1992; McEnroe, 1992; and Wycoff and Singh, 1976). More sophis ticated procedures using optimization have also been proposed (Bennett and Mays, 1985; May and Bedient, 1982; Nix and Heaney, 1988; Taur et al., 1987).

The stormwater detained during and after a storm is a function of the runoff volume, the deten tion basin outlet(s), and the available storage volume of the detention basin. The objective of siz ing the pond is to determine the storage volume V_s, which mathematically is the time integral o the difference between the detention basin inflow and outflow hydrographs; i.e., the storage vol ume is

$$V_s = \int (Q_{in} - Q_{out})\, dt \tag{15.4.1}$$

where Q_{in} is the inflow rate and Q_{out} is the outflow rate. Figure 15.4.9 illustrates this integratioı showing the V_{max}.

Simple methods based upon regression equations, the rational method, or the modified rationa method can be used to estimate preliminary sizes. This is followed up by iteratively routing on or more hydrographs through the preliminary sized pond to refine the size and outlet structures The classic detention sizing procedure consists of the following steps (Urbonas and Roesner 1993):

1. Estimate the preliminary storage volume V_s using a simplified procedure (see sections 15.4.3.1 and 15.4.3.2).
2. Use site topography to prepare a preliminary layout of a detention basin that has the desired volume and outlet configuration.
3. Determine stage-storage-outflow characteristics of the trial pond size.

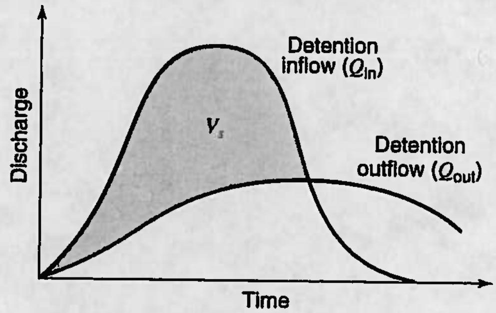


Figure 15.4.9 Detention storage volume V_s.

4. Perform routing of input hydrographs through the pond. Steps 3 and 4 can be accomplished using computer models.
5. If the trial pond (size and outlet configuration) does not satisfy desired goals and design criteria, resize the basin and or reconfigure the outlet(s) and repeat steps 3–5 until design goals and design criteria are satisfied (optimized).

The inflow hydrographs can be generated using any of a number of rainfall-runoff models (also see Akan, 1993; Chow et al., 1988; Kibler, 1982; and Urbonas and Roesner, 1993).

The American Association of State Highway Transportation Officials (AASHTO) (1991) recommended using triangular-shaped inflow and outflow hydrographs (see Figure 15.4.10) to determine preliminary estimates of storage volume V_s. The required storage volume is simply the cross-hatched area shown in Figure 15.4.10, which is computed using

$$V_s = 0.5t_b(Q_p - Q_A) \tag{15.4.2}$$

Any consistent units may be used in equation (15.4.2). The time to peak inflow hydrograph t_p is half of the total time base of this hydrograph.

Abt and Grigg (1978) considered a triangular inflow hydrograph and a trapezoidal outflow hydrograph to develop the following relationship to estimate the required storage volume V_s using consistent units:

$$\frac{V_s}{V_r} = \left(1 - \frac{Q_A}{Q_P}\right)^2 \tag{15.4.3}$$

where V_r is the runoff volume, Q_A is the allowable peak outflow rate, and Q_P is the peak inflow rate. This procedure assumes that the rising limbs of the inflow and outflow hydrographs coincide up to the allowable peak outflow rate (see Figure 15.4.11).

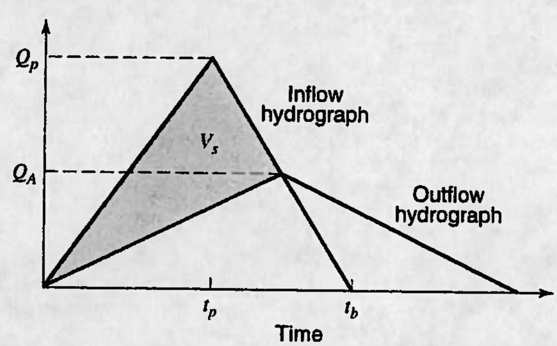


Figure 15.4.10 Inflow and outflow hydrographs for AASHTO (1991) procedure.

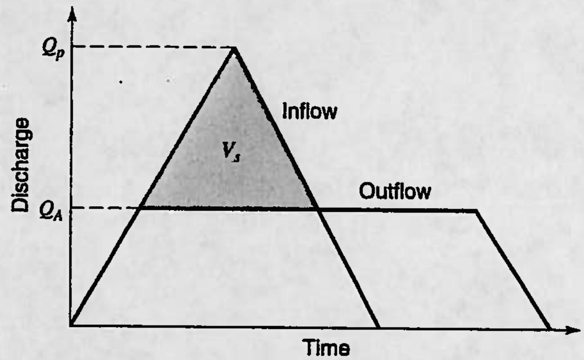


Figure 15.4.11 Inflow and outflow hydrographs for procedure by Abt and Grigg (1978).

Aron and Kibler (1990) developed an approximate method considering trapezoidal inflc hydrographs. They assumed (1) that the peak outflow hydrograph falls on the recession limb of tl inflow hydrograph and (2) that the rising limb of the outflow hydrograph can be approximated l a straight line (see Figure 15.4.12). The volume of storage is computed using

$$V_s = Q_P t_D - Q_A\left(\frac{t_D + t_C}{2}\right) \qquad (15.4.4$$

where t_D is the storm duration and t_C is the time of concentration of the watershed. The desig storm duration is the one that maximizes the detention storage volume V_s for a given return perio This method uses a trial-and-error procedure to find the storm duration using the local intensit duration-frequency (IDF) relationships. The rational formula ($Q_P = CiA$) is used to compute tl peak flow rate Q_P.

AASHTO (1991) recommended an alternate estimate of storage volume using the regressi equation developed by Wycoff and Singh (1986) as

$$\frac{V_s}{V_r} = \frac{1.29\left(1 - \dfrac{Q_A}{Q_P}\right)^{0.153}}{\left(t_b/t_p\right)^{0.411}} \qquad (15.4.$$

where V_s is the volume of storage in inches, V_r is the volume of runoff in inches, Q_A is the pe outflow in cfs, Q_P is the peak inflow in cfs, t_b is the time base of the inflow hydrograph in hou (determined as the time from the beginning of rise to a point on the recession limb where the flc is 5 percent of the peak), and t_p is the time to peak of the inflow hydrograph in hour. A prelin nary estimate of the potential peak flow reduction for a selected storage volume is

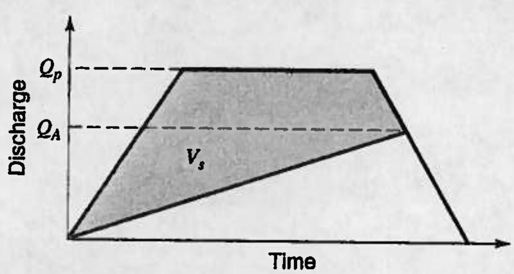


Figure 15.4.12 Inflow and outflow hydrographs for procedure by Aron and Kibler (1990).

$$\frac{Q_A}{Q_P} = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546} \tag{15.4.6}$$

EXAMPLE 15.4.1

The peak runoff rate for a 10-year storm of 133 ft³/s is to be limited to a peak of 40 ft³/s through the use of a detention basin. Determine a preliminary estimate of storage using the AASHTO (1991) method, equation (15.4.5); t_p = 30 minutes.

OLUTION

First, by definition t_b = time base of inflow hydrograph in hours determined as the time from the beginning to a point on the recession limb where the flow is 5 percent of the peak. So

$$t_b = 60 - 30\left[\frac{0.05(133)}{133}\right] = 58.5 \text{ min} = 0.98 \text{ hr}$$

Using equation (15.4.5) yields

$$\frac{V_s}{V_r} = 1.29\frac{\left(1-\frac{40}{133}\right)^{0.153}}{\left(\frac{0.98}{0.5}\right)^{0.411}} = \frac{1.29(0.95)}{1.32} = 0.93$$

$$V_r = \frac{1}{2}(60\,\text{min})\left(133\frac{\text{ft}^3}{\text{s}}\right)\times\frac{60\text{ sec}}{\text{min}} - \frac{1}{2}(60\text{ min})\left(40\frac{\text{ft}^3}{\text{s}}\right)\left(\frac{60\text{ sec}}{\text{min}}\right)$$

$$= 239{,}400 - 72{,}000 = 167{,}400\text{ ft}^3 = 3.84\text{ ac-ft}$$

The volume of storage is

$$V_s = V_r(0.93) = 3.84\,(0.93) = 3.57\text{ ac-ft}$$

EXAMPLE 15.4.2

Solve example 15.4.1 using the Abt and Grigg (1978) method.

OLUTION

Using equation (15.4.3) yields

$$\frac{V_s}{V_r} = \left(1-\frac{Q_A}{Q_P}\right)^2 = (1-0.3)^2$$

$$V_s = 0.49V_r = 0.49(3.84) = 1.88\text{ ac-ft}$$

The procedure adopted by the Federal Aviation Agency (FAA) (1966) is a simple mass-balance technique that is intensity-duration-frequency (IDF) based. The procedure assumes that rainfall volume accumulates with time and is a time integral of the desired IDF curve. The rainfall volume-duration curve is transformed into a runoff volume-duration curve using

$$V_{in} = K_u CiAt_D \tag{15.4.7}$$

where V_{in} is the cumulative runoff volume, ft³(m³), K_u is 1.0 (for U.S. customary units) or 0.28 (for SI units), C is the runoff coefficient, i is the storm intensity from the IDF curve at time t_D in in/h (mm/h), A is the tributary area in acres (km²), and t_D is the storm duration in seconds. The cumulative volume leaving the detention basin, V_{out}, is estimated by

$$V_{out} = kQ_{out}t_D \tag{15.4.8}$$

where V_{out} is in ft³(m³), Q_{out} is the maximum outflow rate, ft³/s (m³/s), and k is an outflow adjustment coefficient from Figure 15.4.13 ($Q_{pin} = CiA$). The FAA procedure assumes a constant outflow rate Q_{out}, which is the rate of discharge when the detention basin is full. Because discharge

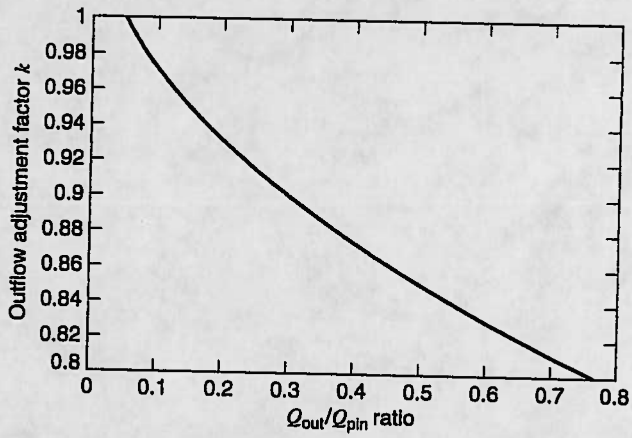


Figure 15.4.13 Outflow adjustment factor versus outflow rate/inflow peak ratio (from Urbonas and Roesner (1993)).

increases with depth of water, the outflow adjustment factor is used. The required detention vo ume is computed using

$$V_{req} = \max\,(V_{in} - V_{out}) \tag{15.4.}$$

which states that the required storage volume is the maximum difference between the cumulati inflow and the cumulative outflow volume.

15.4.3.1 Modified Rational Method

The *modified rational method* can be used to determine the preliminary design, which is the dete tion pond volume requirement for contributing drainage areas of 30 acres or less (Chow et a 1988). For larger contributing areas, a more detailed rainfall-runoff analysis with a detention bas flow routing procedure should be used. The modified rational method is an extension of t rational method to develop hydrographs for storage design, rather than only peak discharges f storm sewer design. The shape of hydrographs produced by the modified rational method is eith triangular or trapezoidal, constructed by setting the duration of the rising and recession limbs equ to the time of concentration t_c and computing the peak discharge assuming various duratio Figure 15.4.14a illustrates hydrographs drawn using the modified rational method.

An allowable discharge Q_A from a proposed detention basin can be the requirement that t peak discharge from the pond be equal to the peak of the runoff hydrograph for predeveloped co ditions. The required detention storage V_s for each rainfall duration can be approximated as t cumulative volume of inflow minus the outflow, as shown in Figure 15.4.14b.

The assumptions of the modified rational method include:

1. The same assumptions as the rational method
2. The period of rainfall intensity averaging is equal to the duration of the storm
3. Because the outflow hydrograph is either triangular or trapezoidal, the effective contribu ing drainage area increases linearly with respect to time

An equation for the *critical storm duration*, that is, the storm duration that provides the large storage volume, can be determined for small watersheds based upon the modified rational metho Consider a rainfall intensity-duration equation of the general form

$$i = \frac{a}{(t_D + b)^c} \tag{15.4.1}$$

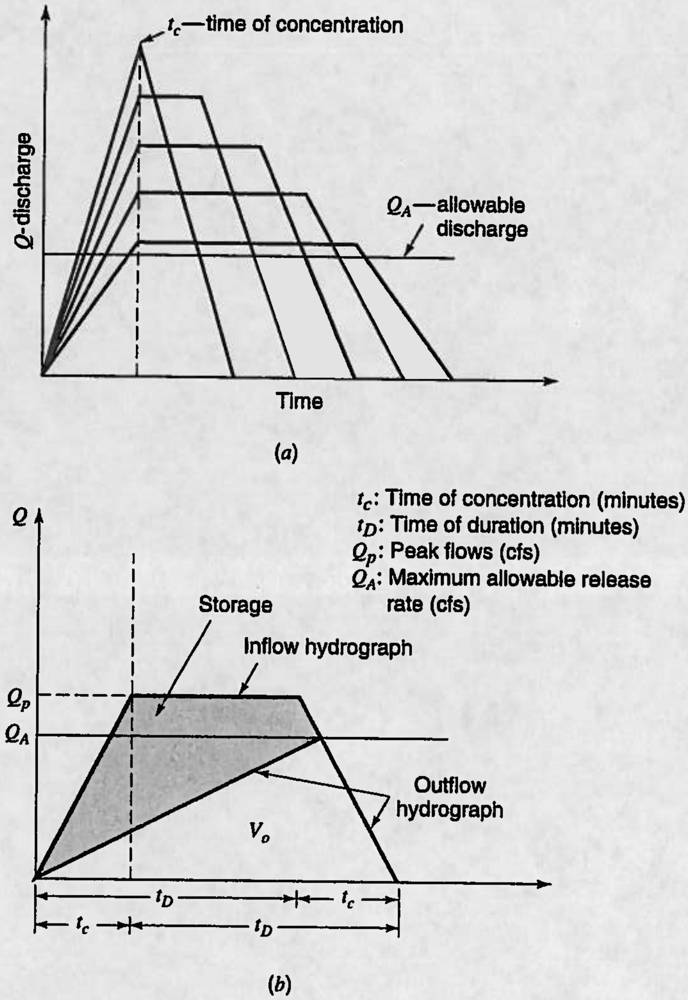


Figure 15.4.14 Modified rational method hydrographs. (*a*) Hydrographs for different durations; (*b*) Storage requirement.

where i is the average rainfall intensity (in/hr) for the specific duration and return period, t_D is storm duration in minutes, and a, b, and c are coefficients for a specific return period and location. Consider the trapezoidal-shaped inflow hydrograph and outflow hydrograph in Figure 15.4.15.

Using the rational formula, the peak discharge can be expressed in terms of the storm duration:

$$Q_p = C_p iA = C_p\left(\frac{a}{(t_D + b)^c}\right)A \tag{15.4.11}$$

The inflow hydrograph volume V_i in ft^3 is expressed as

$$V_i = 60\ (0.5)\ Q_p[(t_D - t_c) + (t_D + t_c)] \tag{15.4.12}$$

where t_c is the time of concentration for proposed conditions. The outflow hydrograph volume V_0 in ft^3 is expressed as

$$V_0 = 60(0.5)Q_A(t_D + t_c) \tag{15.4.13}$$

where Q_A is the allowable peak flow release in ft^3. The storage volume V_s in ft^3 is computed using the above expressions for V_i and V_0:

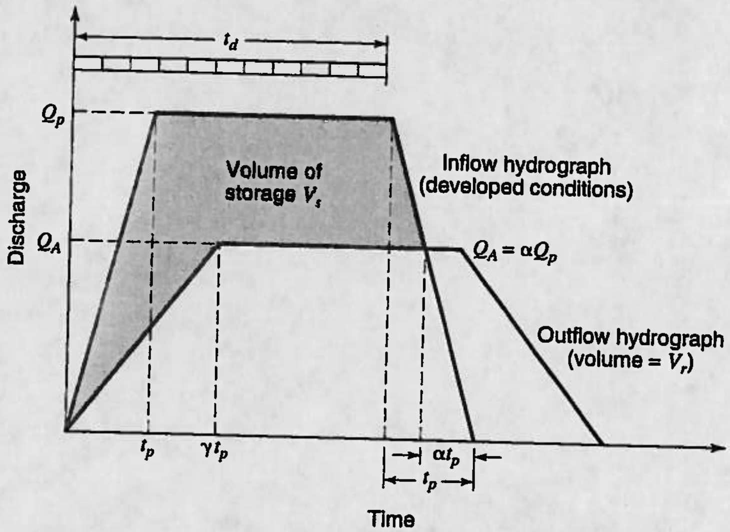


Figure 15.4.15 Inflow and outflow hydrographs for detention design. The outflow hydrograph is based on the inflow hydrograph for predeveloped conditions or on other more restrictive outflow criteria (from Donahue et al. (1981)).

$$V_s = V_i - V_0 = 60(0.5)Q_p[(t_D - t_c) + (t_D + t_c)] - 60\ (0.5)\ Q_A(t_D + t_c)$$

$$= 60\ Q_p t_D - 30Q_A(t_D + t_c) \qquad (15.4.1$$

The duration for the maximum detention is determined by differentiating equation (15.4.1 with respect to t_D and setting the derivative equal to zero:

$$\frac{dV_s}{dt_D} = 0 = 60t_D + \frac{dQ_p}{dt_D} = 0 = 60t_D + 60Q_p - 30Q_A$$

$$= 60t_D C_p A \frac{di}{dt_D} + 60C_p iA - 30Q_A \qquad (15.4.1$$

where

$$\frac{di}{dt_D} = \frac{d}{dt_D}\left[\frac{a}{(t_D + b)^c}\right] = \frac{-ac}{(t_D + b)^{c+1}} \qquad (15.4.1$$

so

$$\frac{dV_s}{dt_D} = 0 = 60C_p A(-ac)\frac{t_D}{(t_D + b)^{c+1}} + 60C_p\left(\frac{a}{(t_D + b)^c}\right)A - 30Q_A \qquad (15.4.1$$

Simplifying results in

$$\frac{a[t_D(1-c) + b]}{(t_D + b)^{c+1}} - \frac{Q_A}{2C_p A} = 0 \qquad (15.4.1$$

t_D in equation (15.4.18) can be solved by using Newton's iteration technique (Appendix A) whe the iterative equation is

$$t_{D_{i+1}} = t_{D_i} - \frac{F(t_{D_i})}{F'(t_{D_i})} \qquad (15.4.1$$

where

$$F(t_D) = \frac{a[t_D(1-c)+b]}{(t_D+b)^{c+1}} - \frac{Q_A}{2C_pA} \tag{15.4.20}$$

and

$$F'(t_{Di}) = \frac{d[F(t_{D_i})]}{dt_D} = -\frac{a[t_D(1-c)+b](c+1)}{(t_D+b)^{c+2}} + \frac{a(1-c)}{(t_D+b)^{c+1}} \tag{15.4.21}$$

EXAMPLE 15.4.3

Determine the critical duration t_D and maximum detention storage for a 31.39-acre fully developed watershed with a runoff coefficient of $C_p = 0.95$. The allowable discharge is the predevelopment discharge of $Q_A = 59.08$ cfs. The time of concentration for proposed conditions is 21.2 minutes. The applicable rainfall intensity duration relationship is

$$i = \frac{97.86}{(t_D+16.4)^{0.76}}$$

OLUTION

The critical storm duration t_D can be obtained by solving equation (15.4.18) using Newton's iteration as stated in equation (15.4.19). From equation (15.4.20) we find

$$\begin{aligned} F(t_D) &= \frac{a[t_D(1-c)+b]}{(t_D+b)^{c+1}} - \frac{Q_A}{2C_pA} \\ &= \frac{a[t_D(1-0.76)+16.4]}{(t_D+16.4)^{1.76}} - \frac{59.08}{2(0.95)(31.39)} \\ &= \frac{97.86[0.24t_D+16.4]}{(t_D+16.4)^{1.76}} - 0.99 = \frac{23.49t_D+1604.90}{(t_D+16.4)^{1.76}} - 0.99 \end{aligned}$$

and from equation (15.4.21) we find

$$\begin{aligned} F'(t_D) &= -\frac{a[t_D(1-c)+b](1+c)}{(t_D+b)^{c+2}} + \frac{a(1-c)}{(t_D+b)^{c+1}} \\ &= -\frac{97.86[t_D(1-0.76)+16.4](1+0.76)}{(t_D+16.4)^{2.76}} + \frac{1-0.76}{(t_D+16.4)^{1.76}} \\ &= -\frac{97.86(1.76)[0.24t_D+16.4]}{(t_D+16.4)^{2.76}} + \frac{97.86(0.24)}{(t_D+16.4)^{1.76}} \\ &= \frac{41.34t_D+2824.63}{(t_D+16.4)^{1.76}} + \frac{23.49}{(t_D+16.4)^{1.76}} \end{aligned}$$

With Newton's algorithm, the value of t_D converges to 92.0 minutes.

Next use equation (15.4.11):

$$Q_p = C_p\left[\frac{a}{(t_D+b)^c}\right]A = 0.95\left(\frac{97.86}{(92.0+16.4)^{0.76}}\right)(31.39) = 82.89 \text{ cfs}$$

Then use equation (15.4.14) to compute the detention storage:

$$\begin{aligned} V_s &= 60Q_pt_D - 30Q_A(t_D + t_c) = 60(82.89)(92.0) - 30(59.08)(92.0 + 21.2) \\ &= 256917 \text{ cfs} = 5.90 \text{ acre-ft} \end{aligned}$$

Figure 15.4.15 is a representation of inflow and outflow hydrographs for a detention basin design. In this figure, α is the ratio of the peak discharge before development Q_A (or peak discharge from the detention basin that is allowable), and the peak discharge after development, Q_p:

$$\alpha = \frac{Q_A}{Q_p} \qquad (15.4.2$$

The ratio of the times to peak in the two hydrographs is γ. V_r is the volume of runoff after deve opment. The volume of storage V_s needed in the basin is the accumulated volume of inflow mini outflow during the period when the inflow rate exceeds the outflow rate, shown shaded in tl figure.

Using the geometry of the trapezoidal hydrographs, the ratio of the volume of storage to the vc ume of runoff V_s/V_r can be determined (Donahue et al., 1981) as:

$$\frac{V_s}{V_r} = 1 - \alpha\left[1 + \frac{t_p}{t_D}\left(1 - \frac{\gamma + \alpha}{2}\right)\right] \qquad (15.4.2$$

where t_D is the duration of the precipitation and t_p is the time to peak of the inflow hydrograph.

Consider a rainfall intensity-duration relationship of the form

$$i = \frac{a}{t_D + b} \qquad (15.4.2$$

where i is rainfall intensity and a and b are coefficients. The volume of runoff after developme is equal to the volume under the inflow hydrograph:

$$V_r = Q_p t_D \qquad (15.4.2$$

The volume of storage is determined by substituting (15.4.25) into (15.4.23) and rearranging to g

$$V_s = Q_p t_D\left\{1 - \alpha\left[1 + \frac{t_p}{t_D}\left(1 - \frac{\gamma + \alpha}{2}\right)\right]\right\} \qquad (15.4.2$$

$$= t_D Q_p - Q_A t_D - Q_A t_p + \frac{\gamma Q_A t_p}{2} + \frac{Q_A^2 t_p}{2}\frac{1}{Q_p} \qquad (15.4.2$$

where α has been replaced by Q_A/Q_p.

The duration that results in the maximum detention is determined by substituting $Q_p = CiA$ $CAa/(t_D + \mathrm{b})$, then differentiating (15.4.27) with respect to t_D and setting the derivative equal zero:

$$\frac{dV_s}{dt_D} = 0 = t_D\frac{dQ_p}{dt_D} + Q_p - Q_A + \frac{Q_A^2 t_p}{2}\left[\frac{d(1/Q_p)}{dt_D}\right] = \frac{bCAa}{(t_D + b)^2} - Q_A + \frac{Q_A^2 t_p}{2CAa}$$

where it is assumed that Q_A, t_p, and γ are constants. Solving for t_D,

$$t_D = \left(\frac{bCAa}{Q_A - \dfrac{Q_A^2 t_p}{2CAa}}\right)^{1/2} - b \qquad (15.4.2$$

The time to peak t_p is set equal to the time of concentration.

EXAMPLE 15.4.4

Determine the maximum storage for a detention pond on a 25-acre watershed for which the develop runoff coefficient is 0.8; the time of concentration before development is 25 min and after developme is 15 min. The allowable discharge is 25 cfs, and $a = 96.6$ and $b = 13.9$.

SOLUTION

The maximum storage is determined using equation (15.4.27) with allowable discharge $Q_A = 25$ c $t_p = 15$ min (developed condition), $C = 0.8$ (developed condition), $a = 96.6$, and $b = 13.9$. First dete mine the critical duration t_D using equation (15.4.28).

$$t_D = \left[\frac{(13.9)(0.8)(25)(96.6)}{25 - \left((25)^2 (15)/2(0.8)(25)(96.6)\right)} \right]^{1/2} - 13.9 = 20.59 \text{ min}$$

The peak discharge is $Q_P = C\, i\, A$. Then using equation (15.4.24) and $i = a/(t_D + b)$, with $t_D = 20.59$ min, we get

$$Q_P = CA\left(\frac{a}{t_D + b}\right) = 0.8(25)\left(\frac{96.6}{20.59 + 13.9}\right) = 56.01 \text{ cfs}$$

The maximum storage is then

$$V_s = t_D Q_P - Q_A t_D - Q_A t_P + \frac{\gamma Q_A t_P}{2} + \frac{Q_A^2 t_P}{2} \frac{1}{Q_P}$$

$$V_s = (20.59)(56.01) - (25)(20.59) - (25)(15) + \frac{(25/15)(25)(15)}{2} + \frac{(25)^2(15)}{2} \frac{1}{56.01}$$

$$= 659.81 \text{ cfs(min} \times 60 \text{ s/min} = 39{,}588 \text{ ft}^3$$

15.4.3.2 Hydrograph Design Methods

A simple design procedure for sizing detention basins is now outlined that is useful in practice.

1. Determine the watershed characteristics and location of the detention basin.
2. Determine the design inflow hydrograph to the detention basin using a rainfall-runoff model.
3. Determine the detention storage-discharge relationship.
   a. Determine the storage-elevation relationship.
   b. Determine the discharge-elevation relationship for the discharge structure (culvert, spillway, etc.).
   c. Using the above relationships, develop the storage-discharge relationship.
4. Perform the computations described in Chapter 9 or section 15.4.4 for routing the inflow hydrograph through the detention basin using hydrologic reservoir routing.
5. Once the routing computations are completed, the reduced peak can be checked to see that the reduction is adequate and also to check the delay of the peak outflow.
6. Steps 3(b) through 5 of this procedure can be repeated for various discharge structures.

5.4.4 Detention Basin Routing

The hydrograph design method presented in section 15.4.3 requires routing of a design inflow hydrograph through the detention/retention basin. The level pool routing can be accomplished using the procedure in Chapter 9. An alternative procedure is presented in this section that does not require development of the storage outflow function. This method, presented in Chow et al. (1988), is based upon the Runge–Kutta method (Carnahan et al., 1969). A third-order scheme which breaks the time interval into three time increments and computes the water surface elevation and reservoir discharge for each increment.

Continuity is expressed as

$$\frac{dS}{dt} = I(t) - Q(H) \tag{15.4.29}$$

where S is the storage volume in the detention pond, $I(t)$ is the inflow into the pond as a functic of time, t, and $Q(H)$ is the discharge from the pond as a function of the head or elevation H in tl pond. The change in storage dS due to a change in elevation dH is

$$dS = A(H)dH \tag{15.4.3}$$

where $A(H)$ is the water surface area at elevation H. Substitution of this expression for dS in equation (15.4.29) and rearranging gives

$$\frac{dH}{dt} = \frac{I(t) - Q(H)}{A(H)} \tag{15.4.3}$$

which is an implicit differential equation.

Equation (15.4.31) is solved at each time step using three approximations of ΔH, ΔH_1, ΔH_2, ar ΔH_3 at times t_j, $t_j + \Delta t/3$, and $t_j + 2\Delta t/3$, respectively. These approximations of ΔH are

$$\Delta H_1 = \left[\frac{I(t_j) - Q(H_j)}{A(H_j)}\right]\Delta t \tag{15.4.3}$$

$$\Delta H_2 = \left[\frac{I\left(t_j + \frac{\Delta t}{3}\right) - Q\left(H_j + \frac{\Delta H_1}{3}\right)}{A\left(H_j + \frac{\Delta H_1}{3}\right)}\right]\Delta t \tag{15.4.3}$$

$$\Delta H_3 = \left[\frac{I\left(t_j + \frac{2\Delta t}{3}\right) - Q\left(H_j + \frac{2\Delta H_2}{3}\right)}{A\left(H_j + \frac{2\Delta H_2}{3}\right)}\right]\Delta t \tag{15.4.3}$$

The value of H_{j+1} at time $t_{j+1} = t_j + \Delta t$ is

$$H_{j+1} = H_j + \Delta H \tag{15.4.3}$$

where

$$\Delta H = \frac{\Delta H_1}{4} + \frac{3\Delta H_3}{4} \tag{15.4.3}$$

This procedure requires the relationship of $Q(H)$ and $A(H)$ and the design detention pond inflo hydrograph.

EXAMPLE 15.4.5

Consider a 2-acre detention basin with vertical walls. The triangular inflow hydrograph increas linearly from zero to a peak of 540 cfs at 60 min and then decreases linearly to a zero discharge 180 min. Route the inflow hydrograph through the detention basin using the head-discharge curv Assuming the basin is initially empty, use the third-order Runge-Kutta method, with a 20-min tin interval to determine the maximum depth.

Elevation (H, ft)	0.0	0.5	1.0	1.5	2.0	2.5	3.0
Discharge (Q, ft^3/s)	0.0	3.0	8.0	17	30	43	60
Elevation (H, ft)	3.5	4.0	4.5	5.0	5.5	6.0	6.5
Discharge (Q, ft^3/s)	78	97	117	137	156	173	190
Elevation (H, ft)	7.0	8.0	9.0	10	11	12	
Discharge (Q, ft^3/s)	205	231	253	275	323	340	

OLUTION

The function $A(H)$ relating the water surface area to the reservoir elevation is simply $A(H) = 2(43{,}560)$ ft^2 = 87,120 ft^2. For all values of H, the reservoir has a base area of 2 acres and vertical sides. The routing procedure begins with determination of $I(0)$, $I(0 + 20/3)$, and $I[0 + (2/3) \times 20]$, which are determined by linear interpolation of values between 0 and 540/3 = 180 cfs, so they are respectively 0, 60, and 120 cfs.

Next ΔH_1 is computed using equation (15.4.32) with Δt = 20 min = 1200 s, A = 87,120 ft^2, and $I(0)$ = 0 cfs. The reservoir is initially empty, so $H_1 = 0$ and $Q(H_1) = 0$, and thus $\Delta H_1 = [(0 - 0)/87120] \times 1200 = 0$. For the next $I(0+20/3)$ = 60 ft^2/s, $\Delta H_2 = [(60 - 0)/87120] \times 1200$ = 0.826 ft, and $\Delta H_3 = [(120 - 3.507)/87120)](1200)$ = 1.605 ft, so

$$H_2 = H_1 + \frac{\Delta H_1}{4} + \frac{3}{4}\Delta H_3 = 0 + \frac{0}{4} + \frac{3}{4}(1.605) = 1.204 \text{ ft}$$

and by linear interpolation $Q(1.204)$ = 11.672 cfs.

In the next iteration, $\Delta H_1 = [(180 - 11.66)/87{,}120](1200) = 2.319$

$$Q\left(1.204 + \frac{2.319}{3}\right) = 29.402 \text{ cfs}$$

$\Delta H_2 = [(240 - 29.402)/87120]1200 = 2.901$ etc.

The routing computations are summarized in Table 15.4.1. The maximum depth in the pond is 12 ft.

Table 15.4.1 Routing Computation for Example 15.4.8

Time (min)	Inflow (cfs)	ΔH_1 (ft)	ΔH_2 (ft)	ΔH_3 (ft)	ΔH (ft)	Depth (H) (ft)	Outflow (cfs)
0	0	—	—	—	—	0	0
20	180	0	0.826	1.605	1.204	1.204	11.66
40	360	2.319	2.901	3.138	2.933	4.137	102.48
60	540	3.547	3.731	4.171	4.015	8.152	234.34
80	450	4.210	3.372	2.559	2.972	11.124	325.11
100	360	1.720	1.173	0.711	0.963	12.087	341.48
120	270	0.255	–0.178	–0.543	–0.344	11.743	335.63
140	180	–0.904	–1.247	–1.498	–1.350	10.393	293.91
160	90	–1.568	–1.682	–1.914	–1.828	8.565	243.43
180	0	–2.113	–2.306	–2.420	–2.343	6.222	180.55

5.4.5 Subsurface Disposal of Stormwater

Subsurface practices can be categorized as follows:
Infiltration practices:

- Swales and filter strips
- Porous pavement
- Infiltration trenches (Figure 15.4.16)
- Infiltration basins (Figure 15.4.17)
- Recharge wells (Figures 15.4.18–15.4.20)
- Underground storage (Figures 15.4.21–15.4.22)

15.4.5.1 Infiltration Practices

First the various types of infiltration practices are discussed. *Swales* are shallow vegetated trenches with nearly flat longitudinal slopes and mild side slopes. *Filter strips* are strips of land that stormwater must flow across before entering a conveyance system. These practices allow some of the runoff to infiltrate into the soil and filter the flow, removing some of the suspended solids ar other pollutants attached to the solids. They also have the effect of reducing the directly connecte impervious area and reducing the runoff velocity. They can be used for stormwater runoff fro streets, parking lots, and roofs.

Wanielista et al. (1988) used mass balance of input and output water in swale systems develop the following estimate of the length of a swale necessary to infiltrate all the input and rai fall excess from a specific storm event for a trapezoidal cross-sectional shape:

$$L = \frac{K\bar{Q}^{5/8}S^{3/16}}{n^{3/8}f} \qquad (15.4.3$$

where L is the length of the swale in ft (m), $\bar{Q}$ is the average runoff flow rate, ft^3/s (m^3/s), S is th longitudinal slope, ft/ft (m/m), n is the Manning roughness coefficient for overland flow (Tabl 15.4.2 and 15.4.3), f is the infiltration rate, in/h (cm/h), and K is a constant that is a function of th side slope parameter Z (1 vertical/Z horizontal), as listed in Table 15.4.4.

Swales should be as flat as possible to maximize infiltration and to minimize resuspension solids caused by high-flow velocities. Table 15.4.2 lists maximum or permissible velocities reduce erosion or resuspension. The swale volume V_{swale}, for situations in which the available lan is not long enough to infiltrate all the runoff, can be estimated using

$$V_{\text{swale}} = V_r - V_f \qquad (15.4.3$$

where V_{swale} is the volume of the swale (m^3), V_r is the volume of runoff (m^3), and V_f is the volum of infiltration (m^3). V_f can be derived using

$$V_f = \bar{Q}t_r \qquad (15.4.3$$

where $\bar{Q}$ is the average infiltration flow rate in ft^3/s (m^3/s) (see equation (15.4.38)) and t_r is th runoff hydrograph time, seconds. With $\bar{Q} = [(Ln^{3/8}f)/(KS^{3/16})]^{8/5}$ from equation (15.4.37), the vo ume of swale is

$$V_{\text{swale}} = V_r - \left[\frac{Ln^{3/8}f}{KS^{3/10}}\right]^{8/5} t_r \qquad (15.4.4$$

Table 15.4.2 Maximum Permissible Design Velocities to Prevent Erosion and Manning's n for Swales

Cover	Manning's n for vR:[a] 0.1	1.0	10	Slope Range (%)	Maximum Permissible Velocity (ft/s)
Tufcote, Midland, and coastal Bermuda grass	0.25	0.150	0.045	0.0–5.0	6.0
				5.1–10.0	5.0
				Over 10.0	4.0
Reed canary grass	0.40	0.250	0.070	0.0–5.0	5.0
Kentucky 31 tall fescue	0.40	0.250	0.070	5.1–10.0	4.0
Kentucky bluegrass (mowed)	0.10	0.055	0.030	Over 10.0	3.0
Red fescue and Argentine Bahia	0.10	0.055	0.030	0.0–5.0	2.5
Annuals[b] and ryegrass	0.10	0.050	0.030	0.0–5.0	2.5

[a]Product of velocity and hydraulic radius (ft^2/s).
[b]Annuals—use only as temporary protection until permanent vegetation is established.

Source: As presented in Wanielista and Yousef (1992).

Table 15.4.3 Overland Flow Manning's n Values for Shallow Flow ($vR < 1.0$)[a]

	Recommended Value	Range of Values
Concrete	0.011	0.01–0.013
Asphalt	0.012	0.01–0.015
Bare sand	0.010	0.010–0.016
Graveled surface	0.012	0.012–0.030
Bare clay-loam (eroded)	0.012	0.012–0.033
Fallow (no residue)	0.05	0.006–0.16
Plow	0.06	0.02–0.10
Range (natural)	0.13	0.01–0.32
Range (clipped)	0.08	0.02–0.24
Grass (bluegrass sod)	0.45	0.39–0.63
Short grass prairie	0.15	0.10–0.20
Dense grass	0.24	0.17–0.30
Bermuda grass	0.41	0.30–0.48

[a]These values were determined specifically for overland flow conditions and are not appropriate for conventional open-channel flow calculations.

Source: As presented in Wanielista and Yousef (1992).

Table 15.4.4 Swale Length Formula Constant

Z (Side Slope (1 vertical/Z horizontal)	K (SI Units) (i = cm/h, Q = m^3/s)	K (U.S. Units) (i = in/h, Q = ft^3/s)
1	98,100	13,650
2	85,400	11,900
3	71,200	9,900
4	61,200	8,500
5	54,000	7,500
6	48,500	6,750
7	44,300	6,150
8	40,850	5,680
9	38,000	5,255
10	35,760	4,955

Source: Wanielista and Yousef (1992).

EXAMPLE 15.4.6

Determine the length of a swale needed to infiltrate an average runoff flow rate of 0.003 m^3/s. The trapezoidal-shaped swale has a slope of 0.02, a Manning $n = 0.05$, an infiltration rate of 10 cm/h, and side slope of 1 vertical to $Z = 5$ horizontal.

OLUTION

Equation (15.4.37) is used to determine the required length of the swale with $K = 54{,}000$ from Table 15.4.4 for $Z = 5$:

$$L = \frac{K\overline{Q}^{5/8}S^{3/16}}{n^{3/8}f} = \frac{54{,}000(0.003)^{5/8}(0.02)^{3/16}}{(0.05)^{3/8}(10)} = 211 \text{ m}$$

EXAMPLE 15.4.7

For the situation in example 15.4.9, only 100 m of swale was needed. How much storage volume required for a runoff time of 120 min?

SOLUTION

Equation (15.4.40) is solved for V_{swale} with the volume of runoff $V_r = (0.003)(60)(120) = 21.6\ \text{m}^3$:

$$V_{\text{swale}} = V_r - \left[\frac{Ln^{3/8} f}{KS^{3/10}}\right]^{8/5} t_r = 21.6 - \left[\frac{100(0.05)^{3/8}(10)}{54{,}000(0.02)^{3/16}}\right]^{8/5} (60)(120)$$

$$= 21.6\ \text{m}^3 - 6.5\ \text{m}^3$$

$V_{\text{swale}} = 15.1\ \text{m}^3$ of storage required

Porous pavement and *modular pavement* (modular porous block pavement) can be used in parl ing areas to help reduce the amount of land needed for runoff quality control.

Percolation (or *infiltration*) *trenches* include both open surface type and underground (covere trenches. Figure 15.4.16 illustrates infiltration trenches for perforated storm sewers and parking l drainage. The perforated pipe allows distribution of stormwater along the entire length of tl trench.

Perforated pipes allow the collection of sediment before it enters the aggregate backfil Trenches are particularly suited for rights-of-way, parking lots, easements, and other areas wi limited space. Their advantages are that they can be placed in narrow bands and in complex aligi ments. Prevention of excessive silt from entering the aggregate backfill and thus clogging the sy tem is a major concern in design and construction. Sediment traps, filtration manholes, deep catcl basins, synthetic fibercloths, and the installation of filter bags in catch basins has proven effecti (American Iron and Steel Institute, 1995).

Infiltration basins are retention facilities in which captured runoff is infiltrated into the groun They are essentially depressions of varying size, either natural or excavated, into which storn water is conveyed and allowed to infiltrate. Figure 15.4.17 illustrates an infiltration basin th serves the dual function of infiltration and storage. Infiltration basins are typically used in parl and urban open spaces, in highway rights-of-way, and in open spaces in freeway interchang loops. Infiltration basins are susceptible to clogging and sedimentation and can require large lan areas. Standing water in these basins can create problems of security and insect breeding.

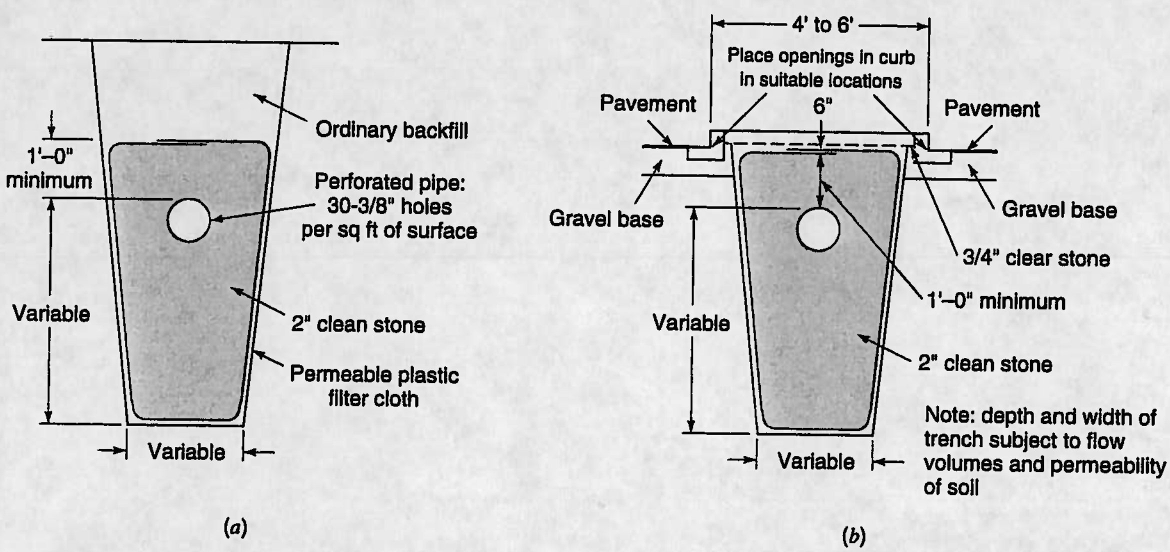


Figure 15.4.16 (*a*) Typical trench for perforated storm sewer; (*b*) Typical trench for parking lot drainage (from American Iron and Steel Institute (1995)).

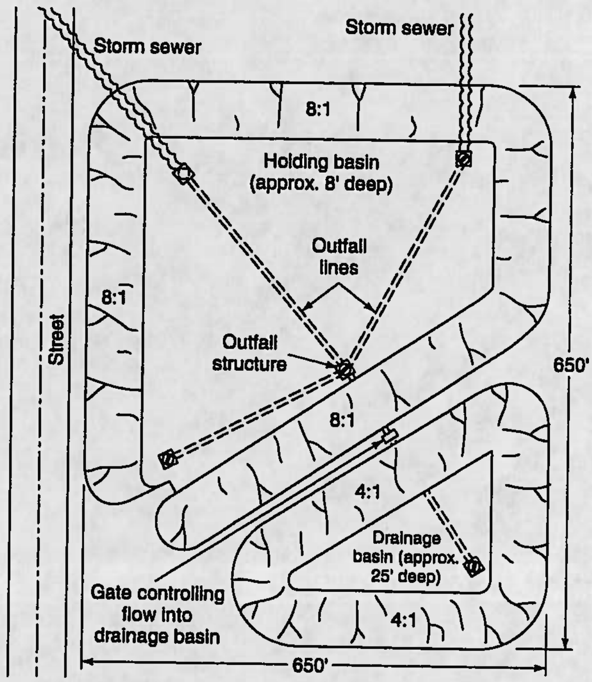


Figure 15.4.17 Infiltration basin (from American Iron and Steel Institute (1995)).

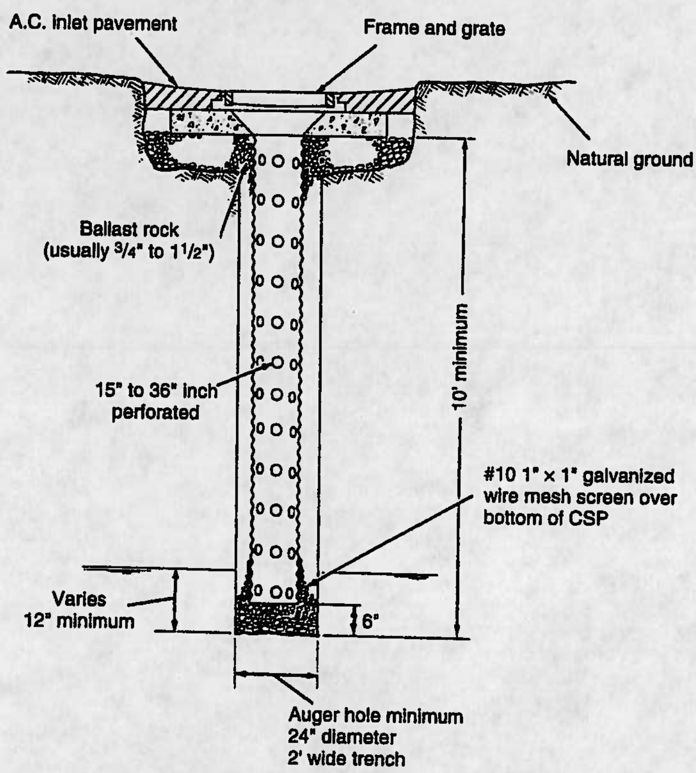


Figure 15.4.18 Recharge well (from American Iron and Steel Institute (1995)).

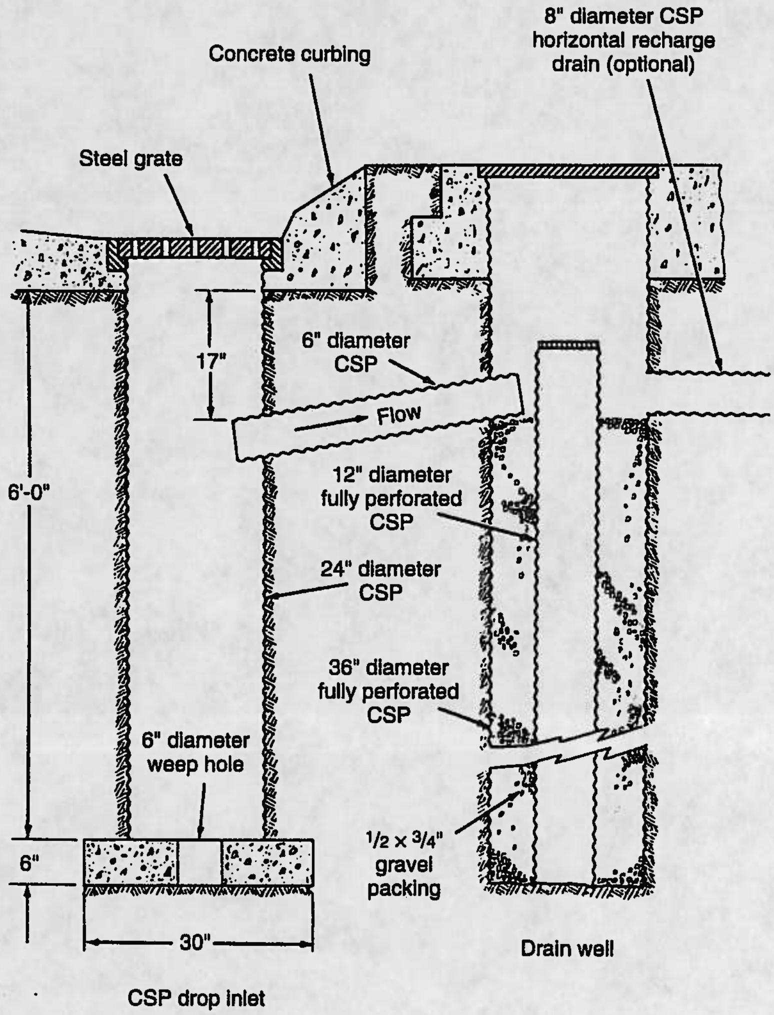


Figure 15.4.19 Typical design for combination catch basin for sand and sediment and recharge well. Catch basin would be periodically cleaned, and recharge well jetted through lower pipe to flush silt and restore permeability (from American Iron and Steel Institute (1995)).

15.4.5.2 Recharge Wells

Recharge wells can be used to dispose of stormwater directly into the subsurface. Figure 15.4.1: illustrates a recharge well. Recharge wells can be used to remove standing water in areas that ar difficult to drain. They can also be used in conjunction with infiltration basins to penetrate imper meable strata. Another use is as a bottomless catchbasin in conventional minor system desigr Typically, recharge wells are used for small areas and can be combined with catchbasins as illus trated in Figure 15.4.19. Figure 15.4.20 illustrates the use of a filter manhole in conjunction witl a recharge well, in order to prevent excess silt entering the recharge well and causing clogging.

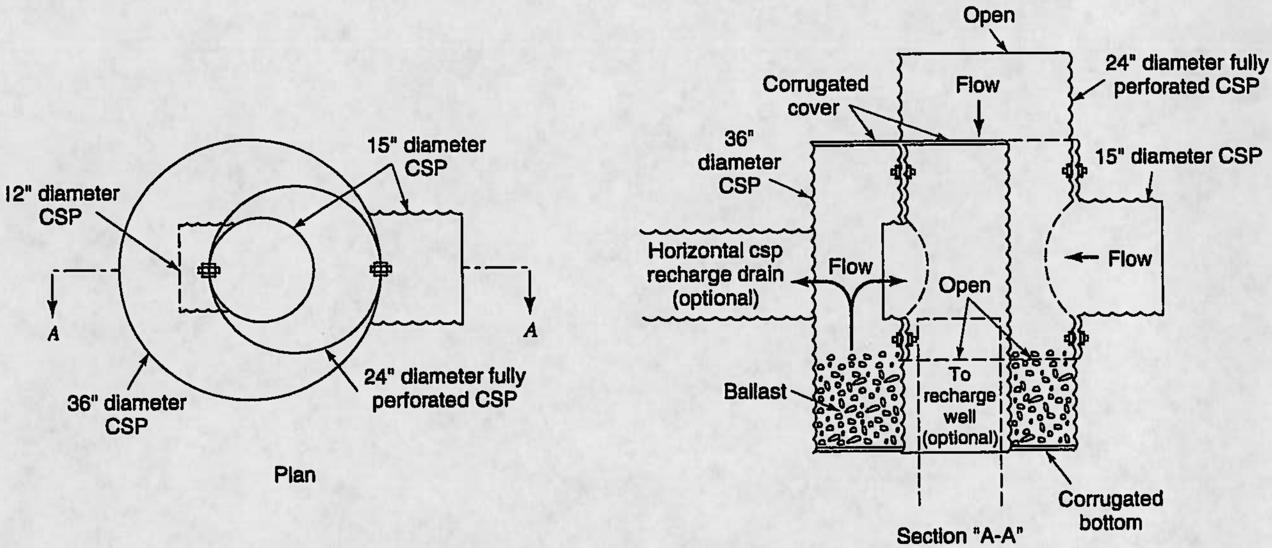


Figure 15.4.20 Typical CSP "Filter Manhole" (from American Iron and Steel Institute (1995)).

15.4.5.3 Underground Storage

Underground storage can be effective where surface ponds are not permitted or feasible. These storage tanks can be either *in-line*, in which the storage is incorporated directly into the sewer system, or *off-line*, in which stormwater is collected before it enters the sewer system and then discharged to either the sewer system or an open water course at a controlled rate. When the capacity of an in-line system is exceeded, surcharging in the sewer can occur. Figure 15.4.21 illustrates an off-line underground stormwater detention tank with an inlet control system.

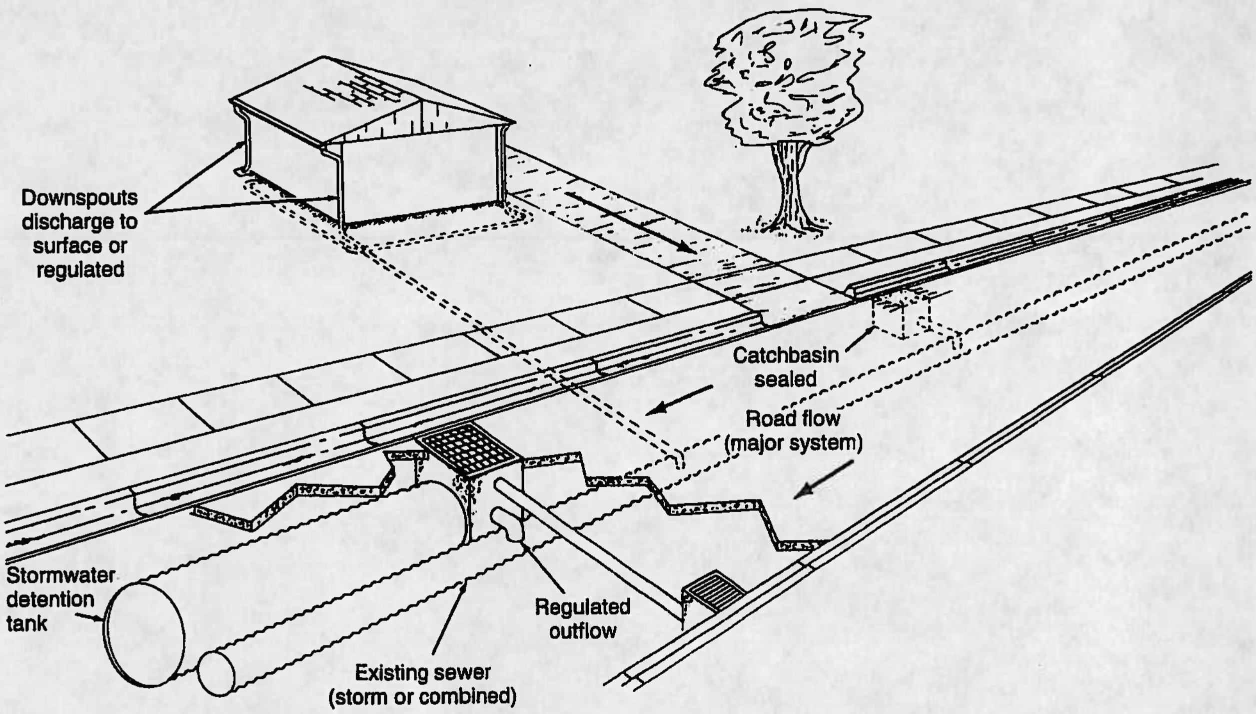


Figure 15.4.21 Inlet control system (from American Iron and Steel Institute (1995)).

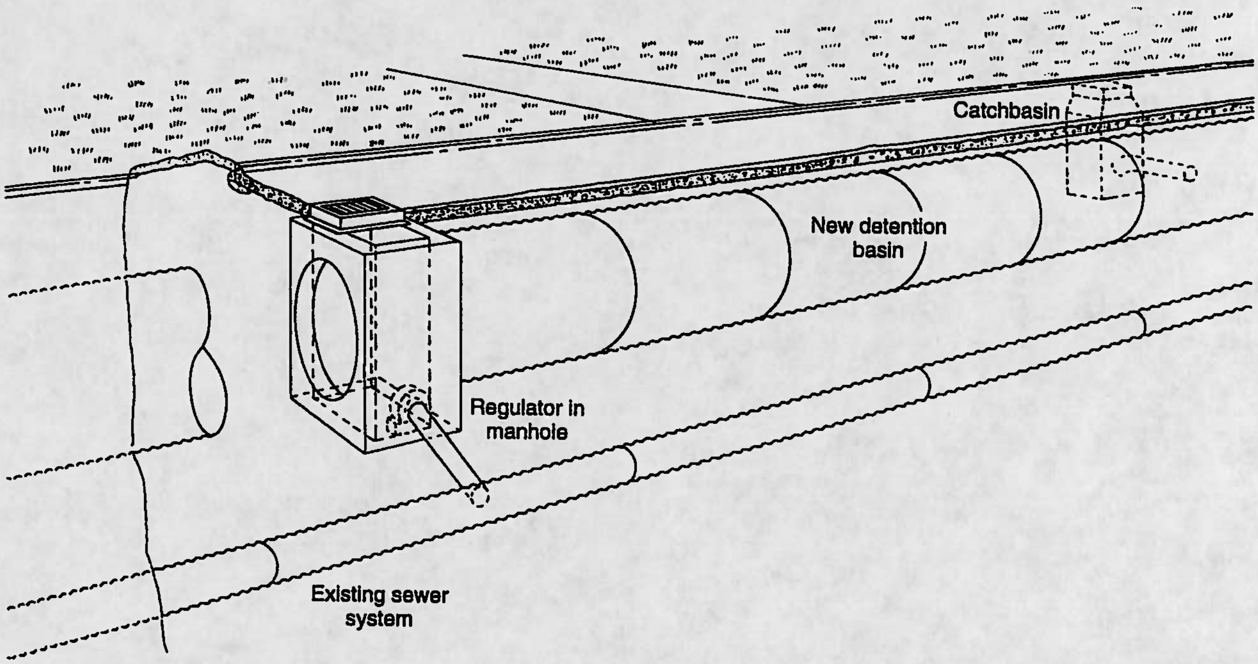


Figure 15.4.22 Typical installation of regulator for underground storage (from American Iron and Steel Institute (1995)).

Figure 15.4.22 illustrates a typical installation of a regulator for underground storage. Flow regulators at inlets to storm sewers are effective in preventing storm sewer surcharging. The simplest form of a flow regulator is an orifice for which the opening has been sized for a given discharge at the maximum head. Regulators in Figure 15.4.22 are designed to handle a discharge that the sewer can handle without excessive surcharging.

PROBLEMS

15.2.1 Determine the pipe diameters for the storm sewer system in Figure P15.2.1a, which is located in Phoenix, Arizona. The rainfall-intensity-duration frequency relationship for the Phoenix metro area is given in Figure P15.2.1b. Characteristics of the catchments are listed in Table P15.2.1. Use a return period of two years ($n = 0.014$).

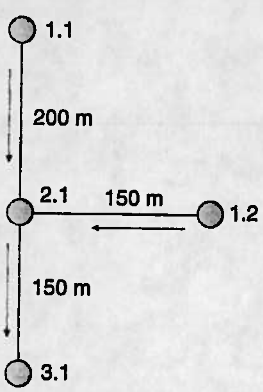


Figure P15.2.1 (*a*)

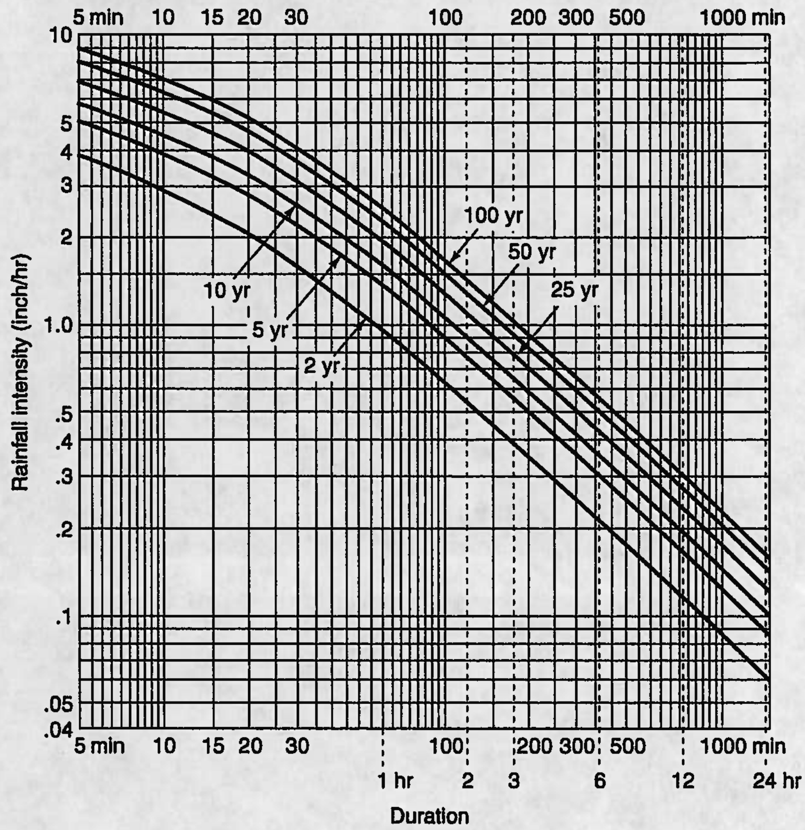
Table P15.2.1 Catchment Characteristics, Problem 15.2.1

Catchment	Ground Elevation (m)	Area (km^2)	Runoff Coefficient	Inlet Time (min)
1.1	300	0.01	0.60	25
1.2	298	0.008	0.75	20
2.1	296	0.005	0.80	15
3.1	294.5			

15.2.2 Rework problem 15.2.1 using a 10-year return period.

15.2.3 The simple storm sewer system below is to be designed using the following data. Assume the use of a 10-year frequency rainfall. The pipe is concrete with $n = 0.014$.

Manhole	Drainage Area (acre)	Time of Conc. (min)	C
1	2	15	0.5
2	3	20	0.8



(*a*)

Figure P15.2.1 (*b*) Rainfall intensity-duration-frequency relation (Phoenix metro area) (from Flood Control District of Maricopa County (1992)).

Pipe	Slope (ft/ft)	Length (ft)
1−2	0.005	1000
2−3	0.006	800

15.2.4 Approximate the sudden expansion loss for a 600-mm sewer pipe connecting to a 700-mm sewer pipe for a design discharge of 0.5 m^3/s.

15.2.5 Compute the bend loss for a 45° bend in a 500-mm sewer pipe with a discharge of 0.45 m^3/s, assuming full-pipe flow. Assume $r/D = 2$.

15.2.6 Rework example 15.2.4 with $Q_L = 30$ cfs, $Q_u = 50$ cfs, and $Q_0 = 80$ cfs. The outfall pressure line elevation is 475.7 ft.

15.2.7 Determine the coefficient of variation of the loading and the capacity for the following parameters. Assume a uniform distribution to define the uncertainty of each parameters.

Parameter		Mode Range
C	0.80	0.75–0.85
i	5.0 in/hr	4.5–5.5
A	10 acres	9.8–10.2
n	0.015	0.014–0.016
d	5 ft	4.98–5.02
S_0	0.0005	0.0004–0.0006

15.2.8 Rework example 15.2.5 using a triangular distribution to define the uncertainty of each parameter.

15.2.9 Using the results of problem 15.2.7, determine the risk of loading exceeding the capacity of the sewer pipe. Assume the use of a safety margin that is normally distributed.

15.2.10 Rework example 15.2.6 using a safety factor approach that is normally distributed; SF = Q_c/Q_L.

15.3.1 Design a nonerodible trapezoidal channel to carry a discharge of 6 m^3/s. Use a Manning's $n = 0.025$ and a slope $S_0 = 0.0005$. Consider the best hydraulic section.

15.3.2 Design a concrete-lined trapezoidal channel to carry a discharge of 8 m^3/s. A slope of $S_0 = 0.0001$ is to be used. Consider the best hydraulic section.

15.3.3 Design a concrete-lined rectangular channel to carry 25 ft^3/s. A slope of $S_0 = 0.0001$ is to be used. Consider the best hydraulic section.

15.3.4 Design a gravel riprap-lined trapezoidal channel to carry a discharge of 11.33 m^3/s. Use a slope $S_0 = 0.0016$ and consider the best hydraulic section.

15.3.5 Design a gravel riprap-lined triangular channel to carry a discharge of 11.33 m^3/s. Use a slope $S_0 = 0.0016$ and consider the best hydraulic section.

15.4.1 Rework example 15.4.1 with the flow peak limited to 30 ft^3/s.

15.4.2 Solve problem 15.4.1 using the Abt and Grigg (1978) method.

15.4.3 Solve example 15.4.6 using a runoff coefficient of $C_p = 0.85$ for a 15.24-acre watershed with $Q_A = 32.17$ cfs.

15.4.4 Solve example 15.4.6 using a developed runoff coefficient of 0.80.

15.4.5 Solve example 15.4.7 using a developed runoff coefficient of 0.85.

15.4.6 Solve example 15.4.7 using a developed runoff coefficient of 0.95.

15.4.7 Solve example 15.4.8 using the level pool routing procedure.

15.4.8 Solve example 15.4.8 using a time interval of 30 minutes.

15.4.9 Consider a 4047 m^2 (0.4047 ha) detention basin with vertical walls. The triangular inflow hydrograph increases linearly from zero to a peak of 10.2 m^3/s at 60 min and then decreases linearly to zero at 150 min. The basin is initially empty and the discharge-elevation relationship is:

Elevation (H, m)	0.0	0.152	0.305	0.457	0.610	0.762	0.914
Discharge (Q, m^3/s)	0.0	0.085	0.230	0.482	0.850	1.220	1.700

Elevation (H, m)	1.067	1.219	1.524	1.830	2.134	2.438	2.743	3.048
Discharge (Q, m^3/s)	2.209	2.750	3.880	4.900	5.806	6.542	7.165	7.788

Use the Runge–Kutta method with a routing interval of 20 min to determine the detention basin discharge at the end of 20 min.

REFERENCES

Abt, S. R., and N. S. Grigg, "An Approximate Method for Sizing Retention Reservoirs," *Water Resources Bulletin*, AWRA, vol. 14, no. 4, pp. 956–965, 1978.

Akan, A. O., "Detention Pond Sizing for Multiple Return Periods," *Journal of Hydraulic Engineering*, vol. 115, no. 5, pp. 650–664, 1989.

Akan, A. O., "Single-Outlet Detention-Pond Analysis and Design," *Journal of Irrigation and Drainage Engineering*, vol. 116, no. 4, pp. 527–536, 1990.

Akan, A. O., *Urban Stormwater Hydrology*, Technomic Publishing, Lancaster, PA, 1993.

American Association of State Highway and Transportation Officials (AASHTO), *Model Drainage Manual*, Washington, DC, 1991.

American Iron and Steel Institute, *Modern Sewer Design*, third edition, Washington, DC, 1995.

American Society of Civil Engineers and Water Pollution Control Federation, *Design and Construction of Urban Stormwater Management Systems*, ASCE Manual and Report of Engineering Practice No.77, WEF Manual of Practice FD-20, New York, 1992.

American Society of Civil Engineering (ASCE) and Water Pollution Control Federation (WPCF), *Design and Construction of Sanitary and Storm Sewers*, ASCE Manual of Practice no. 37 and WPCF Manual of Practice no. 9, 1969.

Ang, A. H.-S., and W. H. Tang, *Probability Concepts in Engineering Planning and Design, I: Basic Principles*, John Wiley, New York, 1975.

Ang, A. H.-S., and W. H. Tang, *Probability Concepts in Engineering Planning and Design, II: Decision, Risk, and Reliability*, John Wiley, New York, 1984.

Aron, G., and C. E. Egborge, "A Practical Feasibility Study of Flood Peak Abatement in Urban Areas," report, U.S. Army Corps of Engineers, Sacramento District, Sacramento, CA, March 1973.

Aron, G., and D. F. Kibler, "Pond Sizing for Rational Formula Hydrographs," *Water Resources Bulletin*, AWRA, vol. 26, no. 2, pp. 255–258, 1990.

Bennett, M. S., and L. W. Mays, "Optimal Design of Detention and Drainage Channel Systems," *Journal of the Water Resources Planning and Management Division, ASCE*, vol. 111, no. 1, pp. 99–112.

Bathurst, J. C., R. M. Li, and D. B.Simons, "Resistance Equation for Large-Scale Roughness," *Journal of the Hydraulics Division, ASCE*, vol. 107, no. HY12, pp.1593–1613, 1981.

Carnahan, B., H. A. Luther, and J. O. Wilkes, *Applied Numerical Methods*, John Wiley, New York, 1969.

Chen, Y. H., and G. K. Cotton, "Design of Roadside Channels with Flexible Linings," Hydraulic Engineering Circular 15, FHWA-IP-87-7, Federal Highway Administration, McLean, VA, 1988.

Chow, V. T., D. R. Maidment, and L. W. Mays, *Applied Hydrology*, McGraw-Hill, New York, 1988.

Cotton, G. K., Hydraulic Design of Flood Control Channels, *Hydraulic Design Handbook*, edited by L. W. Mays, McGraw-Hill, New York, 1999.

Donahue, J. R., R. H. McCuen, and T. R. Bondelid, "Comparison of Detention Basin Planning and Design Models," *Journal of the Water Resource Planning and Management Division, ASCE*, vol. 107, no. 2, pp. 385–400, 1981.

Federal Aviation Administration, Department of Transportation, circular on airport drainage, report A/C 050-5320-5B, Washington, DC, 1970.

Federal Aviation Agency (FAA), *Airport Drainage*, Washington, DC, 1966.

Federal Highway Administration (FHWA), "HYDRAIN—Integrated Drainage Design Computer System. Vol. III. HYDRA—Storm Drains," Structure Division, Federal Highway Administration, Washington, DC, 1993.

Flood Control District of Maricopa County, Drainage Design Manual ɔr Maricopa County, Arizona, vol. I–Hydrology, Phoenix, Arizona, 992.

Flood Control District of Maricopa County, Drainage Design Manual ɔr Maricopa County, Arizona, vol. II–Hydraulics, Phoenix, Arizona, 992.

Frederick, R. H., V. A. Myers, and E. P. Auciello, "Five to 60-minute recipitation Frequency for the Eastern and Central United States," IOAA Technical Memo NWS HYDRO-35, National Weather Service, ilver Spring, MD, June 1977.

Great Lakes–Upper Mississippi River Board of State Sanitary ngineers, Ten-State Standards, "Recommended Standards for Sewage Vorks," Health Education Service, Albany, NY, 1978.

Harr, M. E., *Reliability-Based Design in Civil Engineering*, McGraw-Iill, New York, 1987.

Izzard, C. F., Hydraulics of Runoff from Developed Surfaces, *Proc. Iighway Research Board*, vol. 26, pp. 129–146, 1946.

Kapur, K. C., and L. R. Lamberson, *Reliability in Engineering Design*, ohn Wiley, New York, 1977.

Kececioglu, D., *Reliability Engineering Handbook*, vols. 1 and 2, rentice-Hall, Englewood Cliffs, NJ, 1991.

Kessler, A., and M. H. Diskin, "The Efficiency Function of Detention Reservoirs in Urban Drainage Systems," *Water Resources Research*, American Geophysical Union, vol. 27, no. 3, pp. 253–258, 1991.

Kibler, D. F., *Urban Stormwater Hydrology*, Water Research Monograph 7, American Geophysical Union, Washington, DC, 1982.

Kirpich, Z. P., "Time of Concentration of Small Agricultural Watersheds," *Civ. Eng.*, vol. 10, no. 6, p. 362, 1940.

Kouwen, N., T. E. Unny, and H. M. Hill, "Flow Retardance in Vegetated Channels," *Journal of the Irrigation Division, ASCE*, vol. 95, no. IR2, pp. 329–342, 1969.

Lakatos, D. F., and R. H. Kropp, "Stormwater Detention—Downstream Effects on Peak Flow Rates," *Stormwater Detention Facilities*, American Society of Civil Engineering, New York, 1982.

Loganathan, G. V., D. F. Kibler, and T. J. Grizzard, "Urban Stormwater Management," *Water Resources Handbook*, edited by L.W. Mays, McGraw-Hill, New York, 1996.

Loganathan, G. V., J. W. Delleur, and R. I. Segana, "Planning Detention Storage for Stormwater Management," *Journal of Water Resources Planning and Management Division, ASCE*, vol. 111, no. 4, pp. 382–398, 1985.

Loganathan, G. V., E. W. Watkins, and D. F. Kibler, "Sizing Stormwater Detention Basins for Pollutant Removal," *Journal of Environmental Engineering, ASCE*, vol. 120, no.6, pp. 1380–1399, 1989.

Mays, L. W., and P. B. Bedient, "Model for Optimal Size and Location of Detention Basins," *Journal of the Water Resources Planning and Management Division, ASCE*, vol. 108, no. WR3, pp. 220–285, 1982.

Mays, L. W., and Y. K. Tung, *Hydrosystems Engineering and Management*, McGraw-Hill, New York, 1992.

McEnroe, B. M., "Preliminary Sizing of Detention Reservoirs to Reduce Peak Discharges," *Journal of Hydraulic Engineering, ASCE*, vol. 118, no. 11, pp. 1450–1549, 1992.

Morgali, J. R., and R. K. Linsley, "Computer Analysis of Overland Flow," *J. Hyd. Div., Am. Soc. Civ. Eng.*, vol. 91, no. HY3, pp. 81–100, May 1965.

National Highway Institute, *Urban Drainage Design*, Participant Notebook, NHI Cause No. 13027, Publication No. FHWA HI-89-035, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1993.

Nix, S. J., and J. P. Heaney, "Optimization of stormwater Storage—Release Strategies," *Water Resources Research*, vol. 24, no. 11, pp. 1831–1838, 1988.

Overton, D. E., and M. E. Meadows, *Stormwater Modeling*, Academic Press, New York, 1976.

Phillips, J. V., and T. L. Ingersoll, "Verification of Roughness Coefficients for Selected Natural and Constructed Stream Channels in Arizona," U.S. Geological Survey Professional Paper 1584, U.S. Geological Survey, Denver, CO, 1998.

Poertner, H. G., "Practices in Detention of Urban stormwater Runoff," American Public Works Association, Special Report No. 43, 1974.

Sangster, W. M., H. M. Wood, E. T. Smerdon, and H. G. Bossy, "Pressure Changes at Storm Drain Junction," Engineering Series Bulletin, No. 41, Engineering Experiment Station, University of Missouri, Columbia, MO, 1958.

Segaua, R. I., and G. V. Loganathan, "Stormwater Detention Storage Design Under Random Pollutant Loading," *Journal of Water Resources Planning and Management, ASCE*, vol. 118, no. 5, pp. 475–491, 1992.

Stahre, P., and B. R. Urbonas, *Stormwater Detention for Drainage, Water Quality and CSO Management*, Prentice Hall, Englewood Cliffs, NJ, 1990.

Taur, C. K., G. Toth, G. E. Oswald, and L. W. Mays, "Austin Detention Basin Optimization Model," *Journal of Hydraulic Engineering, ASCE*, vol. 113, no. 7, pp. 860–878, 1987.

University of Colorado, *HYDRO POND User's Manual*, Department of Civil Engineering, University of Colorado at Denver, 1991.

U.S. Bureau of Reclamation (USBR), *Design of Small Dams*, second Edition, U.S. Government Printing Office, Denver, CO, 1973.

U.S. Bureau of Reclamation (USBR), *Design of Small Dams*, third Edition, U.S. Government Printing Office, Denver, CO, 1987.

U.S. Soil Conservation Service, "Urban Hydrology for Small Watersheds, Tech. Release 55," Washington, DC, 1975 (updated, 1986).

U.S. Soil Conservation Service (SCS), *Handbook of Channel Design for Soil and Water Conservation, SCS-TP-61*, Stillwater, OK., 1954.

Urbonas, B. R., and L. A. Roesner, "Hydrologic Design of Urban Drainage and Flood Control," *Handbook of Hydrology* (edited by D. R. Maidment), McGraw-Hill, New York, 1993.

Wang, S. Y., and H. W. Shen, "Incipient Sediment Motion and Riprap Design," *Journal of the Hydraulics Division, ASCE*, vol. 111, no. 3, pp. 521–538, 1985.

Wanielista, M. P., and Y. A. Yousef, *Stormwater Management*, Wiley Interscience, New York, 1993.

Whipple, W., N. S. Grigg, T. Grizzard, C. W. Randall, R. P. Shubinski, and L. S. Tucker, *Stormwater Management in Urbanizing Areas*, Prentice-Hall, Englewood Cliffs, NJ, 1983.

Whipple, W., R. Kropp, and S. Burke, "Implementing Dual Purpose Stormwater Detention Program," *Journal of Water Resources Planning and Management, ASCE*, vol. 113, no. 6, pp. 779–792, 1987.

Wycoff, R. L., and V. P. Singh, "Preliminary Hydrologic Design of Small Flood Detention Reservoirs," *Water Resources Bulletin*, AWRA, vol. 12, no. 2, pp. 337–349, 1976.

Yen, B. C., and A. O. Akan, "Hydraulic Design of Urban Drainage Systems," *Hydraulic Design Handbook*, edited by L. W. Mays, McGraw-Hill, New York, 1999.

Yen, B. C., "Hydraulics for Excess Water Management," *Water Resources Handbook*, edited by L.W. Mays, McGraw-Hill, New York, 1996.

Yen, B. C., ed., Storm Sewer System Design, Department of Civil Engineering, University of Illinois at Urbana-Champaign, 1978.

Yen, B. C., ed., *Stochastic and Risk Analysis in Hydraulic Engineering*, Water Resources Publications, Littleton, CO, 1986.