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 and Water Pollution Control Federation, 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.

15.2.2 Rational Method Design

From an engineering viewpoint the design can be divided into two main aspects: runoff prediction and pipe sizing. The rational method, which can be traced back to the mid-nineteenth century, is 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 methods have been proposed, the rational method, because of its simplicity, is still in continued use for sewer design when high accuracy of runoff rate is not essential.

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)
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).

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

$$Q = KCiA \tag{15.2.1}$$

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) from intensity-duration frequency relationships for a specific return period and duration t_c in min, and A is the area of the tributary drainage area in acres (km^2). The duration is taken as the time of concentration t_c of the drainage area.

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:

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

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,

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

Table 15.2.3 Runoff Coefficients for Use in the Rational Method

Character of surface	Return period (years)						
	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
Undeveloped							
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	0.36	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

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

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

where the flow time is

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

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 the computation of sewer flow time) and the corresponding rainfall intensity i is computed repeatedly for the area drained by the sewer. For a given sewer, all the different areas drained by this sewer have the same i. Thus, as the design progresses towards the downstream sewers, the drainage area increases and usually the time of concentration increases accordingly. This increasing t_c in turn 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 using a number of methods, some of which are presented in Table 15.2.4. The longest time of concentration among the times for the various flow routes in the drainage area is the critical time of concentration 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 mountainous 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 analysis 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 ac; found generally good where area is completely paved; for mixed areas it tends 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.
SCS average velocity charts (U.S. Soil Conservation Service 1975, 1986)	$t_c = \dfrac{1}{60}\sum \dfrac{L}{V}$ 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.

Source: 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).

SOLUTION

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

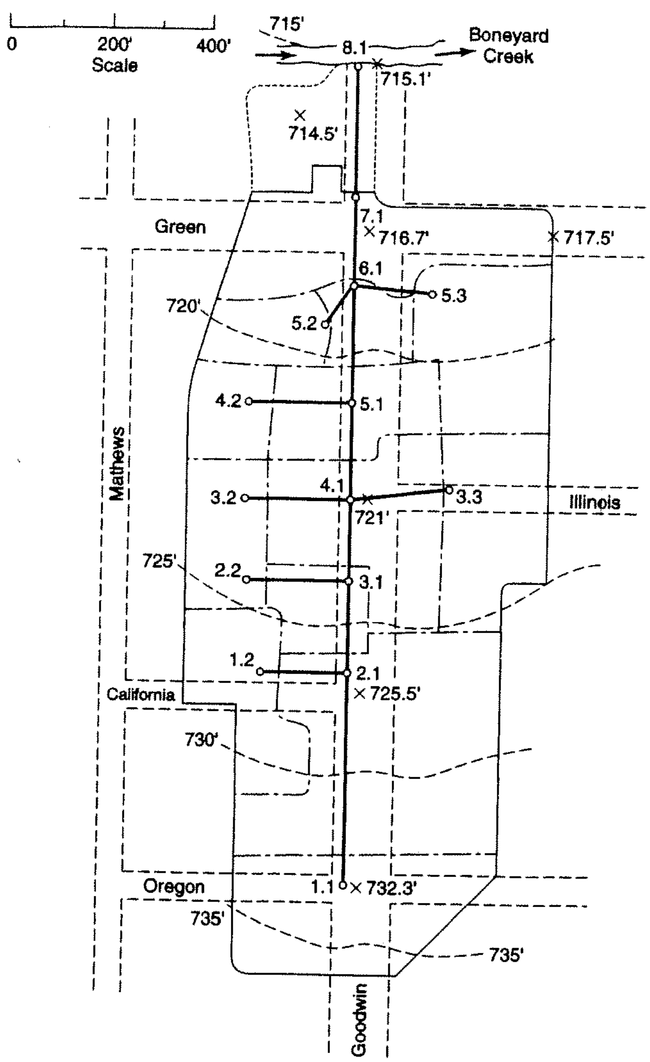


Figure 15.2.1 Goodwin Avenue drainage basin at Urbana, Illinois (from Yen (1978)).

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).

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).

Column (5): The identification number of the incremental subcatchments that drain directly through manhole 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 contributing 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 methods in Table 15.2.4), i.e., the overland flow inlet time if the upstream subcatchment is no more than one sewer away from the sewer being designed (e.g., in designing sewer 3.1, 5.2 min for subcatchment 2.2 and 8.7 min for subcatchment 3.1); otherwise it is the total flow time to the entrance of the immediate 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 (column (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 concentration of different flow paths to arrive at the entrance of the sewer being considered; e.g., for sewer 3.1, 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 subcatchment 3.1 (8.7 min).

Column (14): The rainfall intensity i for the duration given in column (13) is based on HYDRO-35 for the 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 (9) and (14).

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).

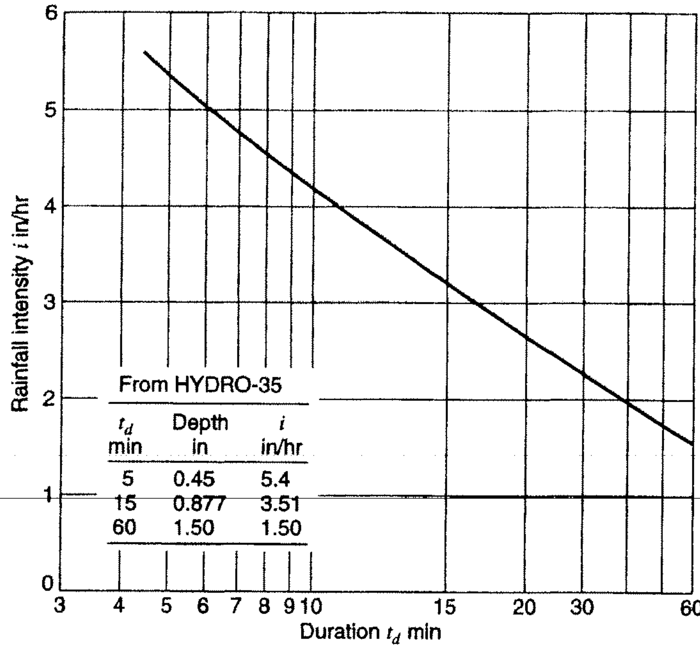


Figure 15.2.2 Variation of rainfall intensity with duration at Urbana, Illinois (from Yen (1978)).

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 is adopted.

Column (18): Flow velocity computed by using $V = 4Q_p/(\pi D^2)$, i.e., column (15) multiplied by $4/\pi$ and 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 converted into minutes.

This example demonstrates that in the rational method each sewer is designed individually and independently (except for the computation of sewer flow time) and the corresponding rainfall intensity i is computed repeatedly for the area drained by the sewer. For a given sewer, all the different areas drained 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 are 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 the above two equations.

Transition losses (pressurized flow)

Contraction losses for pressurized flow are expressed as

$$H_c = K\left(\frac{V_2^2}{2g}\right)\left[1 - \frac{A_2}{A_1}\right]^2 \tag{15.2.11}$$

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

$$H_e = K\left[\frac{(V_1 - V_2)^2}{2g}\right] \tag{15.2.12}$$

where $K = 1.0$ for a sudden expansion and $K = 0.2$ for a well-designed transition. These K values for the contractions and expansions are for approximation. For detailed analysis, Tables 15.2.7–15.2.10

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 (ft/s)												
	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

D_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.

Source: 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 (ft/s)												
	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
10.0	.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

D_2/D_1 = ratio of larger pipe to smaller diameter; V_2 = velocity in smaller pipe.

Source: 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).

can be used in conjunction with the following form of the headloss equation:

$$H = K\left(\frac{V^2}{2g}\right) \tag{15.2.13}$$

Exit losses can be computed with the following equation:

$$H_{\text{ext}} = K_e\left(\frac{V^2}{2g}\right) \tag{15.2.14}$$

Manhole losses

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

$$H_m = 0.05\frac{V^2}{2g} \tag{15.2.15}$$

Losses at *terminal manholes* can be estimated using

$$H_m = \frac{V^2}{2g} \tag{15.2.16}$$

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

$$H_m = K\frac{V^2}{2g} \tag{15.2.17}$$

where Figure 15.2.3 shows manhole junction types and nomenclature. Values of K for various 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

$$H_b = K_b\frac{V^2}{2g} \tag{15.2.18}$$

where

$$K_b = 0.25\sqrt{\frac{\Phi}{90}} \tag{15.2.19}$$

for curved sewer segments where the angle of deflection Φ is less than 40°. For greater angles of 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 pipe for a design discharge of 0.3 m^3/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 $K = 1.0$:

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

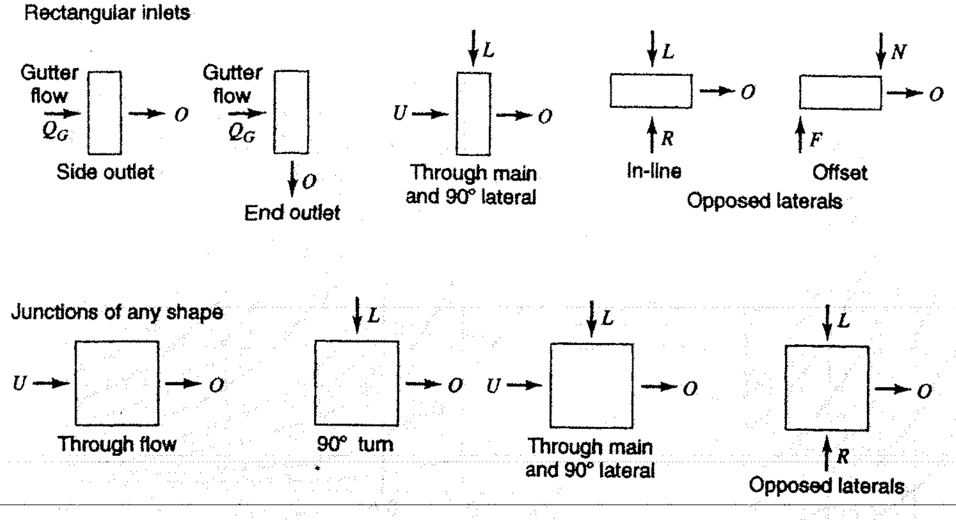
EXAMPLE 15.2.3

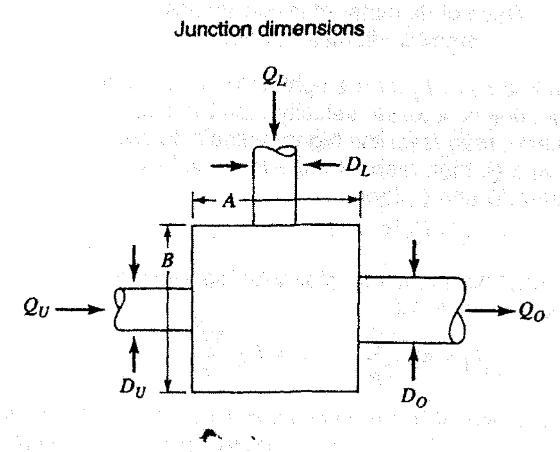
Compute the bend loss for a 30° bend in a 400-mm sewer pipe with a discharge of 0.3 m^3/s assuming full-pipe flow.

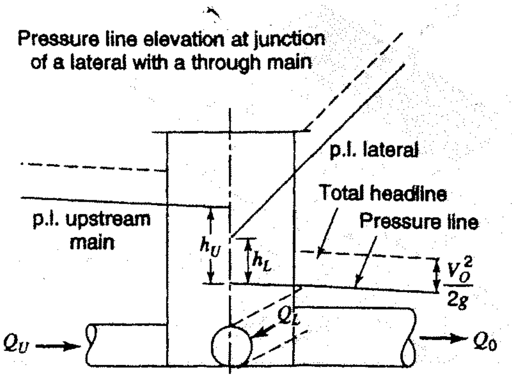
SOLUTION

First compute the flow velocity in the sewer pipe:

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







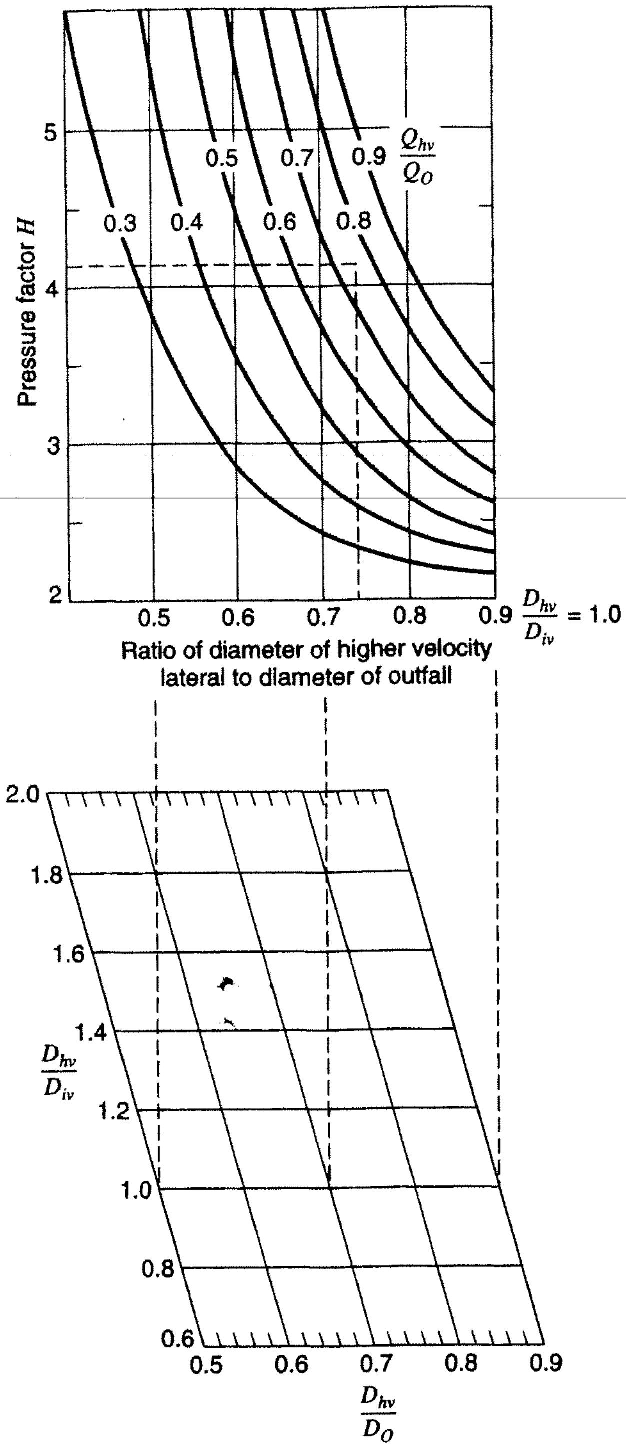
Nomenclature

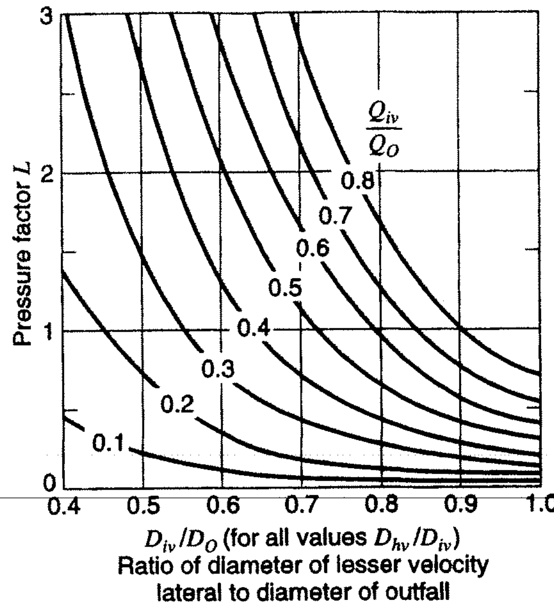
Q	rate of flow
D	diameter of pipe
A	dimension of junction in direction of outfall pipe
B	dimension of junction at right angles to outfall pipe
d	depth of water in inlet
S	slope of pipe
S_f	friction slope
Q_G	flow into inlet through top grate
$D_O\ Q_O$	diameter and flow in outfall
$D_U\ Q_U$	diameter 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)
$\bar{K}_U$, $\bar{K}_L$	pressure coefficient at $Q_L = Q_O$
M_U, M_L	multiplier for $\bar{K}_U$ or $\bar{K}_L$ to obtain K_U or K_L

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

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

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



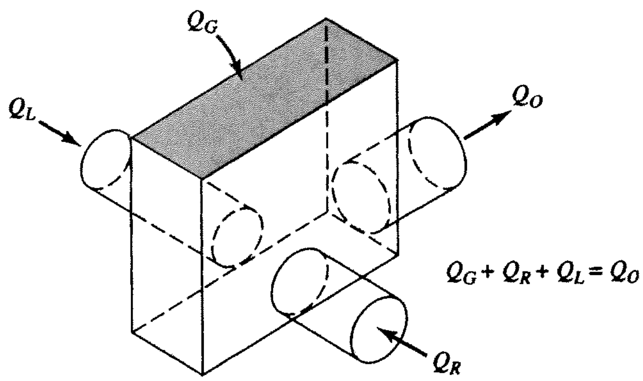


To find K_R or K_L for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q; then:

$$K_R \text{ (or } K_L) = H - L$$

K_R or K_L for the lateral pipe with higher velocity flow is always 1.8

$$h_L = K_L \frac{V_O^2}{2g} \qquad h_R = K_R \frac{V_O^2}{2g}$$



Elevation sketch

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)).

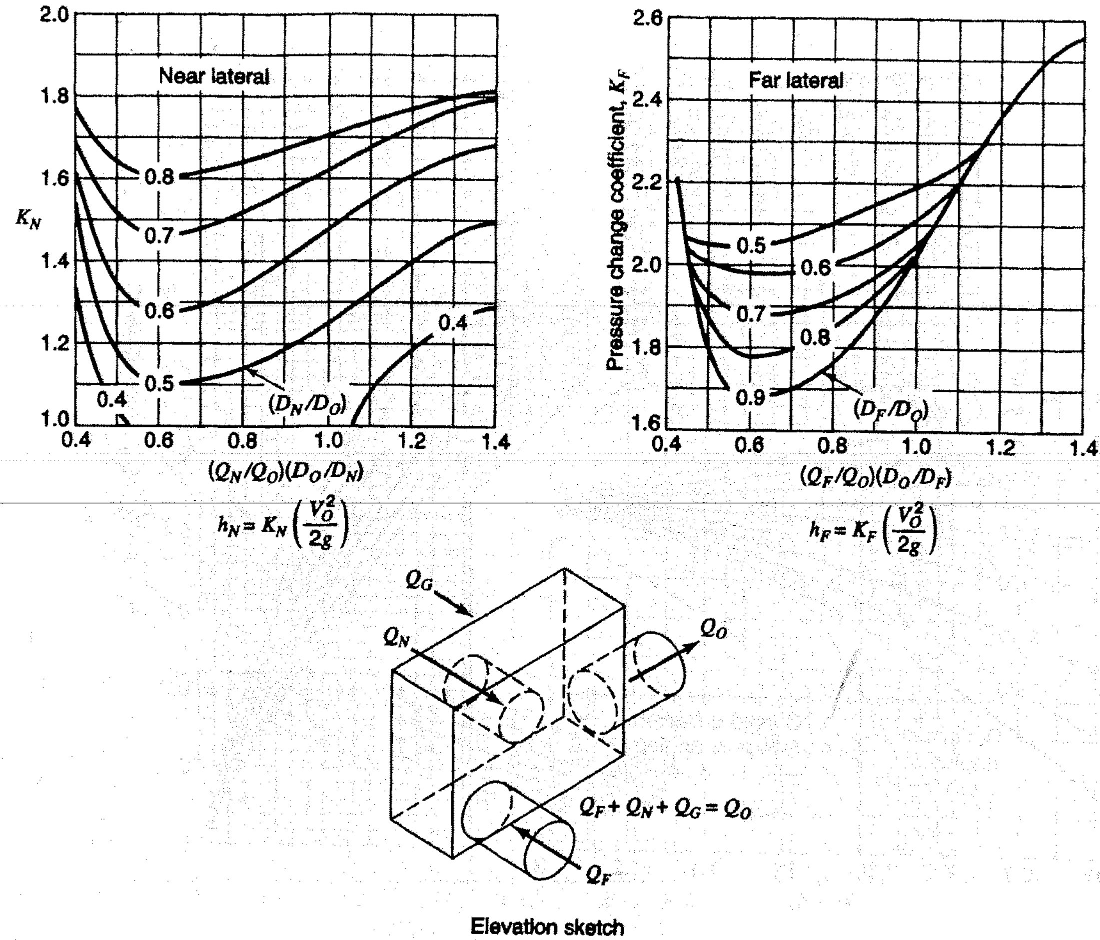


Figure 15.2.5 Rectangular manhole with offset opposed lateral pipes—each at 90° to outfall (with or without inlet flow) (from Sangster et al. (1958)).

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

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

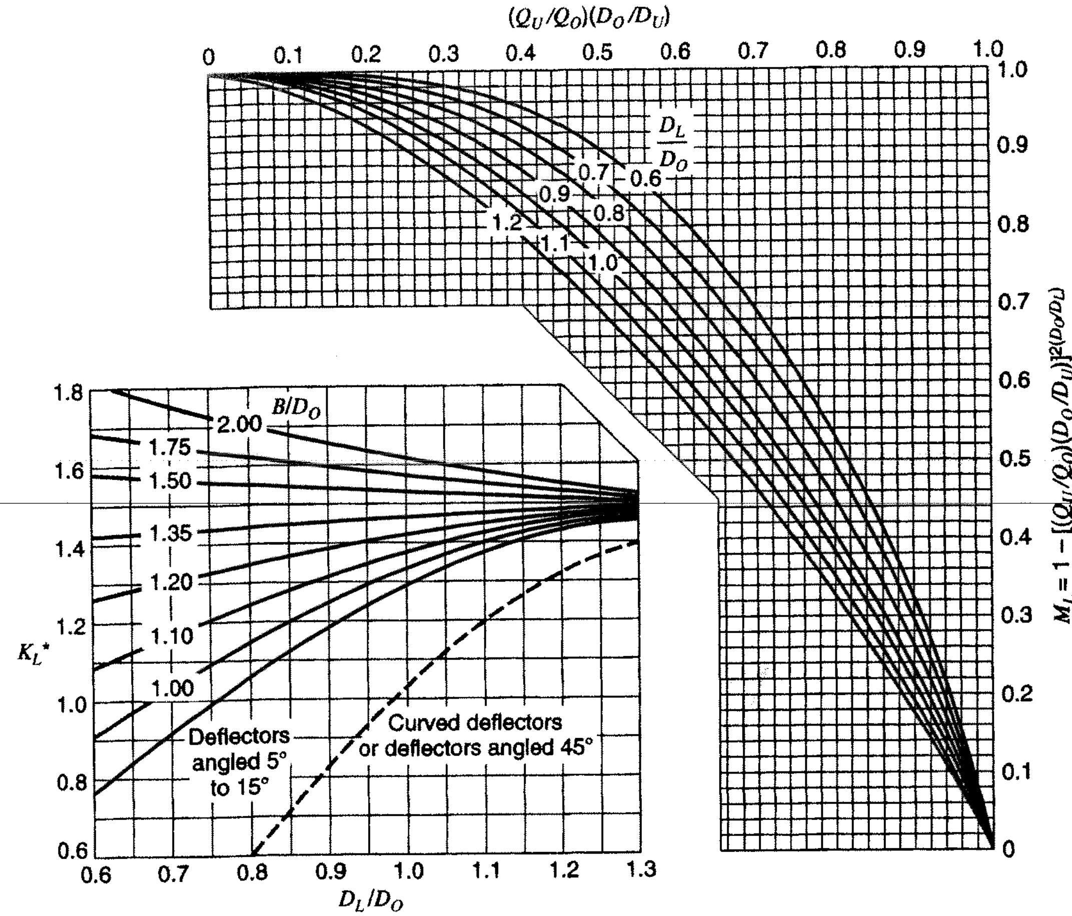
Use equation (15.2.18) to compute the bend loss:

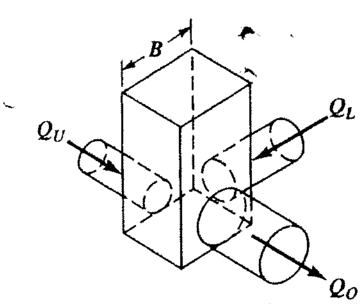
$$H_b = K_b\frac{V^2}{2g} = 0.144\frac{(2.39)^2}{2(9.81)} = 0.0419\,\mathrm{m}$$

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





Elevation sketch

To find K_L for the for the lateral pipe, first read $\overline{K}_L{}^*$ 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.

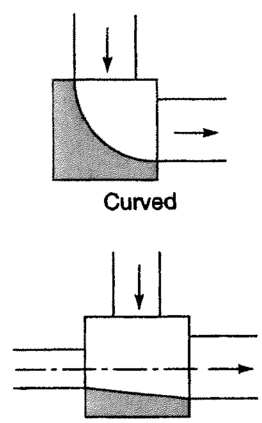
For rounded entrance to outfall pipe, reduce chart values of $K_L{}^*$ by 0.2 for combining flow.

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

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

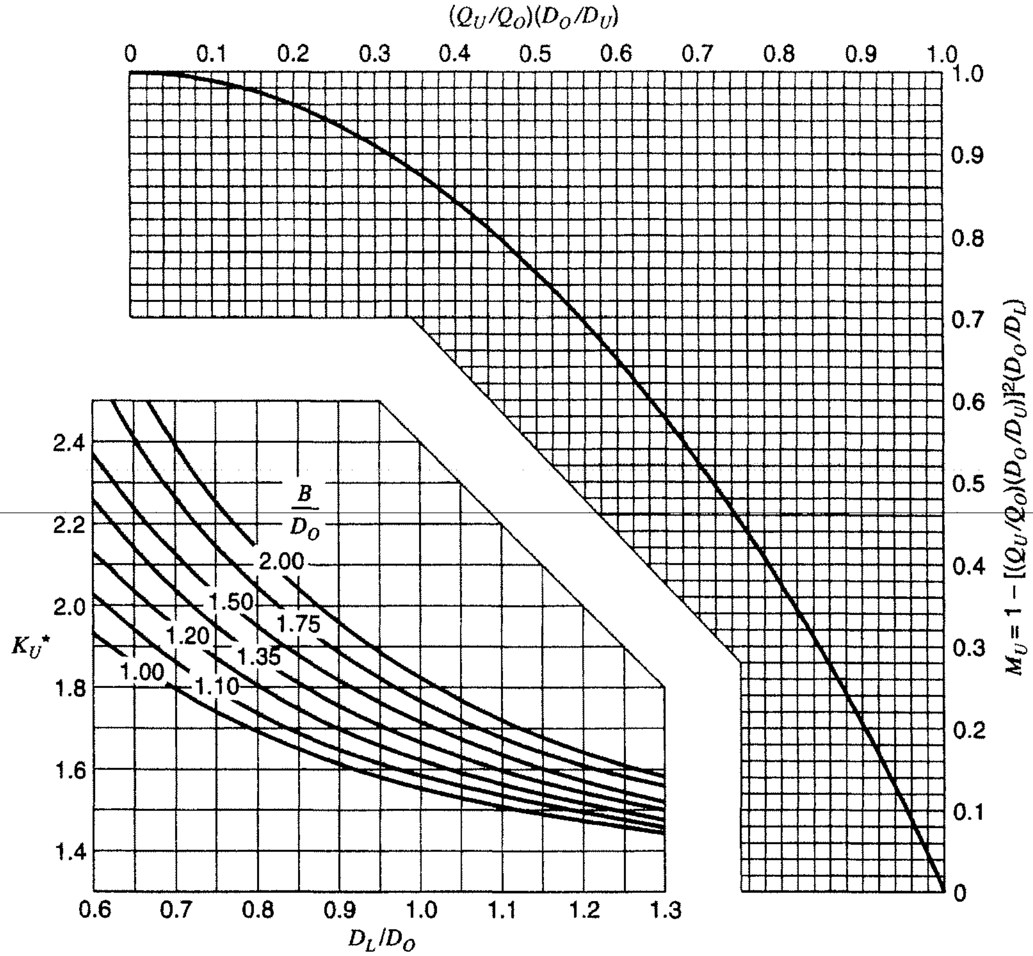
For $D_L = D_O < 0.6$ use

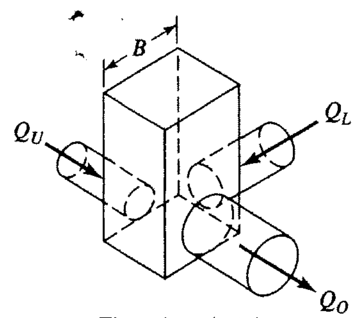
$$h_L = K_L \frac{V_O^2}{2g} \text{ from Figure 15.2.8}$$



Plan of deflectors

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)).





Elevation sketch

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

$$h_U = K_U \frac{V_O^2}{2g} \text{ from Figure 15.2.8}$$

For $D_L/D_O < 0.6$ use

$$h_U = K_U \frac{V_O^2}{2g} \text{ from Figure 15.2.8}$$

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

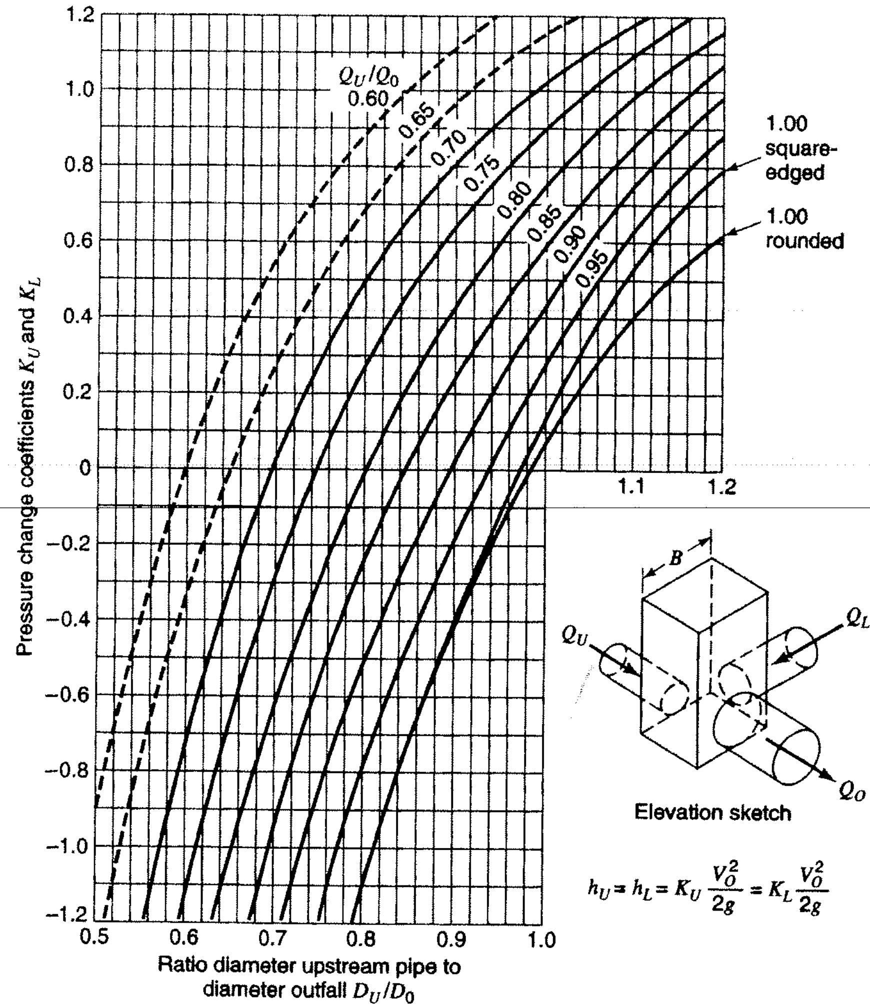


Figure 15.2.8 Manhole on through pipeline at junction of a 90° lateral pipe (for conditions outside the range of Figures 15.2.6 and 15.2.7 (from Sangster et al. (1958)).

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 − 4	475.08 ft

SOLUTION

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

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

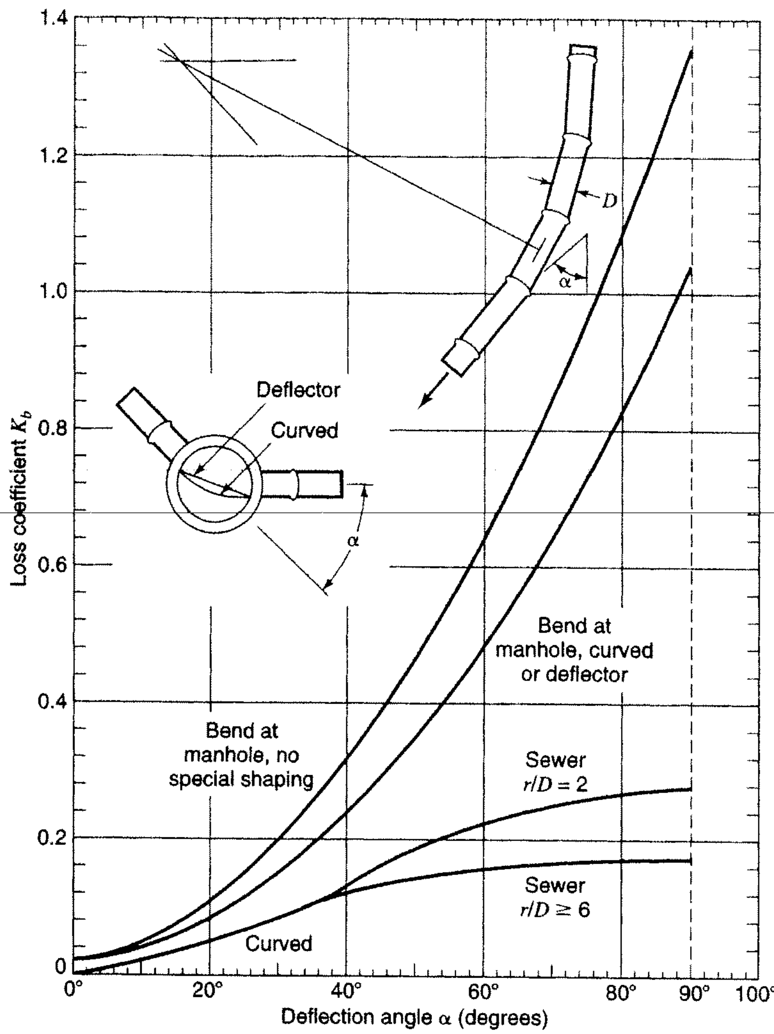


Figure 15.2.9 Sewer bend loss coefficient (from Sangster et al. (1958)).

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.
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.

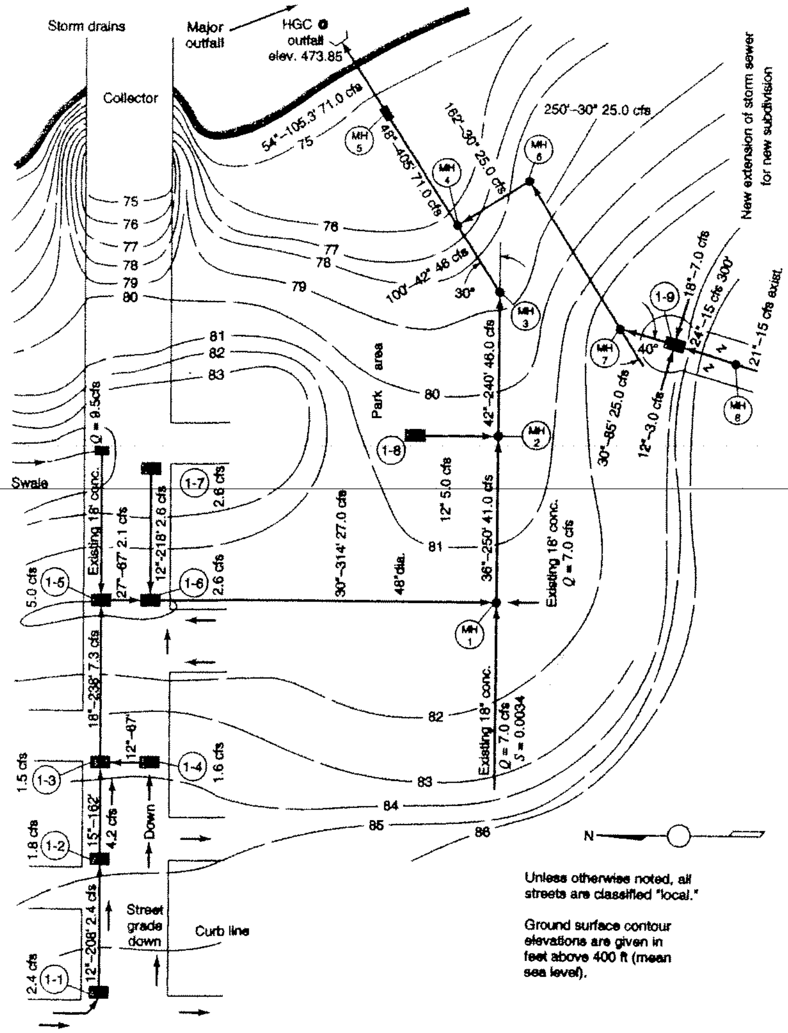


Figure 15.2.10 Storm drain design example (from Flood Control District of Maricopa County, 1992a).

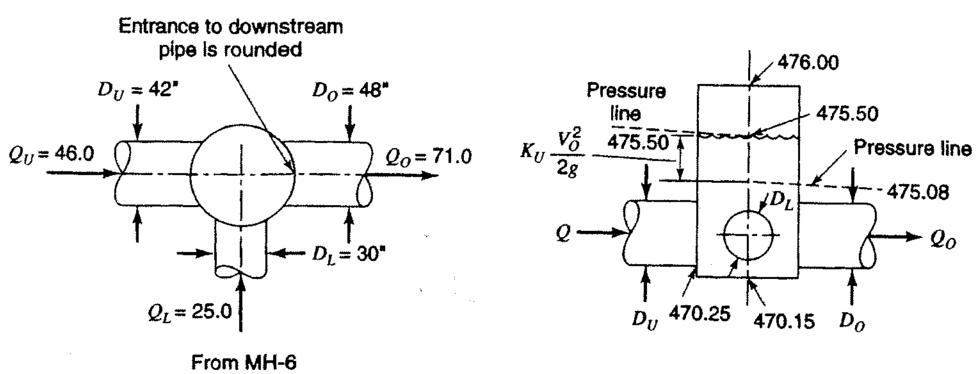


Figure 15.2.11 Storm drain design example for manhole no. 4 (from Flood Control District of Maricopa County, 1992b).

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 (1992b).

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 be 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 surface 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 accommodated without causing flows to exit the system and creating flooding conditions. Figure 15.2.12 illustrates the difference between an improper design and a proper design. Note the energy and 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 flow calculations are indicated; if it is below the pipe crown, then open-channel flow calculations should be used at the upstream manhole. The process is repeated throughout the storm drain system. 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 water surface elevation. Computer programs such as HYDRA (FHWA, 1993) are recommended for the 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 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.

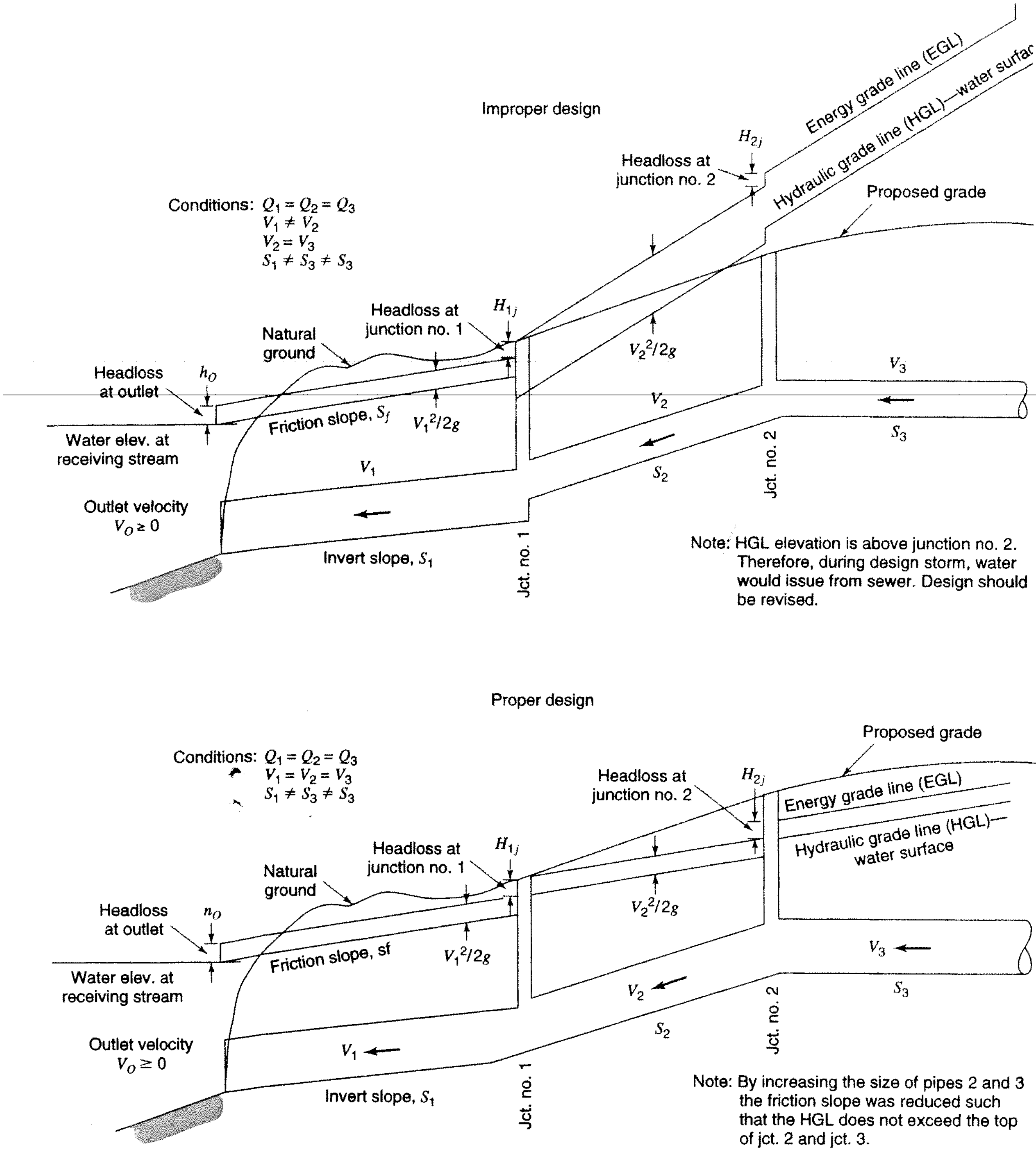


Figure 15.2.12 Use of energy losses in developing a storm drain system: energy and hydraulic grade lines for storm sewer under constant discharge (from AASHTO (1991)).

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

$$\sum Q_{ij} + Q_j - Q_0 = \frac{ds}{dt} \qquad (15.2.20)$$

in which Q_{ij} is the inflow from the ith 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

$$t_f = \frac{L}{V} \qquad (15.2.21)$$

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

$$V = \frac{4Q_p}{\pi D^2} \qquad (15.2.22)$$

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

In this method, the continuity relationship of the flow within the sewer is not directly considered. The routing of the sewer flow is done by shifting the inflow hydrograph by t_f and no consideration is given to the unsteady and nonuniform nature of the sewer flow. Shifting of hydrographs approximately accounts for the sewer flow translation time but offers no wave attenuation. 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 manholes, bends, inlets, and catch basins. The discussion here briefly explains some of the common appurtenances; 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 alignment, with street intersections being typical locations. Typical manholes for small sewers (sewer diameters less than 2 ft) are shown in Figure 15.2.13 and for intermediate sized sewers (sewer diameters greater than 2 ft) are shown in Figure 15.2.14. Manholes provide convenient access to the sewer for observation and maintenance operation. They should be designed to cause a minimum 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 to 760 mm) (Great Lakes–Upper Mississippi Board of State Sanitary Engineers, 1978). For larger 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 elevation of 24 in (0.6 m) or more above the manhole invert. These manholes are constructed with either an 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 detailed 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 engine ing problems—to an element of uncertainty. The uncertainty inherent in storm sewer des