7.0 URBAN DRAINAGE

Several aspects of highway drainage design are specific to urban areas. Urban highways commonly have curb and gutter edge treatments with inlets to underground drainage pipes. These components are designed to limit the spread of water on the roadway. This section presents guidance for the design of urban highway drainage systems.

7.1 STREET DRAINAGE

7.1.1 Design Criteria

The recurrence interval for street drainage is 10 years. Table 7.1.1-1 specifies the allowable spread of water according to the road classification and the design speed.

Table 7.1.1-1 Design Criteria for Allowable Spread for 10-Year Recurrence Interval

Road/Street Classification	Design Speed	Allowable Spread on *Driving Lane
Local and collector	< 45 mi/h	1/2 driving lane
Arterial	< 45 mi/h	1/2 driving lane
High speed, any classification, with shoulders	> 45 mi/h	None
High speed, non- freeways, without shoulders **	> 45 mi/h (2 lanes total with curb & gutter)	3.0 ft.
High speed, non- freeways, without shoulders **	> 45 mi/h (3 or more lanes total with curb & gutter)	1/2 driving lane

* Driving Lane is considered to be right side through lane. ** For urban and developing suburban locations.

Curb and gutter inlets should be located so that the allowable spread is not exceeded in the 10-year event.

7.1.2 Types of Inlets

KDOT has four standard designs for curb and gutter inlets: Type 22 curb inlet, Type 12 combination inlet, Type B gutter inlet, and the concrete gutter inlet. The Type 22 curb inlet, the Type 12 combination inlet and the Type B gutter inlet are normally used on roadways with the Type I combined curb and gutter or a similar curb and gutter. The concrete gutter inlet is used on roadways with the standard gutter or a similar shallow gutter. The Type 22 curb inlet is available in three sizes, with opening lengths of 5.0 ft, 10.0 ft and 15.0 ft.

The Type 22 curb inlets perform better on mild grades than on steep grades. The grade of the roadway does not have a significant effect on the performance of the three inlets with gutter openings (the Type 12 combination inlet, the Type B gutter inlet, and the concrete gutter inlet).

Gutter openings tend to collect debris and become partially clogged in sag locations. For this reason, the Type 22 curb inlet or the Type 12 combination inlet should be used in sag locations on roadways with curb and gutter. Design capacities of gutter inlets in sag locations should be reduced by 30 to 50 percent to compensate for the likelihood of partial clogging.

7.1.3 Location of Inlets

Curb and gutter inlets should be located as needed so that the spread of water on the roadway does not exceed the design limits specified in Table 7.1.1-1. Inlets should also be located directly upgrade of street intersections and cross walks. Runoff from areas draining toward the highway should be intercepted before it reaches the highway. For example, inlets should be located on side streets directly upgrade from intersections.

Inlets on grade should be sized and spaced so that a small fraction of the design flow bypasses the inlet, except at locations such as intersections and cross walks where bypassed flow is undesirable. The allowance of up to 20 percent bypass at acceptable locations leads to more economical designs.

In sag locations, it is good practice to maintain a minimum grade of 0.3% in the gutter within 50 ft of the low point to provide for adequate drainage. In sag locations on high speed roads or streets, flanking inlets should be located on each side of the inlet at the low point. Flanking inlets are located and sized to capture the design flow and to act in relief of primary inlet at the low point if it should become clogged or if the design spread is exceeded. HEC No.22 provides guidance for location of flanking inlets.

The recurrence interval should be increased to 50 years for depressed sections and underpasses where ponded water can be removed only through the storm drainage system.

7.1.4 Discharge Capacity of Gutter and Street

The discharge that can be conveyed in the gutter and street depends on the allowable spread, the type of curb and gutter, and the grade and cross-slope of the street. To find the discharge for a given spread, determine the hydraulic conveyance from Table 7.1.4-1 and then compute the discharge with Equation 7-1:

$$Q = K\sqrt{S} \tag{7-1}$$

where: Q = discharge (cfs)

K = hydraulic conveyance (cfs), from Table 7.1.4-1

S = slope of energy grade line (ft/ft)

For streets on grade, the slope of the energy grade line is the grade of the street. In sag locations, the slope of the energy grade line is assumed to be 0.003 ft/ft.

Table 7.1.4-1 Hydraulic Conveyance of Gutter and Street

Spread on Street (ft)	Hydraulic Conveyance, K (cfs)			
	Type I Combined Curb and Gutter		Standard Gutter	
	$S_x = 1.6\%$	$S_x = 3.1\%$	$S_x = 1.6\%$	$S_x = 3.1\%$
0	2.90	2.90	2.76	2.76
1	3.84	4.94	3.81	5.06
2	5.06	7.95	5.18	8.51
3	6.65	12.20	6.97	13.36
4	8.69	17.91	9.24	19.86
5	11.24	25.29	12.07	
6	14.36	34.53	15.52	
7	18.12	45.81	19.65	
8	22.57	59.30	24.52	
9	27.76	75.18	30.17	
10	33.73	93.59		
11	40.55	114.70		
12	48.26			
13	56.90			
14	66.52			
15	77.16			
16	88.87			
17	101.69			
18	115.65			
19	130.81			
20	147.20			
21	164.86			
22	183.82			

Notes: S_x = cross-slope of roadway. Tabulated K are based on n = 0.015.

Source: Report No. K-TRAN: KU-99-1

7.1.5 Capacity of Inlets on Grade

Table 7.1.5-1 Captured Discharges for Concrete Gutter Inlet, Type B Gutter Inlet and Type 12 Combination Inlet

Total Discharge (cfs)	Captured Discharge (cfs)					
	Concrete Gutter Inlet		Type B Gutter Inlet		Type 12 Combination Inlet	
	S_x=1.6%	S_x=3.1%	S_x=1.6%	S_x=3.1%	S_x=1.6%	S_x=3.1%
0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.20	0.20	0.20	0.17	0.19	0.20	0.20
0.40	0.39	0.38	0.33	0.37	0.40	0.40
0.60	0.54	0.56	0.48	0.54	0.57	0.60
0.80	0.67	0.73	0.61	0.71	0.73	0.79
1.00	0.80	0.89	0.75	0.87	0.87	0.96
1.20	0.91	1.05	0.87	1.03	1.02	1.12
1.40	1.02	1.19	1.00	1.18	1.15	1.27
1.60	1.13	1.33	1.11	1.33	1.28	1.42
1.80	1.23	1.47	1.23	1.47	1.41	1.55
2.00	1.32	1.59	1.34	1.61	1.53	1.69
2.20	1.42	1.72	1.44	1.75	1.65	1.82
2.40	1.50	1.83	1.55	1.88	1.76	1.94
2.60	1.59	1.94	1.65	2.01	1.87	2.06
2.80	1.67	2.05	1.74	2.14	1.98	2.18
3.00	1.75	2.15	1.84	2.26	2.09	2.29
3.20	1.83	2.25	1.93	2.39	2.19	2.40
3.40	1.90	2.34	2.02	2.50	2.28	2.51
3.60	1.97	2.43	2.11	2.62	2.38	2.61
3.80	2.04	2.51	2.19	2.73	2.47	2.71
4.00	2.11	2.59	2.27	2.84	2.56	2.81

Note: This table is applicable to grades from 0.5% to 5.0%. S_x = cross-slope of roadway

Source: Report No. K-TRAN: KU-99-1

Table 7.1.5-2 Captured Discharges for 5.0-ft Type 22 Curb Inlet on Roadway with Cross-slope of 1.6%

Total Discharge (cfs)	Captured Discharge (cfs)				
	0.5% Grade	1% Grade	2% Grade	3% Grade	5% Grade
0.00	0.00	0.00	0.00	0.00	0.00
0.20	0.20	0.20	0.20	0.20	0.20
0.40	0.40	0.40	0.40	0.40	0.40
0.60	0.59	0.59	0.59	0.58	0.58
0.80	0.77	0.77	0.75	0.75	0.74
1.00	0.95	0.93	0.91	0.89	0.87
1.20	1.11	1.08	1.05	1.03	0.99
1.40	1.26	1.22	1.17	1.14	1.09
1.60	1.41	1.35	1.29	1.25	1.18
1.80	1.55	1.48	1.39	1.34	1.26
2.00	1.68	1.59	1.48	1.42	1.33
2.20	1.80	1.69	1.57	1.50	1.38
2.40	1.92	1.79	1.65	1.56	1.43
2.60	2.03	1.88	1.72	1.62	1.48
2.80	2.14	1.97	1.78	1.68	1.52
3.00	2.24	2.05	1.84	1.72	1.55
3.20	2.33	2.12	1.89	1.77	1.58
3.40	2.42	2.19	1.94	1.80	1.60
3.60	2.51	2.25	1.98	1.84	1.62
3.80	2.59	2.31	2.02	1.87	1.64
4.00	2.67	2.36	2.06	1.90	1.66

Source: Report No. K-TRAN: KU-99-1

Table 7.1.5-3 Captured Discharges for 5.0-ft Type 22 Curb Inlet on Roadway with Cross-slope of 3.1%

Total Discharge (cfs)	Captured Discharge (cfs)				
	0.5% Grade	1% Grade	2% Grade	3% Grade	5% Grade
0.00	0.00	0.00	0.00	0.00	0.00
0.20	0.20	0.20	0.20	0.20	0.20
0.40	0.40	0.40	0.40	0.40	0.40
0.60	0.59	0.59	0.59	0.58	0.58
0.80	0.78	0.77	0.75	0.75	0.73
1.00	0.95	0.93	0.91	0.89	0.86
1.20	1.12	1.09	1.05	1.03	0.97
1.40	1.27	1.23	1.17	1.14	1.06
1.60	1.42	1.37	1.29	1.25	1.14
1.80	1.56	1.49	1.39	1.34	1.21
2.00	1.70	1.61	1.48	1.42	1.26
2.20	1.83	1.72	1.57	1.50	1.31
2.40	1.95	1.82	1.65	1.56	1.35
2.60	2.07	1.92	1.72	1.62	1.39
2.80	2.18	2.00	1.78	1.68	1.42
3.00	2.28	2.09	1.84	1.72	1.44
3.20	2.38	2.17	1.89	1.77	1.47
3.40	2.48	2.24	1.94	1.80	1.48
3.60	2.57	2.31	1.98	1.84	1.50
3.80	2.65	2.37	2.02	1.87	1.51
4.00	2.74	2.43	2.06	1.90	1.52

Source: Report No. K-TRAN: KU-99-1

Table 7.1.5-4 Captured Discharges for 10.0-ft Type 22 Curb Inlet

Total Discharge (cfs)	Captured Discharge (cfs)					
	S_x = 1.6%	S_x = 3.1%				
	0.5% to 5% Grade	0.5% Grade	1% Grade	2% Grade	3% Grade	5% Grade
0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.50	0.48	0.50	0.50	0.50	0.50	0.50
1.00	0.93	1.00	0.99	0.99	0.99	0.99
1.50	1.35	1.47	1.46	1.45	1.45	1.44
2.00	1.74	1.91	1.90	1.87	1.86	1.84
2.50	2.10	2.33	2.31	2.26	2.24	2.20
3.00	2.43	2.73	2.69	2.62	2.58	2.53
3.50	2.74	3.10	3.05	2.95	2.89	2.82
4.00	3.03	3.46	3.39	3.26	3.18	3.08
4.50	3.30	3.79	3.71	3.54	3.44	3.32
5.00	3.55	4.11	4.00	3.80	3.68	3.54
5.50	3.79	4.41	4.28	4.04	3.90	3.73
6.00	4.01	4.69	4.54	4.26	4.10	3.90
6.50	4.21	4.95	4.79	4.47	4.28	4.06
7.00	4.39	5.21	5.02	4.65	4.45	4.20
7.50	4.57	5.45	5.24	4.83	4.60	4.33
8.00	4.73	5.67	5.44	4.99	4.74	4.44
8.50	4.88	5.88	5.63	5.14	4.86	4.54
9.00	5.02	6.08	5.80	5.27	4.98	4.64
9.50	5.15	6.27	5.97	5.40	5.08	4.72
10.00	5.27	6.45	6.13	5.51	5.18	4.80

Note: S_x = cross-slope of roadway

Source: Report No. K-TRAN: KU-99-1

Table 7.1.5-5 Captured Discharges for 15.0-ft Type 22 Curb Inlet

Total Discharge (cfs)	Captured Discharge (cfs)	
	S_x = 1.6%	S_x = 3.1%
0.00	0.00	0.00
0.50	0.50	0.50
1.00	1.00	1.00
1.50	1.47	1.50
2.00	1.91	2.00
2.50	2.31	2.48
3.00	2.70	2.93
3.50	3.06	3.36
4.00	3.40	3.76
4.50	3.72	4.14
5.00	4.03	4.50
5.50	4.32	4.85
6.00	4.60	5.17
6.50	4.86	5.48
7.00	5.10	5.78
7.50	5.33	6.06
8.00	5.55	6.32
8.50	5.75	6.57
9.00	5.94	6.81
9.50	6.12	7.03
10.00	6.29	7.24

Note: This table is applicable to grades from 0.5% to 5.0%. S_x = cross-slope of roadway

Source: Report No. K-TRAN: KU-99-1

7.1.6 Capacity of Inlets in Sag Locations

Table 7.1.6-1 provides the captured discharge as a function of the depth of water for the standard inlets in sag locations.

Table 7.1.6-1 Capacities of KDOT Inlets in Sag Locations

Depth at edge of pavement (in.)	Captured Discharge (cfs)					
	Concrete Gutter Inlet	Type B	Type 12	Type 22, 5.0 ft	Type 22, 10.0 ft	Type 22, 15.0 ft
0.00	0.26	0.56	1.28	1.28	1.28	1.28
0.25	0.41	0.72	1.48	1.50	1.55	1.60
0.50	0.61	0.92	1.72	1.77	1.91	2.04
0.75	0.84	1.14	1.98	2.08	2.32	2.56
1.00	1.10	1.39	2.26	2.41	2.78	3.16
1.25	1.39	1.65	2.57	2.77	3.29	3.81
1.50	1.71	1.94	2.88	3.16	3.84	4.52
1.75	2.05	2.24	3.22	3.56	4.42	5.28
2.00		2.55	3.57	3.99	5.04	6.09
2.25		2.88	3.94	4.44	5.69	6.95
2.50		3.23	4.31	4.90	6.37	7.84
2.75		3.59	4.71	5.39	7.08	8.77
3.00		3.96	5.11	5.89	7.82	9.75
3.25		4.35	5.53	6.40	8.58	10.76
3.50		4.74	5.96	6.93	9.37	11.80
3.75		5.15	6.40	7.48	10.18	12.88
4.00		5.58	6.86	7.98	11.02	13.99
4.25		6.01	7.32	8.12	11.88	15.13

Source: Report No. K-TRAN: KU-99-1

7.1.7 Discharge for Inlet Design

Design discharges for curb and gutter inlets should be computed by the Rational method. The time of concentration to the inlet should be no less than 5 minutes.

7.1.8 Example: Design of Street Drainage

An arterial street in Shawnee County has a design speed of 40 mi/h. The edge treatment is the Type I combined curb and gutter. The total width (back of curb to back of curb) is 53.0 ft (4 -lanes @ 12 ft. + C & G @ 2.5 ft.) and the cross-slope is 1.6%. The longitudinal slope is 1.0%. The street drainage system is being designed to handle the runoff from the street. Runoff from adjacent areas does not flow into the street. The preferred inlet is the 5.0-ft Type 22 combination inlet.

Problem 1:

Determine whether the 5.0-ft Type 22 combination inlet would perform satisfactorily in this situation.

Solution:

Obtain the allowable spread on the driving lane from Table 7.1.1-1.

Allowable spread = 6.0 ft

Obtain the hydraulic conveyance of the gutter and street at the allowable spread from Table 7.1.4-1.

K = 14.36 cfs

Compute the discharge in the gutter and street at the allowable spread with Equation 7-1.

$$Q = K\sqrt{S} = 14.36\sqrt{0.01} = 1.44 \text{ cfs}$$

Determine the discharge captured by the inlet at the allowable spread from Table 7.1.5-2 by interpolation.

Captured Q = 1.25 cfs

The captured discharge exceeds 80% of the discharge in the street and gutter at the allowable spread, so the inlet type and size are satisfactory.

Problem 2:

Determine the maximum inlet spacing for the 5.0-ft Type 22 combination inlets from Problem 1.

Solution:

Find the maximum drainage area between inlets by the Rational method (Section 3.2).

The discharge from drainage area between inlets should equal the discharge captured by the inlet at the allowable spread.

$$Q = 1.25 \text{ cfs}$$

Obtain the runoff coefficient for the street from Table 3.2.4-1.

$$C = 0.90$$

The recurrence interval is 10 years (Section 7.1.1).

The time of concentration to the inlet is 5 minutes (Section 7.1.7).

Obtain the 10-year, 5-minute rainfall intensity for Shawnee County from KDOT's *Rainfall Intensity Tables for Counties in Kansas (2014)*.

$$i = 8.34 \text{ in./hr}$$

Compute the drainage area between inlets with Equation 2-1.

$$Q = C\,i\,A$$

$$A = \frac{Q}{C\,i} = \frac{1.25}{0.90(8.34)} = 0.167 \text{ ac} = 7{,}275 \text{ ft}^2$$

Compute the maximum inlet spacing.

$$\text{Maximum inlet spacing} = \frac{\text{maximum drainage area}}{\text{half width of street}} = \frac{7275}{(53.0/2)} = 274 \text{ ft}$$

Use a maximum inlet spacing of 270 ft.

Problem 3:

Determine whether the 5.0-ft Type 22 combination inlet would perform satisfactorily in a sag location.

Solution:

Allowable spread on driving lane = 6.0 ft (Table 7.1.1-1)

Hydraulic conveyance of gutter and street at allowable spread = 14.36 cfs (Table 7.1.4-1)

Assume that the minimum slope of the energy grade line in vicinity of the low point is 0.003 ft/ft. (Section 7.1.4)

Compute the discharge in the gutter and street at the allowable spread with Equation 7-1.

$$Q = K\sqrt{S} = 14.36\sqrt{0.003} = 0.79 \text{ cfs}$$

Compute the total discharge to the low-point inlet from two directions at the allowable spread.

2 x 0.79 cfs = 1.58 cfs

Compute the depth at the edge of the pavement for the allowable spread.

Depth = spread x cross-slope = 6.0 (0.016) = 0.10 ft = 1.20 in.

Obtain the capacity of the inlet in a sag location at the allowable spread from Table 7.1.6-1 by interpolation.

Captured Q = 2.70 cfs

The captured discharge exceeds the design flow at the allowable spread, so the inlet type and size are satisfactory.

Problem 4:

Determine the maximum inlet spacing in the vicinity of the low point.

Solution:

Find the maximum drainage area between inlets by the Rational method (Section 3.2).

Q = 0.79 cfs (discharge at the allowable spread)

$C = 0.90$; $T_c = 5$ min; $i = 8.34$in./h (from Problem 2)

Compute the drainage area between inlets with Equation 3-1.

$$Q = C\,i\,A$$

$$A = \frac{Q}{C\,i} = \frac{0.79}{0.90(8.34)} = 0.105 \text{ ac} = 4574 \text{ ft}^2$$

Compute the maximum inlet spacing.

$$\text{Maximum inlet spacing} = \frac{\text{maximum drainage area}}{\text{half width of street}} = \frac{4574}{(53.0/2)} = 173 \text{ ft}$$

Use a maximum inlet spacing of 170 ft in the vicinity of the low point.

7.2 STORM SEWERS

7.2.1 Design Criteria

The recurrence interval for storm sewers is generally 10 years. Storm sewers should be designed to flow nearly full, but not surcharged, at the 10-year recurrence interval. Minor, localized surcharging may be acceptable provided that the hydraulic grade line remains below the inlet openings. It is desirable for the velocity at full flow to be at least 3.0 ft/s to provide for transport of sediment and debris. The pipe size should not decrease in the downstream direction.

7.2.2 System Layout

1. Pipes should be placed no deeper than necessary. Refer to Section 8 for minimum cover requirements.

2. The slope of a pipe should be approximately equal to the average slope of the ground over the pipe, except where a steeper slope is needed to attain the minimum velocity.

3. The tops of the pipes should be aligned at junctions where practical.

7.2.3 Preliminary Design

The preliminary sizing of pipes starts at the upper end of the system and proceeds in the downstream direction. Before sizing the pipe directly downstream of a junction, the pipes upstream of the junction should be sized. Refer to Section 2.7 for the policy on minimum pipe sizes.

For each pipe, the basic steps are:

1. Compute the design discharge for the pipe by the Rational Method.
2. Determine the required pipe size.
3. Compute the travel time through the pipe.

The time of concentration in the Rational Method is the sum of the time of concentration for the inlet and the total pipe-flow time to the design point (the upstream end of the pipe) by the longest route.

Use Equation 7-2 to compute the minimum pipe size required to carry the design flow.

$$D = 16.0\left(\frac{Qn}{\sqrt{S}}\right)^{3/8} \qquad (7\text{-}2)$$

where: D = minimum pipe diameter required to convey design flow (in.)

Q = design flow (cfs)

n = Manning's roughness coefficient (see Table 6.3.4-2)

S = pipe slope (ft/ft)

Select the smallest standard pipe size that exceeds this minimum size. Use Equation 7-3 to compute the velocity for full flow in a pipe of the selected size.

$$V = \frac{0.1128}{n}D^{2/3}S^{1/2} \qquad (7\text{-}3)$$

where: V = velocity (ft/s)

D = diameter of pipe (standard size) (in.)

S = pipe slope (ft/ft)

n = Manning's roughness coefficient (see Table 6.3.4-2)

If this velocity is less than 3.0 ft/s, increase the slope of the pipe as needed to obtain the minimum velocity of 3.0 ft/s. Use Equation 7-4 to compute the travel time in the pipe.

$$T_t = \frac{1}{60}\,\frac{L}{V} \qquad (7\text{-}4)$$

where: T_t = travel time in pipe (minutes)

L = length of pipe (ft)

V = velocity for full flow, from Equation 7-3 (ft/s)

7.2.4 Hydraulic Grade Line and Final Design

Compute an approximate hydraulic grade line (HGL) for the preliminary design, starting from the outfall and proceeding upstream. The design should be such that HGL is below all inlet openings and below the ground surface along the entire length of the storm sewer. Modify the preliminary design as needed to meet this requirement.

Note: The HGL is typically determined by computing the energy grade line (EGL) and then subtracting the velocity head from the EGL. Because of the difficultly in computing the velocity head in an access manhole, it is acceptable to use the energy grade line as a conservative estimate of the hydraulic grade line for storm drain design. The following procedure is the standard procedure for computing the EGL; however, it is presented here as an acceptable method for determining the approximate HGL.

Compute the approximate HGL as follows:

1. Determine whether a 10-year discharge in the receiving stream would submerge the storm-sewer outfall. If so, determine the depth of submergence. A separate study of the receiving stream may be required. If the drainage area of the receiving stream at the storm-sewer outfall exceeds 100 times the drainage area of the storm sewer, the depth of submergence should be computed for a more frequent event; refer to the guidelines for frequency mixing in the Bridge Design Manual. The HGL at the downstream end of the terminal pipe should be set at the crown of the pipe or the 10-year water level in the receiving stream, whichever is higher.

2. Compute the friction loss through each pipe using Equation 7-5.

$$h_f = 2.64 \times 10^6 \, L \left(\frac{Q\,n}{D^{8/3}}\right)^2 \qquad (7\text{-}5)$$

where: h_f = friction loss for surcharged flow (ft),

L = length of pipe (ft),

Q = design discharge (cfs)

n = Manning's roughness coefficient

D = diameter of pipe (in.)

3. Compute the local head loss through each inlet, access manhole, and junction box using Equation 15-6. Determine the local-loss coefficient from Table 7.2.4-1. If the structure is a junction box or an inlet with multiple incoming pipes, compute separate local losses for each branch.

$$h_m = K \frac{V^2}{2g} \qquad (7\text{-}6)$$

where: h_m = local loss through structure (ft)

K = local-loss coefficient (Table 7.2.4-1)

V = velocity for surcharged flow in outgoing pipe (ft/s)
= (design discharge) / (cross-sectional area of pipe)

g = gravitational constant (32.2 ft/s^2)

Table 7.2.4-1 Local-Loss Coefficients for Flow through Inlet Boxes, Access Manholes, and Junction Boxes

Structure Type	K
Inlet box with no incoming pipe	1.5
Inlet box with one or more incoming pipes	$0.5 + 1.0 \sin \theta$
Access manhole with one incoming pipe	$0.2 + 0.8 \sin \theta$
Junction box with multiple incoming pipes	$0.5 + 1.0 \sin \theta$

Note: θ = angle between incoming outgoing pipes (180º for no change in direction)

4. Compute the HGL at all structures, starting from the outfall and proceeding upstream, by applying the following rules repeatedly.

   a. The HGL at the upstream end of a pipe is the sum of the HGL at the downstream end of the pipe and the friction loss through the pipe. If this calculation yields an HGL that is below the crown of the pipe, set the HGL at the crown of the pipe. It is not necessary to compute a non-uniform water-surface profile through the pipe.

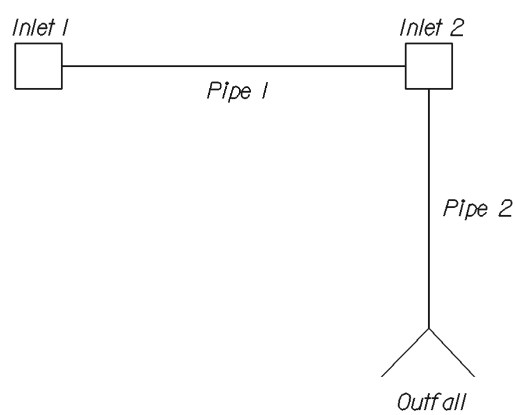
   b. The HGL at a structure is the sum of the HGL at the upstream end of the outgoing pipe and the local loss through the structure. The HGL at the downstream end of the incoming pipe is the same as the HGL at the structure.

7.2.5 Example: Design of a Storm Sewer

Problem:

Figure 7.2.5-1 shows the layout for a planned storm sewer that will discharge to an open channel. The location is Shawnee County. Pipe 1 is 200 ft long and pipe 2 is 170 ft long. Inlet 1 has a drainage area of 0.98 ac, 70% of which is impervious. Inlet 2 has a drainage area of 0.74 ac, 70% of which is impervious. The ground elevations are 854.90 ft at inlet 1 (top of inlet), 853.92 ft at inlet 2 (top of inlet), and 852.28 ft at the outfall (top of bank). The minimum cover for the pipes is 2.5 ft. The 10-year water level in the channel at the outfall is 850.04 ft. The pipe material is reinforced concrete. Size the pipes, determine the invert elevations, and compute the hydraulic grade line.

Figure 7.2.5-1 Layout of Storm Sewer



Solution:

1. Pipe 1

   a. Compute the design flow by the Rational method (Section 3.2).

      Pipe 1 conveys flow from inlet 1 (0.98 ac, 70% impervious).

      Drainage area = 0.98 ac

      Impervious drainage area = 0.70 (0.98 ac) = 0.69 ac

      Recurrence interval = 10 years (Section 7.2.1)

      Runoff coefficients for impervious and pervious surfaces from Table 3.2.4-1.

      C = 0.90 for impervious surfaces

      C = 0.40 for pervious surfaces

      Area-weighted average runoff coefficient

$$C = \frac{0.90(0.69\ ac) + 0.40(0.98\ ac - 0.69\ ac)}{0.98ac} = 0.75$$

Time of concentration: T_c for pipe 1 = T_c for inlet 1 = 5 min (Section 7.1.7)

Obtain the 10-year, 5-minute rainfall intensity for Shawnee County from KDOT's *Rainfall Intensity Tables for Counties in Kansas (2014).*

i = 8.34 in./h

Compute the design flow with Equation 3-1.

Q = C i A = 0.75 (8.34) (0.98) = 6.1 cfs

b. Compute required pipe size.

Obtain the Manning's n for RCP pipe from Table 6.3.4-2.

n = 0.012

Set the pipe slope equal to the average ground slope between inlets 1 and 2.

S = (854.90 – 853.92) / 200 = 0.0049 ft/ft

Compute the required pipe diameter with Equation 7-2.

$$D = 16.0\left(\frac{Qn}{\sqrt{S}}\right)^{3/8} = 16.0\left[\frac{6.1(0.012)}{\sqrt{0.0049}}\right]^{3/8} = 16.3\text{ in.}$$

Specify an 18-in. pipe, the smallest standard size that exceeds the required diameter.

Check the velocity for full, non-surcharged flow with Equation 7-3.

$$V = \frac{0.1128}{n}D^{2/3}S^{1/2} = \frac{0.1128}{0.012}(18)^{2/3}(0.0049)^{1/2} = 4.5\text{ ft/s}$$

This velocity exceeds 3.0 ft/s, so the pipe slope is sufficiently steep.

Compute the travel time in the pipe with Equation 7-4.

$$T_t = \frac{1}{60}\frac{L}{V} = \frac{1}{60}\left(\frac{200}{4.5}\right) = 0.7 \text{ min}$$

c. Compute the elevations of the pipe crown (inside top) and flowline.

Obtain wall thickness for 18-in. RCP pipe from Table 8.2-1.

Wall thickness = 2.5 in.

Crown elevation = (ground elev.) – (cover) – (wall thickness)

Flowline elev. = (crown elevation) – (pipe diameter)

Upstream crown elevation = 854.90 – 2.50 – 2.5/12 = 852.19 ft

Upstream flowline elevation = 852.19 – 18/12 = 850.69 ft

Downstream crown elevation = 853.92 – 2.50 – 2.5/12 = 851.21 ft

Downstream flowline elevation = 851.21 – 18/12 = 849.71 ft

2. Pipe 2

a. Compute the design flow by the Rational Method.

Pipe 2 conveys flow from inlet 1 (0.98 ac, 70% impervious) and inlet 2 (0.74 ac, 70% impervious).

Total drainage area for pipe 2 = 0.98 ac + 0.74 ac = 1.72 ac

Impervious drainage area for pipe 2 = 0.70 (0.98) + 0.70 (0.74) = 1.20 ac

Compute the area-weighted average runoff coefficient.

$$C = \frac{0.90(1.20\,ac) + 0.40(1.72\,ac - 1.20\,ac)}{1.72\,ac} = 0.75$$

Determine the time of concentration and rainfall intensity

T_c for pipe 2 = T_c for inlet 1 + travel time in pipe 1

T_c = 5.0 min + 0.7 min = 5.7 min

i = 7.71 in./h (10-year, 6-minute rainfall intensity for Shawnee County)

Compute the design flow with Equation 3-1.

Q = C i A = 0.75 (7.71) (1.72) = 9.9 cfs

b. Compute required pipe size.

Set the pipe slope equal to the average ground slope between inlet 2 and outfall.

S = (853.92 – 852.28) / 170 = 0.0096 ft/ft

Compute the required pipe diameter with Equation 7-2.

$$D = 16.0\left(\frac{Qn}{\sqrt{S}}\right)^{3/8} = 16.0\left[\frac{9.9(0.012)}{\sqrt{0.0096}}\right]^{3/8} = 17.2 \text{ in.}$$

Specify an 18-in. pipe, the smallest standard size that exceeds the required diameter.

Check the velocity for full, non-surcharged flow with Equation 7-3.

$$V = \frac{0.1128}{n}D^{2/3}S^{1/2} = \frac{0.1128}{0.012}(18)^{2/3}(0.0096)^{1/2} = 6.3 \text{ ft/s}$$

This velocity exceeds 3.0 ft/s, so the pipe slope is sufficiently steep.

c. Compute the elevations of the pipe crown and flowline.

Crown elev. = (ground elev.) – (cover) – (wall thickness)

Flowline elev. = (crown elev.)– (pipe diameter)

Upstream crown elevation = 853.92 – 2.50 – 2.5/12 = 851.21 ft

Upstream flowline elevation = 851.21 – 18/12 = 849.71 ft

Downstream crown elevation = 852.28 – 2.50 – 2.5/12 = 849.57 ft

Downstream flowline elevation = 849.57– 18/12 = 848.07 ft

3. Hydraulic Grade Line

a. Determine the HGL at the downstream end of the terminal pipe (pipe 2).

10-year water level in outfall channel = 850.04 ft (given)

Pipe crown elev. at outfall = flowline elev. + pipe diam. = 848.07 + 18/12 = 849.57 ft

The 10-year water level in the receiving stream is above the pipe crown at the outfall, so the HGL at the downstream end of pipe 2 is the 10-year water level in the receiving stream.

HGL at downstream end of pipe 2 = 850.04 ft

b. Compute the friction loss through each pipe using Equation 7-5.

$$\text{Pipe 1: } h_f = 2.64 \times 10^6 \, L \left(\frac{Qn}{D^{8/3}}\right)^2 = 2.64 \times 10^6 \,(200) \left[\frac{6.1(0.012)}{18^{8/3}}\right]^2 = 0.57 \text{ ft}$$

$$\text{Pipe 2: } h_f = 2.64 \times 10^6 \,(170) \left[\frac{9.9(0.012)}{18^{8/3}}\right]^2 = 1.28 \text{ ft}$$

c. Compute the local head loss through each inlet using Equation 7-6. Determine the local-loss coefficients from Table 7.2.4-1.

Inlet 1:

K = 1.5 (Table 7.2.4-1; inlet box with no incoming pipes)

Compute the velocity in the outgoing pipe (pipe 1)

$$V = \frac{Q}{A} = \frac{Q}{\left(\frac{\pi}{4}D^2\right)} = \frac{6.1}{\frac{\pi}{4}\left(\frac{18}{12}\right)^2} = 3.45 \text{ ft/s}$$

Compute the local head loss with Equation 7-6.

$$h_m = K\frac{V^2}{2g} = 1.5\frac{(3.45)^2}{2(32.2)} = 0.28 \text{ ft}$$

Inlet 2:

Angle between incoming and outgoing pipes is 90°

K = 0.5 + 1.0 sin 90° (Table 7.2.4-1; inlet box with one incoming pipes)

Compute the velocity in the outgoing pipe (pipe 1)

$$V = \frac{Q}{A} = \frac{Q}{\left(\frac{\pi}{4}D^2\right)} = \frac{9.9}{\frac{\pi}{4}\left(\frac{18}{12}\right)^2} = 5.60 \text{ ft/s}$$

Compute the local head loss with Equation 7-6.

$$h_m = K\frac{V^2}{2g} = 1.5\frac{(5.60)^2}{2(32.2)} = 0.73 \text{ ft}$$

d. Compute the HGL at all structures, starting from the outfall and proceeding upstream.

HGL at upstream end of pipe 2

= (HGL at downstream end of pipe 2) + (friction loss through pipe 2)

= 850.04 + 1.28 = 851.32 ft (0.11 ft above crown)

HGL at inlet 2

= (HGL at upstream end of pipe 2) + (local loss through inlet 2)

= 851.32 + 0.73 = 852.05 ft (1.87 ft below ground level)

HGL at downstream end of pipe 1 = HGL at inlet 2 = 852.05 ft

HGL at upstream end of pipe 1

= (HGL at downstream end of pipe 1) + (friction loss through pipe 1)

= 852.05 + 0.57 = 852.62 ft (0.43 ft above crown)

HGL at inlet 1

= (HGL at upstream end of pipe 1) + (local loss through inlet 1)

= 852.62 + 0.28 = 852.90 ft   (2.00 ft below ground level)

HGL is below the openings of both inlets, so the design is satisfactory.

7.3 REFERENCES

AASHTO (2014). *Drainage Manual*, Chapter 13, "Storm Drainage Systems."

AASHTO (2007). *Highway Drainage Guidelines*, Chapter 9, "Storm Drain Systems."

Chow, V. T. (1959). *Open Channel Hydraulics*, McGraw-Hill.

Federal Highway Administration (2009). *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22.

McEnroe, B. M., R. P. Wade and A. K. Smith (1999). *Hydraulic Performance of Curb and Gutter Inlets*, Report No. K-TRAN: KU-99-1, KDOT.