CHAPTER five

Hydraulic Design of Storm Sewers

INTRODUCTION

Storm sewers may be designed as either open channels, where there is a free water surface, or as pipes, where the flow is under surcharged conditions (pressure flow). When the storm sewer system is to be designed as a pressure pipe, the hydraulic grade line can not exceed the floor level of adjacent basements, or catch basin grate opening elevations, if surcharge conditions would create unacceptable flooding or structural damage.

Regardless of whether the sewer system is to be designed as an open channel or pressure system, a thorough hydraulic analysis should be performed to assure that the system operates efficiently. A simplistic approach to the design of storm sewers, with the design and sizing of pipes and appurtenances derived from nomographs or basic hydraulic flow equations, has too often been used.

As a result, excessive surcharging has been experienced in many instances due to improper design of the hydraulic structures. This in turn has led to flood damage, both surface and structural, when service connections have been made to the storm sewer. Overloading of the sewer system may occur in upper reaches, while lower segments may be flowing well below capacity, because of the inability of the upper reaches to transport the flow. Conversely, downstream surcharging can create problems while upper segments are flowing well below capacity.

An efficient, cost-effective storm drain system cannot be designed without a complete and proper hydraulic analysis.

This chapter outlines the basic hydraulic principles for open channel and pipe flow. Losses (friction and form) within the sewer system, the hydraulics of storm water inlets, and the hydraulics of subdrains are also discussed. Manual calculations for designing a storm drainage system are presented and an overview of several commonly used computer programs that may be used to design sewer systems is also given.

HYDRAULICS OF STORM SEWERS

Classification of Channel Flow

Channel flow is distinguished from closed-conduit or pipe flow by the fact that the cross-section of flow is not dependent solely on the geometry of the pipe. It also depends on the free surface (or depth), which varies with respect to space and time and is a function of discharge. As a result, various categories of flow can be identified:

STEADY flow exhibits characteristics at a point that are constant with respect to time. Flow subject to very slow change may be assumed to be steady with little error.

UNSTEADY flow results when a time-dependent boundary condition (tide, flood-wave or gate movement, for example) causes a change in flow and/or depth to be propagated through the system.

UNIFORM flow occurs when the velocity is the same in magnitude and direction at every point in the pipe. Uniform flow is also usually assumed to occur when the velocity at corresponding points in the cross-section is the same along the length of the channel. Note that uniform flow is possible only if:

- flow is steady, or nearly so;
- the channel is prismatic (i.e., has the same cross-sectional shape at all sections);
- depth is constant along the length of the channel; and
- the slope is equal to the energy gradient.

NON-UNIFORM or VARIED flow occurs when any of the requirements for uniform flow are not satisfied. Varied flow may be further sub-classified depending on the abruptness of the variation:

GRADUALLY VARIED flow occurs when depth changes occur over long distances such as the flow profiles or backwater profiles that occur between distinct reaches of uniform flow.

RAPIDLY VARIED flow occurs in the vicinity of transitions caused by relatively abrupt changes in channel geometry or where a hydraulic jump occurs.

In the design of sewer systems, the flow, except where backwater or surcharging may occur, is generally assumed to be steady and uniform.

Figure 5.1 illustrates various typical occurrences of the different classes of flow.

Laws of Conservation

Fluid mechanics is based on the law of conservation applied to the mass, energy and momentum of a fluid in motion. Full details can be found in any text on the subject. At this point, it is sufficient to note that:

a) Conservation of mass reduces to a simple statement of continuity for fluids in which the density is essentially constant.
b) Conservation of energy is usually stated as the Bernoulli equation, which is discussed below.
c) Conservation of momentum is significant in transitions where there are local and significant losses of energy, such as across a hydraulic jump.

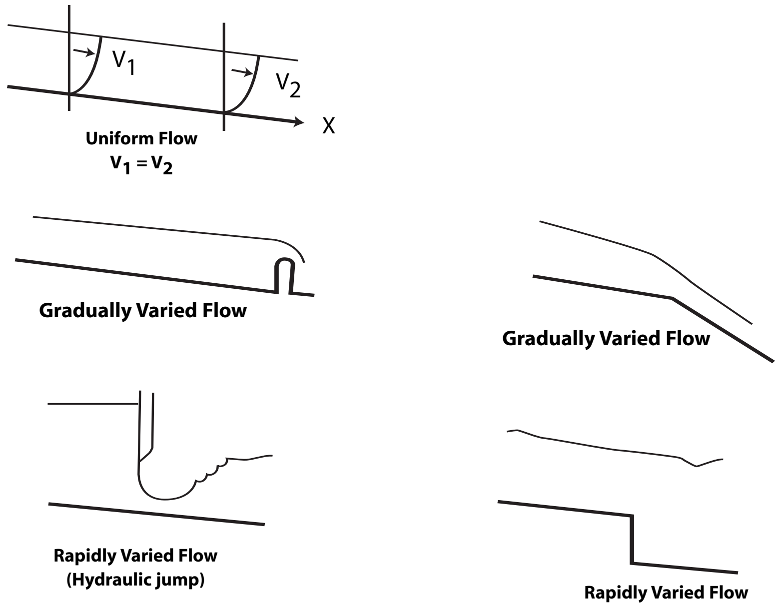


Figure 5.1 Different classes of open channel flow.

Bernoulli Equation

The law of conservation of energy as expressed by the Bernoulli Equation is the basic principle most often used in hydraulics. This equation may be applied to any pipe with a constant discharge. All the friction flow formulae, such as those by Manning, Cutter, and Hazen-Williams, have been developed to express the rate of energy dissipation as it applies to the Bernoulli Equation. The theorem states that the energy head at any cross-section must equal that in any other downstream section plus the intervening losses.

In open channels, the flow is primarily controlled by the gravitational action on moving fluid, which overcomes the hydraulic energy losses. The Bernoulli Equation, defining the hydraulic principles involved in open channel flow (Figure 5.2), is as follows:

$$H = y + \frac{V^2}{2g} + Z + h_f$$

where:
- H = Total energy head, ft
- y = Water depth, ft
- V = Mean velocity, ft/s
- g = Gravitational constant = 32.2 ft/s^2
- Z = Height above datum, ft
- h_f = Head loss, ft

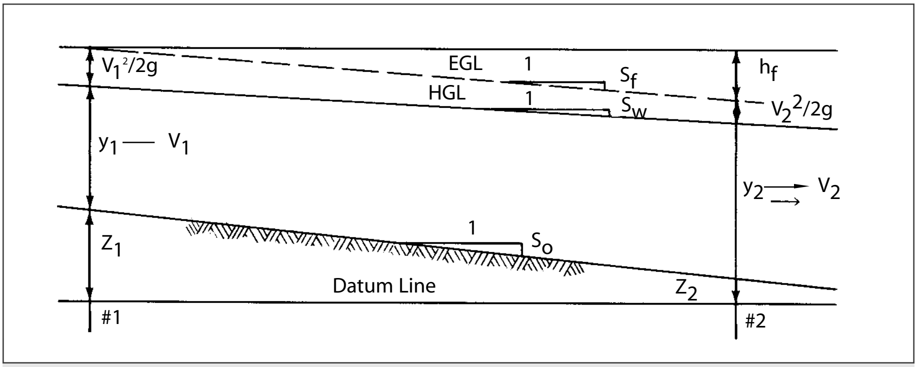


Figure 5.2 Energy in open channel flow.

Other terms shown in Figure 5.2 are defined as follows:

EGL = Energy Grade Line
HGL = Hydraulic Grade Line
S_o = Slope of channel bottom
S_f = Slope of the EGL
S_w = Slope of the HGL
$V^2/2g$ = Velocity head

In Figure 5.2, the total energy at point 1 is equal to the total energy at point 2, and thus:

$$y_1 + Z_1 + \frac{V_1^2}{2g} = y_2 + Z_2 + \frac{V_2^2}{2g} + h_f$$

For pressure or closed pipe flow, as shown in Figure 5.3, the Bernoulli Equation can be written as follows:

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_1}{\gamma} + Z_2 + h_f$$

where: P = Pressure at a given point, lb/in^2
γ = Specific weight of fluid, lb/in^3

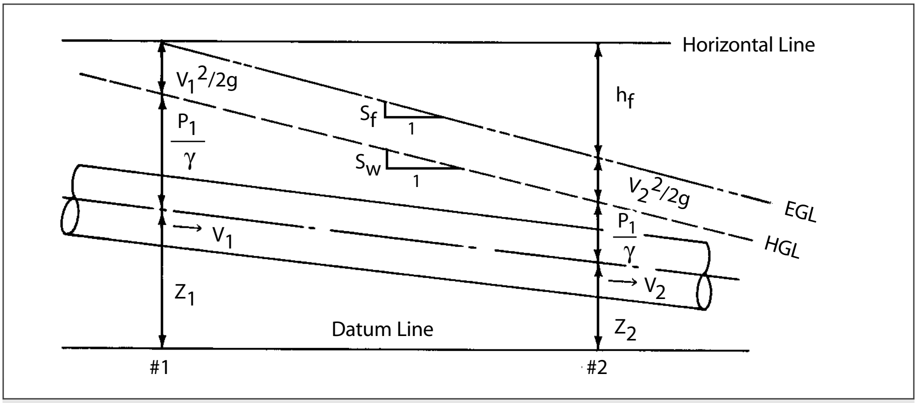


Figure 5.3 Energy in closed pipe flow.

Specific Energy

An understanding of open channel flow is aided by the concept of Specific Energy, E, which is simply the total energy when the channel bottom is taken to be the datum. This may be expressed as:

$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2}$$

Figure 5.4 shows a plot of depth of flow, y, and specific energy, E, as a function of depth of flow for a known cross-sectional shape and constant discharge Q. The minimum value of E occurs at a depth of flow termed the critical depth, y_{cr}. The critical depth is defined by setting dE/dy = 0, from which it can be shown that:

$$\frac{Q^2\ T}{gA^3} = 1$$

where the surface width, T, and cross-sectional area, A, are functions of the depth, y. Table 5.1 lists the cross-sectional areas for a number of standard pipe shapes and sizes. Tables 5.2 through 5.4 provide the means to determine the flow area, hydraulic radius and top width for round pipes flowing partly full. Tables 5.5 through 5.7 provide the same for pipe arches. Note that pipe arch values are approximate since actual pipe arch shapes vary in cross-sectional geometry. An explanation of the use of these tables is provided under Table 5.4

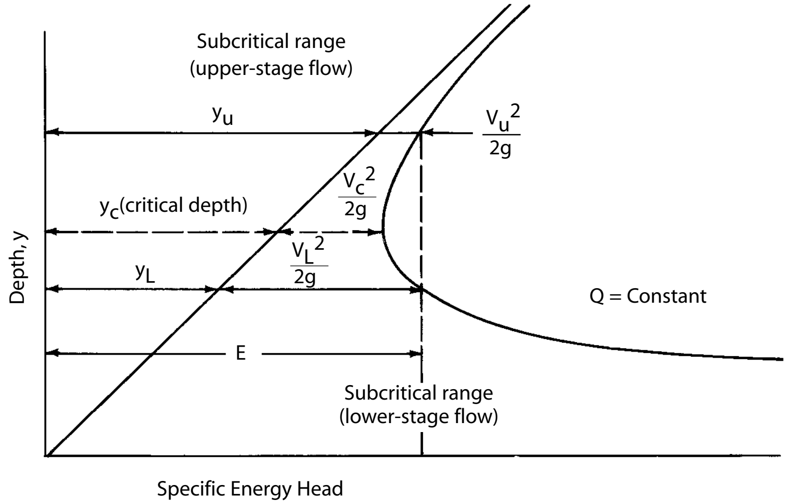


Figure 5.4 Specific energy as a function of depth.

The velocity corresponding to the critical depth is called the critical velocity and is given by:

$$\frac{V^2_{cr}\,T}{gA} = 1 \quad \text{or} \quad V_{cr} = \left(\frac{gA}{T}\right)^{\frac{1}{2}}$$

The critical velocity, and hence the critical depth, is unique to a known cross-sectional shape and constant discharge, Q.

For the special case of rectangular cross-sections, A = B•y and T = B, where B is the channel width. In this case, the above equation for critical depth reduces to:

$$\frac{Q^2}{g \bullet B^2 \bullet y^3} = 1$$

from which the critical depth is found as:

$$y_{cr} = \left(\frac{Q^2}{g \bullet B^2}\right)^{\frac{1}{3}}$$

and the corresponding critical velocity is $V_{cr} = (g \bullet y)^{\frac{1}{2}}$.

Table 5.1

Waterway areas for standard sizes of corrugated steel conduits

Round Pipe Diameter	Round Pipe Area
(in.)	(ft^2)
12	0.785
15	1.227
18	1.767
21	2.405
24	3.142
30	4.909
36	7.069
42	9.621
48	12.566
54	15.904
60	19.635
66	23.758
72	28.27
78	33.18
84	38.49
90	44.18
96	50.27
108	63.62
114	70.88
120	78.54
132	95.03
138	103.87
144	113.10
150	122.7
156	132.7
162	143.1
168	153.9
174	165.1
180	176.7
186	188.7
192	201.1
198	213.8
204	227.0
210	240.5
216	254.0
222	268.8
228	283.5
234	298.6
240	314.2
246	330.1
252	346.4
258	363.1
264	380.1
270	397.6
276	415.5
282	433.7
288	452.4
294	471.4
300	490.9

Pipe Arch (1/2 in. deep corrugation) Size	Area
(in.)	(ft^2)
17 x 13	1.1
21 x 15	1.6
24 x 18	2.2
28 x 20	2.9
35 x 24	4.5
42 x 29	6.5
49 x 33	8.9
57 x 38	11.6
64 x 43	14.7
71 x 47	18.1
77 x 52	21.9
83 x 57	26.0
Pipe Arch (1 in. deep corrugation)	
Size	Area
60 x 46	15.6
66 x 51	19.3
73 x 55	23.2
81 x 59	27.4
87 x 63	32.1
95 x 67	37.0
103 x 71	42.4
112 x 75	48.0
117 x 79	54.2
128 x 83	60.5
137 x 87	67.4
142 x 91	74.5
Structural Plate Arch (2 in. deep corrugation)	
Size	Area
6.0 x 3-2	15
7.0 x 3-8	20
8.0 x 4-2	26
9.0 x 4-8.5	33
10.0 x 5-3	41
11.0 x 5-9	50
12.0 x 6-3	59
13.0 x 6-9	70
14.0 x 7-3	80
15.0 x 7-9	92
16.0 x 8-3	105
17.0 x 8-10	119
18.0 x 8-11	126
19.0 x 9-5.5	140
20.0 x 10-0	157
21.0 x 10-6	172
22.0 x 11-0	190
23.0 x 11-6	208
24.0 x 12-0	226
25.0 x 12-6	247

Structural Plate Pipe Arch Size	Area
18-inch Corner Radius	
(ft-in.)	(ft^2)
6-1 x 4-7	22
6-4 x 4-9	24
6-9 x 4-11	26
7-0 x 5-1	28
7-3 x 5-3	31
7-8 x 5-5	33
7-11 x 5-7	35
8-2 x 5-9	38
8-7 x 5-11	40
8-10 x 6-1	43
9-4 x 6-3	46
9-6 x 6-5	49
9-9 x 6-7	52
10-3 x 6-9	55
10-8 x 6-11	58
10-11 x 7-1	61
11-5 x 7.3	64
11-7 x 7.5	67
11-10 x 7-7	71
12-4 x 7-9	74
12-6 x 7-11	78
12-8 x 8-1	81
12-10 x 8-4	85
13-5 x 8-5	89
13-11 x 8-7	93
14-1 x 8-9	97
14-3 x 8-11	101
14-10 x 9-1	105
15-4 x 9-3	109
15-6 x 9-5	113
15-8 x 9-7	118
15-10 x 9-10	122
16-5 x 9-11	126
16-7 x 10-1	131
31 in. Corner Radius	
13-3 x 9-4	97
13-6 x 102	102
14-0 x 9-8	105
14-2 x 9-10	109
14-5 x 10-0	114
14-11 x 10-2	118
15-4 x 10-4	123
15-7 x 10-6	127
15-10 x 10-8	132
16-3 x 10-10	137
16-6 x 11-0	142
17-0 x 11-2	146
17-2 x 11-4	151
17-5 x 11-6	157
17-11 x 11-8	161
18-1 x 11-10	167
18-7 x 12-0	172
18-0 x 12-2	177
19-3 x 12-4	182
19-6 x 12-6	188
19-8 x 12-8	194
19-11 x 12-10	200
20-5 x 13-0	205
20-7 x 13-2	211

Hydraulic Properties of Circular Conduits Flowing Part Full*

D = Diameter
y = Depth of flow
A = Area of flow
R = Hydraulic radius
T = Top width

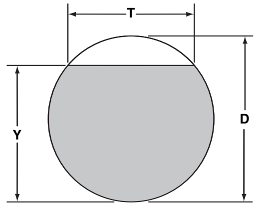


Table 5.2

Determination of area — Values of $\frac{A}{D^2}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.001	.004	.007	.011	.015	.019	.024	.029	.035
.1	.041	.047	.053	.060	.067	.074	.081	.089	.096	.104
.2	.112	.120	.128	.136	.145	.154	.162	.171	.180	.189
.3	.198	.207	.217	.226	.236	.245	.255	.264	.274	.284
.4	.293	.303	.313	.323	.333	.343	.353	.363	.373	.383
.5	.393	.403	.413	.423	.433	.443	.453	.462	.472	.482
.6	.492	.502	.512	.521	.531	.540	.550	.559	.569	.578
.7	.587	.596	.605	.614	.623	.632	.640	.649	.657	.666
.8	.674	.681	.689	.697	.704	.712	.719	.725	.732	.738
.9	.745	.750	.756	.761	.766	.771	.775	.779	.782	.784
1.0	.785									

Table 5.3

Determination of hydraulic radius — Values of $\frac{R}{D}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.007	.013	.020	.026	.033	.039	.045	.051	.057
.1	.063	.070	.075	.081	.087	.093	.099	.104	.110	.115
.2	.121	.126	.131	.136	.142	.147	.152	.157	.161	.166
.3	.171	.176	.180	.185	.189	.193	.198	.202	.206	.210
.4	.214	.218	.222	.226	.229	.233	.236	.240	.243	.247
.5	.250	.253	.256	.259	.262	.265	.268	.270	.273	.275
.6	.278	.280	.282	.284	.286	.288	.290	.292	.293	.295
.7	.296	.298	.299	.300	.301	.302	.302	.303	.304	.304
.8	.304	.304	.304	.304	.304	.303	.303	.302	.301	.299
.9	.298	.296	.294	.292	.289	.286	.283	.279	.274	.267
1.0	.250									

Table 5.4

Determination of top width — Values of $\frac{T}{D}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.199	.280	.341	.392	.436	.457	.510	.543	.572
.1	.600	.626	.650	.673	.694	.714	.733	.751	.768	.785
.2	.800	.815	.828	.842	.854	.866	.877	.888	.898	.908
.3	.917	.925	.933	.940	.947	.954	.960	.966	.971	.975
.4	.980	.984	.987	.990	.993	.995	.997	.998	.999	1.000
.5	1.000	1.000	.999	.998	.997	.995	.993	.990	.987	.984
.6	.980	.975	.971	.966	.960	.954	.947	.940	.933	.925
.7	.917	.908	.898	.888	.877	.866	.854	.842	.828	.815
.8	.800	.785	.768	.751	.733	.714	.694	.673	.650	.626
.9	.600	.572	.543	.510	.475	.436	.392	.341	.280	.199
1.0	.000									

*Example: y = 36 in.; D = 48 in.; From tables, $\frac{A}{D^2} = 0.632$, $\frac{R}{D} = 0.302$, $\frac{T}{D} = 0.866$

Hydraulic Properties of Pipe Arch Conduits Flowing Part Full

y = Depth of flow
D = Rise of conduit
B = Span of conduit

A = Area of flow
R = Hydraulic radius
T = Top width of flow

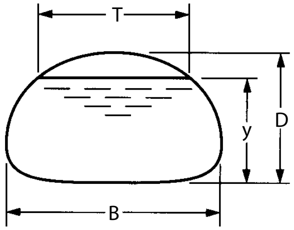


Table 5.5

Determination of area — Values of $\frac{A}{BD}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.1		.072	.081	.090	.100	.109	.119	.128	.138	.148
.2	.157	.167	.177	.187	.197	.207	.217	.227	.237	.247
.3	.257	.267	.277	.287	.297	.307	.316	.326	.336	.346
.4	.356	.365	.375	.385	.394	.404	.413	.423	.432	.442
.5	.451	.460	.470	.479	.488	.497	.506	.515	.524	.533
.6	.541	.550	.559	.567	.576	.584	.592	.600	.608	.616
.7	.624	.632	.640	.647	.655	.662	.670	.677	.684	.690
.8	.697	.704	.710	.716	.722	.728	.734	.740	.745	.750
.9	.755	.760	.764	.769	.772	.776	.780	.783	.785	.787
1.0	.788									

Table 5.6

Determination of hydraulic radius — Values of $\frac{R}{D}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.1		.078	.086	.094	.102	.110	.118	.126	.133	.141
.2	.148	.156	.163	.170	.177	.184	.191	.197	.204	.210
.3	.216	.222	.228	.234	.240	.245	.250	.256	.261	.266
.4	.271	.275	.280	.284	.289	.293	.297	.301	.305	.308
.5	.312	.315	.319	.322	.325	.328	.331	.334	.337	.339
.6	.342	.344	.346	.348	.350	.352	.354	.355	.357	.358
.7	.360	.361	.362	.363	.363	.364	.364	.365	.365	.365
.8	.365	.365	.364	.364	.363	.362	.361	.360	.359	.357
.9	.355	.353	.350	.348	.344	.341	.337	.332	.326	.318
1.0	.299									

Table 5.7

Determination of top width — Values of $\frac{T}{B}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
1		.900	.914	.927	.938	.948	.956	.964	.971	.976
.2	.982	.986	.990	.993	.995	.997	.998	.998	.998	.999
.3	.997	.996	.995	.993	.991	.989	.987	.985	.982	.979
.4	.976	.971	.967	.964	.960	.956	.951	.947	.942	.937
.5	.932	.927	.921	.916	.910	.904	.897	.891	.884	.877
.6	.870	.863	.855	.847	.839	.830	.822	.813	.803	.794
.7	.784	.773	.763	.752	.741	.729	.717	.704	.691	.678
.8	.664	.649	.634	.618	.602	.585	.567	.548	.528	.508
.9	.486	.462	.437	.410	.381	.349	.313	.272	.223	.158

The critical depth serves to distinguish two more classes of open channel flow:

$y > y_{cr}$ - The specific energy is predominantly potential energy (y) and the kinetic energy is small. The velocity is less than V_{cr} and the flow is called SUBCRITICAL (i.e., with respect to velocity) or TRANQUIL.

$y < y_{cr}$ - Most of the specific energy is kinetic energy and the depth or potential energy is small. The velocity is greater than V_{cr} and the flow is therefore called SUPERCRITICAL or RAPID.

For circular pipes, Figure 5.5 provides a nomograph for calculating y_{cr}. For pipe arch CSP, Figures 5.6 and 5.7 provide a graphical method of determining critical flow depths.

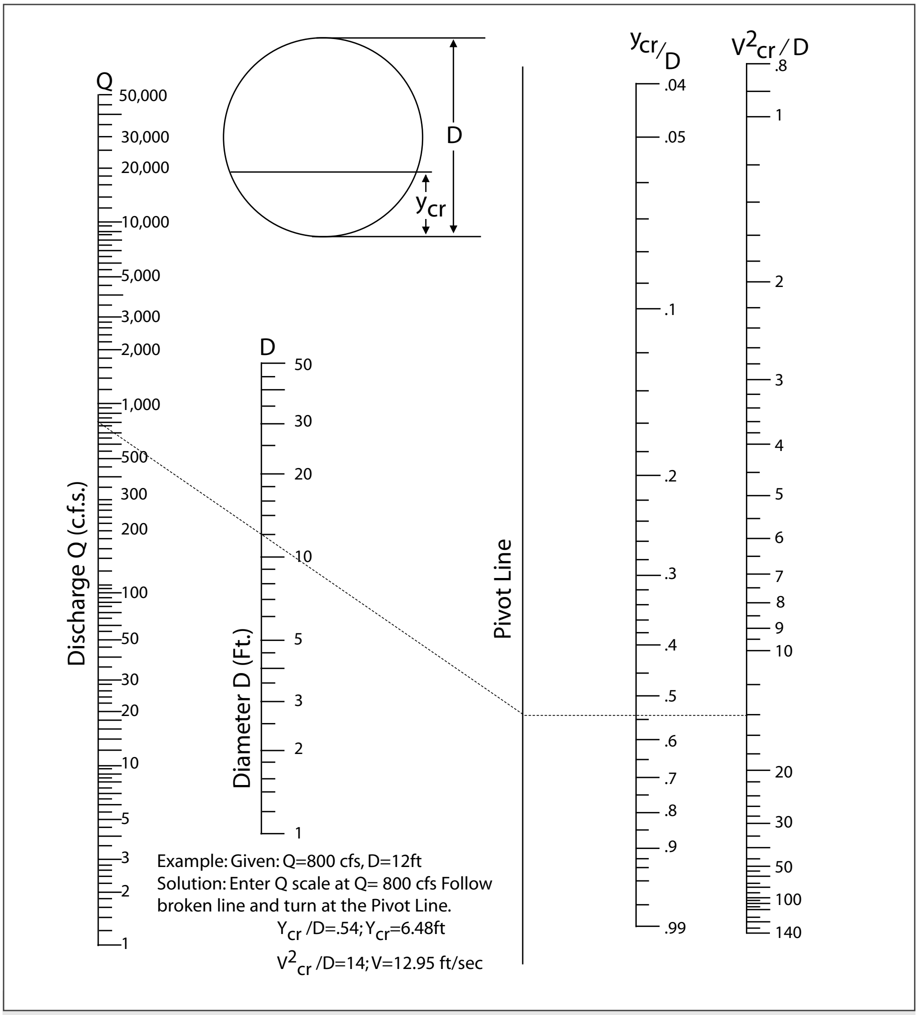
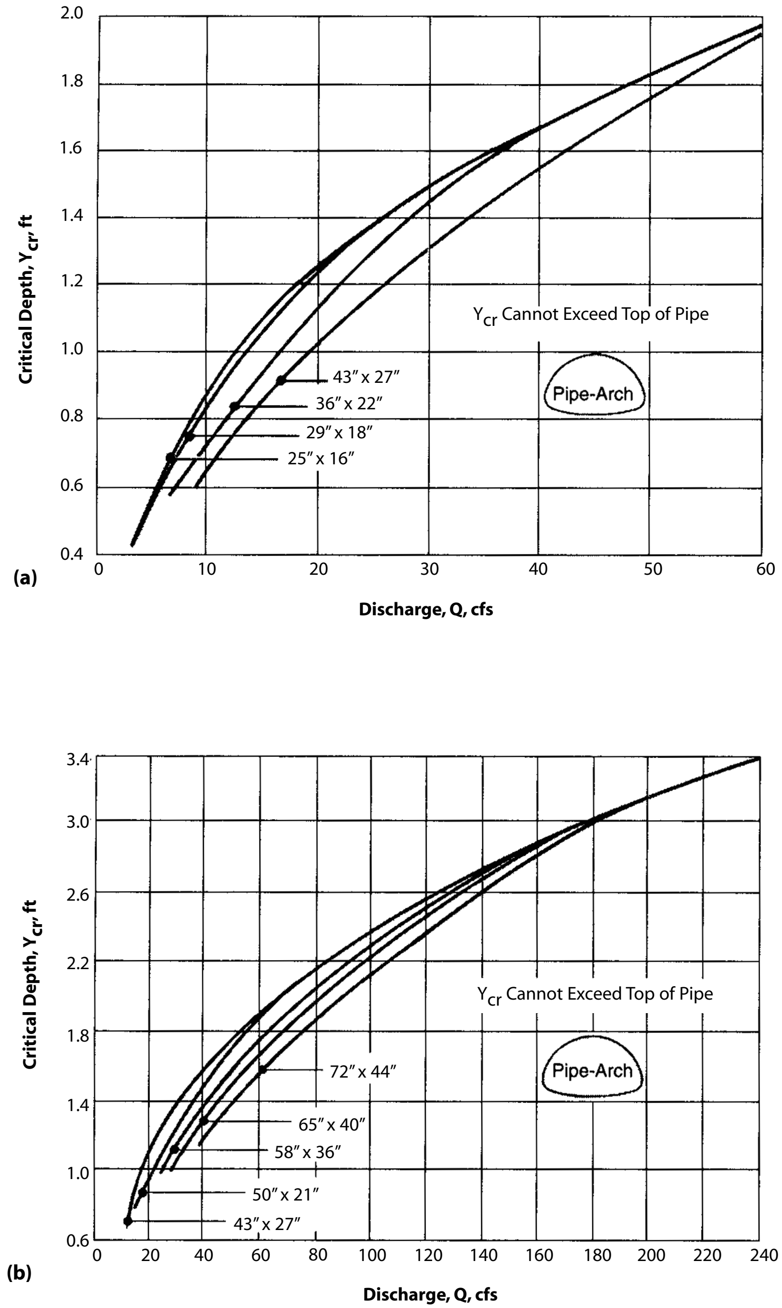
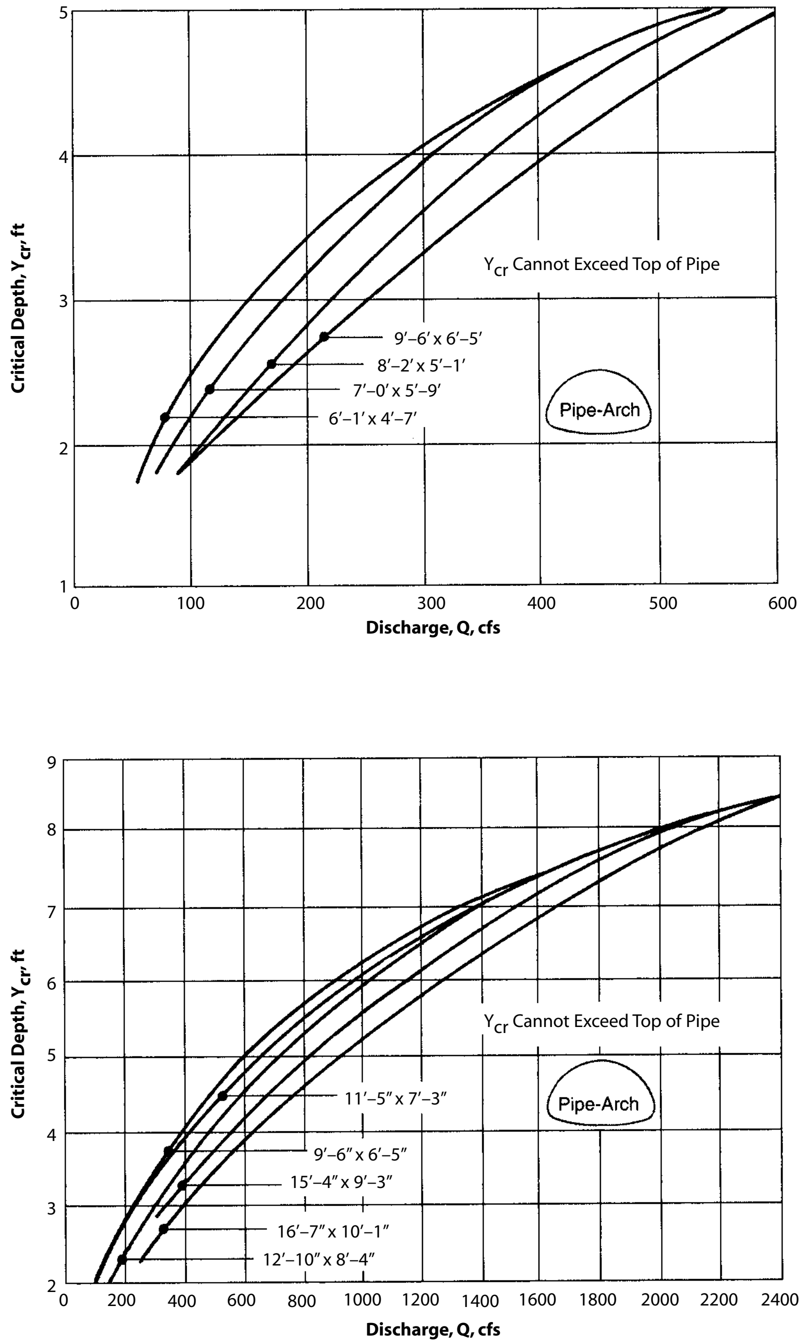


Figure 5.5 Critical flow and critical velocity in circular pipes.



■ **Figure 5.6** Critical depth for standard corrugated steel pipe arches. (Adapted from Federal Highway Administration.)



■ **Figure 5.7** Critical depth for structural plate pipe arches. (Adapted from Federal Highway Administration.)

Energy Losses

When using the Bernoulli Equation for hydraulic design, it is necessary to make allowance for energy losses as illustrated in Figure 5.2. The losses are expressed in terms of head and may be classified as:

friction losses - losses due to the shear stress between the moving fluid and the boundary material; and

form losses - losses caused by abrupt transitions resulting from the geometry of manholes, bends, expansions and contractions.

It is a common mistake to include only friction losses in the hydraulic analysis. Form losses can constitute a major portion of the total head loss and, although estimates of form losses are generally based on empirical equations, it is important to make allowance for them in the design.

Friction Losses

In North America, the Manning and Kutter equations are commonly used to estimate the friction gradient for turbulent flow in storm sewers. In both equations, fully developed rough turbulent flow is assumed so that the head loss per unit length of pipe is approximately proportional to the square of the discharge (or velocity). Both equations use an empirical coefficient, typically termed Manning's 'n', to describe the roughness of the channel boundary. Table 5.8 provides suggested values of 'n' for a variety of pipes and channels. Tables 5.9 and 5.10 provide suggested values of 'n' for various corrugated steel pipe corrugation profiles and linings.

Table 5.8

Coefficient of roughness (Manning's n) for pipes and channels

Closed conduits	
Asbestos-cement pipe	0.011-0.015
Brick	0.013-0.017
Cast iron pipe	
Uncoated (new)	–
Asphalt dipped (new)	–
Cement-lined & seal coated	0.011-0.015
Concrete (monolithic)	
Smooth forms	0.012-0.014
Rough forms	0.015-0.017
Concrete pipe	0.011-0.015
Plastic pipe (smooth)	0.011-0.015
Vitrified clay	
Pipes	0.011-0.015
Liner plates	0.013-0.017
Open channels	
Lined channels	
a. Asphalt	0.013-0.017
b. Brick	0.012-0.018
c. Concrete	0.011-0.020
d. Rubble or riprap	0.020-0.035
e. Vegetal	0.030-0.400
Excavated or dredged	
Earth, straight and uniform	0.020-0.030
Earth, winding, fairly uniform	0.025-0.040
Rock	0.030-0.045
Unmaintained	0.050-0.140
Natural Channels (minor streams, top width at flood stage < 30m, 100 ft)	
Fairly regular section	0.030-0.0700
Irregular section with pools	0.040-0.100

Table 5.9

Coefficient of Roughness (Manning's n) for Standard Corrugated Steel Pipe

		2-2/3 x 1/2	Helical Corrugation, Pitch x Rise (in.)										
		Annular	1-1/2 x 1/4			2-2/3 x 1/2							
Flowing	Finish	Corrugation	Diameter (in.)										
		All Dia.	8	10	12	15	18	24	30	36	42	48	≥ 54
Full	Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021
Full	25% paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019
Part Full	Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023
		All	Pipe Arch Span x Rise (in.)										
		Pipe Arches				17 x 13	21 x 15	28 x 20	35 x 24	42 x 29	49 x 33	57 x 38	≥ 54 x 43
Full	Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022
Part Full	Unpaved	0.029				0.018	0.016	0.021	0.023	0.024	0.025	0.025	0.026
		3 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	3 x 1										
		Corrugation	Diameter (in.)										
		All Dia.				36	42	48	54	60	66	72	≥78
Full	Unpaved	0.027				0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
Full	25% Paved	0.023				0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023
		5 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	5 x 1										
		Corrugation	Diameter (in.)										
		All Dia.						48	54	60	66	72	≥78
Full	Unpaved	0.025						0.022	0.022	0.023	0.024	0.024	0.025
Full	25% Paved	0.022						0.019	0.019	0.020	0.021	0.021	0.022
			All Diameters										
Smooth Interior Pipe (1)			0.012										

Note (1): Includes fully paved, concrete lined, spiral rib pipe, ribbed pipe with inserts, and double wall pipe.

Table 5.10

Coefficient of roughness (Manning's n) for structural plate pipe (6 x 2 in. corrugations)

Corrugation	Diameters (ft)			
6 x 2 in.	5	7	10	15
Plain-unpaved	0.033	0.032	0.030	0.028
25% Paved	0.028	0.027	0.036	0.024

Manning Equation

The Manning Equation is one of a number of so-called empirical solutions. It is widely used for open channel flow calculations but can also be applied to closed pipe flow. The equation is as follows:

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2}$$

where: V = Average velocity, ft/s
R = Hydraulic radius = A/WP, ft
A = Cross-sectional area, ft^2
WP = Wetted perimeter, ft
S_f = Friction gradient or slope of energy line, ft/ft
n = Manning's roughness coefficient

The discharge, Q, is simply determined by multiplying the resulting velocity by the cross-sectional area of the pipe.

Figure 5.8 provides a nomograph for estimating steady uniform flow for pipe flowing full, using the Manning equation. In cases where pipes are flowing only partly full, the corresponding hydraulic section parameters may be determined from Figures 5.9 and 5.10. These figures provide hydraulic section parameters, as a fraction of the value for the pipe flowing full, for ratios of water depth to pipe diameter or rise.

Kutter Equation

The Kutter Equation is used for open channel calculations in certain areas of the United States. It is an empirically derived relation between the Chezy coefficient 'C' and the Manning roughness coefficient 'n'. The equation is as follows:

$$Q = A \bullet C \bullet R^{1/2} \bullet S_f^{1/2}$$

$$\text{where} \quad C = \frac{23 + \frac{0.00155}{S_f} + \frac{1}{n}}{1 + \frac{n}{\sqrt{R}}\left(23 + \frac{0.00155}{S_f}\right)}$$

Although the friction slope, S_f, appears as a second order term in the expression for 'C', the resulting discharge is not sensitive to this term. Table 5.11 shows the difference (%)

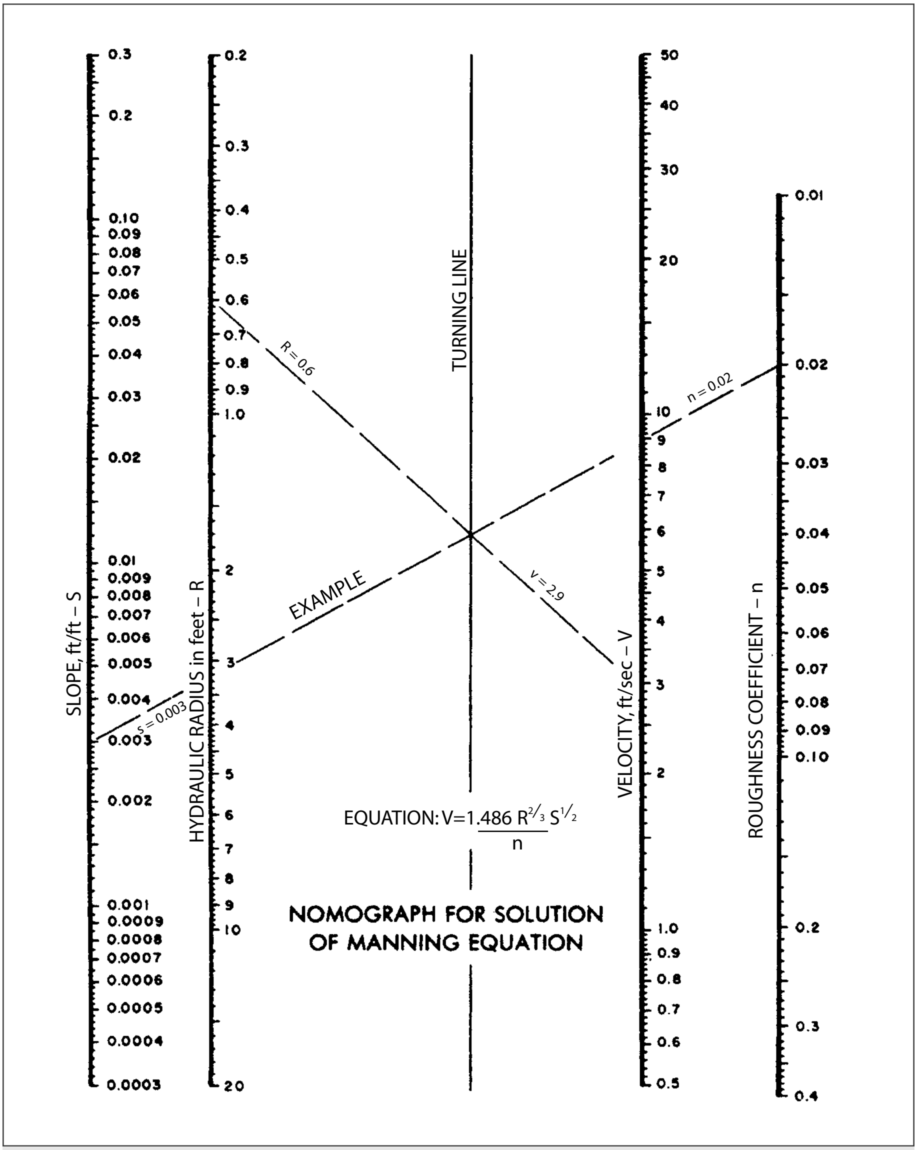


Figure 5.8 Nomograph for solution of Manning's formula.

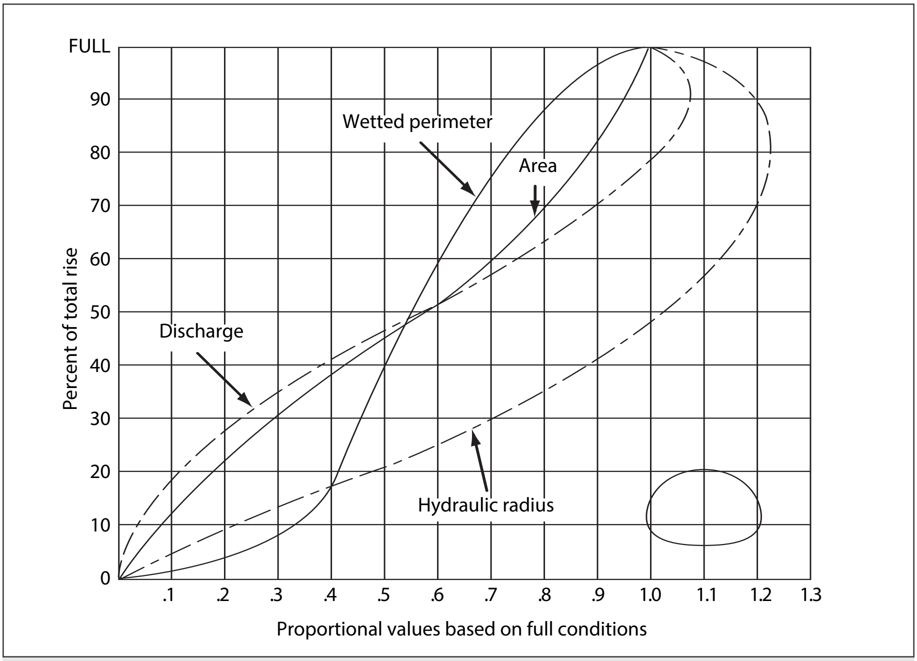


Figure 5.9 Hydraulic section parameters for corrugated steel and structural plate pipe arches.

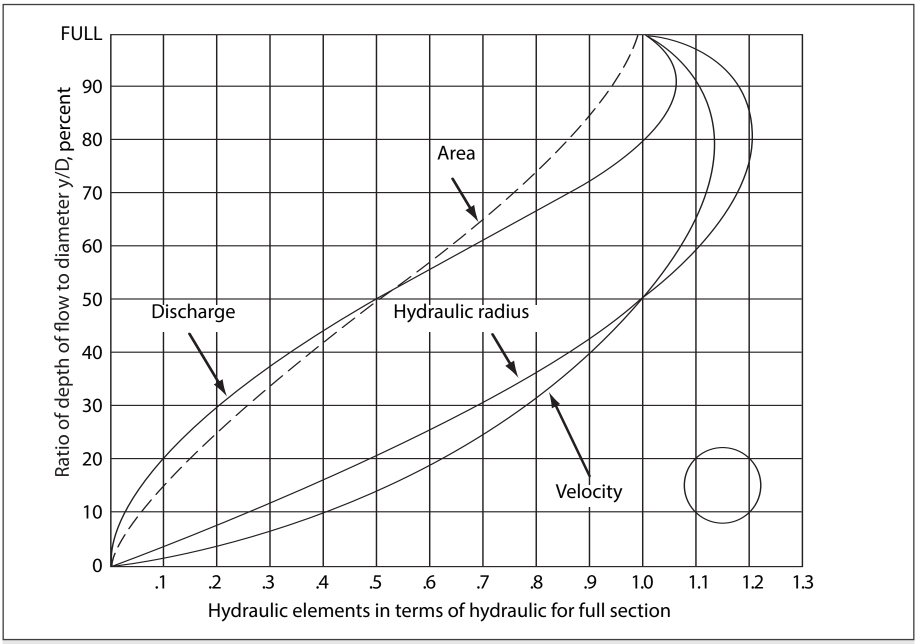


Figure 5.10 Hydraulic section parameters for circular corrugated steel and structural plate corrugated steel pipes.

in discharge computed using the Kutter equation compared with that obtained using the Manning equation. The table gives the relationship between the diameter, D, and the hydraulic radius, R, assuming full flow in a circular pipe. The values in Table 5.11 are also valid for noncircular pipes flowing partially full.

Table 5.11

Percent Difference of Kutter Equation Compared With Manning Equation (Grade = 1.0%)

Diameter D-(ft)	Hydraulic Radius R-(ft)	n = 0.013	n = 0.020	n = 0.030
1.0	0.25	-4.46	-16.18	-26.13
2.0	0.50	-0.46	-8.54	-16.74
3.0	0.75	2.05	-5.07	-11.82
4.0	1.00	2.58	-3.12	-8.70
5.0	1.25	2.66	-1.94	-6.54
6.0	1.50	2.51	-1.18	-4.95
7.0	1.75	2.25	-0.70	-3.74
8.0	2.00	1.92	-0.39	-2.80
9.0	2.25	1.55	-0.20	-2.05
10.0	2.50	1.17	-0.10	-1.45
11.0	2.75	0.78	-0.06	-0.96
12.0	3.00	0.38	-0.07	-0.56
13.0	3.25	-0.01	-0.12	-0.23
14.0	3.50	-0.39	-0.19	0.04
15.0	3.75	-0.77	-0.28	0.26
16.0	4.00	-1.14	-0.39	0.44

The two equations give identical results for values of R close to 3 feet, which represents a very large pipe of perhaps 144 inches diameter. For smaller sized pipes, the difference is significant, especially where the roughness coefficient is large.

Solving the Friction Loss Equation

Of the three parameters of greatest interest in open channel flow analysis (Q, S_f, y_o), the discharge, Q, and the friction slope, S_f, are easily obtained as they appear explicitly in the equations. Because of the exponential form of the Manning equation, it is a simple matter to compute the friction slope as a function of either velocity or discharge for known cross-sectional properties. Even with the Kutter equation, the second order term is of little importance and can be safely ignored as a first iteration when solving for S_f.

The third parameter of interest is the normal depth, y_o, which is the depth at which uniform flow would take place in a very long reach of channel. The normal depth is more difficult to determine as it appears in the expressions for both area, A, and hydraulic radius, R. A trial and error solution is required, except for sections of straightforward geometry.

For partially-full circular channels, a convenient semi-graphical method of solution is provided by the curves describing proportional ratios of discharge, hydraulic radius, area and velocity, expressed as a function of the relative depth y/D. The following two simple examples show how these curves can be used:

Example 1: Finding the normal depth.

A pipe with a diameter of 3 feet and a Manning's n of 0.013 has a gradient of 1.0%. Find the normal depth, y_o, for a discharge of 40 ft^3/s.

Step 1: Calculate the full-pipe capacity using Manning's equation for D = 3 ft
$R = D/4 = 0.75$ ft
$Q = 1.486 / n \bullet R^{2/3} \bullet S^{1/2} \bullet (\pi \bullet D^2 / 4) = 1.486 / 0.013 \bullet (0.75)^{2/3} \bullet (0.01)^{1/2} \bullet (\pi \bullet (3)^2 / 4) = 66.7\ ft^3/s$

Step 2: Calculate the proportional discharge
$Q_{act}/Q_{full} = 40 / 66.7 = 0.60$

Step 3: From the Discharge curve of Figure 5.10, the proportional depth, y/D, is 0.56 for a proportional discharge of 0.60.
The normal depth, y_o, is 0.56 * 3 = 1.68 ft

Example 2: Design for a range of flows.

A pipe, with a Manning's n of 0.013, is designed to carry a minimum discharge of 4.24 ft^3/s with a velocity not less than 3.28 ft/s, and a maximum discharge of 21.2 ft^3/s without surcharging. Use the flattest gradient possible. Find the pipe diameter and slope to satisfy these design criteria.

Step 1: Assume $Q_{full} = Q_{max} = 21.2$
Then $Q_{min} / Q_{full} = 4.24 / 21.2 = 0.2$

Step 2: From the Discharge curve of Figure 5.10, this corresponds to a proportional depth, y/D, of 0.30, which in turn corresponds to a proportional velocity (from the Velocity curve) of V_{min} / V_{full} of 0.78.
The full pipe velocity, corresponding to the specified minimum velocity of 3.28 ft/s, is 3.28 / 0.78 = 4.21 ft/s

Step 3: For full pipe flow, the required section area is given by:
$A = Q_{max} / V_{full} = 21.2 / 4.21 = 5.04\ ft^2$
or $D = (4 \bullet A / \pi)^{1/2} = (4 \bullet 5.04 / \pi)^{1/2} = 2.53$ ft = 30.4 in.

Step 4: Assuming that the next smallest commercial size is 30 inches, the selected diameter must be rounded down to 30 inches to ensure that the minimum velocity is greater than 3.28 ft/s.

Step 5: The necessary slope is then obtained from the Manning equation as

$$S_o = S_f = \frac{Q^2 n^2}{A^2 R^{4/3} (1.486)^2}$$

where $A = \pi \bullet D^2 / 4 = \pi \bullet (2.5)^2 / 4 = 4.91\ ft^2$

and $R = D / 4 = 2.5 / 4 = 0.625$ ft

The required grade is $S_o = Q^2 \bullet n^2 / (1.486)^2 / A^2 / R^{4/3} =$

$(21.1)^2 \bullet (0.013)^2 / (1.486)^2 / (4.91)^2 / (0.625)^{4/3} = 0.0019$ or 1.9%

Water Surface Profiles

Uniform flow is seldom attained except in very long reaches, free from any form of transition. Gradually varied flow occurs as a form of gentle transition from one stage of uniform flow to another, and non-uniform flow is found to be the rule rather than the exception.

The flow profiles of gradually varied flow can be classified in relation to the normal depth, y_o, the critical depth, y_{cr}, and the slope of the channel.

Channel slope is described as:

(1) MILD when $y_o > y_{cr}$ i.e. $S_o < S_{cr}$

(2) STEEP when $y_o < y_{cr}$ i.e. $S_o > S_{cr}$

Note that the critical slope, S_{cr}, is slightly dependent on the stage or magnitude of flow, so that strictly speaking the description of Mild or Steep should not be applied to the channel without regard to the flow conditions.

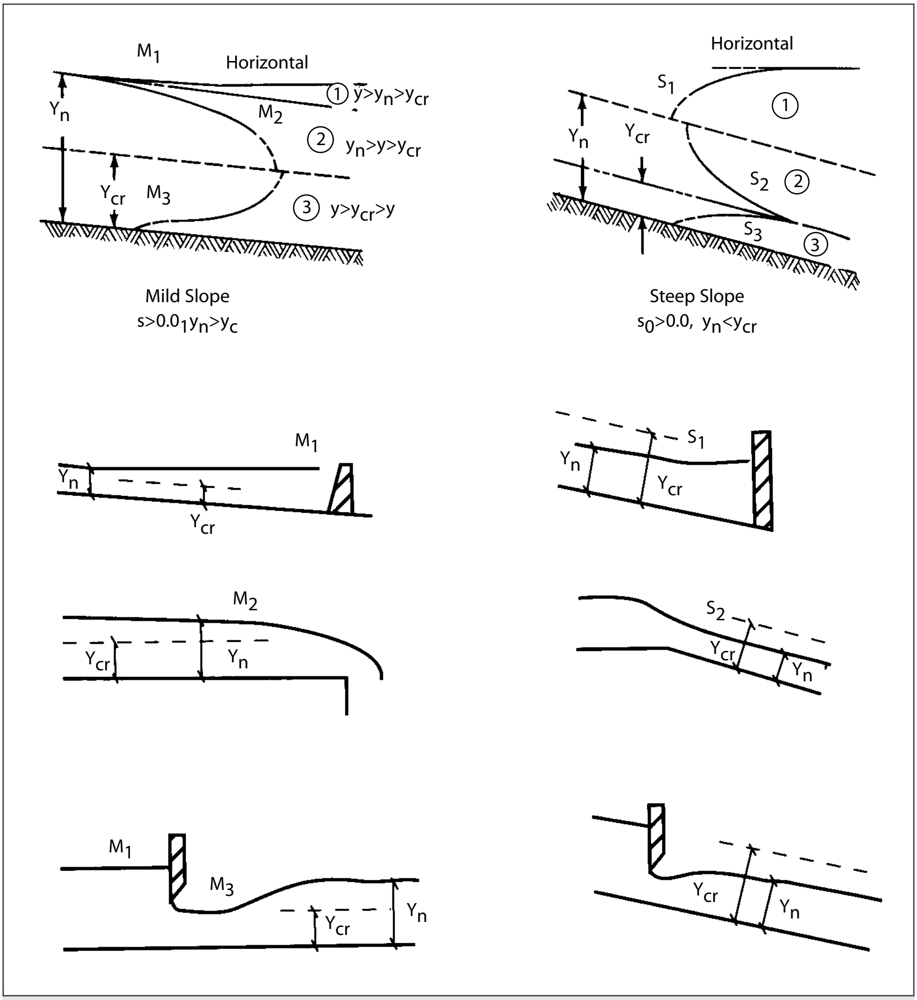
Most textbooks show five classes of channel slope: Mild, Steep, Critical, Horizontal and Adverse. In practice, the last three categories are special cases of the first two and it is sufficient to consider them. In addition to the channel slope, a profile of gradually varied flow can be classified depending on whether it lies above, below or between the normal and critical depths. The three zones may be defined as follows.

Zone 1 — Profile lies above both y_o and y_{cr}

Zone 2 — Profile lies between y_o and y_{cr}

Zone 3 — Profile lies below both y_o and y_{cr}

Using ‘M’ and ‘S’ to denote Mild or Steep channel and the Zone numbers ‘1’, ‘2’ or ‘3’, profiles may be classified as ‘M1’ or ‘S3’, for example. Figure 5.11 shows the idealized cases of the six basic profile types along with typical circumstances in which they can occur.



■ **Figure 5.11** Idealized flow profiles.

Hydraulic Jump

When supercritical flow enters a reach in which the flow is subcritical, an abrupt transition is formed that takes the form of a surface roller or undular wave. The wave tries to move upstream but is held in check by the velocity of the supercritical flow. Figure 5.12 shows a typical situation in which supercritical uniform flow from a steep reach enters a reach of mild slope in which the normal depth is subcritical.

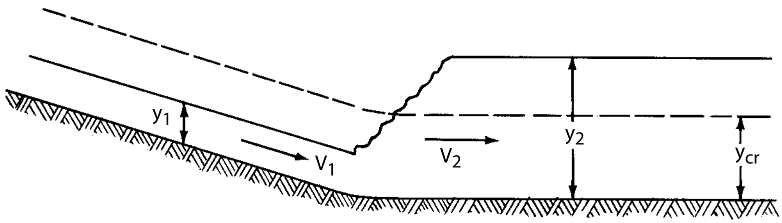


Figure 5.12 Hydraulic jump.

The energy losses associated with the violent turbulence of the hydraulic jump make application of the Bernoulli equation impossible. Instead, the control volume of fluid containing the jump can be analyzed using the equation of conservation of momentum. For a prismatic channel of arbitrary cross-section, this can be expressed as follows:

$$\frac{Q^2}{gA_1} + A_1 y_1 = \frac{Q^2}{gA_2} + A_2 y_2$$

where: y = Depth to the centroid of the cross-section, ft
A = Cross-sectional area, ft^2
Q = Total discharge, ft^3/s
g = Gravitational constant = 32.2 ft/s^2

For the special case of a rectangular cross-section, the solution can be obtained directly using the discharge per unit width:

$$y_2 = -\left(\frac{y_1}{2}\right) + \left(\frac{y_1^{\,2}}{4} + \frac{2q^2}{gy_1}\right)^{\frac{1}{2}}$$

where: y_2 = Depth downstream of the jump, ft
y_1 = Depth upstream of the jump, ft
q = Discharge per unit breadth of channel, $ft^3/s/ft$
g = Gravitational constant = 32.2 ft/s^2

The above equation is reversible so that y_1 may be found as a function of y_2 using a similar relationship.

Form Losses In Junctions, Bends And Other Structures

From the time storm water first enters the sewer system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as manholes, bends, contractions, enlargements and transitions, which will cause velocity head losses. These losses have sometimes been called "minor losses." This is misleading. In some situations these losses are as important as those arising from pipe friction. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisbach equations:

$$H = K\frac{V^2}{2g}$$

where:
- H = Velocity head loss, ft
- K = Structure specific coefficient
- V = Average velocity, ft/s
- g = Gravitational constant = 32.2 ft/s^2

The following are useful velocity head loss equations for hydraulic structures commonly found in sewer systems. They are primarily based on experiments.

Transition Losses (open channel)

The energy losses may be expressed in terms of the kinetic energy at the two ends:

$$H_t = K_t\,\Delta\left[\frac{V^2}{2g}\right]$$

where: K_t = Transition loss coefficient

Contraction:

$$H_t = .1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \qquad V_1 < V_2$$

where:
- V_1 = upstream velocity
- V_2 = downstream velocity

Expansion:

$$H_t = .2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad V_1 > V_2$$

A simple transition from one pipe size to another, in a manhole with straight through flow, may be analyzed with the above equations.

Transition Losses (pressure flow)

Contraction:

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

where: K = 0.5 for sudden contraction
K = 0.1 for well designed transition
A_1, A_2 = cross-sectional area of flow of incoming and outgoing pipe from transition.

Expansion:

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

where: K = 1.0 for sudden expansion
K = 0.2 for well designed transition

The above K values are for estimating purposes. If a more detailed analysis of the transition losses is required, then Tables 5.12 through 5.14, in conjunction with the energy losses equation in the form below, should be used for pressure flow.

$$H_t = K \left(\frac{V^2}{2g}\right)$$

Table 5.12

Values of K_2 for Determining Loss of Head Due to a Sudden Enlargement in a Pipe, for the Formula $H_2 = K_2 (V_1^2/2g)$

D_2/D_1 = Ratio of Larger Pipe to Smaller Pipe — V_1 = Velocity in Smaller Pipe

	Velocity, V_1, in feet per second											
D_2/D_1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	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

Table 5.13

Values of K_2 for Determining Loss of Head Due to a Gradual Enlargement in a Pipe, for the Formula $H_2 = K_2 (V_1^2/2g)$

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.

	Angle of Cone													
D_2/D_1	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

Table 5.14

Values of K_3 for Determining Loss of Head Due to a Sudden Contraction in a Pipe, for the Formula $H_3 = K_3 (V_2^2/2g)$

D_2/D_1 = Ratio of Larger Pipe to Smaller Pipe　　V_2 = Velocity in Smaller Pipe

	Velocity, V_2, ft/s												
D_2/D_1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	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

Entrance Losses

$$H = K_e \frac{V^2}{2g}$$

where: K_e = Entrance loss coefficient (Table 5.15)

Table 5.15

Entrance Loss Coefficients For Corrugated Steel Pipes or Pipe Arches

Inlet End of Culvert	Coefficient K_e
Projecting from fill (no headwall)	0.9
Headwall, or headwall and wingwalls square-edged	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

Notes: *End sections available from manufacturers.

Manhole Losses

Manhole losses, in many cases, comprise a significant percentage of the overall losses within a sewer system. Consequently, if these losses are ignored, or underestimated, the sewer system may surcharge and cause basement flooding or sewer overflows. Losses at sewer junctions are dependent upon flow characteristics, junction geometry and relative sewer diameters. General problems with respect to flow through junctions have been discussed by Chow, who concluded that the losses could be best estimated by experimental analysis as opposed to analytical procedures.

Marsalek, in a study for three junction designs, found the following:

a) In pressurized flow, the most important flow variable is the relative lateral inflow for junctions with more than two pipes. The losses increase as the ratio of the lateral discharge to main line discharge increases.
b) Among the junction geometrical parameters, the important ones are: relative pipe sizes, junction benching and pipe alignment. Base shape and relative manhole sizes were less influential.
c) Full benching to the crown of the pipe significantly reduces losses as compared to benching to the mid-section of the pipe or no benching.
d) In junctions where two lateral inflows occur, the head losses increase as the difference in flows between the two lateral sewers increases. The head loss is minimized when the lateral flows were equal.

Various experimental studies have been performed to estimate manhole losses. These works should be referred to whenever possible. In cases where no applicable results are available, the following may be used as a guideline to estimate manhole losses.

In a straight through manhole, where there is no change in pipe size, losses can be estimated by:

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

Losses at terminal manholes may be estimated by:

$$H_{tm} = \frac{V^2}{2g}$$

Losses at junctions where one or more incoming laterals occur may be estimated by combining the laws of pressure plus momentum:

$$H_j = K_j \frac{V^2}{2g}$$

where: H_j = Junction losses

Using the laws of pressure plus momentum:

$$(H_j + D_1 - D_2)\frac{(A_1 + A_2)}{2} = \frac{Q_2^{\,2}}{A_2 g} - \frac{Q_1^{\,2}}{A_1 g} - \frac{Q_3^{\,2}}{A_3 g}\cos\theta$$

where the variable numbering is as shown in Figure 5.13.

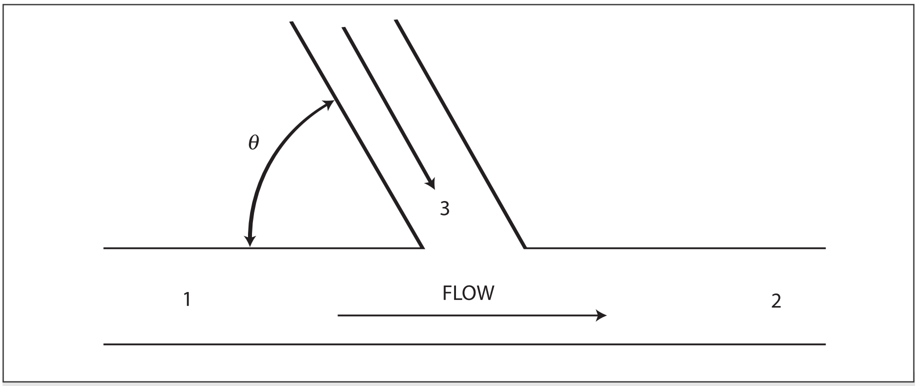


Figure 5.13 Manhole junction losses; variable numbering.

Bend Losses

Bend losses may be estimated from the equation:

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

For curved sewer segments where the angle is less than 40° the bend loss coefficient may be estimated as:

$$K_b = .25 \sqrt{\frac{\Delta}{90}}$$

where: Δ = central angle of bend in degrees

For greater angles of deflection and bends in manholes, the bend loss coefficient may be determined from Figure 5.14.

HYDRAULICS OF STORM WATER INLETS

Storm water inlets are the means by which storm runoff enters the sewer system. Their design is often neglected or receives very little attention during the design of storm drainage systems. Inlets play an important role in road drainage and storm sewer design because of their effect on both the rate of water removal from the road surface and the degree of utilization of the sewer system.

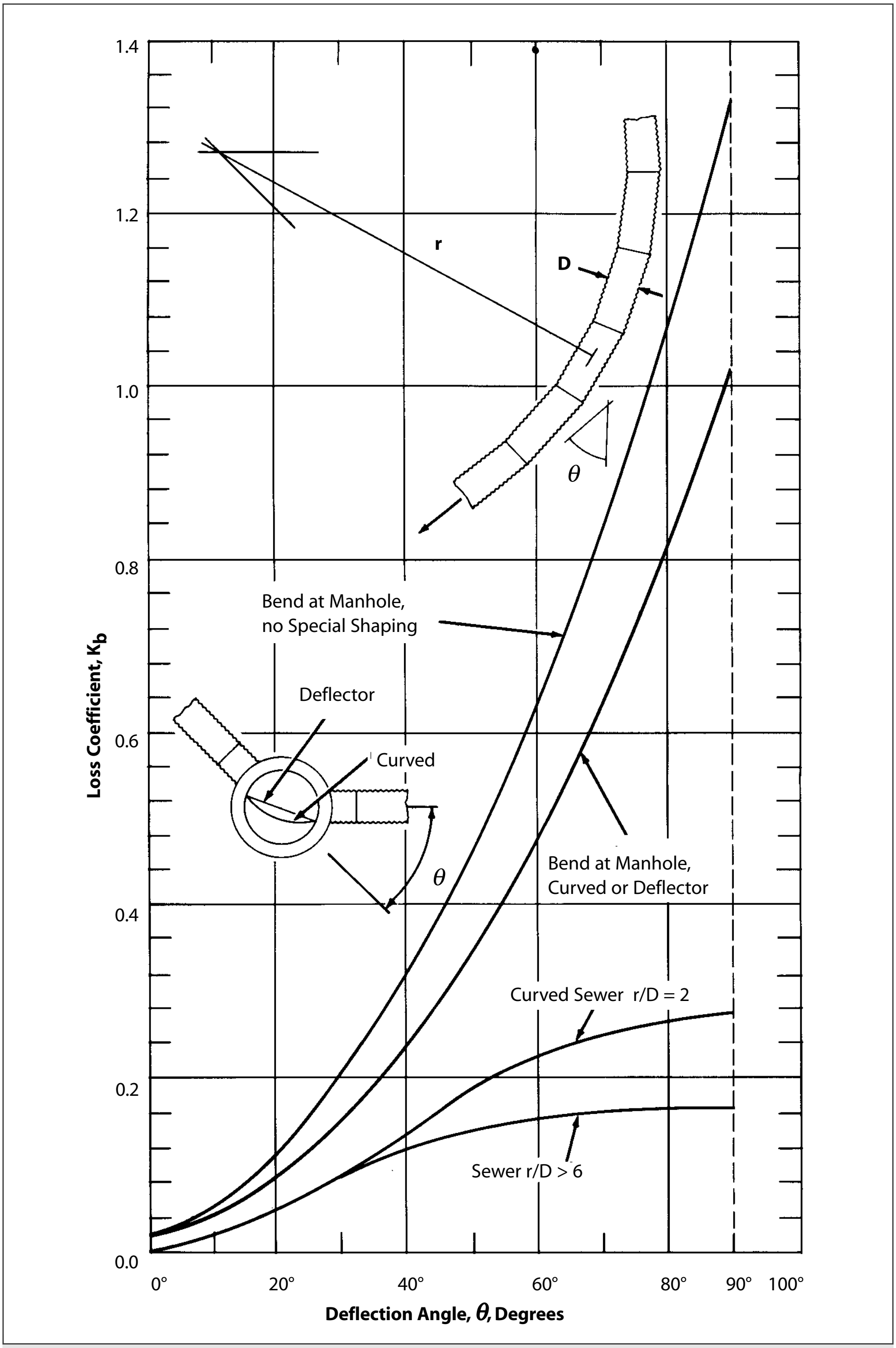
If inlets are unable to accept the design inflow into the sewer system, it may result in a lower level of roadway convenience and conditions hazardous to traffic. It may also lead to overdesign of the sewer pipes downstream of the inlet. In some cases, however, the limited capacity of the inlets may be desirable as a storm water management alternative, thereby offering a greater level of protection from excessive sewer surcharging. In such cases, both the quantity of runoff intercepted and the resulting level of roadway convenience must be known. Furthermore, overdesign in the number of inlets results in higher costs and could result in overuse of the sewer system.

It is imperative that more emphasis be placed on inlet design to assure that the inlet type, location and capacity are adequately determined to achieve the overall drainage requirements.

No one inlet type is best suited for all conditions. Many different types of inlets have thus been developed, as shown in Figure 5.15. In the past, the hydraulic capacities of some of these inlets were often unknown, sometimes resulting in erroneous capacity estimates.

Storm water inlets may not intercept all runoff due to the velocity of flow over the inlet and the spread of flow across the roadway and gutter. This leads to the concept of carryover flow. As carryover flow progresses downstream, it may accumulate, resulting in a greater demand for interception.

The hydraulic efficiency of inlets is a function of street grade, cross slope, inlet geometry, and curb and gutter design. Generally, an increased street cross-slope will result in increased inlet capacity as the flow is concentrated within the gutter. The depth of flow in the gutter may be estimated from Figure 5.16.



■ **Figure 5.14** Sewer bend loss coefficient.

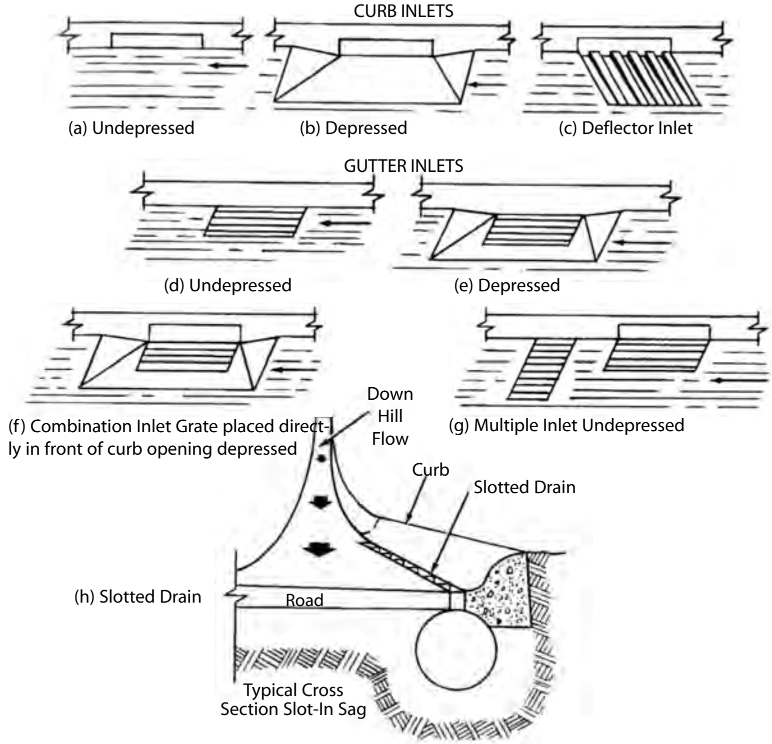
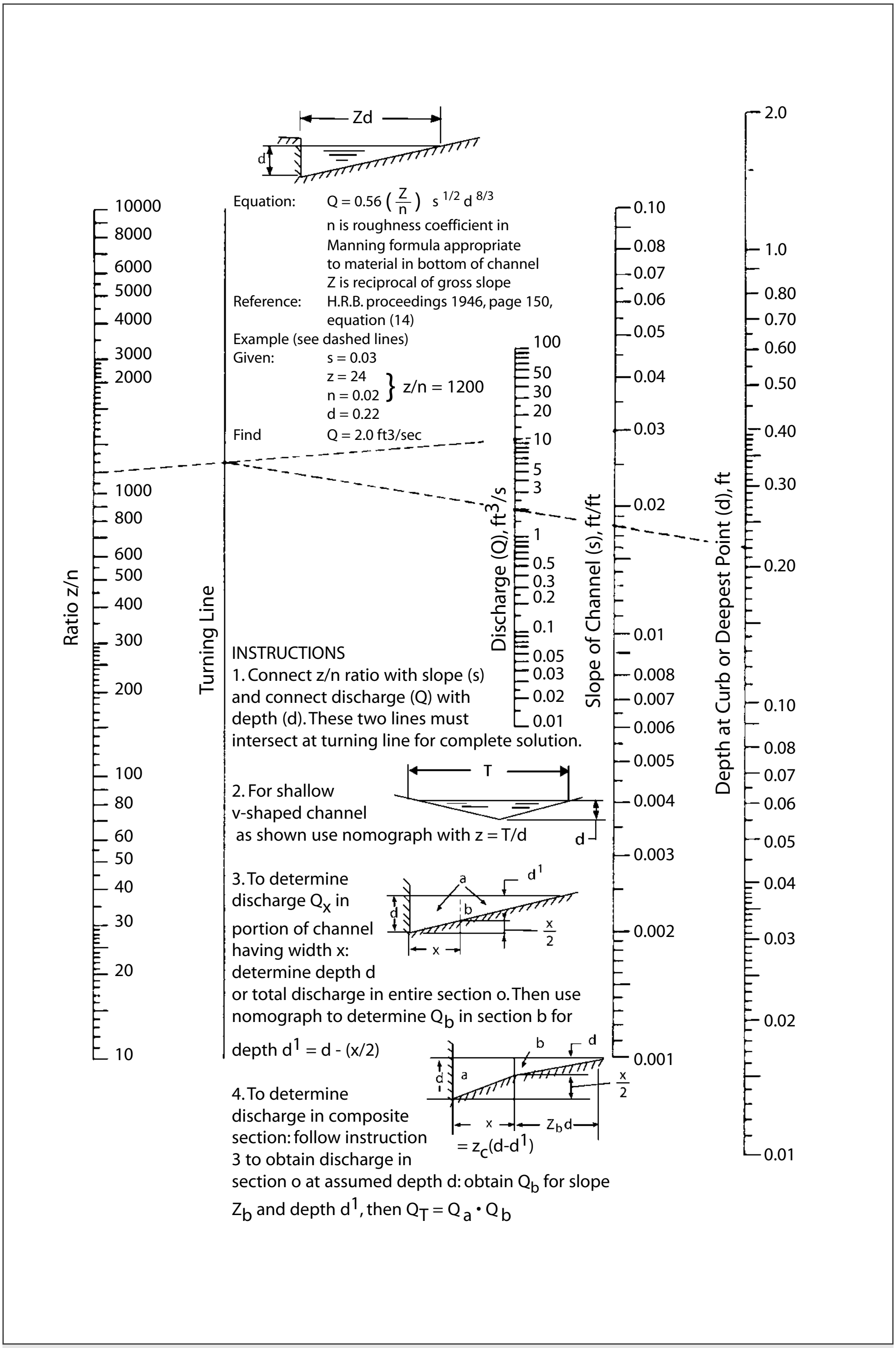


Figure 5.15 Storm water inlet types.

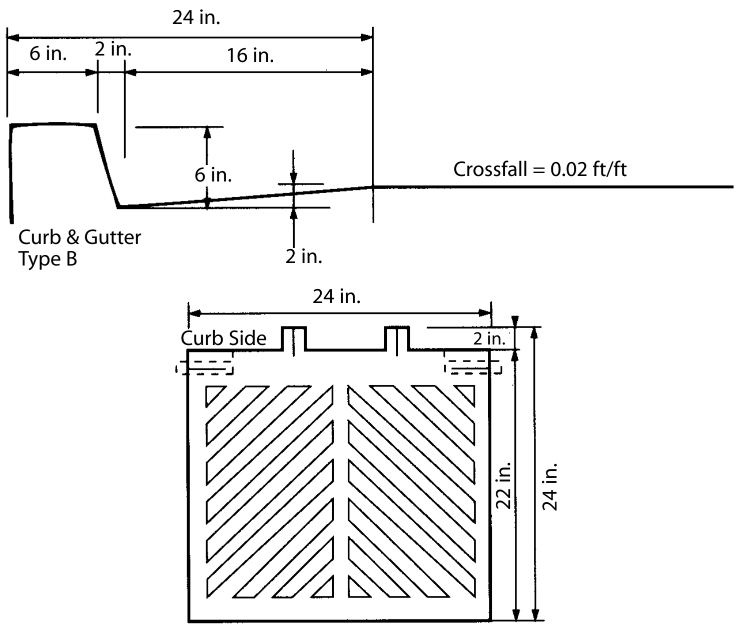
The effect of street grades on inlet capacities varies. Initially as the street grade increases there is an increase in gutter flow velocity, which allows a greater flow to reach the inlets for interception. However, as street grades continue to increase, there is a threshold where the velocity is so high that less flow can be intercepted. This threshold velocity depends upon the geometry of the inlet and characteristics of the gutter.

Experimental determination of inlet capacities have resulted in a set of tables and charts to aid the designer in storm water inlet selection and sewer system design. A sample of the results is shown in Figures 5.17 and 5.18 and Tables 5.16 and 5.17.

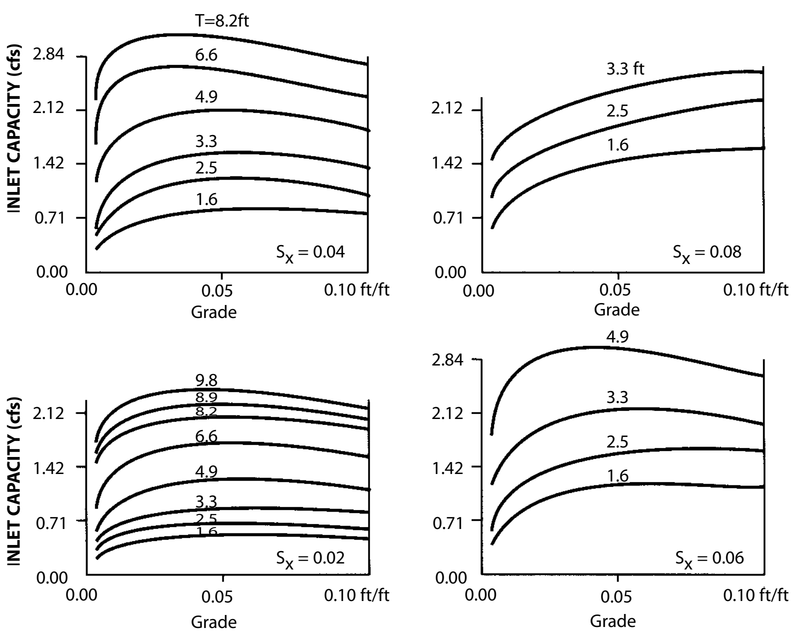
To use these charts or tables, the designer determines the overland flow and the resulting spread in gutter flow from a pre-determined road grade and crossfall, gutter design and inlet type (Table 5.16). This value is then used with Table 5.17 to obtain the storm water



■ **Figure 5.16** Nomograph for flow in triangular channels.



■ **Figure 5.17** Curb and gutter catch basin grate.



■ **Figure 5.18** Sewer inlet capacity for curb and gutter catch basin grate shown in Figure 5.17.

Table 5.16

Gutter Flow Rate (cfs)

Crossfall	Spread	Depth	Grade (ft/ft)							
(ft/ft)	(ft)	(ft)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	0.00	0.16	0.16	0.29	0.41	0.50	0.58	0.71	0.81	0.91
	1.64	0.20	.027	0.49	0.69	0.84	0.98	1.19	1.38	1.54
	2.46	0.21	0.35	0.64	0.90	1.11	1.28	1.56	1.81	2.02
	3.28	0.23	0.46	0.83	1.18	1.44	1.66	2.04	2.35	2.63
0.02	4.92	0.26	0.76	1.38	1.95	2.39	2.76	3.38	3.90	4.36
	6.56	0.30	1.19	2.18	3.08	3.78	4.36	5.34	6.17	6.89
	8.20	0.33	1.80	3.28	4.64	5.68	6.56	8.03	9.28	10.37
	8.86	0.34	2.09	3.81	5.39	6.60	7.62	9.34	10.78	12.05
	9.84	0.36	2.58	4.72	6.67	8.17	9.43	11.55	13.34	14.92
	1.64	0.23	0.41	0.76	1.07	1.31	1.51	1.86	2.14	2.39
	2.46	0.26	0.64	1.16	1.64	2.01	2.32	2.84	3.28	3.66
0.04	3.28	0.30	0.93	1.70	2.41	2.95	3.41	4.17	4.82	5.39
	4.92	0.36	1.81	3.31	4.69	5.73	6.63	8.11	9.37	10.47
	6.56	0.43	3.14	5.74	8.11	9.94	11.47	14.05	16.23	18.14
	8.20	0.49	5.00	9.12	12.90	15.80	18.24	22.34	25.80	28.84
	1.64	0.26	0.59	1.08	1.53	1.88	2.17	2.66	3.07	3.43
	2.46	0.31	1.00	1.82	2.58	3.15	3.64	4.46	5.15	5.76
0.06	3.28	0.36	1.56	2.84	4.02	4.93	5.69	6.96	8.04	8.99
	4.92	0.46	3.24	5.92	8.37	10.25	11.84	14.50	16.75	18.72
	5.48	0.49	3.99	7.29	10.31	12.63	14.59	17.86	20.63	23.06
	1.64	0.30	0.80	1.47	2.07	2.54	2.93	3.59	4.14	4.64
0.08	2.46	0.36	1.43	2.61	3.69	4.52	5.22	6.39	7.38	8.25
	3.28	0.43	2.31	4.23	5.98	7.32	8.45	10.35	11.95	13.36
	4.10	0.49	3.49	6.38	9.02	11.05	12.76	15.62	18.04	20.17

Table 5.17

Catch Basin Grate Inlet Capacity (cfs)

Crossfall	Spread	Depth	Grade (ft/ft)						
(ft/ft)	(ft)	0.00	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	1.64	0.17	0.26	0.34	0.39	0.41	0.44	0.45	0.43
	2.46	0.28	0.41	0.50	0.59	0.63	0.66	0.68	0.61
	3.28	0.36	0.51	0.64	0.74	0.79	0.82	0.83	0.77
0.02	4.92	0.46	0.80	1.01	1.11	1.18	1.22	1.21	1.13
	6.56	0.81	1.25	1.42	1.53	1.55	1.54	1.51	1.45
	8.20	1.21	1.63	1.84	1.92	1.92	1.89	1.83	1.75
	8.86	1.29	1.77	1.97	2.03	2.04	2.02	1.96	1.84
	9.84	1.48	1.94	2.14	2.19	2.18	2.14	2.09	2.02
	1.64	0.24	0.45	0.60	0.69	0.76	0.84	0.83	0.75
	2.46	0.43	0.74	0.96	1.07	1.11	1.14	1.10	0.99
0.04	3.28	0.55	0.96	1.22	1.36	1.41	1.47	1.42	1.34
	4.92	0.97	1.63	1.90	2.01	2.04	1.98	1.87	1.77
	6.56	1.48	2.27	2.46	2.50	2.51	2.47	2.39	2.25
	8.20	2.03	2.75	2.85	2.85	2.82	2.70	2.59	2.54
	1.64	0.34	0.54	0.74	0.86	0.93	0.99	1.07	1.06
0.06	2.46	0.66	0.99	1.16	1.27	1.39	1.50	1.57	1.53
	3.28	1.07	1.49	1.69	1.83	1.90	1.96	1.94	1.80
	4.92	1.69	2.19	2.43	2.52	2.56	2.52	2.40	2.21
	1.64	0.46	0.81	1.04	1.14	1.24	1.33	1.35	1.34
0.08	2.46	0.96	1.33	1.49	1.61	1.73	1.89	2.00	2.02
	3.28	1.34	1.78	1.65	2.15	2.24	2.41	2.55	2.63

inlet or grate inlet capacity. The difference between the flow on the roadway and the inlet capacity is referred to as the carryover. An illustrative example is presented below:

Design Parameter Road crossfall = 0.02 ft/ft
Road grade = 0.02 ft/ft
Gutter type B
Inlet grate type per Figure 5.17
One inlet on each side of the road
There is no upstream carryover flow (0 ft^3/s)
Catchment Runoff = 6.2 ft^3/s

The flow in each gutter will be half of the total runoff.
Gutter Flow = 6.2 / 2 + 0 = 3.1 ft^3/s

From Table 5.16 the resulting spread in flow for a depth of 0.30 feet is 6.56 feet.

From Table 5.17, 6.56 feet of spread results in an inlet capacity of 1.42 ft^3/s.

Therefore, the total flow intercepted = 2 x 1.42 = 2.84 ft^3/s.

The carryover flow = 6.2 – 2.84 = 3.36 ft^3/s.

For roads where few restrictions to inlet location may exist (i.e., highways and arterial roads), these charts can be used to establish minimum spacing between inlets. This is done by controlling the catchment area for each inlet. The area is simplified to a rectangular shape of width and length where the length represents the distance between inlets.

Under special circumstances, it may be necessary to install twin or double inlets to increase the inlet capacity. For reasons of interference by traffic, such installations are usually installed in series, parallel to the curb. Studies have shown that where such installations exist on a continuous grade, the increases in inlet capacity rarely exceed 50 percent of the single inlet capacity.

The capacity of storm water inlets at a sag in the roadway is typically expressed by weir and orifice equations. Flow into the inlets initially operates as a weir having a crest length equal to the length of the inlet perimeter that the flow crosses. The inlet operates under these conditions to a depth of about 4 inches. The quantity intercepted is expressed by the following:

$$Q = C\,L\,D^{1.5}$$

where: Q = Rate of discharge into the grate opening, ft^3/s
C = 3.0
L = Perimeter length of the grate, disregarding bars and neglecting the side against the curb, ft
D = Depth of water at the grate, ft

When the depth exceeds 0.4 feet, the inlet begins to operate as an orifice and its discharge is expressed by the following:

$$Q = C\,A\,D^{0.5}$$

where: Q = Rate of discharge into the grate opening, ft^3/s
A = Clear opening of the grate, ft^2
C = 3.0
D = Depth of water ponding above the top of the grate, ft

The inlet capacity of an non-depressed curb inlet may be expressed by the equation:

$$Q/l = C\,(1x10^{-3})\,d\,(g/d)^{1/2}$$

where: Q = Discharge into inlet, ft^3/s
C = 4.82
l = Length of opening, ft
g = Gravitational constant = 32.2 ft/s^2
d = Depth of flow in gutter, ft

Another equation can be used, for the same calculation, which assumes a gutter of wedge shaped cross-section with a cross-sectional street slope of 10^{-3} to 10^{-1}:

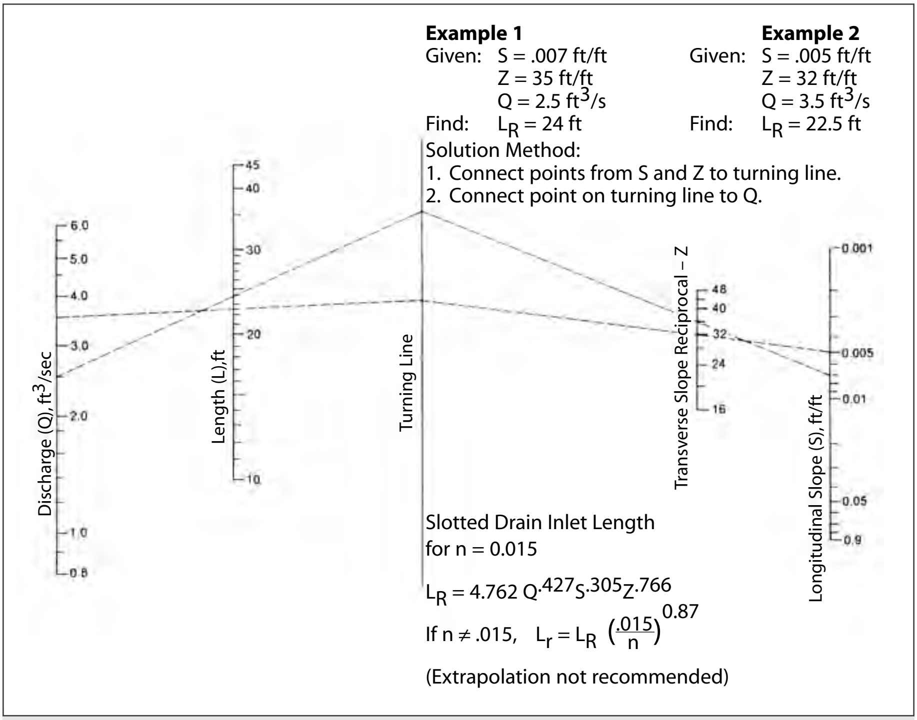
$$Q/l = C\,i^{0.579}\left(\frac{Q_o}{\sqrt{s}/n}\right)^{0.563}$$

where: Q_o = Flow in the gutter, ft^3/s
i = Transverse slope, ft/ft
s = Hydraulic gradient of gutter, ft/ft
n = Coefficient of roughness of gutter
C = 1.87

Slotted Drain

Slotted drain inlets are typically located as spaced curb inlets on a grade (sloping roadway) to collect downhill flow, or located in a sag (low point). The inlet capacity of a slotted drain may be determined from Figure 5.19,

where: S = Longitudinal gutter or channel slope, ft/ft
S_x = Transverse slope, ft/ft
Z = Transverse slope reciprocal = 1 / S_x, ft/ft
Q = Discharge, ft^3/s
L = Length of Slot, ft



■ **Figure 5.19** Slotted drain design nomograph.

It is suggested that the length of a slotted drain be in increments of 5 or 10 feet to facilitate fabrication, construction and inspection. Pipe diameter is usually not a factor but it is recommended that it be at least 18 inches

For a series of slotted drain curb inlets on a grade, each inlet will collect all or a major portion of the flow to it. The anticipated flow at points along the curb can be determined by the methods described previously in this chapter.

Once the initial upstream inlet flow is established, Figure 5.19 is used to determine the required length of slot to accommodate the total flow at the inlet.

The length of slot actually used may be less than required by Figure 5.19. Carryover is that portion of the flow that does not form part of the flow captured by the slotted drain. While some of the flow enters the drain, some flows past the drain to the next inlet. The efficiency of a slotted drain, required in order to consider carryover, is shown in Figure 5.20,

where: Q_d = Total discharge at an inlet, ft^3/s
Q_a = An assumed discharge, ft^3/s

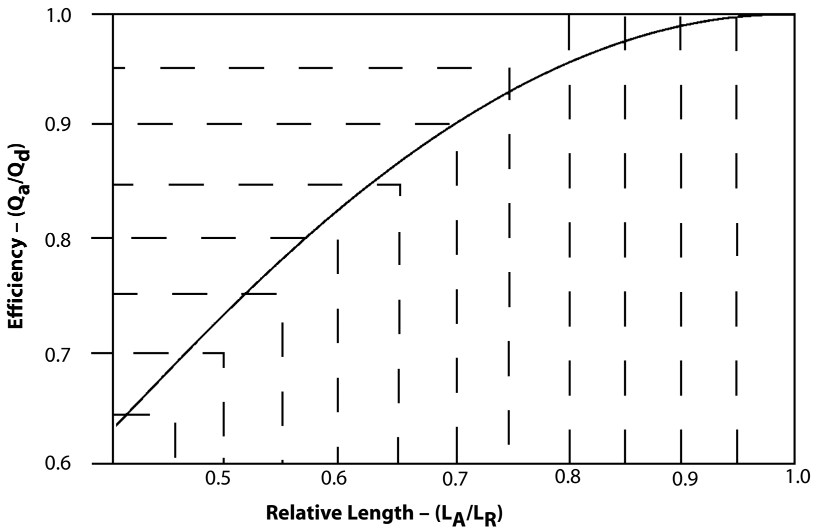


Figure 5.20 Slotted drain carryover efficiency.

If carryover is permitted, the designer must assume a length of slot such that the ratio of the assumed length of slot to the length of slot required for total interception and no carryover (L_A/L_R) is greater than 0.4 but less than 1.0. In other words, the designer must decide on a length of slot that will provide an acceptable carryover efficiency. Where carryover is not permitted, L_A must be at least the length L_R.

Economics usually favor slotted drain pipe inlets designed with carryover rather than for total flow interception. There must be a feasible location to which the carryover may be directed.

The actual length of slotted drain required, when carryover is allowed, can be determined using Figure 5.20. For example, if 20% carryover (slotted drain efficiency, Q_a / Q_d = 80%) is allowed, then only 58% (L_A/L_R) of the total slotted drain length is required, resulting in a 42% savings in material and installation costs.

The slotted drain efficiency can also be calculated using the following equation:

$$E = 1 - 0.918\left(1 - \frac{L_A}{L_R}\right)^{1.769}$$

where:
- E = Efficiency, fraction
- L_A = Actual slot length, ft
- L_R = Slot length required for no carryover, ft

The amount of carryover can be calculated using the following equation:

$$CO = Q_d\,(1 - E)$$

where: CO = Carryover flow, ft^3/s
Q_d = Total design flow, ft^3/s

Combining the above two equations results in the following equation for carryover flow:

$$CO = 0.918\,Q_d \left(1 - \frac{L_A}{L_R}\right)^{1.769}$$

At sag inlets, the required length of slotted drain for total interception should be based on the orifice equation, which is:

$$Q_d = C\,A\sqrt{2gd}$$

where: C = Orifice coefficient = 0.61
A = Open area of slot based on the width of the slot and the length L_R, ft^2
g = Gravitational constant = 32.2 ft/s^2
d = Maximum allowable depth of water in the gutter, ft

Solving for the required slot length, L_R, assuming a slot width of 1.75 inches, results in the following equation:

$$L_R = \frac{1.401\,Q_d}{\sqrt{d}}$$

For a slotted drain in a sag at the end of a series of drains on a grade, the flow to the drain will include any carryover from the immediately adjacent drain upgrade. Unlike a drain-on-grade situation, a slotted drain in a sag will produce significant ponding if its capacity will not accommodate the design flow. Therefore, the actual length of sag inlets should be at least 2 times the calculated required length. This helps ensure against a hazard caused by plugging from debris. L_A should never be less than 20 feet for sag inlet cases.

Carryover is not usually permitted at level grade inlets. In that case, the actual slotted drain length must be at least the required length.

In addition to applications where slotted drain is located parallel to a curb and gutter and in the gutter, it is also used effectively to intercept runoff from wide, flat areas such as parking lots, highway medians and even tennis courts and airport loading ramps. The water is not, in these applications, collected and channeled against a berm (or curb), but rather the drain is placed transverse to the direction of flow so that the open slot acts as a weir intercepting all of the flow uniformly along the entire length of the drain.

Slotted drain has been tested for overland flow (sheet flow). The tests included flows up to 0.04 ft^3/s per foot of slot. The slotted drain used in the test was designed to accommodate at least 0.025 ft^3/s per foot, which corresponds to a rainstorm of 15 in./hr over a 72 foot wide roadway (6 lanes). Slopes ranged from a longitudinal slope of 9% and a transverse slope reciprocal of 16 ft/ft, to a longitudinal slope of 0.5% and a transverse slope reciprocal of 48 ft/ft. At the design discharge of 0.025 ft^3/s per foot, it was reported that the total flow fell through the slot as weir flow, without hitting the curb side of the slot. Even at the maximum discharge of 0.04 ft^3/s per foot and maximum slopes, nearly all the flow passed through the slot.

HYDRAULICS OF SUBDRAINS

Ground water may be in the form of an underground reservoir or it may be flowing through a seam of pervious material. If it is flowing, it may be seeping or percolating through a seam between impervious strata, or be concentrated in the form of a spring.

Free water moves through the ground by gravity. It may consist of storm water seeping through cracks in the pavement or entering the ground along the edges of the pavement or road. It may be ground water percolating from a higher water-bearing stratum to a lower one, or from a water-bearing layer into the open as in the case of an excavation.

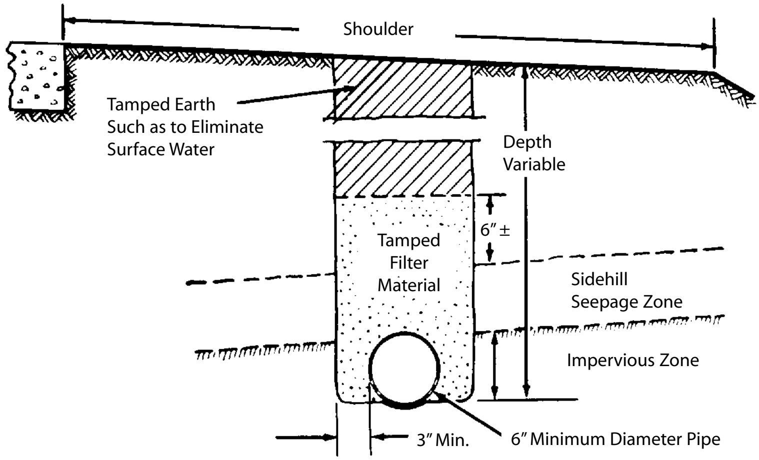
A number of subdrainage applications are discussed in Chapter 1.

Water seeping through cracks in the pavement is especially noticeable in springtime and also visible shortly after rains when the remainder of the road has dried. Passing traffic pumps some of this water, sometimes mixed with subgrade soil, up through the cracks or joints onto the road surface. This water is harmful because it may freeze on the surface and become an unexpected traffic hazard, and it can also destabilize the road subgrade. It can and should be removed in order to establish a stable subgrade and to prevent potential problems.

Subsurface Runoff Computation

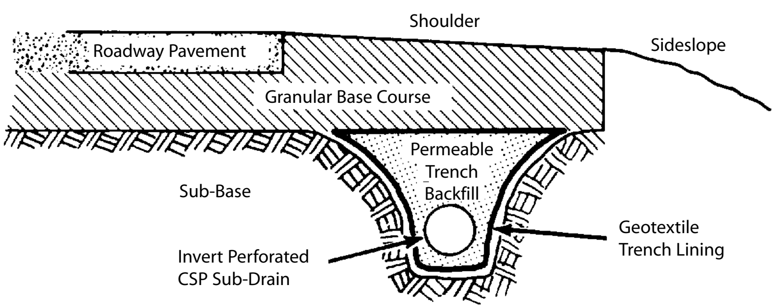
In general, the amount of available ground water is equivalent to the amount of water that soaks into the ground from the surface less the amounts that are used by plants or lost by evaporation. The nature of the terrain and the catchment area size, shape and slopes, as well as the character and slopes of the substrata, are contributing factors to the amount of ground water available and the volume of subsurface runoff.

A practical way to determine the presence of ground water, and the potential flow rate, is to dig a trench or test pit. This is helpful especially where an intercepting drain is to be placed across a seepage zone to intercept the ground water and divert the flow, as shown in Figure 5.21.



■ **Figure 5.21** Intercepting Drain

Geotextiles are sometimes used in addition to, or instead of, free draining granular backfill. The fabric serves as a filter, preventing fine erodible soils from entering the subdrain system while allowing the free flow of water. A typical cross-section of a trench design utilizing a geosynthetic filter fabric as a separator/filter is shown in Figure 5.22.



■ **Figure 5.22** Intercepting Drain

Determining a correct size for subdrainage pipe requires an indirect approach. For problems other than those involving large flat areas, size determination becomes a matter of personal judgment and local experience. The following procedure applies to relatively flat areas.

The rate of runoff for average agricultural soils has been determined by agricultural engineering experiment stations to be about 0.375 inches in 24 hours. For areas of heavy rainfall or more pervious soils, this factor may be increased to 0.75 or 1.0 inch. The runoff expressed in inches per 24 hours is converted to $ft^3/s/acre$ for design discharge calculations. Table 5.18 provides a conversion table.

Table 5.18

Subsurface Runoff Constants for Various Soil Permeability Types

Soil Permeability Type	Depths of Water Removed in 24 Hours in.		Quantity of Water per Lateral, $ft^3/s/acre$ Constant C
	Fraction	Decimal	
Slow to Moderate	1/16	0.0625	0.0026
Slow to Moderate	1/8	0.1250	0.0052
Slow to Moderate	3/16	0.1875	0.0079
Slow to Moderate	1/4	0.2500	0.0105
Moderate	5/16	0.3125	0.0131
Moderate	3/8	0.3750	0.0157
Moderate	7/16	0.4375	0.0184
Moderate	1/2	0.5000	0.0210
Moderate	9/16	0.5625	0.0236
Moderate to Fast	5/8	0.6250	0.0262
Moderate to Fast	11/16	0.6875	0.0289
Moderate to Fast	3/4	0.7500	0.0315
Moderate to Fast	13/16	0.8125	0.0341
Moderate to Fast	7/8	0.8750	0.0367
Moderate to Fast	15/16	0.9375	0.0394
Moderate to Fast	1	1.0000	0.0420

The design discharge can be calculated from the following:

$$Q = CA$$

where: Q = Discharge or required capacity, ft^3/s
C = Subsurface runoff factor, $ft^3/s/acre$
A = Area to be drained, acres

Example:
Assuming a drainage runoff rate of 0.375 inches in 24 hours (runoff factor, C = 0.0157) and laterals 600 feet long spaced on 50 foot centers, the following result is obtained:

$$Q = 0.0157 \text{ x } \frac{600 \text{ x } 50}{43{,}560 \text{ ft}^2\text{/acre}} = 0.0108 \text{ ft}^3\text{/sec}$$

Size of Pipe

The size of pipe can be determined using Manning's formula, or by the use of a nomograph. For standard subdrainage applications, approximately 500 feet of 6 inch diameter perforated steel pipe may be used before increasing the pipe size to the next diameter.

Where possible, a minimum slope of 0.15 percent should be used for subdrainage lines. It is often permissible to use an even flatter slope to achieve a free outlet, but the steeper slope provides a self-cleansing flow velocity.

HYDRAULIC DESIGN OF STORM SEWERS

The hydraulic design of a sewer system may have to take into account the effect of backwater (the limiting effect on flows that a downstream sewer has on upstream sewers), surcharging, inlet capacity and all energy losses in the system. Whether each, or all, of these factors have to be considered depends on the complexity of the sewer system and the objectives of the analysis (i.e., whether the sizing of the system is preliminary or final). Furthermore, the degree of analysis will also depend on the potential impact should the sewer system capacity be exceeded. For example, consideration should be given to whether surcharging would result in damage to private property as a result of foundation drains being connected to the system, and whether the depth of flooding on a roadway would impact access by emergency vehicles that depend on safe access along the street. By evaluating the above factors, the designer will be in a position to select the level of analysis required.

The two hand calculation methods that follow assume all flows enter the sewer system. In other words, the inlet capacity of the system is not a limiting factor.

Flow charts and nomographs, such as those previously presented in this chapter, provide quick answers for the friction head losses in a given run of straight pipe between structures (manholes, junctions). These design aids do not consider the additional head losses associated with other structures and appurtenances common in sewer systems.

In most instances, when designing with common friction flow relationships such as the Manning equation, the hydraulic grade is assumed to be equal to the pipe slope at an elevation equal to the crown of the pipe. Consideration must therefore also be given to the changes in hydraulic grade line due to pressure changes, elevation changes, manholes and junctions. The design should not only be based on the pipe slope, but on the hydraulic grade line.

A comprehensive storm sewer design must therefore proceed on the basis of one run of pipe or channel at a time, progressing with the design methodically through the system. Only in this way can the free flow conditions be known and the hydraulic grade controlled, thus assuring performance of the system.

Making such an analysis requires backwater calculations for each run of pipe. This is a detailed process, which is demonstrated on the following pages. However, it is recognized that a reasonable conservative "estimate" or "shortcut" will sometimes be required. This can be done and is demonstrated in a discussion titled, "Method of Determining Equivalent Hydraulic Alternatives".

When using the backwater curve approach, the designer should first establish the type of flow (sub-critical or supercritical) to determine the direction in which the calculations should proceed.

- Super critical flow – a designer works downstream with flow
- Sub-critical flow – a designer works against the flow
- Note that a hydraulic jump may form if there is super and sub-critical flow in the same sewer

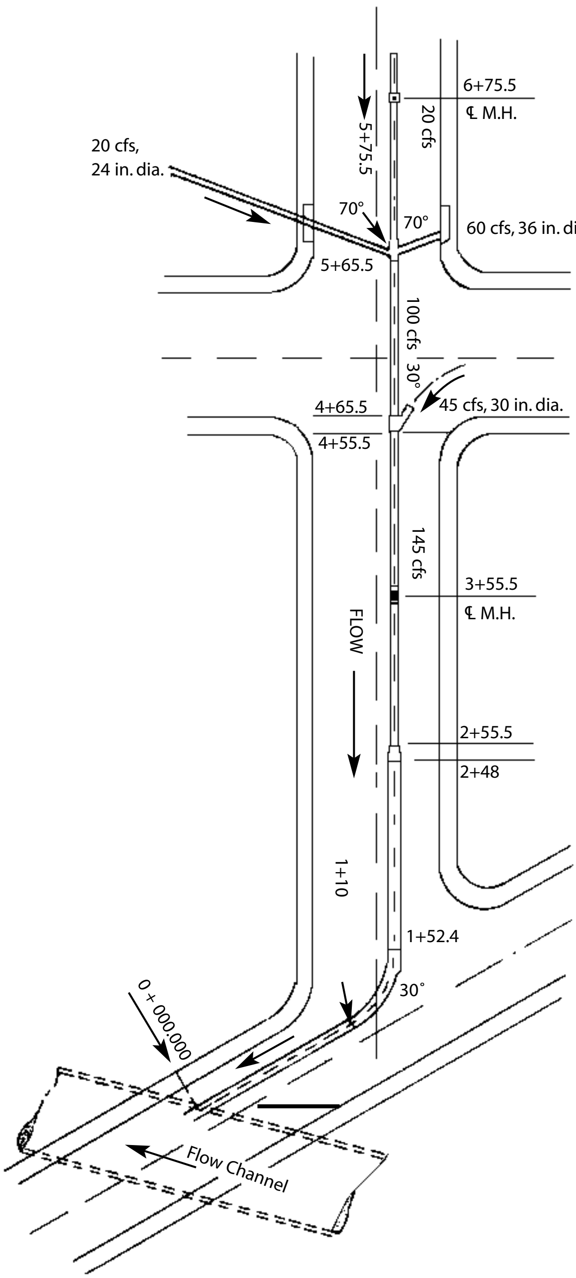
Backwater Analysis

A plan and profile of a storm drainage system is presented in Figures 5.23 and 5.24. A flow profile is shown where the hydraulic grade is set at the crown of the outlet pipe. Hydrological computations have been made, and preliminary design for the initial pipe sizing has been completed.

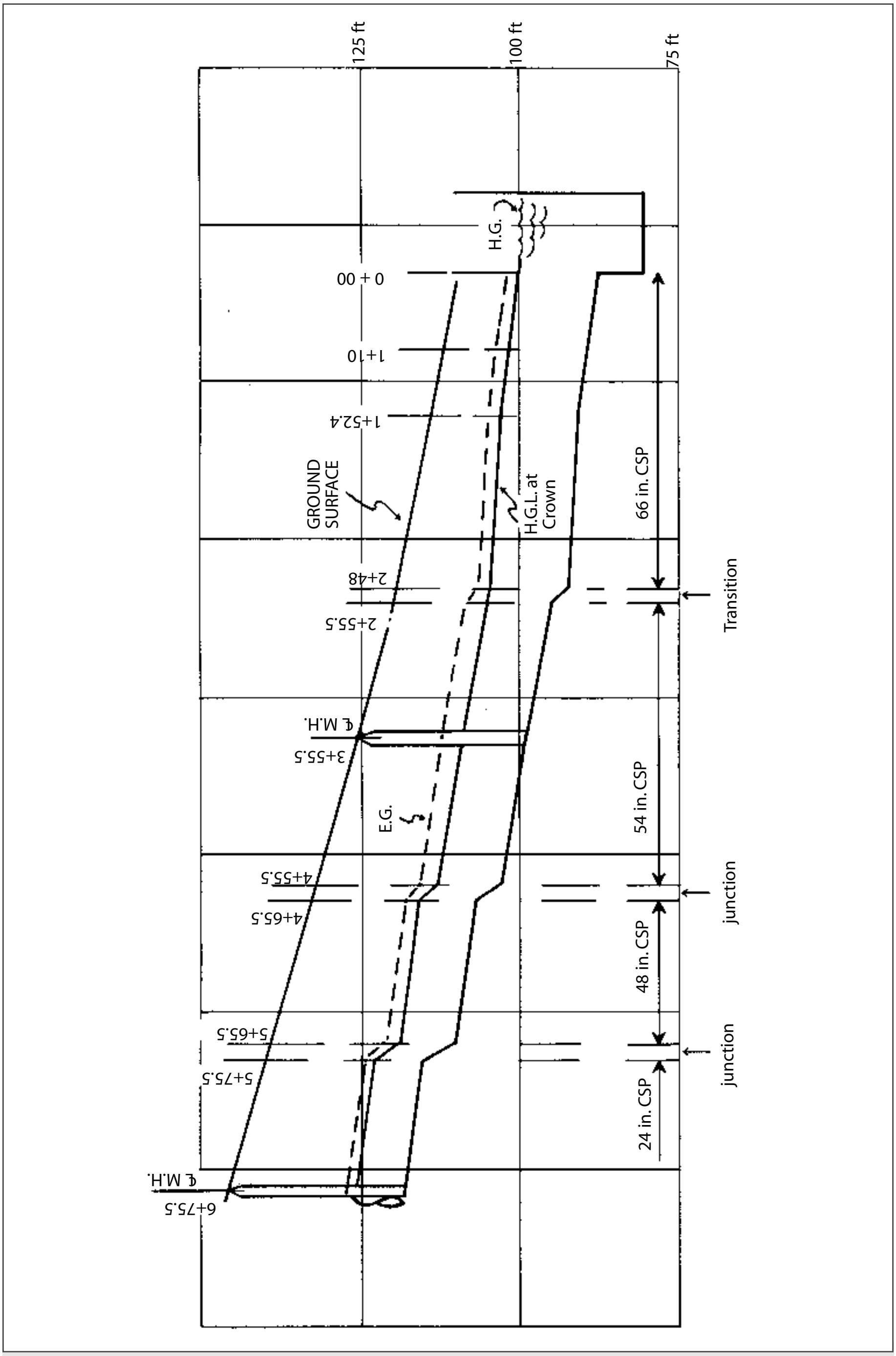
A backwater calculation will be performed in this example, using helical corrugated steel pipe, to demonstrate the significance of energy losses in sewer design. The calculation details are provided in Table 5.19.

Solution Steps:

1. Draw a plan and surface profile of the trunk storm sewer (as in Figures 5.23 and 5.24).
2. For this example, the following information is known: design discharges, Q; areas, A; and the diameters of pipes, D, have been calculated in a preliminary design.



■ **Figure 5.23** Typical plan view of storm sewer.



■ **Figure 5.24** Typical profile view of storm sewer.

Table 5.19

Sample Hydraulic Calculation Sheet

1	2	3	4	5	6		7	8	9	10	11	12	13	14	15	16	17	18	19	20
Station	Invert Elevation	Pipe Size	H.G.	Section	Area	Manning's n	K	V	Q	$V^2/2g$	E.G.	S_f	Average S_f	Length	H_f	H_b	H_j	H_m	H_t	E.G.
	(ft)	(in.)	(ft)		(ft^2)			(ft/s)	(ft^3/s)	(ft)	(ft)	(ft/ft)	(ft/ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0+00	94.50	66	100.00	Main	23.76	0.024	0.01680	6.10	145.00	0.58	100.58	0.0064	0.0064	110	0.70					100.58
1+10	95.20	66	100.70	Main	23.76	0.024	0.01680	6.10	145.00	0.58	101.28	0.0064	0.0064	42.4	0.27	0.08				101.28
1+52.4	95.55	66	101.05	Main	23.76	0.024	0.01680	6.10	145.00	0.58	101.63	0.0064	0.0064	95.6	0.61					101.63
2+48	96.16	66	101.66	Main	23.76	0.024	0.01680	6.10	145.00	0.58	102.24	0.0064	0.0110	7.5	0.08				0.14	102.24
2+55.5	97.38	54	101.88	Main	15.90	0.022	0.01412	9.12	145.00	1.29	103.17	0.0156	0.0156	100	1.56			0.06		103.17
3+55.5	99.00	54	103.50	Main	15.90	0.022	0.01412	9.12	145.00	1.29	104.79	0.0156	0.0156	100	1.56					104.79
4+55.5	100.56	54	105.06	Main	15.90	0.022	0.01412	9.12	145.00	1.29	106.35	0.0156	0.0135	10	0.14		0.87			106.35
4+65.5	102.07	48	106.07	Main	12.57	0.020	0.01167	7.96	100.00	0.98	107.05	0.0114	0.0114	100	1.14					107.05
5+65.5	103.21	48	107.21	Main	12.57	0.020	0.01167	7.96	100.00	0.98	108.19	0.0114	0.0117	10	0.12		3.78			108.19
5+75.5	109.11	24	111.11	Main	3.14	0.016	0.00747	6.37	20.00	0.63	111.74	0.0119	0.0119	100	1.19			0.03		111.74
6+75.5	110.33	24	112.33	Main	3.14	0.016	0.00747	6.37	20.00	0.63	112.96	0.0119								112.96

n = Variable $K = \frac{2g(n^2)}{2.21}$ $S_f = K\left(\frac{V^2}{2g}\right) + R^{4/3}$ $\Sigma H_{friction} = 7.37$ ft $\Sigma H_{energy} = 4.96$ ft

3. Perform calculations for the first section of pipe. Note that the normal depth is greater than the critical depth ($y_n > y_c$)so calculations begin at the outfall and move upstream. The design conditions at the "point of control" (outfall) are shown on the profile and calculation sheets:

Point of Control: Station 0 + 00 (outfall)

Note that the numbers in parentheses refer to the columns in Table 5.19.

Given:

Design discharge	Q	= 145 ft^3/s	(9)
Invert elevation of pipe		= 94.50 ft	(2)
Pipe diameter	D	= 66 in.	(3)
Hydraulic grade elevation	H.G.	= 100 ft	(4)
Area of pipe	A	= 23.76 ft^2	(6)
Velocity = Q/A	V	= 6.1 ft/s	(8)

Compute:

a. K (7): $K = (2g)\, n^2 / (1.486)^2$ (Derived from Manning-Chezy equation)

b. S_f (12):

$$S_f = K \frac{V^2}{2g} \div R^{4/3}$$

The friction slope (S_f) may be estimated from the relationships and formula in Table 5.20 for the expected flow, Q, and a given diameter of pipe with a known 'n' value.

S_f is a "point slope" at each station set by the designer. Therefore, the average friction slope, (Avg. S_f) (13), for each reach of pipe, L (14), is the average of the two point slopes S_f being considered.

c. Velocity Head (l0):

$$H_v = \frac{V^2}{2g}$$

d. Energy grade point, E.G. (11): H.G. (4) plus the velocity head (10)

e. Friction loss (15): H_f (15) = Avg. S_f (13) multiplied by the length of sewer section, L (14)

f. Calculate energy losses: H_b (16), H_j (17), H_m (18), H_t (19), using equations presented in this chapter (detailed calculations follow this design step summary)

g. Compute new H.G. (4) by adding all energy loss columns, (15) through (19), to the previous H.G.

h. Set new E.G. (20) equal to E.G. (11)

Note: If the sewer system is designed to operate under pressure (surcharging), then energy losses must be added (or subtracted, depending on whether working upstream or downstream) to the energy grade line, E.G.

i. Determine pipe invert elevation (2): In this example, we are designing for full flow conditions Therefore, H.G. (4) is at the crown of the pipe and the pipe invert (2) is set by subtracting the pipe diameter, D (3), from H.G. (4).

j. Continue to follow the above procedure taking into account all form losses

k. Complete the profile drawing showing line, grade and pipe sizes. This saves time and usually helps in spotting design errors

Energy Losses Calculations (step f above)

Station 1+10 to 1+52.4 (Bend)

$$H_b = K_b\left(\frac{V^2}{2g}\right) \text{ where } K_b = 0.25\sqrt{\frac{\text{Ø}}{90}}$$

Ø, central angle of bend = 30°

$$K_b = 0.25\sqrt{\frac{30}{90}} = 0.1443$$

$$\therefore H_b = 0.1443\ (0.58) = 0.08 \text{ ft}$$

Station 2+48 to 2+55.5 (Transition)

$$H_t = 0.2\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right)$$

$$= 0.2(1.29 - 0.58)$$

$$= 0.14 \text{ ft}$$

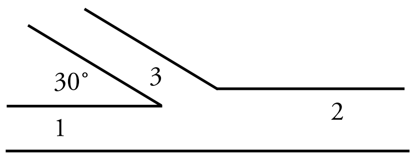
Station 3+55.5 (Manhole)

$$H_m = 0.05\left(\frac{V^2}{2g}\right)$$

$$= 0.05\ (1.29)$$

$$= 0.06 \text{ ft}$$

Station 4+55.5 to 4+65.5 (Junction)



$Q_1 = 100\ ft^3/s$	$Q_2 = 145\ ft^3/s$	$Q_3 = 45\ ft^3/s$
$A_1 = 12.57\ ft^2$	$A_2 = 15.90\ ft^2$	$A_3 = 4.91\ ft^2$
$D_1 = 48$ in.	$D_2 = 54$ in.	$D_3 = 30$ in.
		$\theta_3 = 30°$

$\sum P = \sum M$ (Pressure plus momentum laws)

$$H_j + D_1 - D_2\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2{}^2}{A_2 g} - \frac{Q_1{}^2}{A_1 g} - \frac{Q_3{}^2 \cos\theta_3}{A_3 g}$$

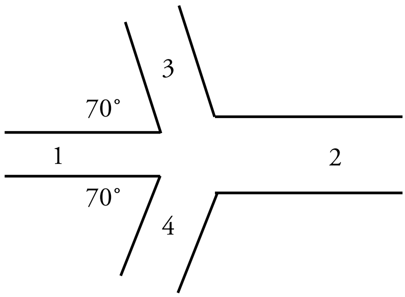
$$(H_j + 4 - 4.5)\,\frac{12.57+15.90}{2} = \frac{(145)^2}{(15.90)(32.2)} - \frac{(100)^2}{(12.57)(32.2)} - \frac{(45)^2 \cos 30°}{(4.91)(32.2)}$$

$$14.24\ H_j - 0.5\ (14.24) = 41.07 - 24.71 - 11.09$$

$$14.24\ H_j - 7.12 = 5.27$$

$$H_j = 0.87 \text{ ft}$$

Station 5+65.5 to 5+75.5 (Junction)



$Q_1 = 20\ ft^3/s$ $\quad Q_2 = 100\ ft^3/s$ $\quad Q_3 = 60\ ft^3/s$ $\quad Q_4 = 20\ ft^3/s$

$A_1 = 3.14\ ft^2$ $\quad A_2 = 12.57\ ft^2$ $\quad A_3 = 7.07\ ft^2$ $\quad A_4 = 3.14\ ft^2$

$D_1 = 24$ in. $\quad D_2 = 48$ in. $\quad D_3 = 36$ in. $\quad D_4 = 24$ in

$\theta_3 = 70°$ $\quad \theta_4 = 70°$

$$H_j + D_1 - D_2 \left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2^2}{A_2 g} - \frac{Q_1^2}{A_1 g} - \frac{Q_3^2 \cos\theta_3}{A_3 g} - \frac{Q_4^2 \cos\theta_4}{A_4 g}$$

$$(H_j + 2 - 4)\ \frac{3.14 + 12.57}{2} = \frac{(100)^2}{(12.57)(32.2)} - \frac{(20)^2}{(3.14)(32.2)} - \frac{(60)^2 \cos 70°}{(7.07)(32.2)} - \frac{(20)^2 \cos 70°}{(3.14)(32.2)}$$

$$7.855\ H_j - 2\ (7.855) = 24.706 - 3.956 - 5.409 - 1.353$$

$$7.855\ H_j - 15.71 = 13.988$$

$$H_j = 3.78\ ft$$

Station 6+75.5 (Manhole)

$$H_m = 0.05 \left(\frac{V^2}{2g}\right)$$

$$= 0.05\ (0.64)$$

$$= 0.03\ ft$$

Total friction loss, H_f, for the system totaled 7.37 feet. Total form energy losses for the system totaled 4.96 feet.

Table 5.20

Energy-loss Solution by Manning's Equation For Pipe Flowing Full

Diameter	Area	Hydraulic Radius			$\left(\frac{n}{1.486 AR^{2/3}}\right)^2 \times 10^{-7}$				
	A	R	$R^{2/3}$	$AR^{2/3}$	n = 0.012	n = 0.015	n = 0.019	n = 0.021	n = 0.024
(in.)	(ft^2)	(ft)							
6	.196	0.125	0.250	0.049	271,600	424,420	681,000	831,940	1,086,350
8	.349	0.167	0.303	0.106	58,000	90,703	145,509	177,730	232,164
10	.545	0.208	0.351	0.191	17,879	27,936	44,802	54,707	71,455
12	.785	0.250	0.397	0.312	6,698	10,466	17,797	20,605	26,791
15	1.227	0.312	0.461	0.566	2,035.6	3,180.8	5,102.5	6,234.4	8,144.6
18	1.767	0.375	0.520	0.919	772.2	1,206.5	1,935.5	2,364.7	3,088.7
21	2.405	0.437	0.576	1.385	340.00	531.24	852.60	1,041.0	1,359.98
24	3.142	0.500	0.630	1.979	166.5	260.04	417.31	510.20	666.39
30	4.909	0.625	0.731	3.588	50.7	79.126	127.01	155.12	202.54
36	7.069	0.750	0.825	5.832	19.20	29.953	48.071	58.713	76.691
42	9.621	0.875	0.915	8.803	8.40	13.148	21.096	25.773	33.667
48	12.566	1.000	1.000	12.566	4.130	6.452	10.353	12.647	16.541
54	15.904	1.125	1.082	17.208	2.202	3.440	5.520	6.741	8.817
60	19.635	1.250	1.160	22.777	1.257	1.965	3.337	3.848	5.030
66	23.758	1.375	1.236	29.365	0.756	1.182	1.895	2.316	3.026
72	28.274	1.500	1.310	37.039	0.475	0.743	1.192	1.456	1.902
78	33.183	1.625	1.382	45.859	0.310	0.485	0.777	0.950	1.241
84	38.485	1.750	1.452	55.880	0.209	0.326	00.524	0.640	0.835
90	44.179	1.875	1.521	67.196	0.144	0.226	0.362	0.442	0.578
96	50.266	2.000	1.587	79.772	0.102	0.160	0.257	0.314	0.410
108	63.617	2.250	1.717	109.230	0.055	0.085	0.137	0.167	0.219
114	70.882	2.375	1.780	126.170	0.041	0.064	0.103	0.125	0.164
120	78.540	2.500	1.842	144.671	0.031	0.049	0.078	0.098	0.125

Manning Flow Equation: $Q = \left(A \times \frac{1.486}{n} \times R^{2/3}\right) \times S^{1/2}$

Energy Loss = $S = Q^2 \left(\frac{n}{1.486 AR^{2/3}}\right)^2$

To find energy loss in pipe friction for a given Q, multiply Q^2 by the factor under the proper value of n

In this example, the head losses at the transition and junctions could also have been accommodated by either increasing the pipe diameter or increasing the slope of the pipe.

This backwater example was designed under full flow conditions but could also have been designed under pressure, allowing surcharging in the manholes, which would have reduced the pipe sizes. Storm sewer systems, in many cases, can be designed under pressure to surcharge to a tolerable hydraulic grade elevation.

Method Of Determining Equivalent Hydraulic Alternatives

A method has been developed to aid the designer in quickly determining equivalent pipe sizes for alternative materials, rather than computing the backwater profiles for each material.

The derivation shown below allows the designer to assign representative values for loss coefficients in the junctions and for average pipe length between the junctions, and develop a relationship for pipes of different roughness coefficients. As a result, the designer need only perform a detailed hydraulic analysis for one material, and then easily determine pipe sizes required for alternative materials. The relationships for hydraulic equivalent alternatives in storm sewer design are derived from the friction loss equation.

The total head loss in a sewer system is composed of junction (form) losses and friction losses:

$$H_T = H_j + H_f$$

where: H_T = Total head loss, ft
H_j = Junction losses, ft
H_f = Friction losses, ft

For the junction losses:

$$H_j = K_j \frac{V^2}{2g}$$

$$= K_j \frac{Q^2}{A^2 2g}$$

$$= K_j \frac{Q^2(16)}{\pi^2 D^4 2g}$$

For the friction losses:

$$V = \frac{(1.486)}{n} R^{2/3} S_f^{1/2}$$

$$S_f^{1/2} = \frac{n\,V}{(1.486)R^{2/3}}$$

$$S_f = \frac{n^2\,V^2}{(1.486)^2 R^{4/3}}$$

$$S_f = \frac{2g\,n^2}{(1.486)^2 R^{4/3}} \frac{V^2}{2g}$$

$$H_f = \frac{2g\;n^2\;L}{(1.486)^2 R^{4/3}} \frac{V^2}{2g}$$

$$H_f = \frac{2g\;n^2\,L\,Q^2}{(1.486)^2 R^{4/3}\,A^2 2g}$$

$$H_f = \frac{2g\;n^2\,L\,Q^2\,(16)}{(1.486)^2 R^{4/3}\,\pi^2 D^4 2g}$$

$$H_f = \frac{2g\;n^2\,L\,Q^2\,(16)\;(4)^{4/3}}{(1.486)^2 D^{4/3}\,\pi^2 D^4 2g}$$

$$H_f = \frac{2g(4)^{4/3}}{(1.486)^2} \frac{n^2\,L}{D^{4/3}} \frac{Q^2(16)}{\pi^2 D^4 2g}$$

Then, for the total head loss:

$$H_T = H_j + H_f$$

$$= K_j \frac{Q^2(16)}{\pi^2 D^4 2g} + \frac{2g(4)^{4/3}}{(1.486)^2} \frac{n^2 L}{D^{4/3}} \frac{Q^2(16)}{\pi^2 D^4 2g}$$

$$= \frac{8Q^2}{g\pi^2}\left[\frac{K_j}{D^4} + \frac{2g(4)^{4/3}}{(1.486)^2} \frac{n^2 L}{D^{16/3}}\right]$$

$$= \frac{8Q^2}{g\pi^2}\left[\frac{K_j D^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n^2 L}{D^{16/3}}\right]$$

Thus, for comparison of a smooth walled pipe and a corrugated steel pipe (the subscripts "s" and "c" denote smooth and corrugated):

$$\frac{8Q^2}{g\pi^2}\left[\frac{K_j D_S{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_S{}^2 L}{D_S{}^{16/3}}\right] = \frac{8Q^2}{g\pi^2}\left[\frac{K_j D_C{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_C{}^2 L}{D_C{}^{16/3}}\right]$$

The flow, Q, for each pipe will be the same, therefore the relationship simplifies to:

$$\frac{K_j D_S{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_S{}^2 L}{D_S{}^{16/3}} = \frac{K_j D_C{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_C{}^2 L}{D_C{}^{16/3}}$$

The pipe length between manholes, L, and the junction loss coefficient, K_j, may be derived from hydraulic calculations already performed for one of the materials. A required pipe diameter, for a specific Manning's n, would have resulted from those calculations. The above equation can then be solved, through trial and error, for the one remaining unknown which is the pipe diameter of the alternate material.

Another approach involves the use of representative values of the average pipe length between manholes, L, and the junction loss coefficient, K_j, derived from hydraulic calculations already performed for one of the materials. Each side of the above equation is solved independently for a number of pipe diameters and the specific Manning's n for the pipe material (and in some cases a specific Manning's n for the diameter of pipe). The results are plotted on a semi-log scale, and the plot is used to select pipe diameters of alternate materials that will provide the same performance.

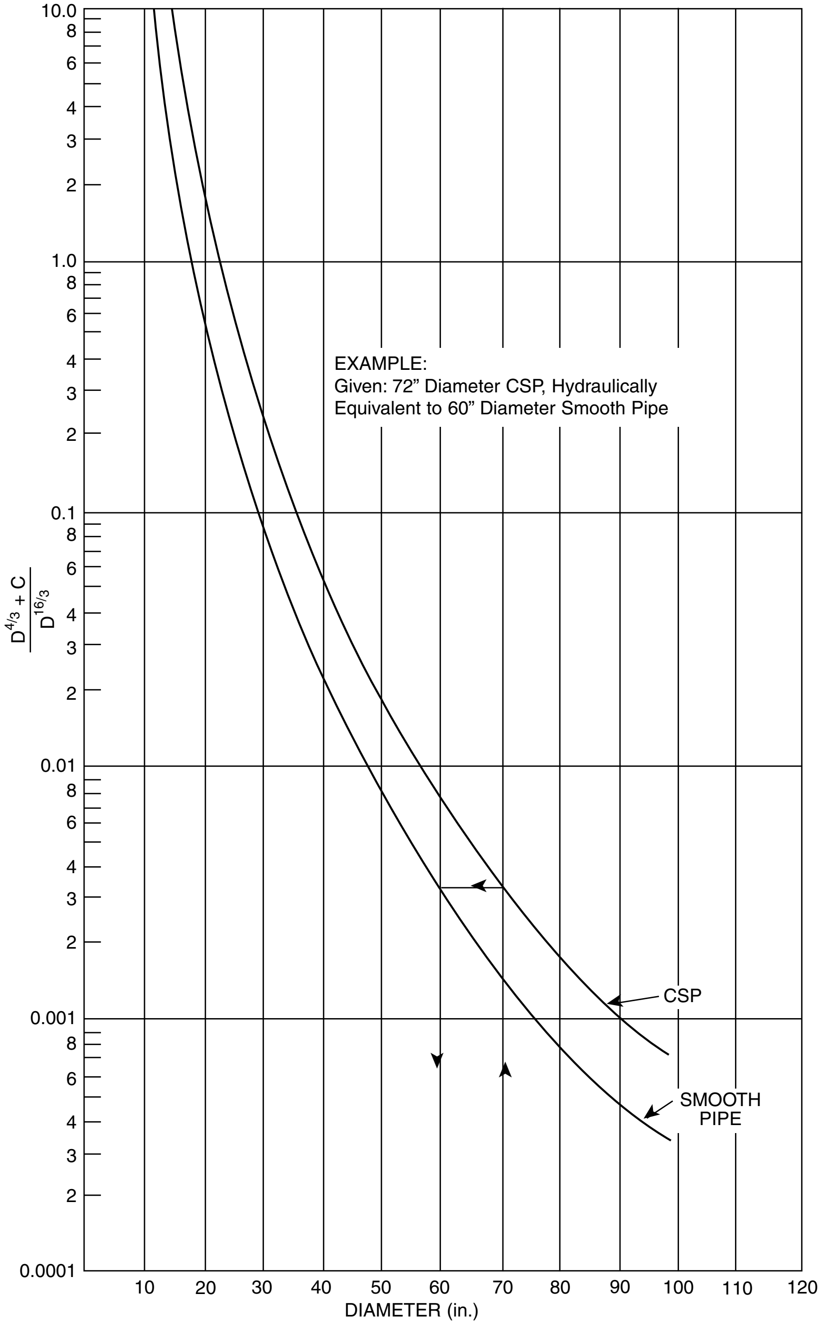
For example, Table 5.21 contains the results of such calculations based on an average pipe length of 300 feet, an average junction loss coefficient of 1.0, and Manning's n values for the pipe materials and sizes shown. The results are plotted, using a semi-log scale, so that hydraulically equivalent materials may be easily selected as demonstrated in Figures 5.25 and 5.26.

Table 5.21

Determining Equivalent Alternatives

	Junction and Friction Losses (K_j = 1.0, L = 300 ft)			
	Smooth Pipe	Annular CSP Pipe	Helical CSP Pipe	
	n = 0.012	n = 0.024		
Diameter (in.)	$\frac{D^{4/3} + 8.00}{D^{16/3}}$	$\frac{D^{4/3} + 32.00}{D^{16/3}}$	n	$\frac{D^{4/3} + 55\,554\,n^2}{D^{16/3}}$
12	9.00	33.00	0.011	7.72
15	2.84	10.14	0.012	2.84
18	1.12	3.88	0.013	1.28
21	0.511	1.72	0.014	0.657
24	0.261	0.856	0.015	0.373
30	0.0860	0.267	0.017	0.147
36	0.0352	0.104	0.018	0.0637
42	0.0167	0.0468	0.019	0.0318
48	0.00883	0.0236	0.020	0.0176
54	0.00506	0.0129	0.021	0.0105
60	0.00310	0.00759	0.021	0.00618
66	0.00199	0.00469	0.021	0.00385
72	0.00134	0.00304	0.021	0.00251
78	0.000930	0.00204	0.021	0.00169
84	0.000665	0.00141	0.021	0.00118
90	0.000488	0.00100	0.021	0.000843
96	0.000366	0.000732	0.021	0.000618

Notes: Pipe diameter in feet in above tabular values.



■ **Figure 5.25** Equivalent alternatives to smooth wall pipe using annular CSP 2-2/3 x 1/2 in. where C = 55,554 n^2.

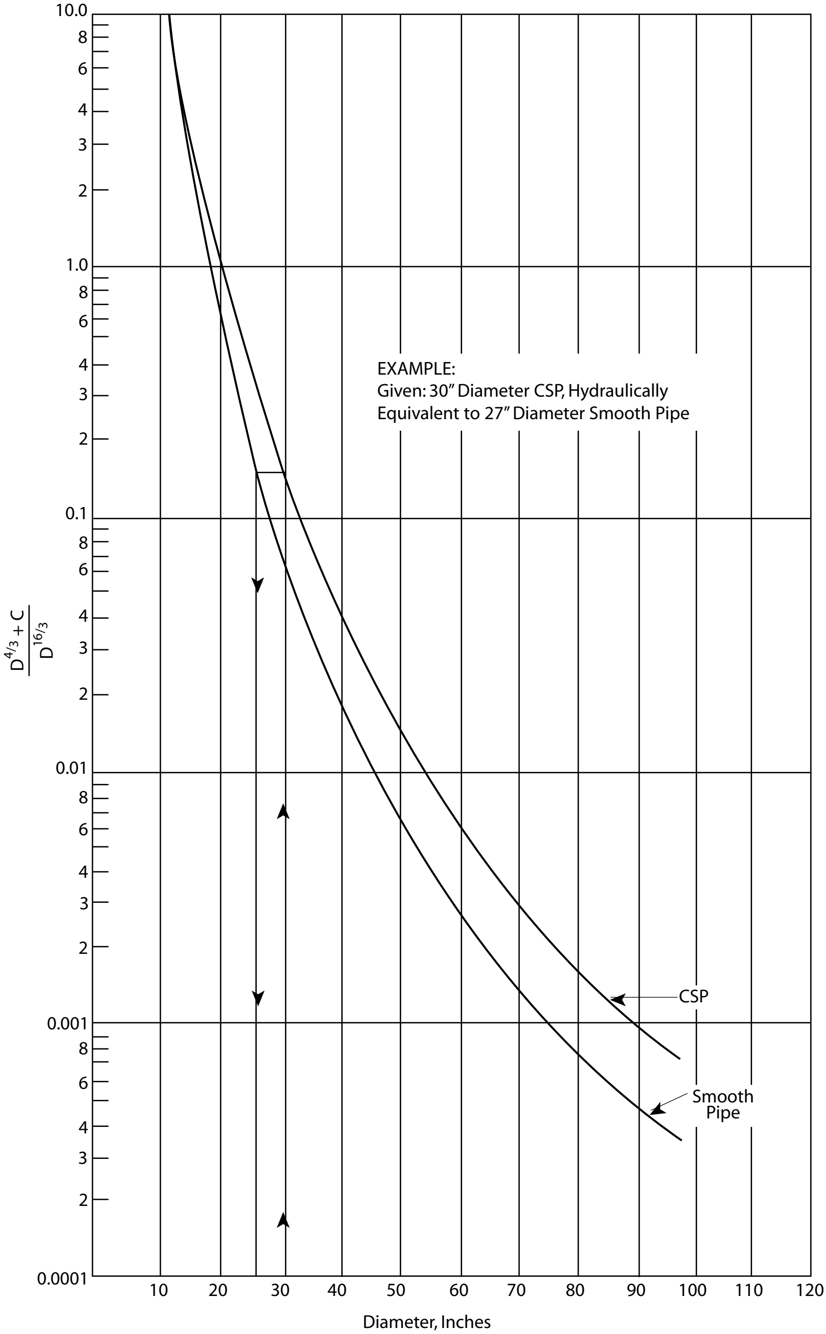


Figure 5.26 Equivalent alternatives to smooth wall pipe using helical CSP (n variable) 2-2/3 x 1/2 in. where C = 55,554 n^2.

Design Of Storm Drainage Facilities

System Layout

The storm drainage system layout should be made in accordance with the urban drainage objectives, following the natural topography as closely as possible. Existing natural drainage paths and watercourses, such as streams and creeks, should be incorporated into the storm drainage system. Thus the storm design should be undertaken prior to finalization of the street layout to effectively incorporate the major-minor drainage concepts.

Topographic maps, aerial photographs, and drawings of existing services are required before a thorough storm drainage design may be undertaken.

Existing outfalls, within the proposed development and adjacent lands for both the minor and major system, should be located. Allowances should be made in the design for runoff from external lands draining through the proposed development both for present conditions and future developments.

The design flows used in sizing the facilities, that will comprise the drainage network, are based on a number of assumptions. Actual flows will be different from those estimated at the design stage; the designer must not be tempted by the inherent limitations of the basic flow data to become sloppy in the hydraulic design. Also, designers should not limit their investigation to system performance under the design storm conditions, but should also assure that in cases where sewer capacities are exceeded, such incidents will not create excessive damage.

This requirement can only be practically achieved if the designer realizes that a dual drainage system exists. A drainage system is comprised of a minor system and a major system. The minor (pipe) system is typically utilized for smaller, more frequent rainfall events, while the major (overland) system is used for extreme rainfall events.

In the layout of an effective storm drainage system, the most important factor is to assure that a drainage path both for the minor and major systems be provided to avoid flooding and ponding in undesirable locations.

Minor System

The minor system consists chiefly of the storm sewer comprised of inlets, pipes, manholes and other appurtenances designed to collect and convey into a satisfactory system outfall, storm runoff for frequently occurring storms (2 to 5-year design).

Storm sewers are usually located in rights-of-way such as roadways and easements for ease of access during repair or maintenance operations.

Major System
The major drainage system will come into operation when the minor system's capacity is exceeded or when inlet capacities significantly control inflow into the minor system. In developments where the major system has been planned, the streets will act as open channels draining the excess storm water. The depth of flow on the streets should be kept within reasonable limits for reasons of safety and convenience. Consideration should be given to the area of flooding and its impact on various street classifications and to public and private property. Typical design considerations are given in Table 5.22.

Table 5.22

Typical Maximum Allowable Flow Depths

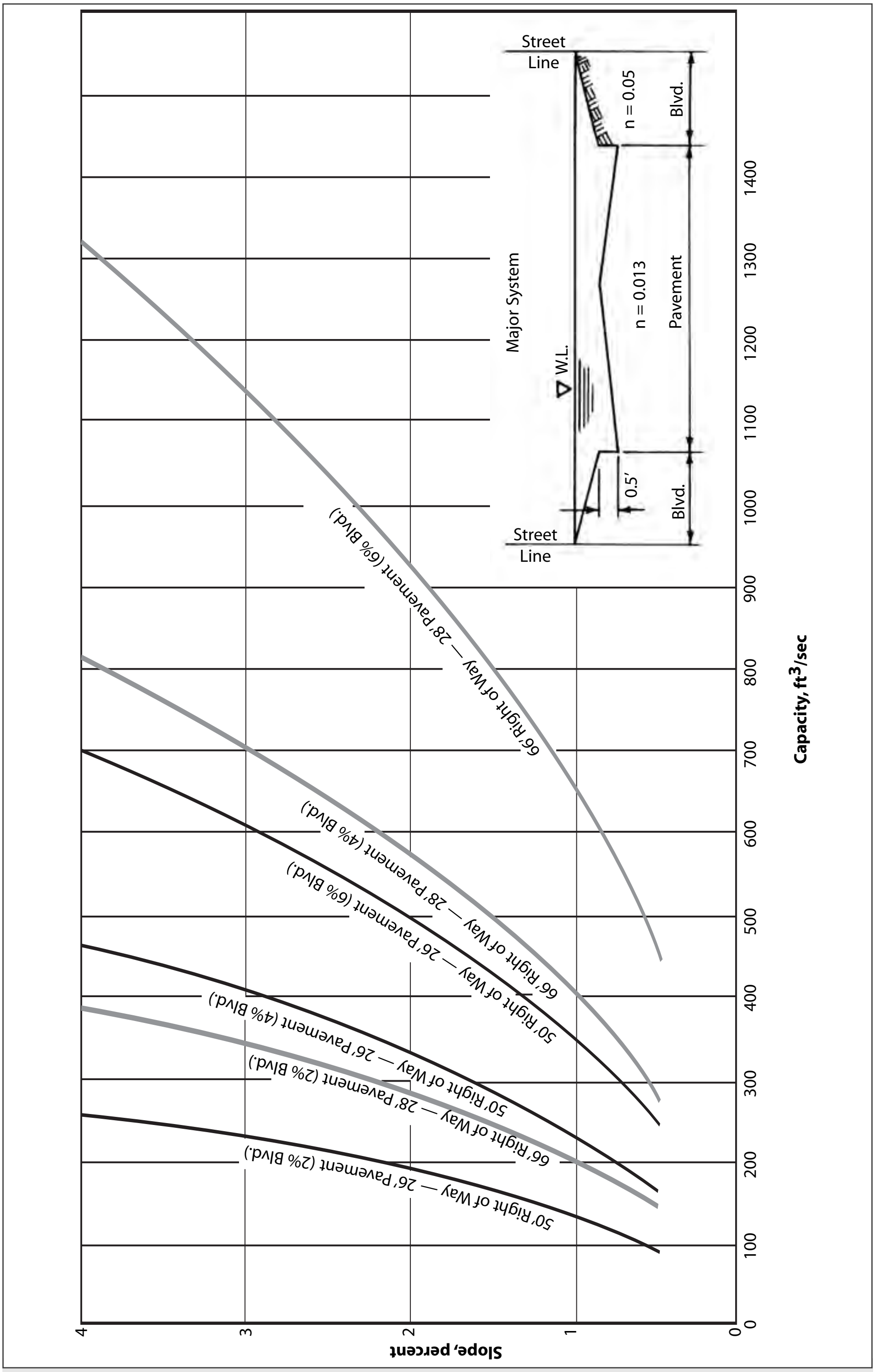
Location*	Storm Return Frequency (Years)		
	5	25	40
Walkways, Open spaces	Minor surface flow up to 1 in. deep on walkways	As required for overland flow outlets	As required for overland flow outlets
Minor, Local and Feeder Roads	3 ft wide in gutters or 4 in. deep at low point catch basins	4 in. above crown	8 in. above crown
Collector and Industrial Roads	Minor surface flow 1 in.	Up to crown	4 in. above crown
Arterial Roads	Minor Surface flow 1 in.	1 lane clear	Up to crown

Notes: *In addition to the above, residential buildings, public, commercial and industrial buildings should not be inundated at the ground line for the 100 year storm, unless buildings are flood-proofed.

To prevent the flooding of basement garages, driveways must meet or exceed the elevations corresponding to the maximum flow depth at the street.

The flow capacity of the streets may be calculated from the Manning equation, or Figure 5.27 may be used to estimate street flows.

When designing the major system, it should be done in consideration of the minor system, with the sum of their capacities being the total system's capacity. The minor system should be first designed to handle a selected high frequency storm (i.e., 2 to 5 or 10-year storm). The major system is then designed for a low frequency storm, (i.e., 100-year storm). If the roadway cannot handle the excess flow, the minor system should be enlarged to compensate.



■ **Figure 5.27** Hydraulic capacity of roadways.

HYDRAULIC DESIGN EXAMPLE OF MINOR-MAJOR SYSTEM

Description of Site

The site for this design example is shown on Figure 5.28.

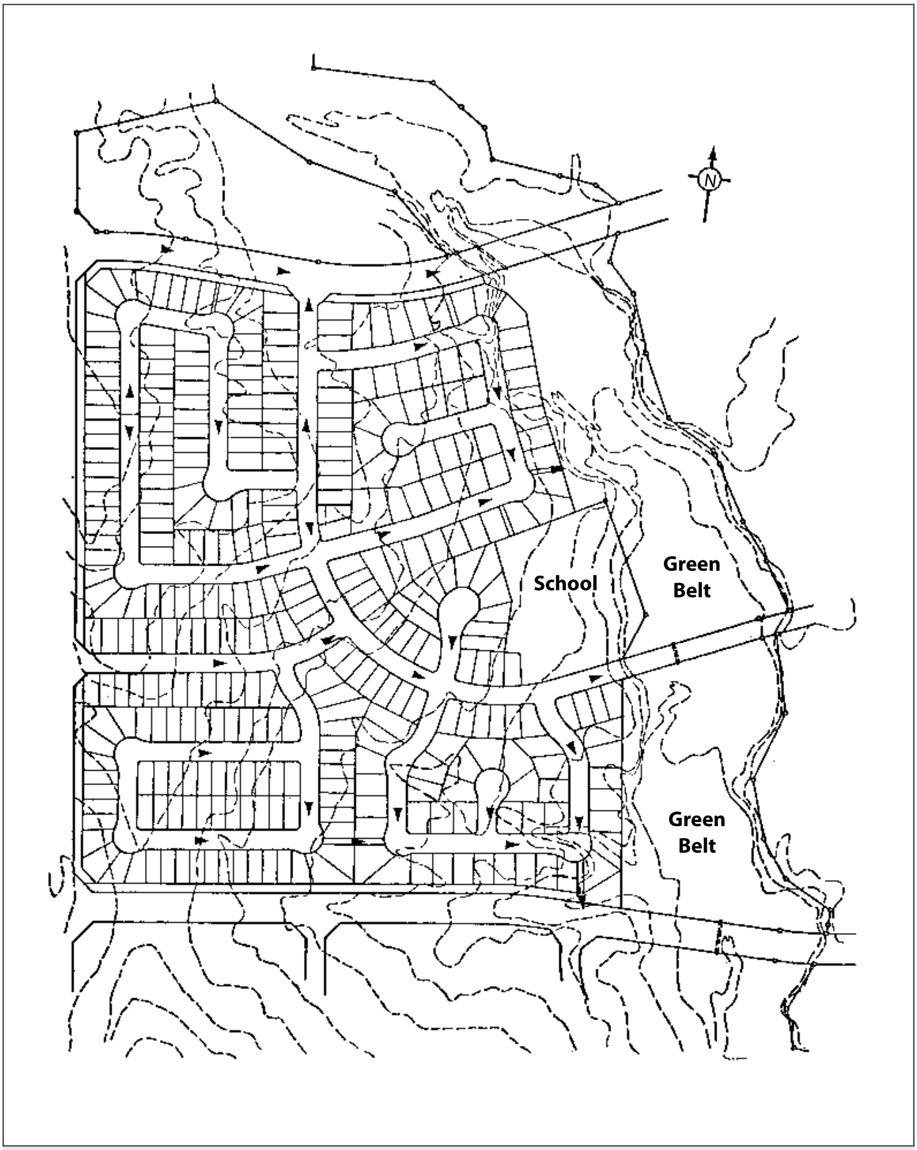
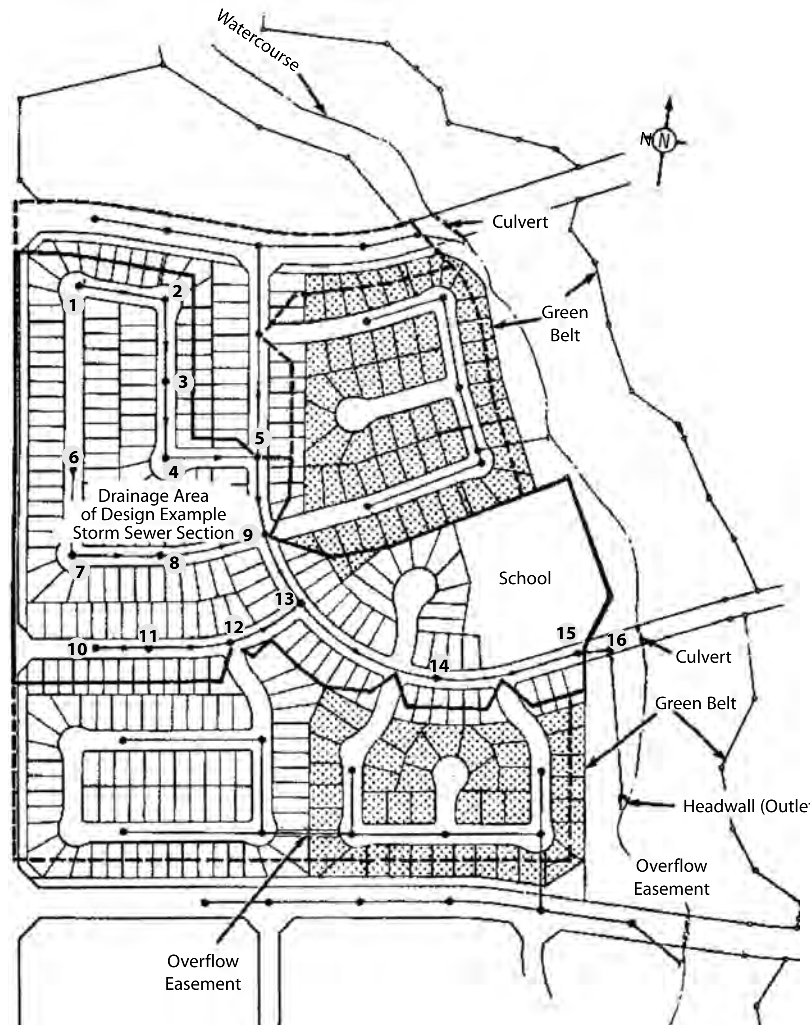


Figure 5.28 Typical site plan with route of surface runoff.

The site is about 38 acres in size consisting of single family and semi-detached housing as well as a site for a public school. The site slopes generally from west to east, where it is bounded by a major open water course. To accommodate the principles of the "minor-major" storm drainage systems, the streets have been planned to conform as much as possible to the natural contours of the lands. Where sags in roadways between intersections could not be avoided, overflow easements or walkways have been provided to permit unobstructed surface runoff during major storms, as shown on Figure 5.29.



■ **Figure 5.29** Typical site plan with storm drainage areas.

Selected Design Criteria

Based on a reasonable level of convenience to the public, a two-year storm is considered adequate as a design basis for the minor system within this development. The major (or overflow) system will be checked together with the minor system against a 100-year storm intensity. The combination of these two systems must be able to accommodate the 100-year storm runoff.

Minor System

For the limited extent of area involved, designing on the principles of the minor-major drainage concept, without gravity connections to foundation drains, permits considerable tolerance in the degree of accuracy of runoff calculations such that the rational equation is considered adequate. The values for the two-year rainfall intensity curve, obtained from local records, are shown in Table 5.23.

Table 5.23

Example Rainfall Intensity-Duration-Frequency Data

Time	2-Year Return	100 Year Return
(Min)	(in./hr)	(in./hr)
5	4.15	10.33
10	2.85	7.04
15	2.25	5.74
20	1.88	4.80
25	1.65	4.30
30	1.47	3.81
35	1.32	3.50
40	1.20	3.20
45	1.10	2.90
50	1.04	2.70
55	0.96	2.50
60	0.98	2.31
65	0.81	2.15
70	0.75	2.00
75	0.69	1.85
80	0.63	1.75
85	0.58	1.63
90	0.53	1.55
95	0.49	1.50
100	0.45	1.30
125	0.40	1.27
150	0.35	1.00
175	0.31	0.90
200	0.27	0.86

The following steps should be followed in the hydraulic design of the minor system:

1. A drainage area map is prepared indicating the drainage limits for the site, external tributary areas, the location of imported minor system and carryover flows, a proposed minor-major system layout, and the direction of surface flow.

2. The drainage area is divided into sub-areas tributary to the proposed storm sewer inlets. In this case, the inlet is located at the upstream end of each pipe segment.

3. The area of each sub-area is calculated.

4. Appropriate runoff coefficients are developed for each sub-area. The example has been simplified in that impervious areas discharging to grass areas have been given a runoff coefficient equal to that of the grassed area. The runoff coefficients chosen for this example are 0.20 for grassed areas and areas discharging to grass such as roof, patios and sidewalks), and 0.95 for impervious surfaces (streets and driveways). This results in an average runoff coefficient of 0.35 for all the sub-areas of this specific site. The area of each sub-area is multiplied by the runoff coefficient for that sub-area, and summed for each section of sewer. An "area x C" value is determined for each inlet.

5. The inflow rate for the first inlet of a sewer section is calculated by the rational method, using the initial time of concentration and the corresponding intensity. The inflow rate is the "area x C" value multiplied by the intensity. As mentioned above, inlets will be located at the upstream manhole for each length of pipe. In this example, the initial values are as flows:

   T_c = Time of concentration = 10 minutes.
   i = 2.85 in./hr for the 2-year storm (Table 5.23).

6. Commencing at the upstream end of the system, the inflow to be carried by each pipe is calculated one at a time. The initial time of concentration, as mentioned above, is 10 minutes at the most up-stream inlet of each section of sewer. In this design example, a helical 2 2/3 x 1/2 in. corrugation CSP with variable roughness coefficients (Table 5.9) has been selected as the pipe material. The capacity of a pipe is calculated using Manning's n, the slope of the pipe and the section properties of the pipe flowing full ($AR^{2/3}$). Once a tentative pipe size is chosen that will accommodate the required flow rate for the first pipe, the flow velocity is calculated (flow rate divided by pipe end area), the travel time is determined (pipe length divided by velocity), and the new time of concentration for the next pipe is the sum of the previous time of concentration and the travel time. The resulting time of concentration is then used to determine a new intensity for the next inflow (required capacity) calculation. At a junction of two or more pipes, the longest time of concentration is selected and the procedure continues downstream. If upon completion of the hydraulic design (and backwater calculations) the times of concentrations have varied enough to alter the discharges, new flow values should be determined. In most cases the slight variance in the time of concentration will not significantly affect the peak flows.
   Note that design velocities in storm sewers should be a minimum of 3 ft/s when

flowing half full to full to prevent deposition by attaining self cleaning velocities. A maximum allowable velocity of 15 ft/s is used so as to avoid erosive damage to the pipe.

The above computations are summarized in Table 5.24.

7. Manhole and junction losses are considered as the design proceeds downstream. Certain rules of thumb may be used in the preliminary hydraulic analysis. In this design example, the following manhole drops are assumed:

   0.05 feet for straight runs
   0.15 feet for junctions with up to a 45° change in direction
   0.25 feet for 45° to 90° junctions

   Also, crowns of incoming and outgoing pipes at manholes are kept equal where the increase in downstream diameter meets or exceeds the above manhole drops.

   The preliminary minor system design is shown in Table 5.24 with the tentative pipe sizes and manhole drops.

8. The hydraulic analysis is then performed on the proposed minor system to ensure that it operates as expected. The hydraulic grade is set at the crown of the outlet pipe, with hydraulic calculations proceeding upstream. The energy loss equations are done using the same procedure as previously detailed. The detailed hydraulic calculations follow this step-by-step design summary, with the results summarized in Table 5.25.

   In this example the initial pipe sizes did not change, but rather manhole drops were adjusted to account for the junction losses. If junction losses had resulted in the elevation of the pipe crown exceeding the minimum cover criteria, then the hydraulic grade line may have been lowered by increasing the pipe size.
   The hydraulic grade line may be permitted to exceed the crown where surcharging in the storm system can be tolerated.

9. The designer can estimate the required pipe sizes for a minor system for an alternative pipe material or roughness coefficient. There is no need to perform a detailed hydraulic analysis for the alternative pipe, but rather use the method of "Equivalent Alternatives" as described earlier in this chapter. In this example, the average length of pipe is estimated to be 300 feet with an average manhole junction loss coefficient of 1.0. The alternative pipe will have constant n = .012. The results are summarized in Table 5.26.

Table 5.24

Example Preliminary Storm Sewer Design

Sewer Description and Data									Runoff Calculation							Pipe Design						Time of Concentration Entry = 10 min.	
							M.H. Invert El.																
Street	From M.H.	To M.H.	Length	Slope	Fall	M.H. Drop	Up-Stream	Down-Stream	Drainage Area A	C	A x C	Total Section A x C	Total Trunk A x C	Intensity I	Flow Q	Pipe Size	Area A	Pipe $AR^{2/3}$	Manning's n	Actual Cap. Q	Velocity V	Sect.ion	Accum.
			(ft)	%	(ft)	(ft)	(ft)	(ft)	(ac)		(ac)	(ac)	(ac)	(in./hr)	(ft^3/s)	(in.)	(ft^2)	($ft^{8/3}$)		(ft^3/s)	(ft/s)	(min.)	(min.)
A	1	2	300	0.84	2.52		771.71	769.19	1.82	0.35	0.64	0.64		2.85	1.82	10	0.545	0.192	0.014	1.87	3.43	1.46	11.46
A	2	3	260	1.30	3.38	0.25	768.94	765.56	2.73	0.35	0.96	1.60		2.67	4.27	12	0.785	0.312	0.012	4.41	5.62	0.77	12.23
A	3	4	265	0.98	2.60	0.25	765.31	762.71	2.57	0.35	0.90	2.50		2.58	6.45	15	1.227	0.565	0.013	6.39	5.21	0.85	13.08
A	4	5	306	1.50	4.59	0.25	762.46	757.87	2.06	0.35	0.72	3.22		2.48	7.99	15	1.227	0.565	0.013	7.91	6.45	0.79	13.87
B	6	7	300	1.70	5.10		771.62	766.52	2.63	0.35	0.92	0.92		2.85	2.62	10	0.545	0.192	0.014	2.66	4.88	1.02	11.02
B	7	8	300	1.70	5.10	0.25	766.27	761.17				0.92			2.62	10	0.545	0.192	0.014	2.66	4.883	1.02	12.04
B	8	9	245	2.20	5.39	0.17	761.00	755.61	3.70	0.35	1.30	2.22		2.61	5.79	12	0.785	0.312	0.012	5.73	7.30	0.56	12.604
C	10	11	300	1.40	4.20		767.54	763.34	4.46	0.35	1.56	1.56		2.85	4.45	12	0.785	0.312	0.012	4.57	5.82	0.86	10.86
C	11	12	275	2.40	6.60	0.05	763.29	756.69	1.76	0.35	0.62	2.18		2.75	6.00	12	0.785	0.312	0.012	5.99	7.63	0.60	11.46
C	12	13	265	1.10	2.92	0.25	756.44	753.52	1.05	0.35	0.37	2.55		2.67	6.81	15	1.227	0.565	0.013	6.77	5.52	0.80	12.26
Main	5	9	265	0.76	2.01	0.25	757.62	755.61	1.06	0.35	0.37		3.59	2.39	8.58	18	1.767	0.919	0.014	8.50	4.81	0.92	14.79
Main	9	13	265	0.60	1.59	0.50	755.11	753.52	1.32	0.35	0.46		6.27	2.28	14.30	24	3.142	1.979	0.016	14.24	4.53	0.97	15.76
Main	13	14	500	1.70	8.50	0.25	753.27	744.77	5.64	0.35	1.97		10.79	2.19	23.63	24	3.142	1.979	0.016	23.96	7.63	1.09	16.85
Main	14	15	500	1.80	9.00	0.05	744.72	735.72	1.37	0.35	0.48		11.27	2.11	23.78	24	3.142	1.979	0.016	24.66	7.854	1.06	17.91
Main	15	16	110	1.20	1.32	0.25	735.47	734.15	5.81	0.20	1.16		12.43	2.03	25.23	27	3.976	2.709	0.017	25.94	6.52	0.28	18.19
Main	16	Outfall	500	.068	3.40	0.25	733.90	730.50							25.23	30	4.909	3.589	0.017	25.87	5.27	1.58	19.77

Table 5.25

Example Hydraulic Calculation Sheet

M.H.	Invert Elevation	Pipe Size	H.G.	Manning's n	Area	K	Q	V	$\frac{V^2}{2g}$	E.G.	S_f	Length	H_f	H_b	H_j	H_m	H_t	E.G.
	(ft)	(in.)	(ft)		(ft^2)		(ft^3/s)	(ft/s)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	
Outlet	730.50	30	733.00	0.017	4.909	0.0084	25.23	5.14	0.41	733.41								733.41
16	733.86	30	736.36	0.017	4.909	0.0084	25.23	5.14	0.41	736.77	0.0064	500	3.20	0.12				736.77
15	735.41	27	737.66	0.017	3.976	0.0084	25.23	6.35	0.63	738.29	0.114	110	1.25				0.04	738.29
14	744.18	24	746.18	0.016	3.142	0.0075	23.78	7.57	0.89	747.07	0.0168	500	8.40	0.07		0.05	0.05	747.07
13	753.69	24	755.69	0.016	3.142	0.0075	23.63	7.52	0.88	756.57	0.0166	500	8.30	0.10	1.11			756.57
9	756.11	24	758.11	0.016	3.142	0.0075	14.30	4.55	0.32	758.43	0.0060	265	1.59	0.03	0.80			758.43
5	759.18	18	760.68	0.014	1.767	0.0057	8.58	4.86	0.37	761.05	0.0078	265	2.07	0.43			0.07	761.05
4	764.99	15	766.24	0.013	1.227	0.0049	7.99	6.51	0.66	766.90	0.0153	306	4.68	0.88				766.90
3	767.64	15	768.89	0.013	1.227	0.0049	6.45	5.26	0.43	769.32	0.0099	265	2.62			0.02	0.01	769.32
2	771.41	12	772.41	0.012	0.785	0.0042	4.27	5.44	0.46	772.87	0.0123	260	3.20	0.29			0.03	772.87
1	774.09	10	774.92	0.014	0.545	0.0057	1.82	3.34	0.17	775.09	0.0078	300	2.34			0.17		775.09
12	757.51	15	758.76	0.013	1.227	0.0049	6.81	5.55	0.48	759.24	0.0111	265	2.94	0.04			0.09	759.24
11	764.49	12	765.49	0.012	0.785	0.0042	6.00	7.64	0.91	766.40	0.0243	275	6.68			0.05		766.40
10	768.98	12	769.98	0.012	0.785	0.0042	4.45	5.67	0.50	770.48	0.0133	300	3.99			0.50		770.48
8	762.82	12	763.82	0.012	0.785	0.0042	5.79	7.38	0.85	764.67	0.0227	245	5.56	0.10			0.05	764.67
7	768.45	10	769.28	0.014	0.545	0.0057	2.62	4.81	0.36	769.64	0.0166	300	4.98	0.48				769.64
6	773.79	10	774.62	0.014	0.545	0.0057	2.62	4.81	0.36	774.98	0.0166	300	4.98			0.36		774.98

Table 5.26

Example Equivalent Alternatives

	Location		Pipe Size	
	M.H.	M.H.	Design	Equivalent
Street	From	To	(in.)	(in.)
A	1	2	10	10
A	2	3	12	12
A	3	4	15	15
A	4	5	15	15
B	6	7	10	10
B	7	8	10	10
B	8	9	12	12
C	10	11	12	12
C	11	12	12	12
C	12	13	15	15
Main	5	9	18	18
Main	9	13	24	24
Main	13	14	24	24
Main	14	15	24	24
Main	15	16	27	24
Main	16	Outfall	30	27

Form Loss Calculations for Minor System Design

M.H. 16 θ = 45° (Bend at manhole, no special shaping)
From Figure 5.14, K_b = 0.3

$$H_b = K_b \left(\frac{V^2}{2g} \right) = 0.3 \text{ x } 0.41 = 0.12 \text{ ft}$$

$$H_t = 0.2 \left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g} \right) = 0.2 \text{ x } (0.63\text{-}0.41) = 0.04 \text{ ft}$$

M.H. 15

$$H_t = 0.2 \left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g} \right) = 0.2 \text{ x } (0.89\text{-}0.63) = 0.05 \text{ ft}$$

M.H. 14 θ = 10° (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x} \left(\frac{10}{90} \right)^{1/2} = 0.083$$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.083 \text{ x } 0.89 = 0.07 \text{ ft}$$

$$H_m = 0.05\left(\frac{V^2}{2g}\right) = 0.05 \text{ x } 0.89 = 0.05 \text{ ft}$$

M.H. 13 $\Delta = 20°$ (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x}\left(\frac{20}{90}\right)^{1/2} = 0.118$$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.118 \text{ x } 0.88 = 0.10 \text{ ft}$$

$$(H_j + D_1 - D_2)\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2{}^2}{A_2 g} - \frac{Q_1{}^2}{A_1 g} - \frac{Q_3{}^2}{A_3 g} \cos\theta$$

$\theta = 90°$, $\cos 90° = 0$

$$(H_j + 2.0 - 2.0)\left(\frac{3.142 + 3.142}{2}\right) = \frac{(23.63)^2}{(3.142)(32.2)} - \frac{(14.30)^2}{(3.142)(32.2)} - 0$$

$H_j = 1.11$ ft

M.H. 9 $\Delta = 10°$ (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x}\left(\frac{20}{90}\right)^{1/2} = 0.118$$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.118 \text{ x } 0.88 = 0.10 \text{ ft}$$

$$(H_j + D_1 - D_2)\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2{}^2}{A_2 g} - \frac{Q_1{}^2}{A_1 g} - \frac{Q_3{}^2}{A_3 g} \cos\theta$$

θ = 90°, cos 90° = 0

$$(H_j + 1.5 - 2.0)\left(\frac{1.767 + 3.142}{2}\right) = \frac{(14.30)^2}{(3.142)(32.2)} - \frac{(8.50)^2}{(1.767)(32.2)} - 0$$

$$H_j \;=\; 0.80 \text{ ft}$$

M.H. 5 θ = 90° (Bend at manhole, no special shaping)
From Figure 5.14, K_b = 1.33

$$H_b \;=\; K_b\left(\frac{V^2}{2g}\right) \;= 1.33 \text{ x } 0.32 = 0.43 \text{ ft}$$

$$H_t \;=\; 0.2\left(\frac{V_1{}^2}{2g} - \frac{V_2{}^2}{2g}\right) = 0.2 \text{ x } (0.66\text{-}0.32) = 0.07 \text{ ft}$$

M.H. 4 θ = 90° (Bend at manhole, no special shaping)
From Figure 5.14, K_b = 1.33

$$H_b \;=\; K_b\left(\frac{V^2}{2g}\right) \;= 1.33 \text{ x } 0.66 = 0.88 \text{ ft}$$

M.H. 3

$$H_t \;=\; 0.2\left(\frac{V_1{}^2}{2g} - \frac{V_2{}^2}{2g}\right) = 0.2 \text{ x } (0.46\text{-}0.43) = 0.01 \text{ ft}$$

$$H_m \;=\; 0.05\left(\frac{V^2}{2g}\right) \;= 0.05 \text{ x } 0.43 = 0.02 \text{ ft}$$

M.H. 2 θ = 60° (Bend at manhole, no special shaping)
From Figure 5.14, K_b = 0.63

$$H_b \;=\; K_b\left(\frac{V^2}{2g}\right) \;= 0.63 \text{ x } 0.46 = 0.29 \text{ ft}$$

$$H_t = 0.1\left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g}\right) = 0.1 \text{ x } (0.46\text{-}0.17) = 0.03 \text{ ft}$$

M.H. 1

$$H_{tm} = \left(\frac{V^2}{2g}\right) = 0.17 \text{ ft}$$

M.H. 12 Δ = 10° (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x}\left(\frac{10}{90}\right)^{1/2} = 0.083$$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.083 \text{ x } 0.48 = 0.04 \text{ ft}$$

$$H_t = 0.2\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right) = 0.2 \text{ x } (0.91\text{-}0.48) = 0.09 \text{ ft}$$

M.H. 11

$$H_m = 0.05\left(\frac{V^2}{2g}\right) = 0.05 \text{ x } 0.91 = 0.05 \text{ ft}$$

M.H. 10

$$H_{tm} = \left(\frac{V^2}{2g}\right) = 0.50 \text{ ft}$$

M.H. 8 Δ = 20° (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x}\left(\frac{20}{90}\right)^{1/2} = 0.118$$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.118 \text{ x } 0.85 = 0.10 \text{ ft}$$

$$H_t = 0.1\left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g}\right) = 0.1 \text{ x } (0.85\text{-}0.36) = 0.05 \text{ ft}$$

M.H. 7 θ = 90° (Bend at manhole, no special shaping)
From Figure 5.14, K_b = 1.33

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 1.33 \text{ x } 0.36 = 0.48 \text{ ft}$$

M.H. 10

$$H_{tm} = \left(\frac{V^2}{2g}\right) = 0.36 \text{ ft}$$

Major System

Various manual methods can be used to estimate the major system flows. As a preliminary estimate, designers often apply the Manning equation, using the rainfall intensity for a 100-year storm and a C factor 60 to 85% higher than what would be used for a 2 or 5-year storm. The increase in value is basically to allow for a change in the antecedent moisture condition; with such large flows, there will be less infiltration and more runoff. Except in special circumstances, a C factor above 0.85 need not be used.

In this design example, the C factor of 0.35 used for the design of the minor system will be increased to 0.60, an increase of about 70 %. The results are shown in Table 5.27.

In cases where this method results in flows in excess of the acceptable roadway capacity, a more detailed method should be applied, such as the SCS Graphical Method or a suitable hydrological computer model.

If properly laid out, the major system can tolerate the variability in flows estimated by the various methods. A minor increase in the depth of surface flow will greatly increase the capacity of the major system, without necessarily causing serious flooding. The designer must also consider the remaining overland flow accumulated at the downstream end of the development. Adequate consideration must be given for its conveyance to the receiving water body. This may involve increasing the minor system and inlet capacities or providing adequate drainage swales.

Table 5.27

Example Major System Flows For 100-Year Storm

Sewer Description and Data									Runoff Calculation							Pipe Design						Time of Concentration		Major System		
Street	From M.H.	To M.H.	Length	Slope	Fall	M.H. Drop	M.H. Invert El. Up-Stream	M.H. Invert El. Down-Stream	Drainage Area A	C	A x C	Total Section A x C	Total Trunk A x C	Intensity I	Flow Q	Pipe Size	Area A	Pipe $AR^{2/3}$	Manning's n	Actual Cap. Q	Velocity V	Entry = 10 min. Sect.ion	Entry = 10 min. Accum.	Flow Q	Road Grade	Surface Capacity
			(ft)	%	(ft)	(ft)	(ft)	(ft)	(ac)		(ac)	(ac)	(ac)	(in./hr)	(ft^3/s)	(in.)	(ft^2)	($ft^{8/3}$)		(ft^3/s)	(ft/s)	(min.)	(min.)	(ft^3/s)	(%)	(ft^3/s)
A	1	2	300	0.89	2.68		774.09	771.41	1.82	0.60	1.09	1.09		7.04	7.67	10	0.545	0.192	0.014	1.93	3.54	1.41	11.41	5.74	2.00	182.0
A	2	3	260	1.35	3.52	0.25	771.16	767.64	2.73	0.60	1.64	2.73		6.67	18.21	12	0.785	0.312	0.011	4.90	6.24	0.69	12.10	13.31	2.00	182.0
A	3	4	265	0.91	2.40	0.25	767.39	764.99	2.57	0.60	1.54	4.27		6.49	27.71	15	1.227	0.565	0.012	6.66	5.43	0.81	12.91	21.05	2.00	182.0
A	4	5	306	1.820	5.56	0.25	764.74	759.18	2.06	0.60	1.24	5.51		6.28	34.60	15	1.227	0.565	0.012	9.43	7.69	0.66	13.57	25.17	1.90	180.0
B	6	7	300	1.78	5.34		773.79	768.45	2.63	0.60	1.58	1.58		7.04	11.12	10	0.545	0.192	0.014	2.72	4.99	1.00	11.00	8.40	2.00	182.0
B	7	8	300	1.79	5.38	0.25	768.20	762.82				1.58			11.12	10	0.545	0.192	0.014	2.73	5.01	1.00	12.00	8.39	2.20	191.0
B	8	9	245	2.67	6.54	0.17	762.65	756.11	3.70	0.60	2.22	3.80		6.52	24.78	12	0.785	0.312	0.011	6.89	8.78	0.47	12.47	17.89	2.00	182.0
C	10	11	300	1.50	4.49		768.98	764.49	4.46	0.60	2.68	2.68		7.04	18.87	12	0.785	0.312	0.011	5.16	6.57	0.76	10.76	13.71	1.85	175.0
C	11	12	275	2.52	6.93	0.05	764.44	757.51	1.76	0.60	1.06	3.74		6.84	25.58	12	0.785	0.312	0.011	6.69	8.52	0.54	11.30	18.89	2.00	182.0
C	12	13	265	1.35	3.57	0.25	757.26	753.69	1.05	0.60	0.63	4.37		6.70	29.28	15	1.227	0.565	0.012	8.12	6.62	0.67	11.97	21.16	2.20	191.0
Main	5	9	265	1.06	2.82	0.25	758.93	756.11	1.06	0.60	0.64		6.15	6.11	37.58	18	1.767	0.919	0.013	10.84	6.13	0.72	14.29	26.74	2.00	275.0
Main	9	13	265	0.72	1.92	0.50	755.61	753.69	1.32	0.60	0.79		10.74	5.92	63.58	24	3.142	1.979	0.015	16.69	5.31	0.83	15.12	46.89	2.00	275.0
Main	13	14	500	1.85	9.26	0.25	753.44	744.18	5.64	0.60	3.38		18.49	5.72	105.78	24	3.142	1.979	0.015	26.68	8.49	0.98	16.10	79.08	2.50	310.0
Main	14	15	500	1.74	8.72	0.05	744.13	735.41	1.37	0.60	0.82		19.31	5.53	106.78	24	3.142	1.979	0.015	25.89	8.24	1.01	17.11	80.89	2.00	275.0
Main	15	16	110	1.18	1.30	0.25	735.16	733.86	5.81	0.60	3.49		22.80	5.34	121.75	27	3.976	2.709	0.016	27.35	6.88	0.27	17.38	94.40	0.50	140.0
Main	16	Outfall	500	.0622	3.11	0.25	733.61	730.50							121.75	30	4.909	3.589	0.017	24.74	5.04	1.65	19.03	97.01	0.50	140.0

Foundation Drains

To establish the groundwater level, piezometer measurements over a 12 month period were taken, indicating the groundwater table would be safely below the footing elevations for the proposed buildings, minimizing the amount of inflow that can be expected into the foundation drains.

The municipal requirements include detailed lot grading control, thus further reducing the possibility of surface water entering the foundation drains. A flow value of 0.0027 ft^3/s per basement is used. Detailed calculations are provided in Table 5.28.

Computer Models

There is a wide range of computer models now available for analyzing sewer networks. The complexity of the models varies from straightforward models that use the rational method to estimate the peak flow, to comprehensive models that are based on the continuity and momentum equations. The latter are capable of modeling surcharge, backwater, orifices, weirs and other sewer components. Table 5.29 lists several of these models and their capabilities.

Table 5.28

Example Foundation Drain Collector Design Sheet

Location	From M.H.	To M.H.	Unit Area	Density	Total Units	Cum. Units	Flow Per Unit	Total Flow	Gradient	Pipe Size	Capacity	Velocity
			(acres)	(per acre)			(ft^3/s)	(ft^3/s)	(%)	(in.)	(ft^3/s)	(ft/s)
Cres.'G'	1A	2A	2.97	6	18	18	0.0027	0.049	0.98	8	1.30	3.7
'G'	2A	3A	1.78	6	11	29	0.0027	0.078	1.51	8	1.61	4.6
'G'	3A	4A	3.68	6	22	51	0.0027	0.138	0.50	8	0.93	2.7
'G'	4A	5A	1.48	6	9	60	0.0027	0.162	0.55	8	0.97	2.8
'G'	1A	6A	3.75	6	23	23	0.0027	0.062	1.39	8	1.54	4.4
'G'	6A	7A	2.30	6	14	37	0.0027	0.023	2.25	8	1.96	5.6
'G'	7A	8A	1.43	6	9	46	0.0027	0.124	1.31	8	1.50	4.3
Street 'F'	9A	10A	3.80	8	30	30	0.0027	0.081	1.20	8	1.43	4.1
'F'	10A	11A	2.10	8	17	47	0.0027	0.127	1.20	8	1.43	4.1
Street 'A'	5A	8A	1.56	8	12	72	0.0027	0.194	1.81	8	1.76	5.0
'A'	8A	11A	1.27	8	10	128	0.0027	0.346	4.34	8	2.73	7.8
'A'	11A	13A	2.33	8	19	194	0.0027	0.524	1.42	8	1.56	4.5

Table 5.29

Computer Models - Sewer System Design and Analysis

Model Characteristics	CE Storm	HVM Dorsch3	ILLUDAS	SWMM-Extran	SWMM-Transport	WASSP-SIM	WSPRO
Model Purpose:							
Hydraulic Design	•		•		•	•	•
Evaluation/Prediction		•		•	•	•	•
Model Capabilities:							
Pipe Sizing	•		•		•	•	
Weirs/Overflows		•		•	•	•	
Surcharging		•		•		•	
Pumping Stations				•	•	•	
Storage				•	•	•	
Open Channel Water Surface Profile							•
Hydraulic Equations:							
Linear Kinematic Wave	•		•				
Non-Linear Kinematic Wave					•	•	
St.Venant's - Explicit				•			
St.Venant's - Implicit		•					
Ease of Use:							
High	•		•				•
Low		•		•	•	•	

Multiple opening installation.

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CSP underground detention system being installed.