SECTION 5 - STORM DRAINS

5.1.0 GENERAL

This ~~The purpose of this~~ section discusses briefly ~~is to consider~~ the hydraulic aspects of ~~storm drains and their appurtenances in~~ a storm drain~~age~~ system. ~~Hydraulically, storm~~ Storm drainage systems include open channels or enclosed ~~consist of~~ conduits ~~(open or enclosed)~~ in which unsteady and non-uniform flow exists. Enclosed conduits may be either reinforced concrete pipe or reinforced concrete box culverts meeting the City Standard Specifications for these products. The design storm shall be the 25-year storm with provisions made for the 100-year storm as noted in Section 3.

5.2.0 DESIGN GUIDELINES

The following rules shall ~~are to~~ be observed in the design of storm drain systems ~~components to be~~ located in public right-of-way or public drainage easements ~~in order~~ to promote proper operation of these systems and to minimize maintenance requirements ~~of those systems~~:

A. Select pipe size and slope so that the velocity of flow will increase progressively or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration.

B. Do not discharge the contents of a larger pipe into a smaller one even though the capacity of the smaller pipe may be greater due to a steeper slope. For projects where a proposed storm drain system associated with the project has greater capacity than an in-place receiving system, and an upgrade to the receiving system is not imminent, a temporary exception may be allowed. For this exception to be allowed, the design of the system must incorporate temporary features as needed to prevent any increase in flooding due to the improved conveyance of the proposed upstream system. The proposed system with interim features shall prevent (a) the capacity of the existing in-place system from being exceeded during the design storm at any location or (b) the exacerbation of existing flooding during the design storm at all points along the receiving system, including the tie-in point. The more stringent of these two requirements shall apply. Supporting calculations, signed and sealed, shall be submitted by the design engineer to demonstrate that there will be no adverse flooding impacts due to the proposed storm drain improvements during the interim condition while the receiving system remains undersized and proposed temporary features are in place to prevent any increase in flooding.

C. Where there is a connection of different conduit sizes on the trunk line, the soffit elevations, rather than the flow line elevations, shall be approximately the same. The soffit elevation of the incoming pipe may be offset (increased) by an amount equal to the headloss at the structure where the conduits meet.

~~C. At connections of two (2) different pipe sizes, match the soffits of the two (2) pipes rather than matching the flow lines.~~

D. For design purposes, wherever two or more incoming conduits intersect at a single location, the incoming conduit having the greatest cross-sectional area shall be considered to be the “trunk line.” If more than one incoming conduit has the same cross-sectional area, then the conduit having both the greatest cross-sectional area and the highest peak flow rate for the design storm shall be the “trunk line.” All other incoming conduits at this location shall be considered to be “lateral conduits” or “laterals.”

E. Where a lateral conduit intersects the trunk line at a manhole, the soffit elevation of the incoming lateral conduit shall be at approximately the same elevation as the soffit of the trunk line conduit. Exceptions may be made when the lateral does not need to be constructed to the depth of the trunk line or when the presence of existing utilities prevents the soffit elevation of the lateral from matching the soffit elevation of the trunk line. Where the soffit elevations of the trunk line and lateral are nearly the same, an offset in the soffit elevations may be provided in an amount equal to the headloss at the junction.

F. Where a lateral conduit intersects the trunk line at a wye junction, the soffit elevation of the incoming lateral conduit shall match the approximate soffit elevation of the trunk line conduit. If the soffit elevations are not proposed to be approximately the same, a request for a waiver will be required. Special consideration may be given in cases where there are design constraints such as the presence of existing major utilities which prevent the matching of soffit elevations without extensive utility relocations. In the case of a lateral pipe intersecting a box culvert, allowances shall be made to allow the wall of the pipe to penetrate the wall of the box without having to notch either the pipe wall or the top slab of the box.

G ~~D~~. For all pipe junctions other than a manhole, the angle of intersection between any two flow paths shall not be greater than 45 degrees. This includes discharges into box culverts and channels.

H ~~E~~. No storm drain system shall discharge into or through an inlet box. Instead, the inlet shall discharge ~~in~~to the trunk line through a lateral line ~~"y" connection~~. A single connection from a one-lot or two-lot commercial subdivision or an irrigation system may discharge ~~be tied in~~ to an existing inlet if it does not impede the function of the inlet, and if the receiving storm drain system has the capacity to convey the additional flows. The inlet shall ~~should~~ then be considered ~~treated as~~ a junction box.

I ~~F~~. No proposed conduit ~~pipe~~ having a diameter or height greater than 50 percent of the minimum dimension of an existing box culvert shall be allowed to discharge into that box culvert. Exceptions must be justified by structural engineering analysis demonstrating a sound structural design.

J ~~G.~~ Conduit ~~Pipe~~ shall be reinforced concrete. Conduit ~~Concrete pipe~~ shall be manufactured and installed in compliance with the City's standard specifications ~~published by the Public Works Department~~.

K ~~H~~. Plastic pipe (schedule 40 PVC or greater strength, 6" minimum diameter) shall be used inside water quality ponds (where the size of pipe required dictates its use) and for retention/re-irrigation systems and may be used within fifty (50) feet of a water quality pond filtration bed (if the pipe is not subject to any type of vehicular loading). End treatment is required for outfall pipe in accordance with City standard specifications. Threaded cleanouts are required within fifty (50) feet of every portion of lateral and collector drain lines and at every bend. Junctions between PVC and RCP shall occur at a manhole or cleanout, as determined by the City.

L ~~I~~. The 25-year hydraulic grade line shall remain a minimum of six (6) inches below the theoretical gutter flow line of inlets.

5.3.0 DESIGN PARAMETERS

5.3.1 Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent deposition of solid material. The controlling velocity is near the bottom of the conduit and is considerably less than the mean velocity. Storm drains should be designed to have a minimum velocity of 2.5 feet per second during the 25-year design storm. When backwater conditions prevent one or more portions of the system from attaining the minimum velocity during the design storm, the portion(s) of the system that fail to meet the criteria shall be checked to ensure that the minimum velocity is attained during a lesser and more frequent storm event. If the minimum velocity is still not achieved, then the storm drain shall be redesigned so that the minimum velocity is attained or a request for waiver submitted explaining why it is not feasible to meet the design criterion.

5.3.2 Maximum Velocities

Maximum velocities in conduits are important because of the possibility of excessive erosion of the storm drain pipe material. Table 5-1 lists the maximum velocities allowed at the peak rate of flow during the 25-year event.

Table 5-1
Maximum Velocity In Storm Drains

Type	Maximum Permissible ~~Permissable~~ Velocity
Storm Drains (inlet laterals)	No limit
Storm Drains (trunk)	20 fps
~~Source: City of Austin, Watershed Engineering Division~~	

5.3.3 Minimum Diameter

Pipes that are to become an integral part of the public storm drain ~~sewer~~ system shall have a minimum diameter of 18 inches.

5.3.4 Roughness Coefficients

The coefficients of roughness listed in Table 5-2 are for use in Manning's Equation.

Table 5-2
Roughness Coefficients "n" For Storm Drains

Materials of Construction	Minimum Design Coefficient
Concrete	0.012
Plastic Pipe (smooth wall) ~~Corrugated-metal Pipe~~	0.010 ~~0.024~~
Plastic Pipe (smooth wall, perforated) ~~Plain or Coated~~ ~~Paved Invert (Asphalt)~~	0.020
~~Plastic Pipe~~ ~~Smooth~~ ~~Perforated~~	~~0.010~~ ~~0.020~~
~~Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January, 1977.~~	

5.4.0 FLOW IN STORM DRAINS

All storm drains shall be designed by direct solution of the ~~the application of the~~ Continuity Equation and Manning's Equation ~~either through the appropriate charts and nomographs or by direct solution of the equations~~ as follows:

5.4.1 Flow Equation Method

$Q \quad = AV$ and (Eq. 5-1)

$Q \quad = (1.49/n)\ AR^{2/3}\ S^{1/2}$ (Eq. 5-2)

Where,

Q = Pipe Flow, cfs
A = Cross-sectional area of flow, ft^2
V = Velocity of flow, ft/sec
n = Coefficient of roughness of pipe
R = Hydraulic radius = A/W_p, ft
S = Friction slope in pipe, ft/ft
W_p = Wetted perimeter, ft

~~5.4.2 Nomograph Method~~

~~Nomographs for determining flow properties in circular pipe, elliptical pipe and pipe-arches are given here as Figures 5-1 through 5-9 in Appendix E of this manual. The nomographs are based upon a value of "n" of 0.012 for concrete and 0.024 for corrugated metal. The charts are self-explanatory, and their use is demonstrated by the following examples in this section.~~

~~For values of "n" other than 0.012, the value of Q should be modified by using the following formula:~~

~~$Q_c = 0.012\ Q_n/n_c$~~

~~Where, Q_c = Flow based upon n_c~~
~~n_c = Value of "n" other than 0.012~~
~~Q_n = Flow from nomograph based on n = 0.012~~

~~This formula can be used in two (2) ways. If n_c = 0.015 and Q_c is unknown, use the known values to find Q_n from the nomograph, and then use the formula to convert Q_n to the required Q_c. If Q_c is one of the known values, use the formula to convert Q_c (based on n_c) to Q_n (based on n = 0.012) first, and then use Q_n and the other known values to find the unknown variable on the nomograph.~~

~~Example 5-1:~~

~~Given: Slope = 0.005 ft/ft~~
~~d = depth of flow = 1.8 feet~~
~~D = diameter = 36 inches~~
~~n = 0.018~~

~~Find: Discharge (Q).~~

~~First determine d/D = 1.8'/3.0' = 0.6. then enter Figure 5-1 (in Appendix E of this manual) to read Q_n = 34 cfs. From the formula, Q_c = 34 (0.012/0.018) = 22.7 cfs.~~

~~Example 5-2:~~

~~Given: Slope = 0.005 ft/ft~~
~~D = diameter = 36 inches~~
~~Q = 22.7 cfs~~
~~n = 0.018~~

~~Find: Velocity of flow (ft/sec).~~

~~First convert Q_c to Q_n so that nomograph can be used. Using the formula Q_n = 22.7 (0.018)/(0.012) = 34 cfs, enter Figure 5-1 (in Appendix E of this manual) to determine d/D = 0.6. Now enter Figure 5-3 (in Appendix E~~

~~of this manual) to determine V = 7.5 ft/sec.~~

5.5.0 HYDRAULIC GRADIENT

In storm drain systems flowing full, all losses of energy are a function of resistance of flow in pipes and ~~or~~ by interference with flow patterns at bends and structures ~~junctions~~. These losses are cumulative and must be accounted for ~~by their accumulation~~ along the entire system from ~~its tailwater elevation at~~ the outlet to the most ~~its~~ upstream inlet. The purpose of determining headlosses ~~head losses~~ is to include these values in a progressive calculation of the hydraulic gradient~~.~~ ~~In this way it is possible to determine the hydraulic gradient line which will exist~~ along the storm drain system. The hydraulic gradient line shall be computed and plotted for all sections of a storm drain system flowing full or under pressure flow. The determination of friction loss and minor loss are important for these calculations.

When computing the hydraulic grade line, the tailwater elevation of the system shall be established after careful consideration of the flow elevation and timing in the receiving stream. When detailed hydrologic and hydraulic studies are available for the receiving stream, the modeling results from these studies must be used to estimate the tailwater elevation. If detailed studies are not available, the design engineer must use his professional judgment in calculating the appropriate tailwater elevation. The use of coincident frequencies is not encouraged, and will only be considered for use on a case-by-case basis when an appropriate means for estimating or calculating the tailwater elevation is not feasible. In no case shall the tailwater elevation of the system be considered to be below the top of pipe (overt elevation).

Storm drain profile drawings submitted for review and final profile drawings for construction shall show directly on the drawing the 25-year and 100-year hydraulic gradeline, flow rates and flow velocities for each segment of the storm drain system.

5.5.1 Friction Losses

Friction loss is the energy required to overcome the roughness of the pipe and is expressed as:

$$h_f = (29n^2/R^{1.33})(V^2/2g)L \qquad \text{(Eq. 5-3)}$$

Where,

- h_f = Friction loss, ft
- n = Manning's Coefficient
- L = Length of pipe, ft
- R = Hydraulic radius, ft
- V = Velocity of flow, ft/sec
- g = Acceleration due to gravity, 32 ft/sec^2

~~In addition to Equation 5-3, Table 5-3 can be used to determine the friction slope and applied in Equation 5-4.~~

$$\cancel{h_f = S_f L} \qquad \cancel{\text{(Eq. 5-4)}}$$

~~Where,~~

- ~~h_f = Friction loss, feet~~
- ~~S_f = Friction slope, feet = $(Q/C)^2$~~
- ~~L = Length of pipe, feet~~
- ~~C = Full flow coefficient from Table 5-3~~
- ~~Q = Discharge, cfs~~

~~**Example 5-3:**~~

~~Given: Discharge Q = 24 cfs, diameter D = 24 inches, the length of pipe L=300 feet and n = 0.013~~

Find: The friction loss H_f

First, from Table 5-3 for D = 24 inches and n = 0.013, the full flow coefficient C = 226.

Second, the friction slope $S_f = (Q/C)^2 = 0.011$

The friction loss $H_f = S_f L = 3.3$ feet

Table 5-3
Full Flow Coefficient Values for Circular Concrete Pipe*

D Pipe Diameter (inches)	A Area (square feet)	R Hydraulic Radius (feet)	Value of C** for			
-	-	-	**n = 0.010**	n = 0.011	n = 0.012	n = 0.013
8	0.349	0.167	15.8	14.3	13.1	12.1
10	0.545	0.208	28.4	25.8	23.6	21.8
12	0.785	0.250	46.4	42.1	38.6	35.7
15	1.227	0.312	84.1	76.5	70.1	64.7
18	1.767	0.375	137	124	114	105
21	2.405	0.437	206	187	172	158
24	3.142	0.500	294	267	245	226
27	3.976	0.562	402	366	335	310
30	4.909	0.625	533	485	444	410
33	5.940	0.688	686	624	574	530
36	7.069	0.750	867	788	722	666
42	9.621	0.875	1308	1189	1090	1006
48	12.566	1.000	1867	1698	1556	1436
54	15.904	1.125	2557	2325	2131	1967
60	19.635	1.250	3385	3077	2821	2604
66	23.758	1.375	4364	3967	3636	3357
72	28.274	1.500	5504	5004	4587	4234
78	33.183	1.625	6815	6195	5679	5242
84	38.485	1.750	8304	7549	6920	6388
90	44.170	1.875	9985	9078	8321	7681

~~96~~	~~50.266~~	~~2.000~~	~~11850~~	~~10780~~	~~9878~~	~~9119~~
~~102~~	~~56.745~~	~~2.125~~	~~13940~~	~~12670~~	~~11620~~	~~10720~~
~~108~~	~~63.617~~	~~2.250~~	~~16230~~	~~14760~~	~~13530~~	~~12490~~
~~114~~	~~70.882~~	~~2.375~~	~~18750~~	~~17040~~	~~15620~~	~~14420~~
~~120~~	~~78.540~~	~~2.500~~	~~21500~~	~~19540~~	~~17920~~	~~16540~~
~~126~~	~~86.590~~	~~2.625~~	~~24480~~	~~22260~~	~~20400~~	~~18830~~
~~132~~	~~95.033~~	~~2.750~~	~~27720~~	~~25200~~	~~23100~~	~~21330~~
~~138~~	~~103.870~~	~~2.875~~	~~31210~~	~~28370~~	~~26010~~	~~24010~~
~~144~~	~~113.100~~	~~3.000~~	~~34960~~	~~31780~~	~~29130~~	~~26890~~
~~** C = (1.486/n)AR^0.667^~~						
~~Source: American Concrete Pipe Association.~~ ~~Concrete Pipe Design Manual.~~						

5.5.2 Minor Losses

From the point at which stormwater enters the drainage system at the inlet until it discharges at the outlet, it encounters a variety of hydraulic structures such as manholes, bends, enlargements, contractions and other transitions. These structures will cause headlosses ~~head losses~~ which are called "minor headlosses ~~head losses~~."

~~The minor head losses are generally expressed in a form derived from the Bernoulli and Darcy-Weisbach Equations:~~

~~$h = KV^2/2g$ (Eq. 5-5)~~

~~Where, h = velocity head loss, feet~~
~~K = coefficient for head loss~~

~~The following are minor head losses of hydraulic structures commonly found in a storm drainage system.~~

~~A. **Junction Losses.** Equation 5-6 is used to determine the head loss at a junction of two (2) pipes, with the various conditions of the coefficient K_j given in Table 5-4.~~

~~$H_j = (V_2^2 - K_j V_1^2)/2g$ (Eq. 5-6)~~

~~Where, V_1 = Velocity for inflowing pipe, ft/sec.~~
~~V_2 = Velocity for outflowing pipe, ft/sec.~~
~~K_j = Junction or structure coefficient of loss.~~

~~The detailed design information for junction losses can be found in Reference 5-10.~~

~~B. **Bend Losses.** The minor head loss at a bend results from a distortion of the velocity distribution, thereby causing additional shear stresses within the fluid. The bend loss is considered to be that in excess of the loss for an equal length of straight pipe. The equation to compute the bend loss is:~~

~~$h_b = K_b V^2/2g$ (Eq. 5-7)~~

~~The coefficient K_b varies with the angle of the bend. Table 5-4 and Figure 5-11 in Appendix E of this manual show the different K_b coefficients used in bend losses.~~

Table 5-3
~~Junction or Structure Coefficient of Loss~~

~~**Cases**~~	~~Reference Figure~~	~~Description of Condition~~	~~Coefficient Kj~~
~~A~~	~~5-10~~	~~Manhole on Main Line with 45° Branch Lateral~~	~~0.50~~
~~B~~	~~5-10~~	~~Manhole on Main Line with 90° Branch Lateral~~	~~0.25~~
~~C~~	~~5-11~~	~~45° Wye Connection or cut-in~~	~~0.75~~
~~D~~	~~5-11~~	~~Inlet or Manhole at Beginning of Main Line or Lateral~~	~~1.25~~
~~E~~	~~5-11~~	~~Conduit on Curves for 90°**~~ ~~Curve radius = diameter~~ ~~Curve radius = (2 to 8) diameter~~ ~~Curve radius = (8 to 20) diameter~~	~~0.50~~ ~~0.40~~ ~~0.25~~
~~F~~	~~5-11~~	~~Bends where radius is equal to diameter~~ ~~90° bend~~ ~~60° bend~~ ~~45° bend~~ ~~22½° bend~~ ~~Manhole on line with 60° Lateral~~ ~~Manhole on line with 22½° Lateral~~	~~0.50~~ ~~0.43~~ ~~0.35~~ ~~0.20~~ ~~0.35~~ ~~0.75~~
~~**Where bends other than 90 degrees are used, the 90 degree bend coefficient can be used with the following percentage factor applied:~~ ~~60° Bend - 85%; 45° Bend - 70%; 22½° Bend - 40%~~			
~~Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.~~			

			Minor Loss Methodology and Coefficients		
		Minor Loss Calculations			
Case	Reference Figure	Description	Headloss Method	Coefficient, K	Reference

1		Exit Losses	$H_o=K[(V_o^2/2g) – (V_d^2/2g)]$ where H_o = exit loss; V_o = average outlet velocity; V_d = channel velocity downstream of outlet in the direction of the pipe flow; and g = acceleration due to gravity, 32.2 ft/ s^2.	1.0	Equation 7-4, FHWA Hydraulic Engineering Circular No. 22, September 2009.
2		Manhole or junction on trunk line with or without a change in flow direction (applicable only when all of the following conditions are met: the deflection angle is less than or equal to 90 degrees, there is only one pipe flowing into the manhole and one pipe flowing out, the inflow and outflow pipes are of equal diameter, and there is no sudden drop in elevation between the inflow pipe and the outflow pipe. When any of these conditions are not met, use the methodology required below for Case 3.)	$H = K(V^2/2g)$	Use loss coefficient determined from Figure 5-1. Use Curve A if the bottom shaping is provided in the manhole. Use Curve B if the manhole does not have bottom shaping.	Reference: Urban Storm Drainage Criteria Manual, Volume 1, Prepared for the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, Revised April 2008.
3		All inlets, manholes and junction boxes not covered by Case 2.	FHWA Inlet and Access Hole Energy Loss Method (FHWA Hydraulic Engineering Circular No. 22, latest edition). Use of the method shall account for energy losses at manholes having either straight or angled runs, and shall account for all branch laterals connected to the manhole. Adjustments shall be made for angled flows, plunging flows, and benching within the structure. Determine the hydraulic grade line elevation by subtracting the velocity head from the energy grade line elevation.		FHWA Hydraulic Engineering Circular No. 22, latest edition.
4		Wye Connection or cut-in at any angle (No junction box or manhole is present)	Where no junction box or manhole is present, use the momentum equation (FHWA Junction Loss Method): $H_j=\{[(Q_oV_o)-(Q_i+V_{i-})-(Q_lV_l\cos THETA_j)]/[0.5g(A_o+A_i)]\}+h_i-h_o$ where the terms are as defined in FHWA Hydraulic Engineering Circular No. 22.		FHWA Hydraulic Engineering Circular No. 22, latest edition.

5		Single-angle mitered bend or elbow.	$H = K(V^2/2g)$	Use loss coefficient determined from Figure 5-2.	Reference: U.S. Bureau of Reclamation, Design of Small Canal Structures, Denver, Colorado, 1978.
6		Loss Coefficients for Bends	$H = K(V^2/2g)$		Reference: Normann, J.M., R.J. Houghtalen, and W.J. Johnston, 2001 (Revised May 2005), Hydraulic Design of Highway Culverts, Second edition, Hydraulic Design Series No. 5, Washington, D.C., Federal Highway Administration (FHWA).
		90-degree bend where radius of bend / equivalent diameter of pipe = 1		0.50	
		90-degree bend where radius of bend / equivalent diameter of pipe = 2		0.30	
		90-degree bend where radius of bend / equivalent diameter of pipe = 4		0.25	
		90-degree bend where radius of bend / equivalent diameter of pipe = 6		0.15	
		90-degree bend where radius of bend / equivalent diameter of pipe = 8		0.15	
		45-degree bend where radius of bend / equivalent diameter of pipe = 1		0.37	
		45-degree bend where radius of bend / equivalent diameter of		0.22	

		pipe = 2			
		45-degree bend where radius of bend / equivalent diameter of pipe = 4		0.19	
		45-degree bend where radius of bend / equivalent diameter of pipe = 6		0.11	
		45-degree bend where radius of bend / equivalent diameter of pipe = 8		0.11	
		22.5-degree bend where radius of bend / equivalent diameter of pipe = 1		0.25	
		22.5-degree bend where radius of bend / equivalent diameter of pipe = 2		0.15	
		22.5-degree bend where radius of bend / equivalent diameter of pipe = 4		0.12	
		22.5-degree bend where radius of bend / equivalent diameter of pipe = 6		0.08	
		22.5-degree bend where radius of bend / equivalent diameter of pipe = 8		0.08	
7		Conduit placed to create a continuous curve	$H = K(V^2/2g)$	Use loss coefficient determined from Figure 5-1.	Reference: Urban Storm Drainage Criteria Manual, Volume 1, Prepared for the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, Revised April 2008.

C. **Transition Losses.** The headlosses ~~head losses~~ resulting from sudden and gradual changes in the cross section or flow direction are included in this category. Four (4) transition losses are discussed here.

1. Sudden Enlargement. Table 5-~~5~~ 4 shows the coefficients used in the different cases for

headlosses ~~head losses~~ due to a sudden enlargement.

2. Gradual Enlargement. Table 5-~~6~~ 5 shows the coefficients for calculating the headlosses ~~head losses~~ based on the angle of the cone transition.

3. Sudden Contraction. Table 5-~~7~~6 illustrates the values of coefficients in determining the headloss ~~head loss~~ due to a sudden contraction.

4. Gradual Contraction. The headlosses ~~head losses~~ due to a gradual contraction are determined by the following equation with a constant headloss ~~head loss~~ coefficient.

$$h_{gc} = 0.04\ V^2/2g \qquad \text{(Eq. 5-}\cancel{8}\text{4)}$$

Where, V = velocity for smaller pipe.

Table 5-~~5~~4
Values of K for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Formula H = K (V^2/2g)

d_1/d_2	Velocity, V, fps									
	2	3	4	5	6	7	8	10	12	15	20
1.2	.11	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84
> 10.0	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86

V = velocity in smaller pipe
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe

Source: Brater, E.F. and H.W. King. Handbook of Hydraulics.

Table 5-~~6~~5
Values of K for Determining Loss of Head Due to Gradual Enlargement in Pipes from the Formula H = K (V^2/2g)

d_2/d_1	Angle of cone*													
	2°	4°	6°	8°	10°	15°	20°	25°	30°	35°	40°	45°	50°	60°
1.1	.01	.01	.01	.02	.03	.05	.10	.13	.16	.18	.19	.20	.21	.23

1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
> 3.0	.03	.04	.04	.06	.08	.16	.31	.40	.49	.56	.60	.64	.67	.72

* Angle of cone is twice the angle between the axis of the cone and its side.
V = velocity in smaller pipe.
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe.

Source: Brater, E.F. and H.W. King. Handbook of Hydraulics.

Table 5-~~7~~6
Values of K for Determining Loss of Head Due to Sudden Contraction in Pipe From the Formula $H = K (V^2/2g)$

d_2/d_1	**Velocity, V in feet per second**									
	2	3	4	5	6	7	8	10	12	15	20
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43
> 10.0	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44

V = velocity in smaller pipe
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe

Source: Brater, E.F. and H.W. King. Handbook of Hydraulics.

~~5.5.3 Hydraulic Gradient Calculation Table~~

~~After computing the quantity of storm runoff entering each inlet, the storm drain system required to convey the runoff can be designed. The ground line profile is now used in conjunction with the previous runoff calculations. Table 5-8 can be used to keep track of the pipe design and corresponding hydraulic grade line calculations. Note that the computations begin at the downstream discharge point and continue upstream through the pipe system. Following is an explanation of each of the columns in Table 5-8.~~

~~Column~~

~~1- Design Point; this point is the first junction point* upstream.~~
~~* "Junction Point" refers to any inlet, manhole, bend, etc. that occurs which would cause a minor head loss.~~

~~2- Junction point immediately downstream of design point.~~

~~3- Distance between 1 and 2 in feet.~~

~~4- Design discharge as determined in inlet calculations. (See Table 4-1).~~

~~5- Size of pipe chosen to carry an amount equal to or greater than the design discharge (Figures 5-12 and 5-15 in Appendix E of this manual can be used to determine this).~~

~~6- Slope of frictional gradient (can be determined from Table 5-3 using~~ $\cancel{(Q/C)^2=S_f}$~~)~~

~~7- Elevation of hydraulic gradient at upstream end of pipe = elevation of downstream end + Column 6 times Column 3, or elevation at upstream end + d/D if pipe is not flowing under pressure flow conditions.~~

~~8- Elevation of hydraulic gradient at downstream end of pipe (Note: at outfall point assume h.g. is at top of pipe or above if actual tailwater elevation exists).~~

~~9- Velocity of flow in incoming pipe at design point (use Q=AV for full flow and Figures 5-1 and 5-3 in Appendix E of this manual for partial flow).~~

~~10- Velocity of flow in outgoing pipe at design point.~~

~~11- Velocity head loss for outgoing pipe at design point.~~

~~12- Velocity head loss for incoming pipe at design point.~~

~~13- Head loss coefficients at junction (see Figures 5-10 and 5-11 in Appendix E of this manual)~~

~~14- Column 12 times Column 13.~~

~~15- Column 11 - Column 14 (Note for bends and inlets or manholes at the beginning of a line~~ $\cancel{V_1 = V_2}$~~. The appropriate~~ $\cancel{K_j}$ ~~value should be used in Column 14 and Column 14 = Column 15.~~

~~16- Column 7 + Column 15.~~

~~17- Invert elevation at design point for incoming pipe.~~

~~18- Invert elevation at design point for outgoing pipe.~~

5.6.0 MANHOLES

Manholes provide a very important access point for maintenance purposes. Due to equipment restraints, every point within the storm drain must be a maximum of 250 feet from an access point for drains 30 inches in diameter or smaller. For storm drains greater than 30 inches in diameter, manholes shall be placed so that there is a maximum distance of 300 feet to an access point. Inlets and storm drain outfalls may be considered as access points for maintenance purposes. Access points must be accessible in accordance with the requirements of Section 1.2.4.E of this Manual and must provide a maintenance path within the storm drain that has no more than one horizontal bend, with that bend having a deflection of no more than forty-five (45) degrees in the direction of the maintenance path, and no vertical bend with a deflection of greater than five (5) degrees. Storm drain slope adjustments of less than five (5) degrees are not subject to this requirement.

Manholes shall also be located where two or more laterals intersect the main line within five (5) feet of each other (See Figure 5-12 in Appendix D ~~E~~ of this manual for examples of possible manhole locations). Manholes shall also be placed at locations where changes in pipe size occur.

5.7.0 DEPTH OF COVER

The design of storm drains for areas that will or could receive vehicular traffic or that will be subject to other loading must be supported by structural engineering calculations or references to structural engineering standards.

Table 5-~~8~~7 Hydraulic Computations – Storm Sewers

Click to view Table 5-~~8~~7

5.8.0 STORM DRAIN OUTFALLS INTO OPEN CHANNELS

Storm drain outfalls into open channels shall conform to the design guidelines in Standards 508S-13 or 508S-16 through 508S-20, as appropriate for site specific conditions.

A. End Treatment: Outfalls into natural channels should utilize flexible armor, such as rock riprap (508S-16 through 508S-20), to allow for adjustment of the receiving channel due to normal creek erosion. Rock riprap shall be sized in accordance with ECM 1.4.6(D) and meet the requirements of Standard Specification 591S. Use of standard concrete headwalls (508S-13) and other rigid end treatments should generally be limited to outfalls into non-erosive or armored channels. Where rigid end treatments are used in natural channels, the design should account for future erosion and channel adjustment such that the structure will not become an obstruction to flow in the channel as adjustments occur. This may include using flexible armor to stabilize the surrounding channel banks and/or setting the structure into the bank such that future erosion will not expose the structure. For both flexible and rigid end treatments, the transition between natural channel banks and the outfall shall be smooth and stable such that erosion at the interface is minimized. The angle of intersection between the outfall flow path and the channel flow path should not be greater than 45-degrees.

B. Drop Height: The recommended outfall drop height is one foot, as measured vertically from the flowline of the outfall to the toe of the channel bank, to reduce erosion and account for potential sedimentation at the outfall location. The outfall drop height shall be minimized to the extent feasible and shall not exceed six feet. A manhole(s) may be used to limit the outfall drop height, as needed per 508S-18. In all cases, non-erosive conveyance shall be provided from the outfall to the flowline (lowest elevation) of the receiving channel. Concentrated discharges on steep embankments, ravine slopes, or high bluffs shall be avoided.

C. Stabilization: Where possible, the outfall should be located away from existing eroded banks in the most stable location available. The surrounding banks and bed shall be appropriately armored or made geotechnically stable, so as to sufficiently resist erosive forces. As outlined in Standards

508S-16 through 508S-20, armor below the outfall shall extend from the toe of the bank into the channel equal to a length ten times the pipe diameter. For channel bottom widths less than ten times the pipe diameter, armoring shall extend up the opposite bank to an elevation equal to that of the top of pipe. Flexible armor, such as rock riprap, is preferred to allow for channel adjustment.