SECTION 5 – STORM DRAINS

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SECTION 5 - STORM DRAINS

5.1.0 GENERAL

The purpose of this section is to consider the hydraulic aspects of storm drains and their appurtenances in a storm drainage system. Hydraulically, storm drainage systems consist of conduits (open or enclosed) in which unsteady and non-uniform flow exists. The design storm shall be the 25 year storm with provisions made for the 100 year storm as noted in Section 3 of this Manual.

5.2.0 DESIGN GUIDELINES

The following rules are to be observed in the design of storm drain system components to be located in public right-of-way or public drainage easements in order to promote proper operation and to minimize maintenance 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.

C. For all pipe junctions other than a manhole, the angle of intersection between any two flow paths shall not be greater than forty-five (45) degrees. This includes discharges into box culverts and channels.

E. No proposed pipe having a diameter greater than fifty (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.

G. Pipe shall be reinforced concrete. Concrete pipe shall be manufactured and installed in compliance with the City of Round Rock DACS - Standard Specifications Manual.

H. The 25 year hydraulic grade line shall remain 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 two and one half (2.5) feet per second (fps).

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.

Table 5-1
Maximum Velocity In Storm Drains

Type	Maximum Permissible Velocity
Storm Drains (inlet laterals)	No limit
Storm Drains (trunk)	20 fps

5.3.3 Minimum Diameter

Pipes that are to become an integral part of the public storm sewer system shall have a minimum diameter of eighteen (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	0.013
Corrugated-metal Pipe	0.024
Plain or Coated Paved Invert (Asphalt)	 0.020
Plastic Pipe Smooth Perforated	 0.010 0.020

5.4.0 FLOW IN STORM DRAINS

All storm drains shall be designed by 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

$$Q = AV \text{ and} \qquad \text{(Eq. 5-1)}$$

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

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 B 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 B 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 B of this Manual) to determine d/D = 0.6. Now enter Figure 5-3 (in Appendix B 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 or by interference with flow patterns at junctions. These losses must be accounted for by their accumulation along the system from its tailwater elevation at the outlet to its upstream inlet. The purpose of determining 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.

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.

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

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
54	15.904	1.125	2557	2325	2131	1967
60	19.635	1.250	3385	3077	2821	2604

Table 5-3 (Continued) 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**
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 head losses which are called "minor head losses."

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

$$h = KV^2/2g \qquad \text{(Eq. 5-5)}$$

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.

$$h_j = (V_2^2 - K_j V_1^2)/2g \qquad \text{(Eq. 5-6)}$$

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 the Bibliography of this Manual, Item 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:

$$h_b = K_b V^2/2g \qquad \text{(Eq. 5-7)}$$

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

Table 5-4 Junction or Structure Coefficient of Loss			
Cases	**Reference Figure**	**Description of Condition**	**Coefficient K_j**
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.			

C. **Transition Losses.** The 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 shows the coefficients used in the different cases for head losses due to a sudden enlargement.

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

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

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

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

where, V = velocity for smaller pipe.

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

d_2/d_1	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 1.00	.99 1.00	.96 .98	.95 .96	.93 .95	.92 .94	.91 .93	.89 .91	.88 .90	.86 .88	.84 .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, 1976.

Table 5-6
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
	.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, 1976.

Table 5-7 Values of K for Determining Loss of Head Due to Sudden Contraction in Pipe From the Formula $H = K (V^2/2g)$											
	Velocity, V in feet per second										
d_2/d_1	**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 .49	.48 .49	.48 .48	.48 .48	.48 .48	.47 .47	.47 .47	.46 .47	.46 .46	.45 .45	.43 .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, 1976.											

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.

The 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.

Column 2. Junction point immediately downstream of design point.

Column 3. Distance between one (1) and two (2) in feet.

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

Column 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 B of this Manual can be used to determine this).

Column 6. Slope of frictional gradient (can be determined from Table 5-3 using $(Q/C)^2=S_f$).

Column 7. Elevation of hydraulic gradient (hg) 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.

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

Column 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 B of this Manual for partial flow).

Column 10. Velocity of flow in outgoing pipe at design point.

Column 11. Velocity head loss for outgoing pipe at design point.

Column 12. Velocity head loss for incoming pipe at design point.

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

Column 14. Column 12 times Column 13.

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

Column 16. Column 7 + Column 15.

Column 17. Invert elevation at design point for incoming pipe.

Column 18. Invert elevation at design point for outgoing pipe.

Table 5-8 Hydraulic Computations Storm Sewers

MANHOLES/ INLETS		DIST.	DISCHARGE	PIPE SIZE	FRICTION GRADE	HYDR. GRADIENT		V_1	V_2	$\frac{V_2^2}{2g}$	$\frac{V_1^2}{2g}$	K_J	$\frac{K_J V_1^2}{2g}$	h	H.G.EL. DESIGN PT.	INV. IN	INV. OUT
						UP STREAM	DOWN STREAM	INFLOW	OUTFLOW								
FROM	TO	FEET	c.f.s.	INCHES	FT/FT	ELEV.	ELEV.	f.p.s.	f.p.s.	FT.	FT.	CON-STANT	FT.	FT.	ELEV.	ELEV.	ELEV.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18

Source: City of Austin Drainage Criteria Manual

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 two hundred fifty (250) feet from an access point for drains thirty (30) inches in diameter or smaller. For storm drains greater than thirty (30) inches in diameter, manholes shall be placed so that there is a maximum distance of three hundred (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 1.2.4D. 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 B 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.