SECTION 5 - STORM DRAINS

5.1.0 GENERAL

This section discusses briefly the hydraulic aspects of storm drains and their appurtenances in a storm drainage system. Hydraulically, storm drainage systems include open channels or enclosed 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 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 these 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 “lateral conduits.”

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. 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. No storm drain system shall discharge into or through an inlet box. Instead, the inlet shall discharge into 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 should then be considered treated as a junction box.

I. 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. 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. 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. 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>
<thead>
<tr>
<th>Type</th>
<th>Maximum Permissible Permissible Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storm Drains (inlet laterals)</td>
<td>No limit</td>
</tr>
<tr>
<td>Storm Drains (trunk)</td>
<td>20 fps</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Materials of Construction</th>
<th>Minimum Design Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>0.012</td>
</tr>
<tr>
<td>Plastic Pipe (smooth wall) Corrugated metal pipe</td>
<td>0.010</td>
</tr>
<tr>
<td>Plastic Pipe (smooth wall, perforated)</td>
<td>0.024</td>
</tr>
<tr>
<td>Plain or Coated Paved Invert (Asphalt)</td>
<td>0.020</td>
</tr>
<tr>
<td>Plastic Pipe Smooth Perforated</td>
<td>0.010 0.020</td>
</tr>
</tbody>
</table>

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 equations as follows:
5.4.1  Flow Equation Method

\[ Q = AV \text{ and} \]
\[ Q = \left(\frac{1.49}{n}\right) AR^{2/3} S^{1/2} \]

(Eq. 5-1)

(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:

\[ Q_c = \frac{0.012}{n_c} Q_n \]

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, \( Q_c = 34 \left(\frac{0.012}{0.018}\right) = 22.7 \) 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 \( Q_n = 22.7 \left(\frac{0.012}{0.018}\right) = 34 \) cfs, enter Figure 5-1 (in Appendix E of this manual) to determine \( d/D = 0.6 \). Now enter Figure 5-3 (in-
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 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 upstream inlet. The purpose of determining headlosses 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:

\[ h_f = \frac{(29n^2/R^{1.33})(V^2/2g)L}{g} \]  \hspace{1cm} (Eq. 5-3)

Where,

\[ h_f \quad \text{= Friction loss, ft} \]
\[ n \quad \text{= Manning's Coefficient} \]
\[ L \quad \text{= Length of pipe, ft} \]
\[ R \quad \text{= Hydraulic radius, ft} \]
\[ V \quad \text{= Velocity of flow, ft/sec} \]
\[ g \quad \text{= Acceleration due to gravity, 32 ft/sec}^2 \]

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

\[ h_f = S_f L \]  \hspace{1cm} (Eq. 5-4)

Where,

\[ h_f \quad \text{= Friction loss, feet} \]
\[ S_f \quad \text{= Friction slope, feet} = (Q/C)^2 \]
\[ L \quad \text{= Length of pipe, feet} \]
\[ C \quad \text{= Full flow coefficient from Table 5-3} \]
\[ Q \quad \text{= Discharge, cfs} \]

**Example 5-3:**

Given: Discharge \( Q = 24 \text{ cfs} \), diameter \( D = 24 \text{ inches} \), the length of pipe \( L = 300 \text{ feet} \) and \( n = 0.013 \)
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 $S_f = \left(\frac{Q}{C}\right)^2 = 0.011$

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

\begin{table}[h]
\centering
\begin{tabular}{|c|c|c|c|c|c|c|}
\hline
$D$ & $A$ & $R$ & Value of $C$ & & & \\
Pipe Diameter & Area (square feet) & Hydraulic Radius (feet) & $n = 0.010$ & $n = 0.011$ & $n = 0.012$ & $n = 0.013$ \\
\hline
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 & 6845 & 6195 & 5679 & 5242 \\
84 & 38.485 & 1.750 & 8304 & 7549 & 6920 & 6388 \\
90 & 44.170 & 1.875 & 9985 & 9078 & 8321 & 7681 \\
\hline
\end{tabular}
\caption{Full Flow Coefficient Values for Circular Concrete Pipe*}
\end{table}
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 which are called "minor headlosses." The minor head losses are generally expressed in a form derived from the Bernoulli and Darcy-Weisbach Equations:

\[ h = \frac{KV^2}{2g} \]  

(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.

\[ H_j = \frac{(V_2^2 - K_j V_1^2)}{2g} \]  

(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

<table>
<thead>
<tr>
<th>Cases</th>
<th>Reference-Figure</th>
<th>Description of Condition</th>
<th>Coefficient $K_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5-10</td>
<td>Manhole on Main Line with 45° Branch Lateral</td>
<td>0.50</td>
</tr>
<tr>
<td>B</td>
<td>5-10</td>
<td>Manhole on Main Line with 90° Branch Lateral</td>
<td>0.25</td>
</tr>
<tr>
<td>C</td>
<td>5-11</td>
<td>45° Wye Connection or cut-in</td>
<td>0.75</td>
</tr>
<tr>
<td>D</td>
<td>5-11</td>
<td>Inlet or Manhole at Beginning of Main Line or Lateral</td>
<td>1.25</td>
</tr>
</tbody>
</table>
| 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 |
| E     | 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%

<table>
<thead>
<tr>
<th></th>
<th>Exit Losses</th>
<th><strong>Formula:</strong> $H_0 = K[(V_o^2/2g) - (V_d^2/2g)]$ where $H_0 = $ 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/ $s^2$.</th>
<th>1.0</th>
<th><strong>Reference:</strong> Equation 7-4, FHWA Hydraulic Engineering Circular No. 22, September 2009.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>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.)</td>
<td>$H = K(V_o^2/2g)$</td>
<td>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.</td>
<td><strong>Reference:</strong> Urban Storm Drainage Criteria Manual, Volume 1, Prepared for the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, Revised April 2008.</td>
</tr>
<tr>
<td>3</td>
<td>All inlets, manholes and junction boxes not covered by Case 2.</td>
<td>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.</td>
<td>FHWA Hydraulic Engineering Circular No. 22, latest edition.</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Wye Connection or cut-in at any angle (No junction box or manhole is present)</td>
<td>Where no junction box or manhole is present, use the momentum equation (FHWA Junction Loss Method): $H_j = \left[ (Q_0 V_o^2) - (Q_1 V_1^2) - (Q, V, \cos \Theta_1) \right] / 0.5g(A_0 + A_1)] + h_i - h_o$ where the terms are as defined in FHWA Hydraulic Engineering Circular No. 22.</td>
<td>FHWA Hydraulic Engineering Circular No. 22, latest edition.</td>
<td></td>
</tr>
<tr>
<td>Page</td>
<td>Section</td>
<td>Formula</td>
<td>Equation</td>
<td>Reference</td>
</tr>
<tr>
<td>------</td>
<td>---------</td>
<td>---------</td>
<td>----------</td>
<td>-----------</td>
</tr>
<tr>
<td>5</td>
<td>Single-angle mitered bend or elbow</td>
<td>$H = K \left( \frac{V^2}{2g} \right)$</td>
<td></td>
<td>Use loss coefficient determined from Figure 5-2. Reference: U.S. Bureau of Reclamation, Design of Small Canal Structures, Denver, Colorado, 1978.</td>
</tr>
<tr>
<td>6</td>
<td>Loss Coefficients for Bends</td>
<td>$H = K \left( \frac{V^2}{2g} \right)$</td>
<td></td>
<td>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).</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bend Angle</th>
<th>Radius of Bend / Equivalent Diameter of Pipe</th>
<th>Loss Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>90-degree</td>
<td>radius of bend / equivalent diameter of pipe = 1</td>
<td>0.50</td>
</tr>
<tr>
<td>90-degree</td>
<td>radius of bend / equivalent diameter of pipe = 2</td>
<td>0.30</td>
</tr>
<tr>
<td>90-degree</td>
<td>radius of bend / equivalent diameter of pipe = 4</td>
<td>0.25</td>
</tr>
<tr>
<td>90-degree</td>
<td>radius of bend / equivalent diameter of pipe = 6</td>
<td>0.15</td>
</tr>
<tr>
<td>90-degree</td>
<td>radius of bend / equivalent diameter of pipe = 8</td>
<td>0.15</td>
</tr>
<tr>
<td>45-degree</td>
<td>radius of bend / equivalent diameter of pipe = 1</td>
<td>0.37</td>
</tr>
<tr>
<td>45-degree</td>
<td>radius of bend / equivalent diameter of pipe = 1</td>
<td>0.22</td>
</tr>
</tbody>
</table>
### Transition Losses

The headlosses 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.

### Table 5-5-4: Transition Losses Coefficients

<table>
<thead>
<tr>
<th>Bend Type</th>
<th>Loss Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>45-degree bend where radius of bend / equivalent diameter of pipe = 4</td>
<td>0.19</td>
</tr>
<tr>
<td>45-degree bend where radius of bend / equivalent diameter of pipe = 6</td>
<td>0.11</td>
</tr>
<tr>
<td>45-degree bend where radius of bend / equivalent diameter of pipe = 8</td>
<td>0.11</td>
</tr>
<tr>
<td>22.5-degree bend where radius of bend / equivalent diameter of pipe = 1</td>
<td>0.25</td>
</tr>
<tr>
<td>22.5-degree bend where radius of bend / equivalent diameter of pipe = 2</td>
<td>0.15</td>
</tr>
<tr>
<td>22.5-degree bend where radius of bend / equivalent diameter of pipe = 4</td>
<td>0.12</td>
</tr>
<tr>
<td>22.5-degree bend where radius of bend / equivalent diameter of pipe = 6</td>
<td>0.08</td>
</tr>
<tr>
<td>22.5-degree bend where radius of bend / equivalent diameter of pipe = 8</td>
<td>0.08</td>
</tr>
</tbody>
</table>

**Equation:**

\[ H = K \left( \frac{V^4}{2g} \right) \]

2. Gradual Enlargement. Table 5-6.5 shows the coefficients for calculating the headlosses 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 headlosses due to a gradual contraction are determined by the following equation with a constant head loss coefficient.

\[ h_{gc} = 0.04 \frac{V^2}{2g} \]  
(Eq. 5-84)

Where, \( V \) = velocity for smaller pipe.

<table>
<thead>
<tr>
<th>Table 5-54</th>
<th>Values of K for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Formula ( H = K \left( \frac{V^2}{2g} \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{d_1}{d_2} )</td>
<td>Velocity, ( V ), fps</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>1.2</td>
<td>.11</td>
</tr>
<tr>
<td>1.6</td>
<td>.40</td>
</tr>
<tr>
<td>1.8</td>
<td>.51</td>
</tr>
<tr>
<td>2.0</td>
<td>.60</td>
</tr>
<tr>
<td>2.5</td>
<td>.74</td>
</tr>
<tr>
<td>3.0</td>
<td>.83</td>
</tr>
<tr>
<td>4.0</td>
<td>.92</td>
</tr>
<tr>
<td>5.0</td>
<td>.96</td>
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<td>10.0</td>
<td>1.00</td>
</tr>
<tr>
<td>&gt; 10.0</td>
<td>1.00</td>
</tr>
</tbody>
</table>

\( V \) = velocity in smaller pipe  
\( \frac{d_2}{d_1} \) = ratio of diameter of larger pipe to diameter of smaller pipe

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

<table>
<thead>
<tr>
<th>Table 5-65</th>
<th>Values of K for Determining Loss of Head Due to Gradual Enlargement in Pipes from the Formula ( H = K \left( \frac{V^2}{2g} \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{d_2}{d_1} )</td>
<td>Angle of cone*</td>
</tr>
<tr>
<td>2°</td>
<td>4°</td>
</tr>
<tr>
<td>1.1</td>
<td>.01</td>
</tr>
</tbody>
</table>
* 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.


Table 5-76
Values of K for Determining Loss of Head Due to Sudden
Contraction in Pipe From the Formula H = K (V^2/2g)

<table>
<thead>
<tr>
<th>d_2/d_1</th>
<th>Velocity, V in feet per second</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td>1.1</td>
<td>.03</td>
</tr>
<tr>
<td>1.2</td>
<td>.07</td>
</tr>
<tr>
<td>1.4</td>
<td>.17</td>
</tr>
<tr>
<td>1.8</td>
<td>.34</td>
</tr>
<tr>
<td>2.0</td>
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<tr>
<td>2.2</td>
<td>.40</td>
</tr>
<tr>
<td>2.5</td>
<td>.42</td>
</tr>
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<td>3.0</td>
<td>.44</td>
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<td>4.0</td>
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<td>5.0</td>
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<td>.49</td>
</tr>
<tr>
<td>&gt; 10.0</td>
<td>.49</td>
</tr>
</tbody>
</table>

V = velocity in smaller pipe

d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe
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 \( (Q/C)^2 = S_f \))

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)


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.


17. Invert elevation at design point for incoming pipe.

18. Invert elevation at design point for outgoing pipe.

5.6.0 MANHOLES
Manholes provide important access points for cleaning and maintenance purposes. Manholes shall be provided in accordance with the following criteria:

1. Provide manholes at all locations where the diameter of the trunk line pipe changes or where the cross-sectional dimensions of the trunk line box change.
2. Provide manholes at all vertical deflections that exceed 80% of the conduit manufacturer’s allowable deflection at a joint.
3. Provide manholes at all locations where the spacing between two or more laterals intersecting the trunk line is equal to or less than ten feet.
4. The maximum distance between manholes within the storm drain system shall not exceed 300 feet for conduits smaller than 72 inches in diameter and should not exceed 500 feet in conduits 72 inches or greater in diameter.
5. When designing a storm drain system, storm drain inlets and outfalls shall not be considered as access points to the trunk line for inspection or cleaning purposes.
6. The location of manholes within the system must provide a maintenance path that has no more than one horizontal bend with that bend having a deflection of no more than forty-five (45) degrees. This allows for two (2) forty-five degree bends within the maximum allowable distance between manholes.
7. Manholes shall not be placed in driveways or in the street in front of or immediately adjacent to a driveway unless approved by the Director of the Watershed Protection Department.
8. New manholes shall not be located within ditches or swales, unless approved by the Director of the Watershed Protection Department.
9. Manholes must be accessible in accordance with the requirements of Section 1.2.4.E of this manual.
10. See Figure 5-12 in Appendix D of this manual for examples of typical manhole locations.

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 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 5-87 Hydraulic Computations – Storm Sewers

Click to view Table 5-87

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.

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