CHAPTER 4

STORM DRAINAGE SYSTEMS

4.1 OVERVIEW

4.1.1 Introduction

In this chapter, guidelines are given for calculating gutter and inlet hydraulics and storm drainage design. Procedures for performing gutter flow calculations are based on a modification of Manning's equation. Inlet capacity calculations for grate and combination inlets are based on information contained in HEC- 22 (USDOT, FHWA, September 2009). Storm drain design is based on the use of the rational formula.

4.1.2 Inlet Definition

There are four storm water inlet categories:

- curb opening inlets
- grated inlets
- combination inlets
- multiple inlets

In addition, inlets may be classified as being on a continuous grade or in a sag. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sag" condition exists when the inlet is located at a low point and water enters from both directions.

4.1.3 Criteria

The following criteria shall be used for drainage system design.

- Maximum spread of 6 feet in a travel lane based on a rainfall intensity of 4 inches/hour.
- For a street with a valley gutter, another foot for the gutter is allowed with a total maximum spread of 7 feet based on a rainfall intensity of 4 inches per hour.
- For a street with a standard 2 feet 6 inch curb and gutter, an additional 2 feet is allowed with a total maximum spread of 8 feet from the face of the curb based on a rainfall intensity of 4 inches per hour.
- See Section 2.3.1 for additional criteria on design storm frequencies.

Other

- In sag areas where relief by curb overflow is not provided the system standard design level (Q_{25} – Q_{50}) is to be used for analysis to ensure traffic flow is not interrupted (NC Division of Highways, Guidelines for Drainage Studies and Hydraulic Design, 1999). Therefore, in a sag condition where relief by overflow for a typical roadway cross section is not provided, inlet capacity and the storm drainage system must be designed for:
    - one dry 8 foot travel lane in the 25-year event for local streets; and
    - one dry 8 foot travel lane in each direction in the 50-year event for thoroughfares.

  All upstream bypass flow must be considered in the design of the inlets and the storm drainage system at the sag. Available overflow at sag locations can be considered when making the determination for available travel lanes in the relevant storm events.

- For local roads the pipe system hydraulic grade line shall not surcharge sag inlets in the 25-year storm. For thoroughfare roads the pipe system hydraulic grade line shall not surcharge in the 50-year storm.
- When checking spread requirements at sag points (0% slope) check spread upstream of sags (at the 0.5% slope point) to verify spread is not exceeded. Additional flanking inlets upstream may need to be added to keep spread criteria from being exceeded at these points.
- Ponding at yard inlets outside the roadway shall be limited to a maximum of one foot above a grated inlet for the 10-year storm.
- No concentrated runoff flowing over City sidewalks except at driveways.
- Roadside ditches, when allowed, shall be a minimum of 18 inches deep and shall provide the capacity designed for a 10-year storm with 6-inches of freeboard. For subdivision streets ditch flow for the 25-year storm shall not encroach onto the pavement. For thoroughfare streets ditch flow for the 50-year storm shall not encroach onto the pavement.
- For subdivision streets the driveway and culvert shall be designed such that the flow from a 25-year storm shall not encroach onto the roadway pavement. For thoroughfare streets the driveway and culvert shall be designed such that the flow from a 50-year storm shall not encroach onto the roadway pavement.

4.2 PAVEMENT DRAINAGE

4.2.1 Introduction

Design factors to be considered during gutter, inlet and pavement drainage calculations include:

- Return period
- Spread
- Storm drain location
- Inlet types and spacing
- Longitudinal slope
- Shoulder gutter
- Cross slope
- Curb and gutter sections
- Roadside and median ditches
- Bridge decks
- Median barriers

4.2.2 Storm Drain Location

For standards related to storm drain location refer to the Charlotte Land Development Standards Manual.

4.2.3 Inlet Types and Spacing

Inlet types shall be selected from the Charlotte Land Development Standards Manual or equivalent North Carolina State Department of Transportation standards. Inlets shall be located or spaced in such a manner that the design curb flow does not exceed the spread limitations. Flow across intersecting streets will be reviewed and approved on a case by case basis.

4.2.4 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement, since it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Curb and gutter grades that are equal to pavement slopes shall not fall below 0.5 percent. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile. For long vertical curves, cross slope may be varied slightly to achieve 0.5 percent minimum gutter grade.

4.2.5 Cross Slope

Refer to the design standards for pavement cross slopes as shown in the Charlotte Land Development Standards Manual.

4.2.6 Curb and Gutter

Curb and gutter installation shall be designed in accordance with the relevant standards for the jurisdiction.

4.2.7 Median Ditches

Large median areas and inside shoulders should be sloped to a center swale, preventing drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

4.2.8 Roadside Ditches

Roadside ditches (when allowed) will be required behind the shoulder of roadways without curb and gutter to convey storm drainage away from the pavement to a discharge point. The steepest side slope allowed is 3:1 (horizontal to vertical) on the roadside of the ditch and 2:1 on the side closest to the right-of-way line. The ditch shall be graded to a minimum longitudinal slope of 1 percent and a maximum velocity of 4 ft/sec. For grass lined channels with velocities up to 7 ft/sec, permanent matting may be approved on a case by case basis. For velocities greater than 7 ft/sec, a concrete lined ditch may be required. Riprap will not be allowed for stabilization within the street right-of-way (except as outlet protection on culverts).

In addition to the design of roadside ditches, a design shall be provided for driveway culverts for each individual lot on the plan. The use of a small driveway culvert, 15 inches minimum diameter, in conjunction with overtopping of the driveway itself will be allowed as further described in Section 4.1.3. Sizes for all driveway culverts shall be shown in tabular form on the plans, and each culvert shall be designed for the highest ditch flow applicable for the lot.

4.2.9 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal. Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 1 percent. When bridges are placed at a vertical curve and the longitudinal slope is less than 1 percent, the gutter spread should be checked to ensure a safe, reasonable design.

Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains.

For situations where traffic under the bridge or environmental concerns prevent the use of scuppers, grated bridge drains should be used.

4.2.10 Median Barriers

Weep holes are often used to prevent ponding of water against median barriers (especially on superelevated curves). In order to minimize flow across traveled lanes, it is preferable to collect the water into a subsurface system connected to the main storm drain system.

4.3 GUTTER FLOW CALCULATIONS

4.3.1 Formula

The following form of Manning's equation should be used to evaluate gutter flow hydraulics:

$$\mathbf{Q = [0.56/n]\, S_T^{5/3}\, S_L^{1/2}\, T^{8/3}} \qquad \textbf{(4.1)}$$

Where: Q = Gutter flow rate (ft^3/sec)

n = Manning's roughness coefficient

S_T = Pavement cross slope (ft/ft)

S_L = Longitudinal slope (ft/ft)

T = Width of flow or spread (ft)

Note: Manning's *n* value for concrete curb and gutter is 0.016.

4.3.2 Procedure

Using Table 4-1, identify the following:

1. Inlet # – Assigned number (or label) of drainage structure.
2. Drainage Area – Area contributing runoff to the inlet (acres).
3. Surface 'Q' Sub. – Flow (in cfs) to the inlet.

   $Q = CiA$

   Where: C = runoff coefficient for the sub-drainage area

   i = 4.0 inches/hour

   A = Area determined in #2 (acres)

4. Surface 'Q' Total = $Q_{sub} + \sum Q_{bypass}$

5. Long. Slope – Longitudinal gutter slope at “Inlet #” (in feet per foot). Generally, this is equivalent to the roadway centerline profile.

6. Trans. Slope – Transverse slope at “Inlet #” (in feet per foot). This is equivalent to the roadway cross-slope.

7. K – This coefficient is used to determine the inlet capacity of a catch basin grate on grade. Refer to Figure 4-1.

8. Inlet Capacity (cfs)

   $$Q_{cap} = KD^{5/3}$$

   Where: $D = S_T \times T$, depth of flow at curb (ft)

   For a “normal crown” street, S_T = 3/8” per 1’ = 0.0313 ft/ft.
   The maximum spread, T, is 8 feet. Therefore D = 0.0313 x 8 = 0.25ft.

9. Spread = Width of flow (feet)
   The maximum width of spread in a travel lane is 6 feet. Total allowable spread with standard curb and gutter is 8 feet, and with valley gutter is 7 feet.

10. Bypass Q = Surface Q_{total} – Inlet Q_{cap}

11. Bypass to Inlet# = List Inlet# directly downstream (bypass destination).
    Remember to add:
    Bypass Q + Surface Q_{sub} =Surface Q_{total}

    Note: Computer software for gutter flow analysis is acceptable. The computer printout should contain the same information that is shown in Table 4-1.

INLET CAPACITY CHART

PROJECT NAME ______________ COMPUTED BY ______________ DATE ______________

PROJECT LOCATION ______________ CHECKED BY ______________

RAINFALL INTENSITY = 4.0 IN/HR

INLET	DRAINAGE	SURFACE 'Q' (CFS)		SLOPE (FT/FT)			INLET		BYPASS		
#	AREA (ACRES)	SUB.	TOTAL	LONG.	TRANS.	K	Q_{CAP} (CFS)	SPREAD (FT)	Q (CFS)	TO INLET #	REMARKS

Table 4-1 Inlet Capacity Chart

4.4 GRATE INLET DESIGN

4.4.1 Grate Inlets on Grade

Following is a discussion of the procedure for the design of grate inlets on grade. Figure 4-1 is used for the design of grate inlets on grade using type ‘F’ and ‘G’ grates.

4.4.1.1 Design Steps

1. Determine the following input data: Q, S_L, S_T, and n

2. D is the depth of water (or head) in the gutter immediately upstream of the grate (in feet). However, before this depth can be calculated, certain parameters must be set. In the case of street design, it is undesirable to have the street inundated and impassable due to the amount of runoff drainage down a given street. Therefore, the maximum allowable top width of water flow, or spread, T, in the gutter and street must be regulated such that flooding does not occur.

3. With the discharge rate, Q, known, T can be solved by applying the modified Manning’s equation:

$$\mathbf{Q = (0.56/n)ZD^{8/3}S_L^{1/2}} \qquad \textbf{(4.2)}$$

Since: $$\mathbf{D = T(S_T)} \qquad \textbf{(4.3)}$$

And: $$\mathbf{S_T = 1/Z} \qquad \textbf{(4.4)}$$

T can be derived:

$$\mathbf{T = [Qn(Z^{5/3})/0.56(S_L^{1/2})]^{3/8}} \qquad \textbf{(4.5)}$$

Where: T = top width, or spread, of water flow (ft)

Q = discharge to the inlet structure (cfs)

Z = reciprocal of the transverse slope (ft/ft)

n = roughness coefficient

S_L = longitudinal street slope (ft/ft)

S_T = transverse street slope (ft/ft)

D=depth of water in gutter (ft)

4. To determine Q_{cap} of inlet:

$$\mathbf{Q_{cap} = KD^{5/3}}$$

Where: D = depth of water in gutter, upstream from the grate (ft)

Q_{cap} = discharge intercepted by grate (cfs)

K= grate inlet coefficient

Note: For S_L> 6%, use the curve for S_L= 6%

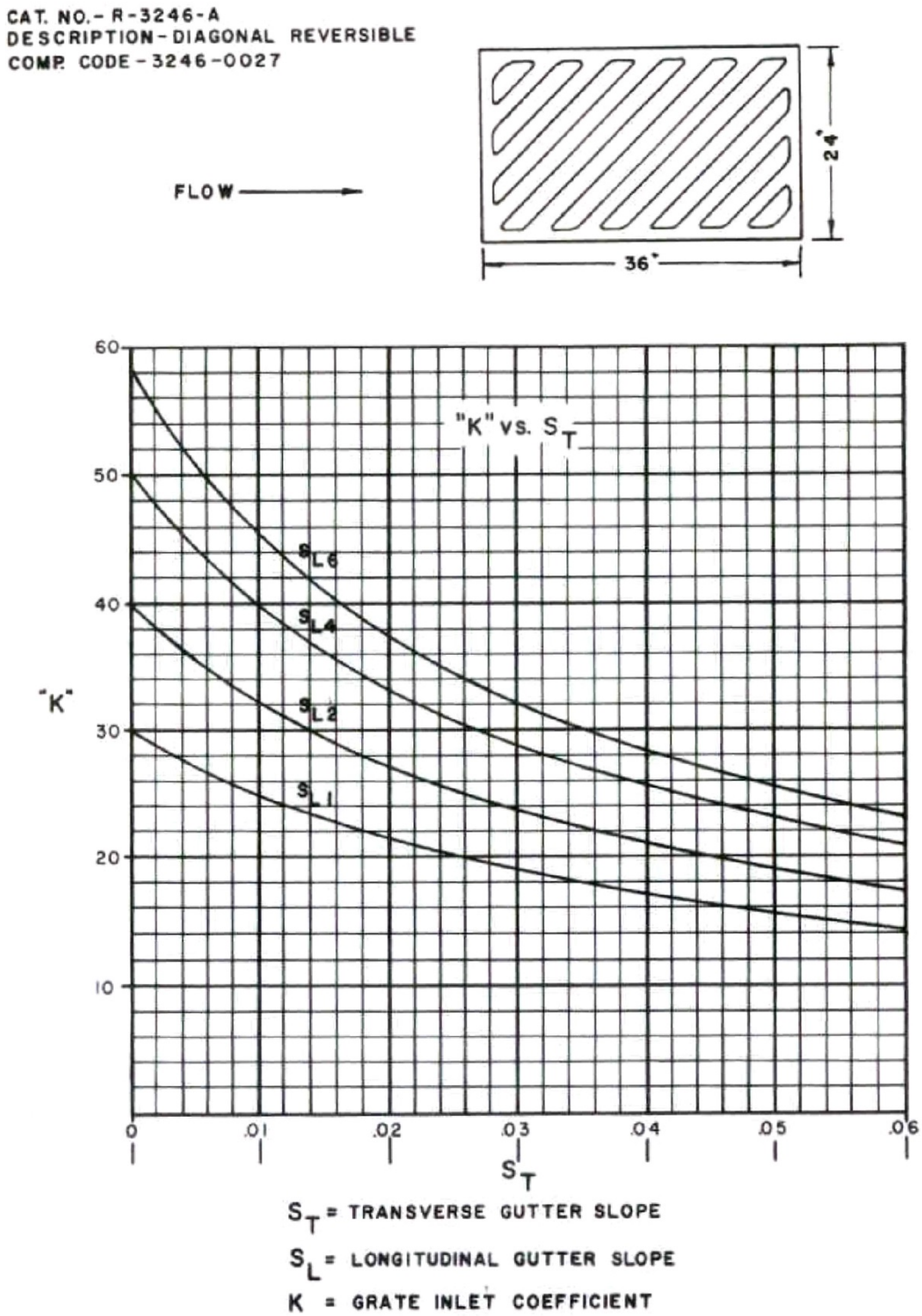


Figure 4-1 Grate Inlet Coefficient – On Grade

5. Once T is calculated and determined to be within its imposed limits, D can be calculated as follows:

$$\mathbf{D = T/Z} \qquad \mathbf{(4.6)}$$

Where: D = depth of water in the gutter, upstream from the grate (ft)

6. The inlet capacity of the grate, Q_{cap}, can then be determined by using Figure 4-1. If 100% interception is not achieved, the overflow must be included in the next downstream inlet.

4.4.1.2 Example 1

A 10-year discharge of 4 cfs (3 cfs off-site and 1 cfs additional roadway drainage) drains to a residential street, with standard curb and gutter, in sheet flow and is to be intercepted in a catch basin midway down the street.

Given: S_L = 4% = 0.04 ft/ft

S_T = 3.125% = 0.03125 ft/ft

T_{max} allowable = 8 ft

Type G grate

Find: Determine whether this runoff is excessive and whether it can be handled by a single grate, type 'G', catch basin.

Solution: 1. $Z = 1/S_T = 1/0.03125 = 32$

$$T = \left(\frac{4\,(0.016)\,(32)^{5/3}}{0.56(0.04)^{1/2}} \right)^{3/8}$$

T = 7.1 ft (which is less than the 8 ft available)

Since T is lower than the maximum allowable top width of 8 ft, the runoff in the street is acceptable.

2. D = T/Z = 7.1/32 = 0.22 ft

3. From Figure 4-1, K = 28.5

Therefore: $\mathbf{Q_{cap} = KD^{5/3} = 28.5(0.22)^{5/3} = 2.3\ cfs} \qquad \mathbf{(4.7)}$

4. Q_{bypass} = 4.0 – 2.3 = 1.7 cfs

Thus, the grate will intercept 2.3 cfs and 1.7 cfs will continue downstream to another structure. If the next downstream structure can handle this 1.7 cfs in addition to any additional runoff that reaches this structure, then the design is adequate. If this additional 1.7 cfs will overload the next downstream structure, additional storm drainage structure can be added upstream.

4.4.2 Grate Inlets in Sag

Because a grate inlet in a sag condition is subject to clogging, a curb opening is required as a supplemental inlet. The capacity of a grate in a sag depends upon the area of the openings and the depth of water at the grate. Figure 4-2 can be used to calculate the head or flow for standard drop inlet grates, and Figure 4-3 for Type 'E' grates in sag or depressed conditions.

4.4.2.1 Type 'E' Grate

For a type 'E' grate, the weir equation will control to a depth (D) of 0.69 feet. Refer to Figure 4-3. Because a depth of 0.69 feet could never be reached without flooding the street, only the weir equation will be used for the analysis of a street sag condition.

1. Knowing the Surface Q_{total}, solve for the required depth:

$$\mathbf{D = \left(\frac{Surface\ Q_{total}}{3.3P}\right)^{2/3}} \qquad \textbf{(4.8)}$$

P = 6.94 ft. (single type 'E' grate)
P = 9.92 ft. (double type 'E' grate)

2. Check the spread:

$$T = \frac{D}{S_T}$$

D = depth solved in #1 (ft)

S_T = transverse slope (ft/ft)

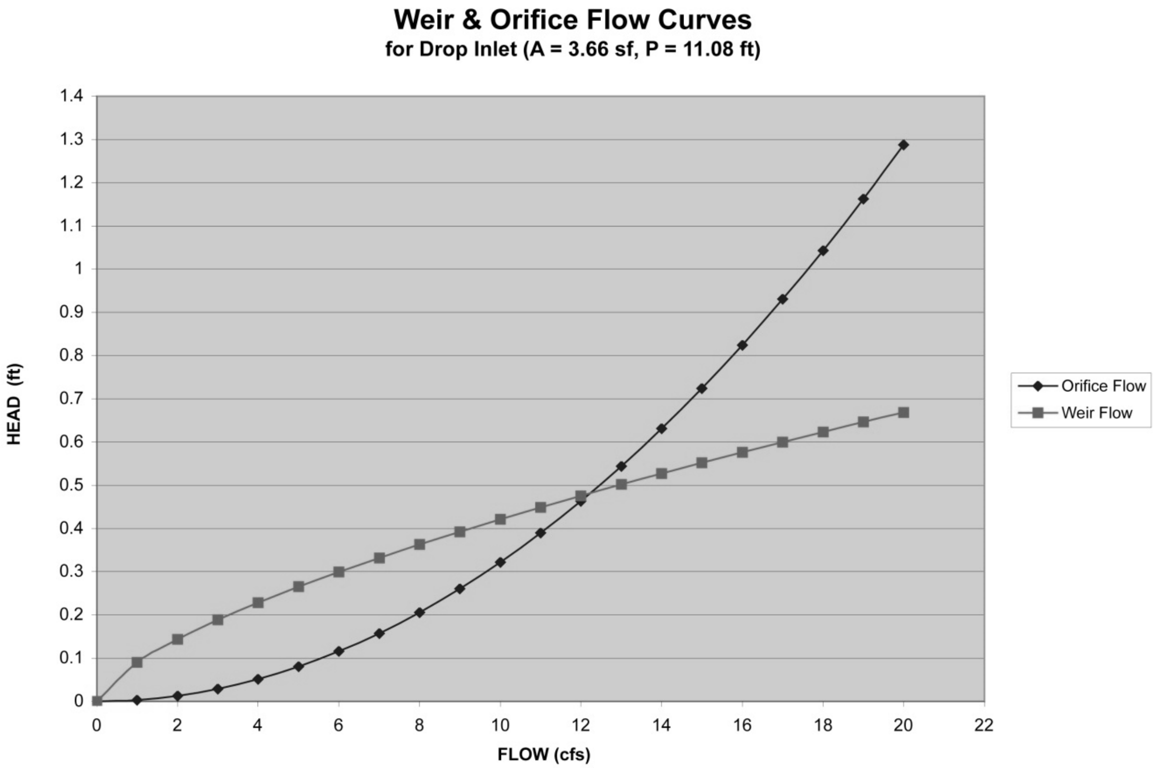


Figure 4-2 Standard Drop Inlet Grate NCDOT Std. 840.16 (2012)

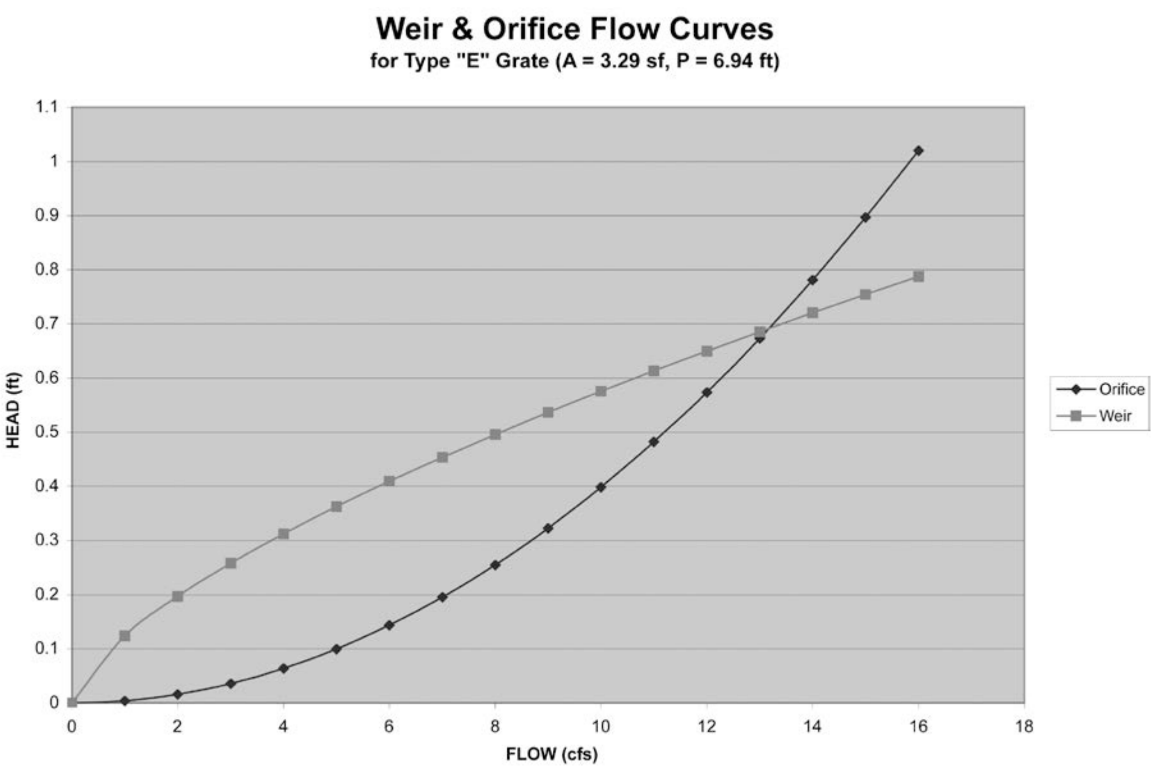


Figure 4-3 Type ‘E’ Grate NCDOT Std. 840.03 (2012)

4.4.2.2 Grated Drop Inlet

For a grated drop inlet, both the weir and orifice equation should be analyzed:

(orifice) $$\mathbf{D} = \left(\frac{\text{Surface } Q_{total}}{(0.6)\ A\ (64.4)^{1/2}} \right)^2 \qquad \textbf{(4.9)}$$

A = 3.66 sq. ft. (open area of standard drop inlet grate-NCDOT Std. 840.16 (2012))

(weir) $$\mathbf{D} = \left(\frac{\text{Surface } Q_{total}}{3.3P} \right)^{2/3} \qquad \textbf{(4.8)}$$

P = 11.08 ft. (perimeter of standard drop inlet grate–NCDOT Std. 840.16 (2012))

Solve both equations 4.8 and 4.9 for D, or refer to Figure 4-3. The larger D controls.

4.4.2.2.1 Example 2

Using the data given for Example 1 in 4.4.1.2, calculate the flow intercepted by type 'E' grate in a sag location.

Given: Same data given for Example 1 in 4.4.1.2

Find: Determine the depth of flow that can be intercepted by a type 'E' grade in a sag location.

Solution:

1. From Example 4.4.1.2, Surface Q_{total} = 4 cfs
2. Solve for D using Figure 4-3 or Equation 4-8

$$D = \left(\frac{4 \text{ cfs}}{3.3\ (6.94 \text{ ft})} \right)^{2/3} = 0.31 \text{ ft}$$

3. Check the spread

$$T = \frac{D}{S_t} = \frac{0.31 \text{ ft}}{0.03125 \text{ ft/ft}} = 9.9 \text{ ft}$$

4. This exceeds the maximum allowable spread of 8 ft. Install additional catch basins above the sag to intercept the additional flow, or try a double catch basin at the sag.

$$D = \left(\frac{4 \text{ cfs}}{3.3\ (9.92 \text{ ft})} \right)^{2/3} = 0.25 \text{ ft}$$

Check the spread

$$T = \frac{D}{S_t} = \left(\frac{0.25 \text{ ft}}{0.03125 \text{ ft/ft}} \right) = 7.9 \text{ ft}$$

This is less than the maximum allowable spread of 8 feet, so a double catch basin will work.

5. Finally, sag location would need to be evaluated in the appropriate storm (25-year for local roads and 50-year storm for thoroughfare roads) to ensure a minimum 8 foot passable lane for vehicles.

4.4.2.3 Open Throat (Slab Top) Catch Basin

Depth of flow for this type of inlet depends on how many sides will be open to flow and what the total length the openings are (L). Typical height for openings for this type of structure is 6 in (0.5 ft)[A]. Both weir and orifice equations should be analyzed to find depth at the inlet:

(weir) $$D = \left(\frac{\text{Surface } Q_{total}}{(3.3)\,(L)}\right)^{2/3}$$

If D> inlet opening height (.5 ft) then use the orifice equation to find the driving head (H) needed to pass the flow into the inlet instead

(orifice) $$H = \frac{[\text{Surface } Q_{total}]^2}{(0.6)\,(.5)\,(L)\,(64.4)^{1/2}}$$

since the driving head (H) is drawn to the centerline of the height of the inlet simply adding 0.25 ft to H will yield depth of flow (D)

$$D = H + (0.5/2)$$

4.4.2.3.1 Example

A four sided slab top open throated drop inlet is planned for a backyard. Surface flow is 25 cfs to this inlet. All four sides of the inlet have openings with each inlet side being 4 ft long (16 ft total). Since the height for inlet openings is 6 in the preceding equations can be used.

Analyzing weir flow for this inlet:

$$D = (25/(3.3*16))^{2/3} = 0.61 \text{ ft}$$

Since this is greater than the inlet opening height of 0.5 ft the orifice equation must be used to find depth of flow:

$$H = (25/(0.6*0.5*16*64.4^{1/2}))^2 = 0.42 \text{ ft}$$

Adding .25 ft to H to find depth of flow to the invert of the opening

$$D = 0.42 \text{ ft} + 0.25 \text{ ft} = 0.67 \text{ ft}$$

Depth of flow is 0.67 ft. Add required freeboard and grade accordingly to allow this ponding depth.

[A] - *For inlets with inlet opening heights different than 0.5 ft adjust these equations accordingly.*

4.5 COMBINATION INLETS

4.5.1 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus, capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figure 4-1.

4.6 HYDRAULIC GRADIENT

4.6.1 Friction Losses

Energy losses from pipe friction may be determined by rewriting the Manning's equation.

$$\mathbf{S_f = [Qn/1.486\ A(R)^{2/3}]^2}$$

Then the head losses due to friction may be determined by the formula:

$$\mathbf{H_f = S_f L}$$

Where: H_f = friction head loss (ft)

S_f = friction slope (ft/ft)

L = length of outflow pipe (ft)

4.6.2 Velocity Head Losses

From the time storm water first enters the storm drainage system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements and transitions, which will cause velocity head losses. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisbach Equations.

$$\mathbf{H = KV^2/2g} \qquad \mathbf{(4.10)}$$

Where: H = velocity head loss (ft)

K = loss coefficient for the particular structure

V = velocity of flow (ft/s)

g = acceleration due to gravity (32.2 ft/s)

4.6.3 Entrance Losses

Following are the equations used for entrance losses for beginning flows.

$$\mathbf{H_{tm} = V^2/2g} \qquad \mathbf{(4.11)}$$

$$\mathbf{H_e = KV^2/2g} \qquad \mathbf{(4.12)}$$

Where: H_{tm} = terminal (beginning of run) loss (ft)

H_e = entrance loss for outlet structure (ft)

K = 0.5 (assuming square-edge)

(Other terms defined above)

4.6.4 Junction Losses

Incoming Opposing Flows

The head loss at a junction, H_{j1} for two almost equal and opposing flows meeting "head on" with the outlet direction perpendicular to both incoming directions, head loss is considered as the total velocity head of outgoing flow.

$$H_{j1} = (V_3^2)\ (\text{outflow})/2g \tag{4.13}$$

Where: H_{j1} = junction losses (ft)

(Other terms are defined above.)

Changes in Direction of Flow

When main storm drain pipes or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90^o) the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$H_b = K(V^2)\ (\text{outlet})/2g \tag{4.14}$$

Where: H_b = bend head loss (ft)

K = junction loss coefficient

Table 4-2 below lists the values of K for various junction angles.

Table 4-2
Values of K for Change in Direction of Flow in Lateral

K	Degrees of Turn (In Junction)
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

K values for other degree of turns can be obtained by interpolating between values.

Several Entering Flows

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy. For a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction.

The total junction losses at the sketched intersections are as follows:

The following equation can be used to calculate these losses:

$$\mathbf{H_{j2} = [(Q_4V_4^2) - (Q_1V_1^2) - (Q_2V_2^2) + (KQ_1V_1^2)]/(2gQ_4)} \qquad \mathbf{(4.15)}$$

Where: H_{j2} = junction losses (ft)

Q = Discharge (cfs)

V = horizontal velocity (ft/s)
(V_3 is assumed to be zero)

g = acceleration due to gravity (32.2 ft/s^2)

K = bend loss factor

Where subscript nomenclature is as follows:

Q_1 = 90^o lateral (cfs)

Q_2 = straight through inflow (cfs)

Q_3 = vertical dropped-in flow from an inlet (cfs)

Q_4 = main outfall = total computed discharge (cfs)

V_1, V_2, V_3, V_4 = horizontal velocities of foregoing flows, respectively (ft/s)

Also assume: $H_b = K(V_1^2)/2g$ for change in direction.

No velocity head of an incoming line is greater than the velocity head of the outgoing line.

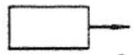
The water surface of inflow and outflow pipes in junction are level.

When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

4.6.5 Summary

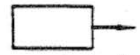
The final step in designing a storm drain system is to check the hydraulic grade line (HGL) as described in the next section of this chapter. Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc.

Figure 4-4 on the following page is a summary of energy losses which should be considered. Following this in Figure 4-5 is a sketch showing the proper and improper use of energy losses in developing a storm drain system.



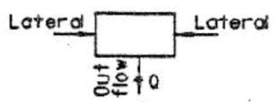
$$H_{tm} = \frac{v^2}{2g}$$

TERMINAL LOSSES
(at beginning of run)
Where g = gravitational constant.
32.2 feet per second
per second.



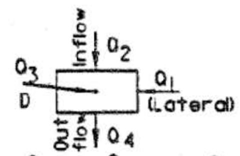
$$H_e = \frac{0.5\,v^2}{2g}$$

ENTRANCE LOSSES
(at end of run)
Assuming square - edge



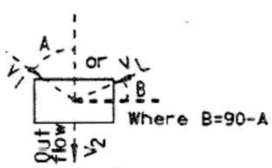
$$H_{J1} = \frac{v^2}{2g}\text{(Outflow)}$$

JUNCTION LOSSES
(Incoming-opposing Flow)
Use only where flows are
identical to above, otherwise
use H_{J2} Equation.



$$H_{J2} = \frac{Q_4 V_4^2 - Q_1 V_1^2 - Q_2 V_2^2 + K Q_1 V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(Several Entering Flows)
Total losses to include H_{J2} plus losses
for changes in direction of less than
90° (h_b).
Where K=Bend loss coefficient
Q_3=Vertical dropped-in flow
from an inlet
V_3=Assumed to be zero



$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSSES (H_f)

$H_f = S_f \times L$

Where H_f=friction head loss
S_f=friction slope
L =length of conduit

$$\text{Where } S = \left(\frac{Qn}{1.486AR^{\frac{2}{3}}}\right)^2$$

Q=Discharge of conduit
n=Mannings coefficient of roughness
A=area of conduit
R=hydraulic radius of conduit

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + (H_{J1} \text{ or } H_{J2}) + H_b + H_f$$

Figure 4-4 Summary of Energy Losses
Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt32.pdf

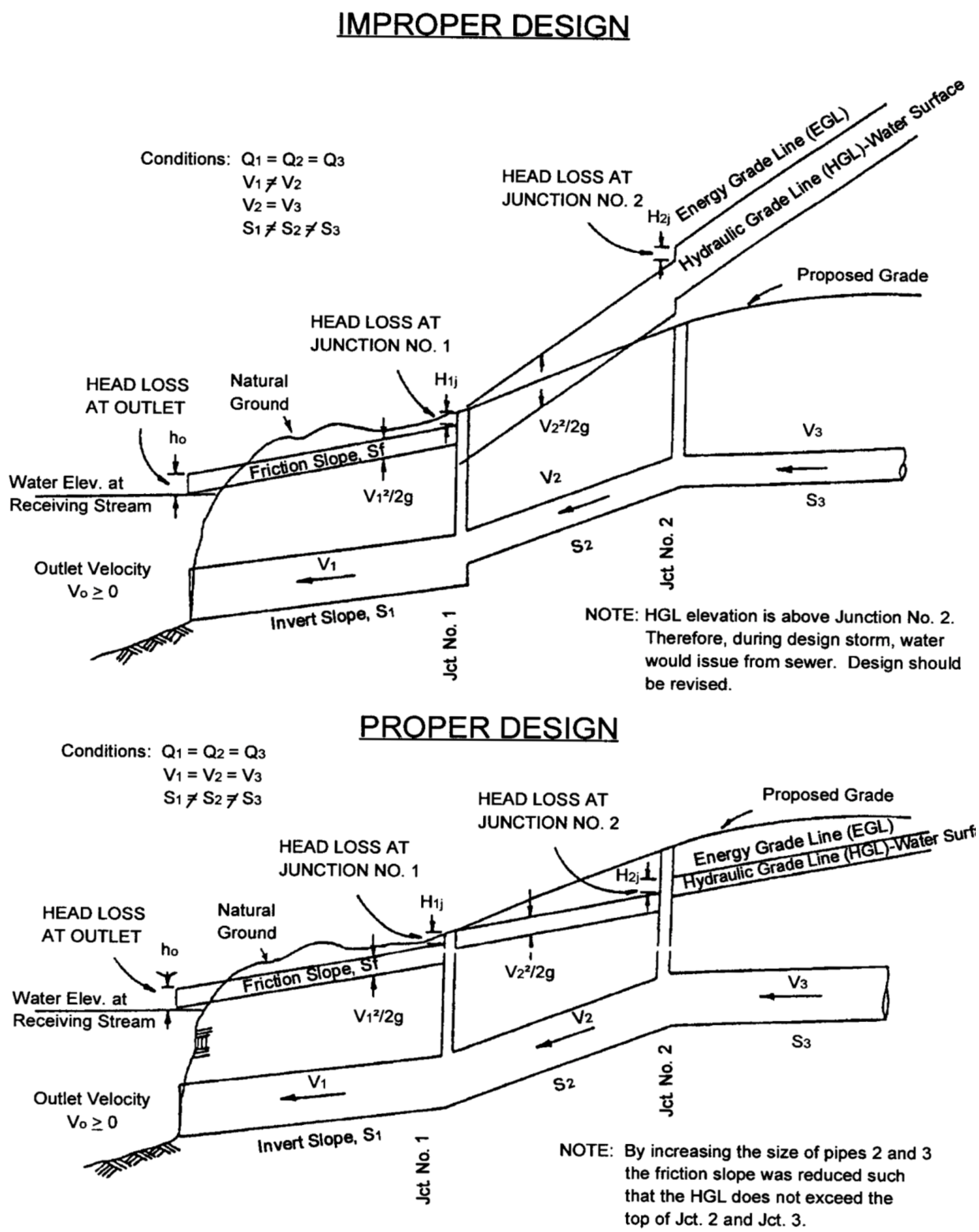


Figure 4-5 Energy and Hydraulic Grade Lines for Storm Drainage Under Constant Discharge
Source: www.ct.gov/dot/lib/dot/documents/ddrainage/11.12.pdf

4.7 STORM DRAIN

4.7.1 Introduction

After the tentative locations of inlets, drain pipes, and outfalls with tailwaters have been determined and the inlets have been sized, the next logical step is the computation for the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to convey this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the drain pipe serving that discharge is sized, and the process is repeated for the next run downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

For ordinary conditions, drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning's equation is recommended for capacity calculations.

4.7.2 Design Criteria

The standard maximum and minimum slopes for storm drains should conform to the following criteria:

1. The maximum pipe velocity shall not exceed 20 feet per second, or 10 feet per second in corrugated metal pipe.

2. The maximum discharge velocity at pipe outlets is 10 fps except for pipes greater than 48 inches in diameter.

3. The minimum allowable slope is 0.5 percent or the slope which will produce a velocity of 2.5 feet per second when the storm drainage system is flowing full, whichever is greater.

Systems should generally be designed for non-pressure conditions by using the Manning's equation. When hydraulic calculations do not consider minor energy losses such as expansion, contraction, bend, junction, and manhole losses, the elevation of the hydraulic gradient for design flood conditions should be at least 1.0 foot below surface inlet elevation. As a general rule, minor losses should be considered when the pipe velocity exceeds 6 feet per second (lower if flooding could cause critical problems). If all minor energy losses are accounted for, it is usually acceptable for the hydraulic gradient to reach 6 inches below the grate elevation.

4.7.3 Capacity

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's equation and it is expressed below:

$$V = [1.486\, R^{2/3} S^{1/2}]/n \qquad (4.16)$$

Where: V = mean velocity of flow (ft/s)

R = The hydraulic radius (ft) – defined as the area of flow divided by the wetted flow surface or wetted perimeter

S = the slope of hydraulic grade line (ft/ft)

n = Manning's roughness coefficient (see Table 4-3)

In terms of discharge, the above formula becomes:

$$\mathbf{Q = [1.486\ AR^{2/3}S^{1/2}]/n} \qquad \mathbf{(4.17)}$$

Where: Q = rate of flow (cfs)

A = cross sectional area of flow (ft^2)

For pipes flowing full, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \qquad \mathbf{(4.18)}$$

$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \qquad \mathbf{(4.19)}$$

Where: D = diameter of pipe (ft)

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = (2.88n^2V^2L)/D^{4/3}} \qquad \mathbf{(4.20)}$$

$$\mathbf{H_f = (29\ n^2LV^2)/(R^{4/3})(2g)} \qquad \mathbf{(4.21)}$$

Where: H_f = total head loss due to friction (ft)

L = length of pipe (ft)

g = acceleration due to gravity = 32.2 ft/sec^2

Table 4-3
Manning's *n* Values

Type of Conduit	Wall and Joint Description	Manning's *n* value
Concrete Pipe & Boxes	Good joints, smooth walls	0.012-0.013
Corrugated Metal Pipe & Boxes Annular Corrugations	68 by 13 mm 2- 2/3 by ½ inch corrugations	0.024
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	68 by 13 mm 2-2/3 by ½ inch corrugated, 24 inch plate width	0.021

Note: For futher information, consult Hydraulic Design of Highways Culverts, Federal Highway Administration, HDS No. 5, page 33.

4.7.4 Hydraulic Grade Lines

In calculating the hydraulic grade line within a closed storm drainage system, all head losses shall be computed to determine the water surface elevation within various structures.

The calculations are begun at the upstream or downstream opening, dependent upon whether the pipe is in inlet or outlet control. If it is inlet control the hydraulic grade line is the headwater elevation minus the entrance loss and the difference in velocity head. If the outlet controls, the tail water surface elevation or 0.8 times the diameter of the pipe, wherever is higher, is the outlet hydraulic grade line. Hydraulic grade lines will be required only as requested on a case by case basis.

If computer models are utilized then results should be consistent with the procedure outlined below. All input data should be supplied, including loss coefficients, and output should be in similar format to Figure 4-6.

4.7.4.1 Design Procedure

The head losses are calculated beginning from the downstream control point to the first junction upstream, and the procedure is repeated for the next junction. The computation for outlet control may be tabulated using Figure 4-6 and the following procedure:

1. Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

2. Enter in Column 2 the outlet water surface elevation or 0.8 diameter plus invert out elevation of the outflow pipe whichever is greater.

3. Enter in Column 3 the diameter (D_o) of the outflow pipe.

4. Enter in Column 4 the design discharge (Q_o) for the outflow pipe.

5. Enter in Column 5 the length (L_o) of the outflow pipe.

6. Enter in Column 6 the friction slope (S_f) in ft/ft of the outflow pipe. This can be determined from the following formula:

$$\mathbf{S_f = \{Qn/1.486AR^{2/3}\}^2} \quad \textbf{(4.22)}$$

7. Multiply the friction slope (S_f) in Column 6 by the length (L_o) in Column 5 and enter the friction loss (H_f) in Column 7.

8. Enter in Column 8 the velocity of the flow (V_o) of the outflow pipe.

9. Enter in Column 9 the contraction loss (H_o) by using the formula:

$$\mathbf{H_o = 0.25\ (V_o^2)/2g} \quad \textbf{(4.23)}$$

   Where: $g = 32.2\ ft/s^2$

10. Enter in Column 10 the design discharge (Q_j) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than ten percent of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

11. Enter in Column 11 the velocity of flow (V_i) for each pipe flowing into the junction (for exception see Step 10).

12. Enter in Column 12 the product of (Q_i x V_i) for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest (Q_i x V_i) product is the line which will produce the greatest expansion loss (H_i). (For exception, see step 10).

13. Enter in Column 13 the controlling expansion loss (H_i) using the formula $H_i = 0.35 (V_i^2)/2g$.

14. Enter in Column 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).

15. Enter in Column 15 the greatest bend loss (H_Δ) calculated by using the formula:

$$\mathbf{H_\Delta = KV_i^2/2g} \quad \textbf{(4.24)}$$

    Where: K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.

16. Enter in Column 16 the total head loss (H_t) by summing the values in Column 9 (H_o), Column 13 (H_i) and Column 15 (H_Δ).

17. If the junction incorporates adjusted surface inflow of ten percent or more of the mainline outflow, i.e., drop inlet, increase H_t by 30 percent and enter the adjusted H_t in Column 17.

18. If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of H_t by 50 percent and enter the adjusted value in column 18.

19. Enter in Column 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of the H_t.

20. Enter in Column 20 the sum of the elevation in Column 2 and the final H in Column 19. This elevation is the potential water surface elevation for the junction under design conditions.

21. Enter in Column 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Column 20. If the potential water surface elevation exceeds the rim elevation or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the H.G.L.

22. Repeat the procedure starting with Step 1 for the next junction upstream.

4.7.5 Minimum Grade

The minimum allowable slope is 0.5 percent or the slope which will produce a velocity of 2.5 feet per second when the storm drainage system is flowing full, whichever is greater.

The minimum slopes are calculated by the modified Manning's equation:

$$\mathbf{S = (nV)^2/(2.208\ R^{4/3})} \qquad \mathbf{(4.25)}$$

4.7.6 Design Procedures

The design of storm drain systems is generally divided into the following operations:

1. The first step is the determination of inlet location and spacing as outlined earlier in this chapter.

2. The second step is the preparation of a plan layout of the storm drainage system establishing the following design area:

   a. Location of storm drains
   b. Direction of flow
   c. Location of manholes
   d. Location of existing facilities such as water, gas, or underground cables

3. The design of the storm drain system is then accomplished by determining drainage areas, computing runoff using the rational method, and computing the hydraulic capacity using Manning's equation. As a final step the hydraulic performance of the system can be checked by calculating the hydraulic grade line.

4. The storm drain design computation sheet (Figure 4-6) and hydraulic grade line computation form (Figure 4-7) can be used to summarize the design computations.

STORM DRAINAGE COMPUTATION FORM

PROJECT NAME ____________ COMPUTED BY ____________ DATE ____________

PROJECT LOCATION CHECKED BY

Structure		Length (feet)	Drainage Area 'A' (Acres)		Runoff Coefficient 'C'	A' X 'C'		Flow Time (min)		Rainfall Intensity (in/hr)	Total Runoff (cfs)	Pipe Diameter (in)	Full Capacity (cfs)	Velocity type		Invert Elevation		Manhole Invert Drop	Slope of Sewer (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Figure 4-6 Storm Drainage Computation Form

HYDRAULIC GRADE LINE COMPUTATION FORM

PROJECT NAME ________________ COMPUTED BY __________ DATE ______________

PROJECT LOCATION ________________ CHECKED BY __________

							Junction Losses													
Station 1	Outlet Water Surface Elevation (ft) 2	D_o (ft) 3	Q_o (cfs) 4	L_o (ft) 5	S_f (ft/ft) 6	H_f (ft) 7	V_o (ft/s) 8	H_o (ft) 9	Q_i (cfs) 10	V_i (ft/s) 11	Q_iV_i 12	$\frac{V_i^2}{2g}$ (ft)	H_i (ft) 13	Angle (°) 14	$H_\wedge$ (ft) 15	H_t (ft) 16	$H_t^{1.3}$ (ft) 17	$H_t^{0.5}$ (ft) 18	Final H (ft) 19	Inlet Water Surface Elevation (ft) 20	Rim Elevation (ft) 21

$S_f = (Qn/1.486AR^{2/3})^2$ $H_f = S_f \times L_o$ $H_o = 0.25(V_o^2)/2g$ $H_i = 0.35\ (V_i^2/2g)$ $H_\wedge = K(V_i^2/2g)$ $H_t = H_o + H_i + H_\wedge$

Final $H = H_f + H_t$

90^o K = 0.70 45^o K = 0.47

75^o K = 0.64 30^o K = 0.35

60^o K = 0.56 15^o K = 0.19

Figure 4-7 Hydraulic Grade Line Computation Form

4.7.7 Rational Method Examples

The following example will illustrate the hydrologic calculations needed for storm drain design using the rational formula (see Hydrology Chapter for Rational Method description and procedures). Figure 4-8 shows a hypothetical storm drain system that will be used in this example. Following is a tabulation of the data needed to use the rational equation to calculate inlet flow rate for the seven inlets shown in the system layout.

Table 4-4
Hydrologic Data

Inlet[a]	Drainage Area (acres)	Time of Concentration (minutes)	Rainfall Intensity (inches/hour)	Runoff Coefficient	Inlet Flow Rate[b] (cfs)
1	2.0	8	6.26	.9	11.3
2	3.0	10	5.84	.9	15.8
3	2.5	9	6.04	.9	13.6
4	2.5	9	6.04	.9	13.6
5	2.0	8	6.26	.9	11.3
6	2.5	9	6.04	.9	13.6
7	2.0	8	6.26	.9	11.3

[a] Inlet and storm drain system configuration are shown in Figure 4-8
[b] Calculated using the Rational Equation (see Chapter 2, Hydrology)

The following table shows the data and results of the calculation needed to determine the design flow rate in each segment of the hypothetical storm drain system.

Table 4-5
Storm Drain System Calculations

Storm Drain Segment	Tributary Area (acres)	Time of Concentration (minutes)	Rainfall Intensity (inches/hour)	Runoff Coefficient	Design Flow Rate (cfs)
I_1-M_1	2.0	8	6.26	.9	11.3
I_2-M_1	3.0	10	5.84	.9	15.8
M_1-M_2	5.0	10.5	5.76	.9	25.9
I_3-M_2	2.5	9	6.04	.9	13.6
I_4-M_2	2.5	9	6.04	.9	13.6
M_2-M_3	10.0	11.5	5.60	.9	50.4
I_5-M_3	2.0	8	6.26	.9	11.3
I_6-M_3	2.5	9	6.04	.9	13.6
M_3-M_4	14.5	13.5	5.27	.9	68.8
I_7-M_4	2.0	8	6.26	.9	11.3
M_4-O	16.5	14.7	5.08	.9	75.4

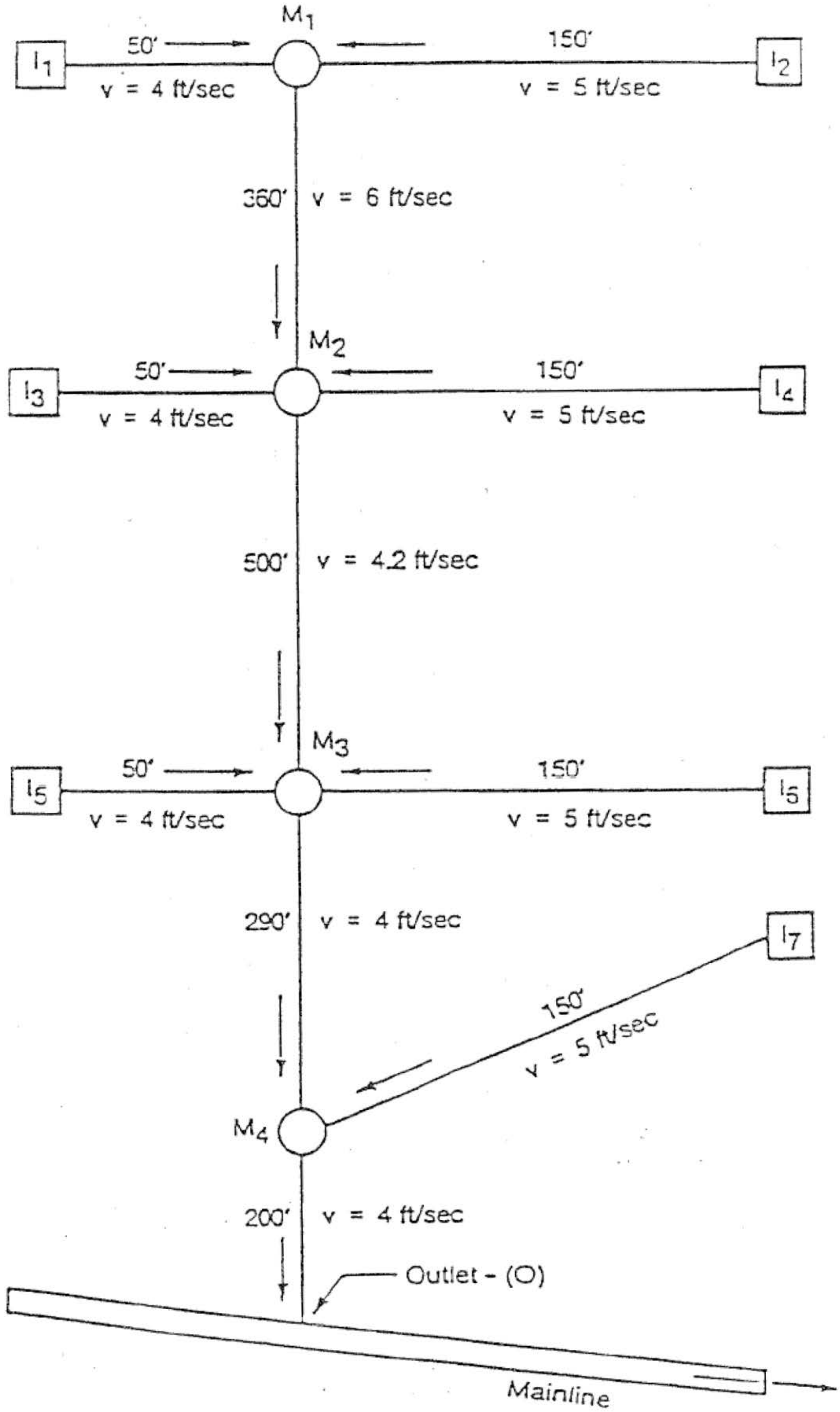


Figure 4-8 Hypothetical Storm Drain System Layout
Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt32.pdf