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## 1.0 INTRODUCTION

### **1.1 Purpose**

The purpose of this chapter is to give concise, practical guidelines for the design of urban stormwater collection and conveyance systems. Procedures and equations are presented for the hydraulic design of street drainage, locating inlets and determining capture capacity, and sizing storm sewers. In addition, examples are provided to illustrate the hydraulic design process. Spreadsheet solutions accompany the hand calculations for most example problems.

The design procedures presented in this chapter are based upon fundamental hydrologic and hydraulic design concepts. The design equations provided are well accepted and widely used. They are presented without derivations or detailed explanation, but are properly referenced if the reader wishes to study their background. Therefore, it is assumed that the reader has a fundamental understanding of basic hydrology and hydraulics. A working knowledge of the Rational equation (RUNOFF chapter) and open channel hydraulics (MAJOR DRAINAGE chapter) is particularly helpful.

### **1.2 Urban Stormwater Collection and Conveyance Systems**

Urban stormwater collection and conveyance systems are critical components of the urban infrastructure. Proper design of these systems is essential to minimize flood damage and disruptions in urban areas during storm events while protecting the urban water resources environment. Their primary function is to collect excess stormwater from street gutters, convey the excess stormwater through storm sewers and along the street right-of-way, and discharge it into a detention basin, water quality best management practice (BMP) or the nearest receiving water body (FHWA 1996).

Urban stormwater collection and conveyance systems must fulfill many objectives. Properly functioning urban drainage systems:

- Minimize disruption to the natural drainage system.
- Promote safe passage of vehicular traffic during minor storm events.
- Maintain public safety and manage flooding during major storm events.
- Preserve and protect the urban stream environment.
- Minimize capital and maintenance costs of the system.

All of these objectives are important, but the public is the most vocal about disruptions to traffic and street flooding when storm drainage systems are not designed properly.



*Photograph ST-1—The critical role that streets play in urban inlet and storm sewer drainage is often not properly taken into account.*



*Photograph ST-2—The capital costs of storm sewer construction are large, emphasizing the importance of sound design.*

### **1.3 Components of Urban Stormwater Collection and Conveyance Systems**

Urban stormwater collection and conveyance systems within the District are comprised of three primary

components: (1) street gutters and roadside swales, (2) stormwater inlets, and (3) storm sewers (and appurtenances like manholes, junctions, etc.). Street gutters and roadside swales collect runoff from the street (and adjacent areas) and convey the runoff to a stormwater inlet while maintaining the street's level-of-service.

Inlets collect stormwater from streets and other land surfaces, transition the flow into storm sewers, and often provide maintenance access to the storm sewer system. Storm sewers convey stormwater in excess of a street's or a swale's capacity along the right-of-way and discharge it into a stormwater management facility or a nearby receiving water body. In rare instances, stormwater pump stations (the design of which is not covered in this *Manual*) are needed to lift and convey stormwater away from low-lying areas where gravity drainage is not possible. All of these components must be designed properly to achieve the stormwater collection and conveyance system's objectives.

#### **1.4 Minor and Major Storms**

Rainfall events vary greatly in magnitude and frequency of occurrence. Major storms produce large flow rates but rarely occur. Minor storms produce smaller flow rates but occur more frequently. For economic reasons, stormwater collection and conveyance systems are not normally designed to pass the peak discharge during major storm events.

Stormwater collection and conveyance systems are designed to pass the peak discharge of the minor storm event (and smaller events) with minimal disruption to street traffic. To accomplish this, the spread of water on the street is limited to some maximum, mandated value during the minor storm event. Inlets must be strategically placed to pick up the excess gutter or swale flow once the limiting spread of water is reached. The inlets direct the water into storm sewers, which are typically sized to pass the peak flow rate from the minor storm without any surcharge. The magnitude of the minor storm is established by local ordinances or criteria, and the 2-, 5-, or 10-year storms are most commonly specified.

On occasion, storms will occur that surpass the magnitude of the minor storm event. When this happens, the spread of water on the street exceeds the allowable spread and the capacity of the storm sewers designed for the minor storm event. Street flooding occurs and traffic is disrupted. However, proper design requires that public safety be maintained and the flooding be managed to minimize flood damage. Thus, local ordinances also often establish the return period for the major storm event, generally the 100-year storm. For this event, the street becomes an open channel and must be analyzed to determine that the consequences of the flood are acceptable with respect to flood damage and public safety.

## 2.0 STREET DRAINAGE

### 2.1 Street Function and Classification

The primary function of a street or roadway is to provide for the safe passage of vehicular traffic at a specified level of service. If stormwater collection and conveyance systems are not designed properly, this primary function can be impaired. To make sure this does not happen, streets are classified for drainage purposes based on their traffic volume, parking practices, and other criteria (Wright-McLaughlin Engineers 1969). The four street classifications are:

- Local (low-speed traffic for residential or industrial area access).
- Collector (low/moderate-speed traffic providing service between local streets and arterials).
- Arterial (moderate/high-speed traffic moving through urban areas and accessing freeways).
- Freeway (high-speed travel, generally over long distances).

Table ST-1 provides additional information on the classification of streets for drainage purposes.

**Table ST-1—Street Classification for Drainage Purposes**

Street Classification	Function	Speed/Number of Lanes	Signalization at Intersections	Street Parking
Local	Provide access to residential and industrial areas	Low speed with 2 moving lanes	Stop signs	One or both sides of the street
Collector	Collect and convey traffic between local and arterial streets	Low to moderate speed with 2 or 4 moving lanes	Stop signs or traffic signals	One or both sides of the street
Arterial	Function as primary through-traffic conduits in urban areas	Moderate to high speeds with 4 to 6 lanes	Traffic signals (controlled access)	Usually prohibited
Freeway	Provide rapid and efficient transport over long distances	High-speed travel with 4 lanes or more	Cloverleaves, access ramps (limited access)	Always prohibited

Streets serve another important function other than traffic flow. They contain the first component in the urban stormwater collection and conveyance system. That component is the street gutter or adjacent swale, which collects excess stormwater from the street and adjacent areas and conveys it to a stormwater inlet. Proper street drainage is essential to:

- Maintain the street's level-of-service.



- Reduce skid potential.
- Minimize the potential for cars to hydroplane.
- Maintain good visibility for drivers (by reducing splash and spray).
- Minimize inconvenience/danger to pedestrians during storm events (FHWA 1984).

**2.2 Design Considerations**

Certain design considerations must be taken into account in order to meet street drainage objectives. The primary design objective is to keep the spread (encroachment) of stormwater on the street below an acceptable value for a given return period of flooding. As mentioned previously, when stormwater collects on the street and flows down the gutter, the top width (or spread) of the water widens as more stormwater is collected. If left unchecked, the spread of water would eventually hinder traffic flow and possibly become hazardous (i.e., reduced skid resistance, hydroplaning, splash, etc.). Based on these considerations, the District has established encroachment (spread) standards for the minor storm event. These standards were given in the POLICY chapter and are repeated in Table ST-2 for convenience.

**Table ST-2—Pavement Encroachment Standards for the Minor Storm**

Street Classification	Maximum Encroachment
Local	No curb overtopping. Flow may spread to crown of street.
Collector	No curb overtopping. Flow spread must leave at least one lane free of water.
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction, but should not flood more than two lanes in each direction.
Freeway	No encroachment is allowed on any traffic lanes.

Standards for the major storm and street cross flows are also required. The major storm needs to be assessed to determine the potential for flooding and public safety. Cross flows also need to be regulated for traffic flow and public safety reasons. The District has established street inundation standards during the major storm event and allowable cross-street flow standards. These standards were given in the POLICY chapter and are repeated in Table ST-3 and Table ST-4 for convenience.

**Table ST-3—Street Inundation Standards for the Major (i.e., 100-Year) Storm**

<b>Street Classification</b>	<b>Maximum Depth and Inundated Area</b>
Local and Collector	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The depth of water over the gutter flow line should not exceed 18 inches.
Arterial and Freeway	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12 inches.

**Table ST-4—Allowable Cross-Street Flow**

<b>Street Classification</b>	<b>Initial Storm Flow</b>	<b>Major (100-Year) Storm Flow</b>
Local	6 inches of depth in cross pan.	18 inches of depth above gutter flow line.
Collector	Where cross pans allowed, depth of flow should not exceed 6 inches.	12 inches of depth above gutter flow line.
Arterial/Freeway	None.	No cross flow. Maximum depth at upstream gutter on road edge of 12 inches.

Once an allowable spread (pavement encroachment) has been established for the minor storm, the placement of inlets can be determined. The inlets will remove some or all of the excess stormwater and thus reduce the spread. The placement of inlets is covered in Section 3.0. It should be noted that proper drainage design utilizes the full allowable capacity of the street gutter in order to limit the cost of inlets and storm sewers.

Another important design consideration is the frequency of occurrence of the minor storm. In other words, how often will the spread of stormwater reach or exceed the maximum encroachment limit. This is addressed by assigning a frequency (or recurrence interval) to the minor storm. The selection of a design frequency is based on many factors including street function, traffic load, vehicle speed, etc. The minor storm is generally between the 2-year and 10-year storm. The major storm is normally defined as the 100-year storm. The minor and major storm return periods are mandated by local governments.

Two additional design considerations of importance in street drainage are gutter (channel) shape and street slope. Most urban streets contain curb and gutter sections. Various types exist which include spill shapes, catch shapes, curb heads, and roll gutters. The shape is chosen for functional, cost, or aesthetic reasons and does not dramatically affect the hydraulic capacity. Swales are common along some urban and semi-urban streets, and roadside ditches are common along rural streets. Their shapes are

important in determining hydraulic capacity and are covered in the next section.

### **2.3 Hydraulic Evaluation**

Hydraulic computations are performed to determine the capacity of roadside swales and street gutters and the encroachment of stormwater onto the street. The design discharge is usually determined using the Rational method (covered in the next two sections). Stormwater runoff ends up in swales, roadside ditches and street gutters where the flow is unsteady and non-uniform. However, uniform, steady flow is usually assumed for the short period of time during peak flow conditions.

#### **2.3.1 Curb and Gutter**

Street slope can be divided into two components: longitudinal slope and cross slope. The longitudinal slope of the gutter essentially mimics the street slope. The hydraulic capacity of a gutter increases as the longitudinal slope increases. The District prescribes a minimum grade of 0.4% (Wright-McLaughlin 1969). The allowable flow capacity of the gutter on steep slopes is limited to provide for public safety. The cross (transverse) slope represents the slope from the street crown to the gutter section. A compromise is struck between large cross slopes that facilitate pavement drainage and small cross slopes for driver safety and comfort. The District prescribes a minimum cross slope of 1% for pavement drainage. Composite sections are often used with gutter cross slopes being steeper than street cross slopes to increase the gutter capacity.

The hydraulic evaluation of street capacity includes the following steps:

1. Calculate the theoretical street gutter flow capacity to convey the minor storm based upon the allowable **spread** defined in [Table ST-2](#).
2. Calculate the theoretical street gutter flow capacity to convey the minor storm based upon the allowable **depth** defined [Table ST-2](#).
3. Calculate the allowable street gutter flow capacity by multiplying the theoretical capacity (calculated in number 2) by a reduction factor. This reduction factor is used for safety considerations. The lesser of the capacities calculated in step 1 and this step is the allowable street gutter capacity.
4. Calculate the theoretical major storm conveyance capacity based upon the road inundation criteria in [Table ST-3](#). Reduce the major storm capacity by a reduction factor to determine the allowable storm conveyance capacity.

##### **2.3.1.1 Gutters With Uniform Cross Slopes (i.e., Where Gutter Cross Slope = Street Cross Slope)**

Since gutter flow is assumed to be uniform for design purposes, Manning's equation is appropriate with a slight modification to account for the effects of a small hydraulic radius. For a triangular cross section

([Figure ST-1a](#)), the Manning formula for gutter flow is written as:

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3} \quad (\text{ST-1})$$

in which:

$Q$  = calculated flow rate for the street (cfs)

$n$  = Manning's roughness coefficient, (typically = 0.016)

$S_x$  = street cross slope for the street (ft/ft)

$S_L$  = longitudinal slope (ft/ft)

$T$  = top width of flow spread (ft)

The flow depth,  $y$ , at the curb can be found using:

$$y = TS_x \quad (\text{ST-2})$$

Note that the flow depth must be less than the curb height during the minor storm based on [Table ST-2](#).

Manning's equation can be written in terms of the flow depth, as:

$$Q = \frac{0.56}{nS_x} S_L^{1/2} y^{8/3} \quad (\text{ST-3})$$

The cross-sectional flow area,  $A$ , can be expressed as:

$$A = (1/2) S_x T^2 \quad (\text{ST-4})$$

The gutter velocity at peak capacity may be found from the continuity equation ( $V = Q/A$ ). Triangular gutter cross-section calculations are illustrated in Example 6.1.

### 2.3.1.2 Gutters With Composite Cross Slopes (i.e., Where Gutter Cross Slope $\neq$ Street Cross Slope)

Gutters with composite cross slopes ([Figure ST-1b](#)) are often used to increase the gutter capacity. For a composite gutter section:

$$Q = Q_w + Q_s \quad (\text{ST-5})$$

in which:

$Q_w$  = flow rate in the depressed section of the gutter (cfs)

$Q_s$  = discharge in the section that is above the depressed section (cfs)

The Federal Highway Administration (FHWA 1996) provides the following equations for obtaining the flow rate in gutters with composite cross slopes. The theoretical flow rate,  $Q$ , is:

$$Q = \frac{Q_s}{1 - E_o} \quad (\text{ST-6})$$

in which:

$$E_o = \frac{1}{1 + \frac{S_w / S_x}{\left[1 + \frac{S_w / S_x}{(T/W) - 1}\right]^{8/3}} - 1} \quad (\text{ST-7})$$

in which  $S_w$  is the gutter cross slope (ft/ft), and,

$$S_w = S_x + \frac{a}{W} \quad (\text{ST-8})$$

in which  $a$  is the gutter depression (feet) and  $W$  is width of the gutter (ft).

[Figure ST-1b](#) depicts all geometric variables. From the geometry, it can be shown that:

$$y = a + TS_x \quad (\text{ST-9})$$

and,

$$A = \frac{1}{2} S_x T^2 + \frac{1}{2} a W \quad (\text{ST-10})$$

in which  $y$  is the flow depth (at the curb) and  $A$  is the flow area. Composite cross-section gutter flow calculations are illustrated in Examples 6.2 and 6.3.

### 2.3.1.3 Allowable Gutter Hydraulic Capacity

Stormwater flows along streets exert momentum forces on cars, pavement, and pedestrians. To limit the hazardous nature of heavy street flows, it is necessary to set limits on flow velocities and depths. As a result, the allowable gutter hydraulic capacity is determined as the lesser of:

$$Q_A = Q_T \quad (\text{ST-11})$$

or

$$Q_A = R Q_F \quad (\text{ST-12})$$

in which  $Q_A$  = allowable street hydraulic capacity,  $Q_T$  = street hydraulic capacity limited by the maximum water spread,  $R$  = reduction factor, and  $Q_F$  = gutter capacity when flow depth equals allowable depth.

There are two sets of reduction factors developed for Denver metropolitan areas (Guo 2000b). One is for the minor event, and another is for the major event. [Figure ST-2](#) shows that the reduction factor remains unity (1.0) for a street slope <1.5%, and then decreases as the street slope increases.

It is important for street drainage designs that the allowable street hydraulic capacity be used instead of the calculated gutter-full capacity. Thus, wherever the accumulated stormwater amount on the street is close to the allowable capacity, a street inlet shall be installed.

### **2.3.2 Swale Sections (V-Shaped With the Same or Different Side Slopes)**

Swales are often used to convey runoff from pavement where curb and gutter sections are not used. It is very important that swale depths and side slopes be as shallow as possible for safety and maintenance reasons. Street-side swales are not the same as roadside ditches that can be considered part of a major drainageway system. Street-side swales serve as collectors of initial runoff and transport it to the nearest inlet or major drainageway. To be effective, they need to be limited to the velocity, depth, and cross-slope geometries considered acceptable. The following limitations shall apply to street-side swales:

- Maximum 2-year flow velocity = 3 ft/sec
- Maximum flow depth = 1.0 ft
- Maximum side slope of each side = 5H:1V.\*

\* Use of flatter side slopes is strongly recommended.

Swales generally have V-sections ([Figure ST-3](#)). Equation ST-1 can be used to calculate the flow rate in a V-section (if the section has a constant Manning's  $n$  value) with an adjusted slope found using:

$$S_x = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \quad (\text{ST-13})$$

in which:

$S_x$  = adjusted side slope (ft/ft)

$S_{x1}$  = right side slope (ft/ft)

$S_{x2}$  = left side slope (ft/ft)

[Figure ST-3](#) shows the geometric variables.

Examples 6.4 and 6.5 show V-shaped swale calculations.

Under no circumstances shall a street-side swale have a longitudinal slope steeper than 2%. Use grade control checks to control the grade if the adjacent street is steeper.

Note that the slope of roadside ditches and swales is often different than the adjacent street. The hydraulic characteristics of the swale can therefore change from one location to another on a given swale. The flow depth and spread limitations of [Tables ST-2](#) and [ST-4](#) are also valid for swales and roadside ditches. There is no capacity reduction for safety considerations for roadside swales.

The designer is cautioned when using swales. If not properly designed and maintained, they can become a nuisance to the local residents.

Manning's equation can be used to calculate flow characteristics.

$$Q = \frac{1.49}{n} AR^{2/3} S_L^{1/2} \quad (\text{ST-14})$$

in which:

$Q$  = flow rate (cfs)

$n$  = Manning's roughness coefficient

$A$  = flow area (ft<sup>2</sup>)

$R$  =  $A/P$  (ft)

$P$  = wetted perimeter (ft)

$S_L$  = longitudinal slope (ft/ft)

## **2.4 Major Storm Hydraulics**

### **2.4.1 Purpose and Objectives**

As previously mentioned, the primary objective of street drainage design is not to exceed the spread (encroachment) criteria during the minor storm event. Since larger storms do occur, it is prudent to determine the consequences of the major storm event. [Table ST-3](#) lists the street inundation standards recommended by this *Manual* for the major storm event. Proper street design requires that the major storm be assessed in the interest of public safety and to minimize the potential for flood damages.

### **2.4.2 Street Hydraulic Capacity**

During major storms, streets typically become wide, open channels that convey stormwater flow in excess of the storm sewer capacity. Manning's equation (Equation ST-14) is generally appropriate to determine flow depths and street capacities assuming uniform flow.

The general form of Manning's equation is the most appropriate solution method for this situation since many different flow situations and channel shapes may be encountered. The allowable street capacity for a major storm is also subject to safety considerations using the reduction factor taken from [Figure ST-2](#).

Major storm street hydraulic capacity calculations are shown in Example 6.6.



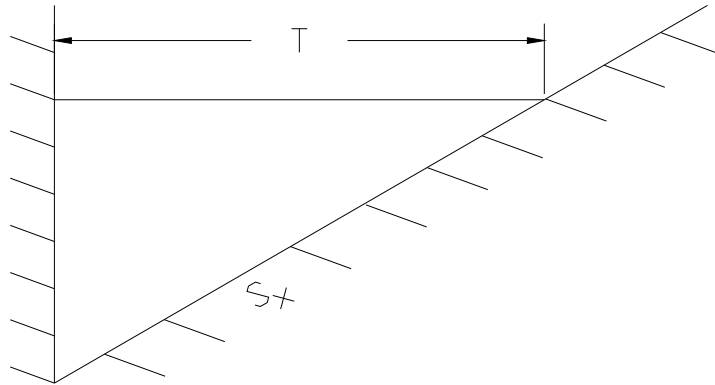


Figure ST-1a—Typical Gutter Sections—Constant Cross Slope

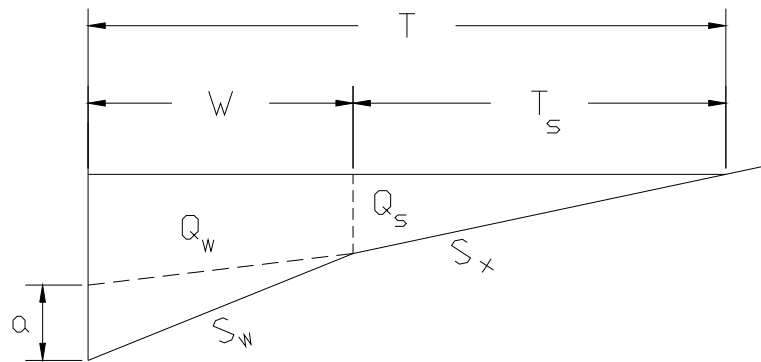


Figure ST-1b—Typical Gutter Sections—Composite Cross Slope

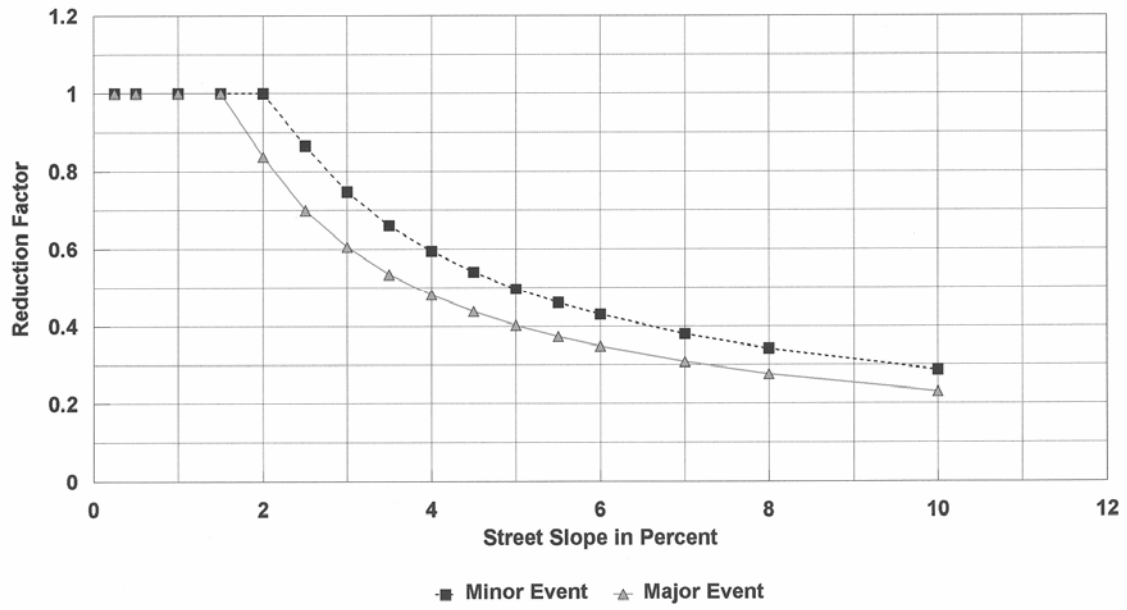
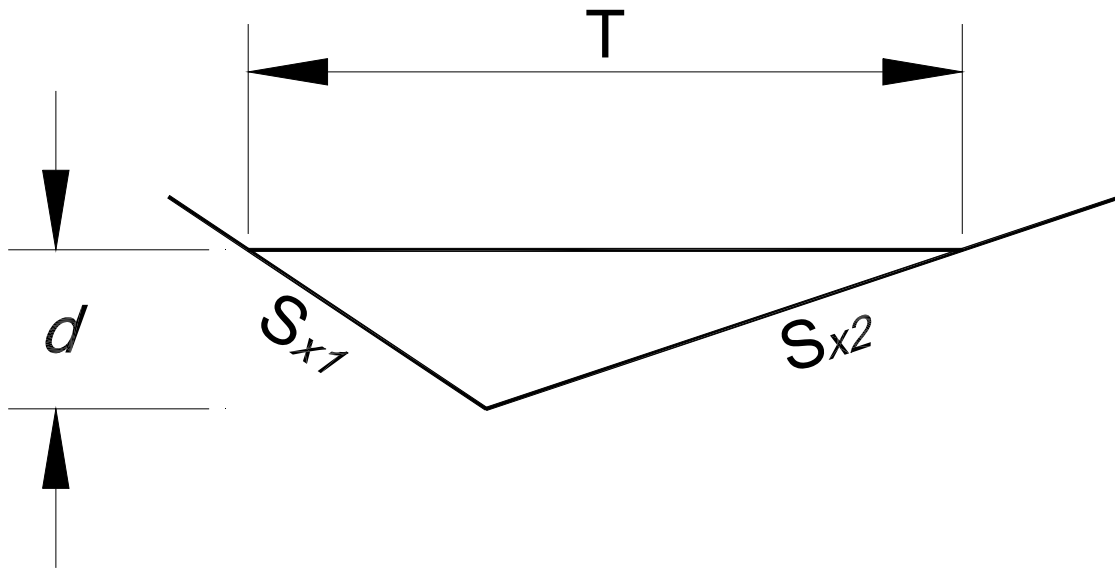


Figure ST-2—Reduction Factor for Gutter Flow



Notes:

1.  $S_{x1}$  and  $S_{x2} \leq 5H:1V$ .
2.  $d \leq 1.0$  feet.
3. Normal flow velocity in a grass-lined swale shall be less than 3 ft/sec during a 2-year storm.
4. Longitudinal grade of a grass-lined swale shall be less than 2%. Use grade control checks if adjacent street is steeper to limit the swale's flow.

**Figure ST-3—Typical Street-Side Swale Sections—V-Shaped**

### 3.0 INLETS

#### **3.1 Inlet Functions, Types and Appropriate Applications**

Stormwater inlets are a vital component of the urban stormwater collection and conveyance system. Inlets collect excess stormwater from the street, transition the flow into storm sewers, and can provide maintenance access to the storm sewer system. They can be made of cast-iron, steel, concrete, and/or pre-cast concrete and are installed on the edge of the street adjacent to the street gutter or in the bottom of a swale.

Roadway geometrical features often dictate the location of pavement drainage inlets. In general, inlets are placed at all low points (sumps or sags) in the gutter grade, median breaks, intersections, and crosswalks. The spacing of inlets placed between those required by geometric controls is governed by the design flow spread (i.e., allowable encroachment). In other words, the drainage inlets are spaced so that the spread under the design (minor) storm conditions will not exceed the allowable flow spread (Akan and Houghtalen 2002).

There are four major types of inlets: grate, curb opening, combination, and slotted. [Figure ST-4](#) depicts the four major types of inlets along with some associated geometric variables. Table ST-5 provides information on the appropriate application of the different inlet types along with advantages and disadvantages of each.

**Table ST-5—Applicable Settings for Various Inlet Types**

<b>Inlet Type</b>	<b>Applicable Setting</b>	<b>Advantages</b>	<b>Disadvantages</b>
Grate	Sumps and continuous grades (should be made bicycle safe)	Perform well over wide range of grades	Can become clogged Lose some capacity with increasing grade
Curb-opening	Sumps and continuous grades (but not steep grades)	Do not clog easily Bicycle safe	Lose capacity with increasing grade
Combination	Sumps and continuous grades (should be made bicycle safe)	High capacity Do not clog easily	More expensive than grate or curb-opening acting alone
Slotted	Locations where sheet flow must be intercepted.	Intercept flow over wide section	Susceptible to clogging

#### **3.2 Design Considerations**

Stormwater inlet design takes two forms: inlet placement location and inlet hydraulic capacity. As previously mentioned, inlets must be placed in sumps to prevent ponding of excess stormwater. On streets with continuous grades, inlets are required periodically to keep the gutter flow from exceeding the encroachment limitations. In both cases, the size and type of inlets need to be designed based upon their hydraulic capacity.

Inlets placed on continuous grades rarely intercept all of the gutter flow during the minor (design) storm. The effectiveness of the inlet is expressed as an efficiency,  $E$ , which is defined as:

$$E = Q_i / Q \quad (\text{ST-15})$$

in which:

$E$  = inlet efficiency

$Q_i$  = intercepted flow rate (cfs)

$Q$  = total gutter flow rate (cfs)

Bypass (or carryover) flow is not intercepted by the inlet. By definition,

$$Q_b = Q - Q_i \quad (\text{ST-16})$$

in which:

$Q_b$  = bypass (or carryover) flow rate (cfs)

The ability of an inlet to intercept flow (i.e., hydraulic capacity) on a continuous grade generally increases with increasing gutter flow, but the capture efficiency decreases. In other words, even though more stormwater is captured, a smaller percentage of the gutter flow is captured. In general, the inlet capacity depends upon:

- The inlet type and geometry.
- The flow rate (depth and spread of water).
- The cross (transverse) slope.
- The longitudinal slope.

The hydraulic capacity of an inlet varies with the type of inlet. For grate inlets, the capacity is largely dependent on the amount of water flowing over the grate, the grate configuration and spacing, and the velocity of flow. For curb opening inlets, the capacity is largely dependent on the length of the opening, the flow velocity, street and gutter cross slope, and the flow depth at the curb. Local gutter depression along the curb opening helps boost the capacity. On the other hand, top slab supports can decrease the capacity. Combination inlets do not intercept much more than their grates alone if they are placed side by side and are of nearly equal lengths but are much less likely to clog. Slotted inlets function in a manner similar to curb opening inlets (FHWA 1996).

Inlets in sumps operate as weirs for shallow pond depths, but eventually will operate as orifices as the depth increases. A transition region exists between weir flow and orifice flow, much like a culvert. Grate

inlets and slotted inlets tend to clog with debris, so calculations should take that into account. Curb opening inlets tend to be more dependable for this reason.

### **3.3 Hydraulic Evaluation**

The hydraulic capacity of an inlet is dependent on the type of inlet (grate, curb opening, combination, or slotted) and the location (on a continuous grade or in a sump). The methodology for determination of hydraulic capacity of the various inlet types is described in the following sections: (a) grate inlets on a continuous grade (Section 3.3.1), (b) curb opening inlets on a continuous grade (Section 3.3.2), (c) combination inlets on a continuous grade (Section 3.3.3), (d) slotted inlets on a continuous grade (Section 3.3.4), and (e) inlets located in sumps (Section 3.3.5).

#### **3.3.1 Grate Inlets (On a Continuous Grade)**

The capture efficiency of a grate inlet is highly dependent on the width and length of the grate and the velocity of gutter flow. If the gutter velocity is low and the spread of water does not exceed the grate width, all of the flow will be captured by the grate inlet. This is not normally the case during the minor (design) storm. The spread of water often exceeds the grate width and the flow velocity can be high. Thus, some water gets by the inlet. Water going over the grate may be capable of “splashing over” the grate, and usually little of the water outside the grate width is captured.

In order to determine the efficiency of a grate inlet, gutter flow is divided into two parts: frontal flow and side flow. Frontal flow is defined as that portion of the flow within the width of the grate. The portion of the flow outside the grate width is called side flow. By using Equation ST-1, the frontal flow can be evaluated and is expressed as:

$$Q_w = Q[1 - (1 - (W/T))]^{2.67} \quad (\text{ST-17})$$

in which:

$Q_w$  = frontal discharge (flow within width  $W$ ) (cfs)

$Q$  = total gutter flow (cfs) found using Equation ST-1

$W$  = width of grate (ft)

$T$  = total spread of water in the gutter (ft)

It should be noted that the grate width is generally equal to the depressed section in a composite gutter section. Now by definition:

$$Q_s = Q - Q_w \quad (\text{ST-18})$$

in which:

$Q_s$  = side discharge (i.e., flow outside the depressed gutter or grate) (cfs)

The ratio of the frontal flow intercepted by the inlet to total frontal flow,  $R_f$ , is expressed as:

$$R_f = Q_{wi} / Q_w = 1.0 - 0.09(V - V_o) \text{ for } V \geq V_o, \text{ otherwise } R_f = 1.0 \quad (\text{ST-19})$$

in which:

$Q_{wi}$  = frontal flow intercepted by the inlet (cfs)

$V$  = velocity of flow in the gutter (ft/sec)

$V_o$  = splash-over velocity (ft/sec)

The splash-over velocity is defined as the minimum velocity causing some water to shoot over the grate. This velocity is a function of the grate length and type. The splash-over velocity can be determined using the empirical formula (Guo 1999):

$$V_o = \alpha + \beta L_e - \gamma L_e^2 + \eta L_e^3 \quad (\text{ST-20})$$

in which:

$V_o$  = splash-over velocity (ft/sec)

$L_e$  = effective unit length of grate inlet (ft)

$\alpha, \beta, \gamma, \eta$  = constants from Table ST-6

**Table ST-6—Splash Velocity Constants for Various Types of Inlet Grates**

Type of Grate	$\alpha$	$\beta$	$\gamma$	$\eta$
Bar P-1-7/8	2.22	4.03	0.65	0.06
Bar P-1-1/8	1.76	3.12	0.45	0.03
Vane Grate	0.30	4.85	1.31	0.15
45-Degree Bar	0.99	2.64	0.36	0.03
Bar P-1-7/8-4	0.74	2.44	0.27	0.02
30-Degree Bar	0.51	2.34	0.20	0.01
Reticuline	0.28	2.28	0.18	0.01

The ratio of the side flow intercepted by the inlet to total side flow,  $R_s$ , is expressed as:

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \quad (\text{ST-21})$$

in which:

$V$  = velocity of flow in the gutter (ft/sec)

$L$  = length of grate (ft)

The capture efficiency,  $E$ , of the grate inlet may now be determined using:

$$E = R_f(Q_w/Q) + R_s(Q_s/Q) \quad (\text{ST-22})$$

Example 6.9 shows grate inlet capacity calculations.

### **3.3.2 Curb-Opening Inlets (On a Continuous Grade)**

The capture efficiency of a curb-opening inlet is dependent on the length of the opening, the depth of flow at the curb, street cross slope and the longitudinal gutter slope (see Photograph ST-3). If the curb opening is long, the flow rate is low, and the longitudinal gutter slope is small, all of the flow will be captured by the inlet. This is not normally the case during the minor (design) storm. In fact, it is generally uneconomical to install a curb opening long enough to capture all of the flow. Thus, some water gets by the inlet, and the inlet efficiency needs to be determined.



*Photograph ST-3—Gutter/street slope is a major design factor for both street and inlet capacity.*



The hydraulics of curb opening inlets are less complicated than grate inlets. The efficiency,  $E$ , of a curb-opening inlet is calculated as:

$$E = 1 - \left[1 - \left(L/L_T\right)\right]^{1.8} \text{ for } L < L_T, \text{ otherwise } E = 1.0 \quad (\text{ST-23})$$

in which:

$L$  = installed (or designed) curb-opening length (ft)

$L_T$  = curb-opening length required to capture 100% of gutter flow (ft)

and, for a curb-opening inlet that is not depressed,

$$L_T = 0.6 Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_x}\right)^{0.6} \quad (\text{ST-24})$$

in which:

$Q$  = gutter flow (cfs)

$S_L$  = longitudinal street slope (ft/ft)

$S_x$  = steel cross slope (ft/ft)

$n$  = Manning's roughness coefficient

For a depressed curb-opening inlet,

$$L_T = 0.6 Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_e}\right)^{0.6} \quad (\text{ST-25})$$

The equivalent cross slope,  $S_e$ , can be determined from

$$S_e = S_x + \frac{a}{W} E_o \quad (\text{ST-26})$$

in which  $a$  = gutter depression and  $W$  = depressed gutter section as shown in [Figure ST-1b](#). The ratio of the flow in the depressed section to total gutter flow,  $E_o$ , can be calculated from Equation ST-7. See Examples 6.8 and 6.9 for curb-opening inlet calculations.

### **3.3.3 Combination Inlets (On a Continuous Grade)**

Combination inlets take advantage of the debris removal capabilities of a curb-opening inlet and the capture efficiency of a grate inlet. If the grate and the curb opening are side-by-side and of approximately equal length, the interception capacity is found by assuming the grate acts alone. If all or part of the curb-opening inlet lies upstream from the grate (a desirable configuration), the inlet capacity is enhanced by

the upstream curb-opening capacity. The appropriate equations have already been presented, but Example 6.10 illustrates the procedure.

### **3.3.4 Slotted Inlets (On a Continuous Grade)**

Slotted inlets can generally be used to intercept sheet flow that is crossing the pavement in an undesirable location. Unlike grate inlets, they have the advantage of intercepting flow over a wide section. They do not interfere with traffic operations and can be used on both curbed and uncurbed sections. Like grate inlets, they are susceptible to clogging.

Slotted inlets function like a side-flow weir, much like curb-opening inlets. The FHWA (1996) suggests the hydraulic capacity of slotted inlets closely corresponds to curb-opening inlets if the slot openings exceed 1.75 inches. Therefore, the equations developed for curb-opening inlets (Equations ST-23 through ST-26) are appropriate for slotted inlets.

### **3.3.5 Inlets Located in Sumps**

All of the stormwater excess that enters a sump (i.e., a depression or low point in grade) must pass through an inlet to enter the stormwater conveyance system. If the stormwater is laden with debris, the inlet is susceptible to clogging. The ponding of water is a nuisance and could be hazardous. Therefore, the capacity of inlets in sumps must account for this clogging potential. Grate inlets acting alone are not recommended for this reason. Curb-opening inlets are more appropriate, as are combination inlets. Photograph ST-4 shows a curb opening inlet in a sump condition.



*Photograph ST-4—Inlets that are located in street sags and sumped can be highly efficient.*

As previously mentioned, inlets in sumps function like weirs for shallow depths, but as the depth of stormwater increases, they begin to function like an orifice. Orifice and weir flows have been exhaustively

studied. Equations are readily available to compute requisite flow rates. However, the transition from weir flow to orifice flow takes place over a relatively small range of depth that is not well defined. The FHWA provides guidance on the transition region based on significant testing.

The hydraulic capacity of grate, curb-opening, and slotted inlets operating as weirs is expressed as:

$$Q_i = C_w L_w d^{1.5} \quad (\text{ST-27})$$

in which:

$Q_i$  = inlet capacity (cfs)

$C_w$  = weir discharge coefficient

$L_w$  = weir length (ft)

$d$  = flow depth (ft)

Values for  $C_w$  and  $L_w$  are presented in Table ST-7 for various inlet types. Note that the expressions given for curb-opening inlets without depression should be used for depressed curb-opening inlets if  $L > 12$  feet.

The hydraulic capacity of grate, curb-opening, and slotted inlets operating as orifices is expressed as:

$$Q_i = C_o A_o (2gd)^{0.5} \quad (\text{ST-28})$$

in which:

$Q_i$  = inlet capacity (cfs)

$C_o$  = orifice discharge coefficient

$A_o$  = orifice area (ft<sup>2</sup>)

$d$  = characteristic depth (ft) as defined in Table ST-7

$g = 32.2 \text{ ft/sec}^2$

Values for  $C_o$  and  $A_o$  are presented in Table ST-7 for different types of inlets.

Combination inlets are commonly used in sumps. The hydraulic capacity of combination inlets in sumps depends on the type of flow and the relative lengths of the curb opening and grate. For weir flow, the capacity of a combination inlet (grate length equal to the curb opening length) is equal to the capacity of the grate portion only. This is because the curb opening does not add any length to the weir equation (Equation ST-27). If the curb opening is longer than the grate, the capacity of the additional curb length should be added to the grate capacity. For orifice flow, the capacity of the curb opening should be added to the capacity of the grate.

**Table ST-7—Sag Inlet Discharge Variables and Coefficients**

(Modified From Akan and Houghtalen 2002)

Inlet Type	$C_w$	$L_w^1$	Weir Equation Valid For	Definitions of Terms
Grate Inlet	3.00	$L + 2W$	$d < 1.79(A_o/L_w)$	$L$ = Length of grate $W$ = Width of grate $d$ = Depth of water over grate $A_o$ = Clear opening area <sup>2</sup>
Curb Opening Inlet	3.00	$L$	$d < h$	$L$ = Length of curb opening $h$ = Height of curb opening $d = d_i - (h/2)$ $d_i$ = Depth of water at curb opening
Depressed Curb Opening Inlet <sup>3</sup>	2.30	$L + 1.8W$	$d < (h + a)$	$W$ = Lateral width of depression $a$ = Depth of curb depression
Slotted Inlets	2.48	$L$	$d < 0.2$ ft	$L$ = Length of slot $d$ = Depth at curb
<sup>1</sup> The weir length should be reduced where clogging is expected. <sup>2</sup> Ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sag locations. <sup>3</sup> If $L > 12$ ft, use the expressions for curb opening inlets without depression.				
	$C_o$	$A_o^4$	Orifice Equation Valid for	Definition of Terms
Grate Inlet	0.67	Clear opening area <sup>5</sup>	$d > 1.79(A_o/L_w)$	$d$ = Depth of water over grate
Curb Opening Inlet (depressed or undepressed, horizontal orifice throat <sup>6</sup> )	0.67	$(h)(L)$	$d_i > 1.4h$	$d = d_i - (h/2)$ $d_i$ = Depth of water at curb opening $h$ = Height of curb opening
Slotted Inlet	0.80	$(L)(W)$	$d > 0.40$ ft	$L$ = Length of slot $W$ = Width of slot $d$ = Depth of water over slot
<sup>4</sup> The orifice area should be reduced where clogging is expected. <sup>5</sup> The ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sag locations. <sup>6</sup> See <a href="#">Figure ST-5</a> for other types of throats.				

**3.3.6 Inlet Clogging**

Inlets are subject to clogging effects (see Photographs ST-5 and ST-6). Selection of a clogging factor reflects the condition of debris and trash on the street. During a storm event, street inlets are usually

loaded with debris by the first-flush runoff volume. As a common practice for street drainage, 50% clogging is considered for the design of a single grate inlet and 10% clogging is considered for a single curb-opening inlet. Often, it takes multiple units to collect the stormwater on the street. Since the amount of debris is largely associated with the first-flush volume in a storm event, the clogging factor applied to a multiple-unit street inlet should be decreased with respect to the length of the inlet. Linearly applying a single-unit clogging factor to a multiple-unit inlet leads to an excessive increase in length. For instance, a six-unit inlet under a 50% clogging factor will function as a three-unit inlet. In fact, continuously applying a 50% reduction to the discharge on the street will always leave 50% of the residual flow on the street. This means that the inlet will never reach a 100% capture and leads to unnecessarily long inlets.



*Photograph ST-5—Clogging is an important consideration when designing inlets.*



*Photograph ST-6—Field inlets frequently need maintenance.*

With the concept of first-flush volume, the decay of clogging factor to curb opening length is described as (Guo 2000a):

$$C = \frac{1}{N}(C_o + eC_o + e^2C_o + e^3C_o + \dots + e^{N-1}C_o) = \frac{C_o}{N} \sum_{i=1}^{i=N} e^{i-1} = \frac{KC_o}{N} \quad (\text{ST-29})$$

in which:

$C$  = multiple-unit clogging factor for an inlet with multiple units

$C_o$  = single-unit clogging factor

$e$  = decay ratio less than unity, 0.5 for grate inlet, 0.25 for curb-opening inlet

$N$  = number of units

$K$  = clogging coefficient from Table ST-8

**Table ST-8—Clogging Coefficients to Convert Clogging Factor From Single to Multiple Units<sup>1</sup>**

$N =$	1	2	3	4	5	6	7	8	>8
Grate Inlet ( $K$ )	1	1.5	1.75	1.88	1.94	1.97	1.98	1.99	2
Curb Opening ( $K$ )	1	1.25	1.31	1.33	1.33	1.33	1.33	1.33	1.33

<sup>1</sup> This table is generated by Equation ST-29 with  $e = 0.5$  and  $e = 0.25$ .

When  $N$  becomes large, Equation ST-29 converges to:

$$C = \frac{C_o}{N(1 - e)} \quad (\text{ST-30})$$

For instance, when  $e = 0.5$  and  $C_o = 50\%$ ,  $C = 1.0/N$  for a large number of units,  $N$ . In other words, only the first unit out of  $N$  units will be clogged. Equation ST-30 complies with the recommended clogging factor for a single-unit inlet and decays on the clogging effect for a multiple-unit inlet.

The interception of an inlet on a grade is proportional to the inlet length, and in a sump is proportional to the inlet opening area. Therefore, a clogging factor shall be applied to the length of the inlet on a grade as:

$$L_e = (1 - C)L \quad (\text{ST-31})$$

in which  $L_e$  = effective (unclogged) length. Similarly, a clogging factor shall be applied to the opening area of an inlet in a sump as:

$$A_e = (1 - C)A \quad (\text{ST-32})$$

in which:

$A_e$  = effective opening area

$A$  = opening area

### **3.4 Inlet Location and Spacing on Continuous Grades**

#### **3.4.1 Introduction**

Locating (or positioning) stormwater inlets rarely requires design computations. They are simply required in certain locations based upon street design considerations, topography (sumps), and local ordinances. The one exception is the location and spacing of inlets on continuous grades. On a long, continuous grade, stormwater flow increases as it moves down the gutter and picks up more drainage area. As the flow increases, so does the spread. Since the spread (encroachment) is not allowed to exceed some specified maximum, inlets must be strategically placed to remove some of the stormwater from the street. Locating these inlets requires design computations by the engineer.

#### **3.4.2 Design Considerations**

The primary design consideration for the location and spacing of inlets on continuous grades is the spread limitation. This was addressed in Section 2.2. [Table ST-2](#) lists pavement encroachment standards for minor storms in the Denver metropolitan area.

Proper design of stormwater collection and conveyance systems makes optimum use of the conveyance capabilities of street gutters. In other words, an inlet is not needed until the spread reaches its allowable limit during the design (minor) storm. To place an inlet prior to that point on the street is not economically efficient. To place an inlet after that point would violate the encroachment standards. Therefore, the primary design objective is to position inlets along a continuous grade at the locations where the allowable spread is about to be exceeded for the design storm.

#### **3.4.3 Design Procedure**

Based on the encroachment standard and street geometry, the allowable street hydraulic capacity can be determined using Equation ST-11 or Equation ST-12. This flow rate is then equated to some hydrologic technique (equation) that contains drainage area. In this way, the inlet is positioned on the street so that it will service the requisite drainage area. The process of locating the inlet is accomplished by trial-and-error. If the inlet is moved downstream (or down gutter), the drainage area increases. If the inlet is moved upstream, the drainage area decreases.

The hydrologic technique most often used in urban drainage design is the Rational method. The Rational method was discussed in the RUNOFF chapter. The Rational equation, repeated here for convenience,

is:

$$Q = CIA \quad (ST-33)$$

in which:

$Q$  = peak discharge (cfs)

$C$  = runoff coefficient described in the RUNOFF chapter

$I$  = design storm rainfall intensity (in/hr) described in the RAINFALL chapter

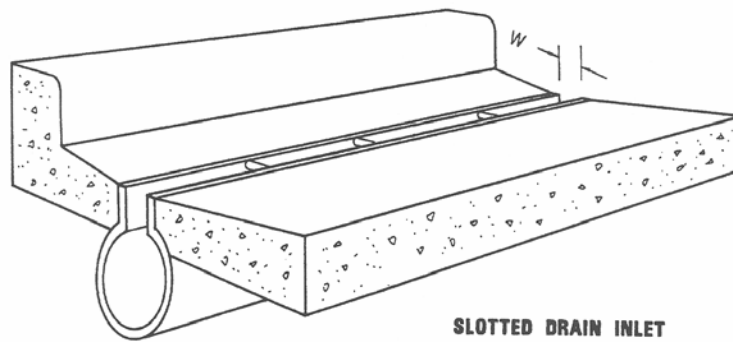
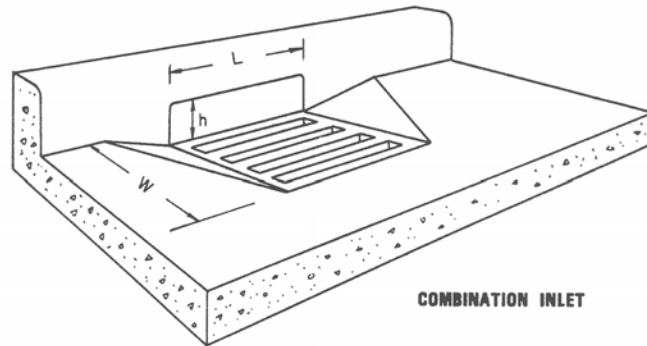
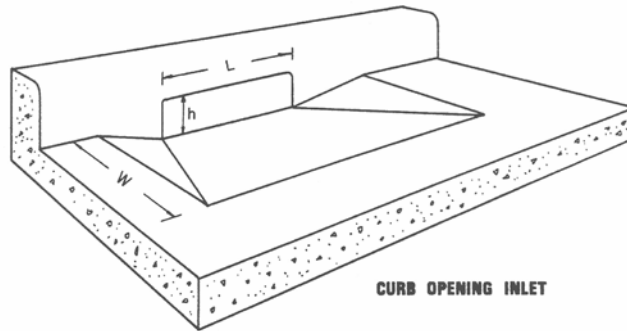
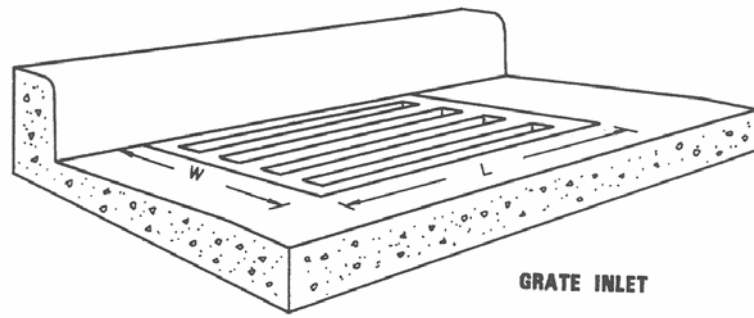
$A$  = drainage area (acres)

As previously mentioned, the peak discharge is found using the allowable spread and street geometry. The runoff coefficient is dependent on the land use as discussed in the RUNOFF chapter. The rainfall intensity is discussed in the RAINFALL chapter. The drainage area is the unknown variable to be solved.

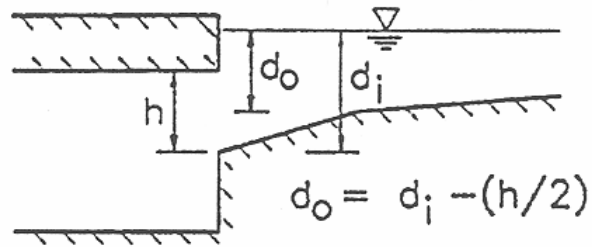
Once the first inlet is positioned along a continuous grade, an inlet type and size can be specified. The first inlet's hydraulic capacity is then assessed. Generally, the inlet will not capture all of the gutter flow. In fact, it is uneconomical to size an inlet (on continuous grades) large enough to capture all of the gutter flow. Instead, some carryover flow is expected. This practice reduces the amount of new flow that can be picked up at the next inlet. However, each inlet should be positioned at the location where the allowable spread is about to reach its allowable limit.

The gutter discharge for inlets, other than the first inlet, consists of the carryover from the upstream inlet plus the stormwater runoff generated from the intervening local drainage area. The carryover flow from the upstream inlet is added to the peak flow rate obtained from the Rational method for the intervening local drainage area. The resulting peak flow is approximate since the carryover flow peak and the local runoff peak do not necessarily coincide.

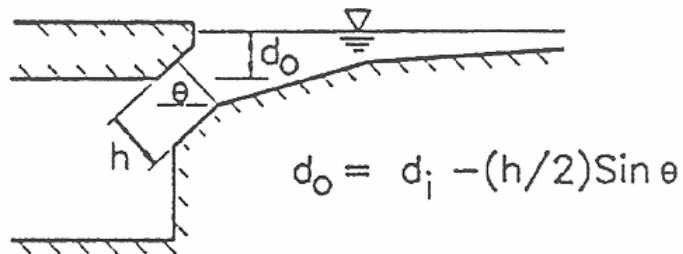




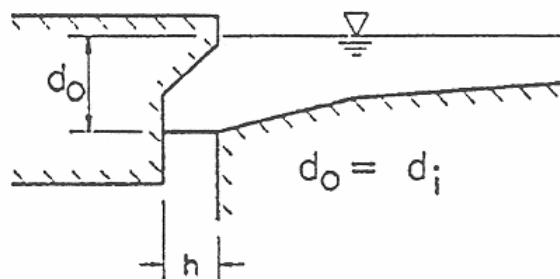
**Figure ST-4—Perspective Views of Grate and Curb-Opening Inlets**



a. Horizontal Throat



b. Inclined Throat



c. Vertical Throat

**Figure ST-5—Curb-Opening Inlets**

## 4.0 STORM SEWERS

### **4.1 Introduction**

Once stormwater is collected from the street surface by an inlet, it is directed into the storm sewer system. The storm sewer system is comprised of inlets, pipes, manholes, bends, outlets, and other appurtenances. The stormwater passes through these components and is discharged into a stormwater management device (e.g., infiltration trench, stormwater pond, constructed wetland, etc.) to mitigate adverse downstream effects or discharged directly to a natural or constructed watercourse. Stormwater management devices are constructed to reduce the peak discharge, decrease the volume of runoff, and/or improve the water quality.

Apart from inlets, manholes are the most common appurtenance in storm sewer systems. Their primary functions include:

- Providing maintenance access.
- Providing ventilation.
- Serving as junctions when two or more pipes merge.
- Providing flow transitions for changes in pipe size, slope, and alignment.

Manholes are generally made of pre-cast or cast-in-place reinforced concrete. They are typically 4 to 5 feet in diameter and are required at regular intervals, even in straight sections, for maintenance reasons. Standard size manholes cannot accommodate large pipes, so junction chambers are used for that application.

Other appurtenances are not as common as manholes, but serve vital functions. Occasionally, bends and transitions are accomplished without manholes, particularly for large pipe sizes. These sections provide gradual transitions in size or alignment to minimize energy losses. Outlet structures are transitions from pipe flow into open channel flow or still water (e.g., ponds, lakes, etc.). Their primary function is to minimize erosion in the receiving water body. Flow splitters separate incoming flow and send it in two or more directions. Flow deflectors are used to minimize energy losses in manholes, junction chambers, and flow splitters. Flap gates are placed on outlets to prevent backflow in areas subject to high tailwater or flood flow.

### **4.2 Design Process, Considerations, and Constraints**

The design of a storm sewer system requires a large data collection effort. The data requirements in the proposed service area include topography, drainage boundaries, soil types, and locations of any existing storm sewers, inlets, and manholes. In addition, identification of the type and location of other utilities is

necessary. Alternative layouts of a new system (or modifications to an existing system) can be investigated using this data.

Alternative system layouts rely largely on street right-of-ways and topography. Most layouts are dendritic (tree) networks that follow the street pattern. Dendritic networks collect stormwater from a broad area and tend to converge in the downstream direction. Looping networks shall be avoided because of their complex hydraulics and potentially higher cost. Each layout should contain inlet and manhole locations, drainage boundaries serviced by the inlets, storm sewer locations, flow directions, and outlet locations. A final layout selection is made from the viable alternatives based on likely system performance and cost.

Once a final layout is chosen, storm sewers are sized using hydrologic techniques (to determine peak flows) and hydraulic analysis (to determine pipe capacities). This is accomplished by designing the upstream pipes first and moving downstream. Pipes sizes smaller than 15 inches are not recommended for storm sewers. Pipes generally increase in size moving downstream since the drainage area is increasing. It is not good design practice to decrease the pipe size moving downstream, even if a steeper slope is encountered that will provide sufficient capacity with a smaller pipe. The potential for clogging is always a concern.

Storm sewers are typically sized to convey the minor storm without surcharging using normal flow techniques. In other words, the flow is in a pipe that is flowing *just full* determined by open channel hydraulics calculations.

The minor storm is defined by the return interval that usually varies from the 2-year to the 10-year storm depending on the importance of the infrastructure being served. Refer to the POLICY chapter for guidance regarding selection of the design storm.

Manholes are located in the system prior to and in conjunction with pipe design. Most manhole locations are dictated by proper design practices. For example, manholes are required whenever there is a change in pipe size, alignment, or slope. In addition, manholes are required at pipe junctions. Manholes are also required along straight sections of pipe for maintenance purposes. The distance between manholes is dependent on pipe size. The invert of a pipe leaving a manhole should be at least 0.1 foot lower than the incoming pipe to ensure positive low flows through the manhole. Whenever possible, match the crown of the pipe elevations when the downstream pipe is larger to minimize backwater effects on the upstream pipe.

Once storm sewers are sized and manhole locations are determined, the performance of the sewer system must be evaluated using energy grade line calculations starting at the downstream terminus of the system. As stormwater flows through the storm sewer system, it encounters many flow transitions. These transitions include changes in pipe size, slope, and alignment, as well as entrance and exit conditions. All of these transitions produce energy losses, usually expressed as head losses. These

losses must be accounted for to ensure that inlets and manholes do not surcharge to a significant degree (i.e., produce street flooding). This is accomplished using hydraulic grade line (HGL) calculations as a check on pipe sizes and system losses. If significant surcharging occurs, the pipe sizes should be increased. High tailwater conditions at the storm sewer outlet may also produce surcharging. This can also be accounted for using HGL calculations.

### **4.3 Storm Sewer Hydrology**

#### **4.3.1 Peak Runoff Prediction**

The Rational method is commonly used to determine the peak flows that storm sewers must be able to convey. It is an appropriate method due to the small drainage areas typically involved. It is also relatively easy to use and provides reasonable estimates of peak runoff. The total drainage area contributing flow to a particular storm sewer is often divided up into smaller subcatchments. The Rational method is described in the RUNOFF chapter of this *Manual*.

The first pipe in a storm sewer system is designed using Equation ST-33 to determine the peak flow. Downstream pipes receive flow from the upstream pipes as well as local inflows. The Rational equation applied to the downstream pipes is:

$$Q_p = I \sum_{j=1}^n C_j A_j \quad (\text{ST-34})$$

in which:

$I$  = design rainfall average intensity, over the time of concentration  $T_c$  (in/hr)

$n$  = number of subareas above the stormwater pipe

$C_j$  = runoff coefficient of subarea  $j$

$A_j$  = drainage area of subarea  $j$  (acres)

In using this equation, it is evident that the peak flow changes at each design point since the time of concentration, and thus the average intensity, changes at each design point. It is also evident that the time of concentration coming from the local inflow may differ from that coming from upstream pipes. Normally, the longest time of concentration is chosen for design purposes. If this is the case, all of the subareas above the design point will be included in Equation ST-34, and it usually produces the largest peak flow. On rare occasions, the peak flow from a shorter path may produce the greater peak discharge if the downstream areas are heavily developed. It is good practice to check all alternative flow paths and tributary areas to determine the tributary zone that produces the biggest design flow.

#### **4.4 Storm Sewer Hydraulics (Gravity Flow in Circular Conduits)**

##### **4.4.1 Flow Equations and Storm Sewer Sizing**

Storm sewer flow is usually unsteady and non-uniform. However, for design purposes it is assumed to be steady and uniform at the peak flow rate. Therefore, Manning's equation is appropriate, which can be stated as:

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2} \quad (\text{ST-35})$$

in which:

$Q$  = flow rate (cfs)

$n$  = Manning's roughness factor

$A$  = flow area (ft<sup>2</sup>)

$R$  = hydraulic radius (ft)

$S_f$  = friction slope (normally the storm sewer slope) (ft/ft)

For full flow in a circular storm sewer,

$$A = A_f = \frac{\pi D^2}{4} \quad (\text{ST-36})$$

$$R = R_f = \frac{D}{4} \quad (\text{ST-37})$$

in which:

$D$  = pipe diameter

$A_f$  = flow area at full flow (ft<sup>2</sup>)

$R_f$  = hydraulic radius at full flow (ft)

If the flow is pressurized (i.e., surcharging at the inlets or manholes is occurring),  $S_f \neq S_o$  where  $S_o$  is the longitudinal bottom slope of the storm sewer. Design of storm sewers assumes *just full flow*, a reference condition referring to steady, uniform flow with a flow depth,  $y$ , nearly equal to the pipe diameter,  $D$ . Just full flow discharge,  $Q_f$ , is calculated using:

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} S_o^{1/2} \quad (\text{ST-38})$$

Computations of flow characteristics for partial depths in circular pipes are tedious. Design aids like [Figure ST-6](#) are very helpful when this is necessary.

Storm sewers are sized to flow *just full* (i.e., as open channels using nearly the full capacity of the pipe). The design discharge is determined first using the Rational equation as previously discussed, then the Manning's equation is used (with  $S_f = S_o$ ) to determine the required pipe size. For circular pipes,

$$D_r = \left[ \frac{2.16nQ_p}{\sqrt{S_o}} \right]^{3/8} \quad (\text{ST-39})$$

in which  $D_r$  is the minimum size pipe required to convey the design flow and  $Q_p$  is peak design flow. However, the pipe diameter that should be used in the field is the next standard pipe size larger than  $D_r$ .

The typical process proceeds as follows. Initial storm sewer sizing is performed first using the Rational equation in conjunction with Manning's equation. The Rational equation is used to determine the peak discharge that storm sewers must convey. The storm sewers are then initially sized using Manning's equation assuming uniform, steady flow at the peak. Finally, these initial pipe sizes are checked using the energy equation by accounting for all head losses. If the energy computations detect surcharging at manholes or inlets, the pipe sizes are increased.

#### **4.4.2 Energy Grade Line and Head Losses**

Head losses must be accounted for in the design of storm sewers in order to find the energy grade line (EGL) and the hydraulic grade line (HGL) at any point in the system. The FHWA (1996) gives the following equation as the basis for calculating the head losses at inlets, manholes, and junctions ( $h_{LM}$ , in feet):

$$h_{LM} = K_o C_D C_d C_Q C_p C_B \left( \frac{V_o^2}{2g} \right)$$

in which:

$K_o$  = initial loss coefficient

$V_o$  = velocity in the outflow pipe (ft/sec)

$g$  = gravitational acceleration (32.2 ft/sec<sup>2</sup>)

$C_D$ ,  $C_d$ ,  $C_Q$ ,  $C_p$ , and  $C_B$  = correction factors for pipe size, flow depth, relative flow, plunging flow and benching

However, this equation is valid only if the water level in the receiving inlet, junction, or manhole is above the invert of the incoming pipe. Otherwise, another protocol has to be used to calculate head losses at

manholes. What follows is a modified FHWA procedure that the engineer can use to calculate the head losses and the EGL along any point in a storm sewer system.

The EGL represents the energy slope between the two adjacent manholes in a storm sewer system. A manhole may have multiple incoming sewers, but only one outgoing sewer. Each sewer and its downstream and upstream manholes form a *sewer-manhole* unit. The entire storm sewer system can be decomposed into a series of sewer-manhole units that satisfy the energy conservation principle. The computation of the EGL does this by repeating the energy-balancing process for each sewer-manhole unit.

As illustrated in [Figure ST-6](#), a *sewer-manhole* unit has four distinctive sections. Section 1 represents the downstream manhole, Section 2 is the point at the exit of the incoming sewer just as enters this manhole, Section 3 is at the entrance to this sewer at the upstream manhole, and Section 4 represents the upstream manhole. For each *sewer-manhole* unit, the head losses are determined separately in two parts as:

- Friction losses through the sewer pipe, and
- Juncture losses at the manhole.

The discussion that follows explains how to apply energy balancing to calculate the EGL through each *sewer-manhole* unit.

#### 4.4.2.1 Losses at the Downstream Manhole—Section 1 to Section 2

The continuity of the EGL is determined between the flow conditions at centerline of the downstream manhole, Section 1, and the exit of the incoming sewer, Section 2, as illustrated in [Figure ST-6](#) and an idealized EGL and HGL profiles in [Figure ST-7](#).

At Section 2 there may be pipe-full flow, critical/supercritical open channel flow, or sub-critical open channel flow. If the sewer crown at the exit is submerged, the EGL at the downstream manhole provides a tailwater condition; otherwise, the manhole drop can create a discontinuity in the EGL. Therefore, it is necessary to evaluate the two possibilities, namely:

$$E_2 = \text{Max}\left(\frac{V_2^2}{2g} + Y_x + Z_2, E_1\right) \quad (\text{ST-40})$$

in which:

$E_2$  = EGL at Section 2

$V_2$  = sewer exit velocity in fps

$Y_2$  = flow depth in feet at the sewer exit



$Z_2$  = invert elevation in feet at the sewer exit

$E_1$  = tailwater at Section 1

Equation ST-40 states that the highest EGL value shall be considered as the downstream condition. If the manhole drop dictates the flow condition at Section 2, a discontinuity is introduced into the EGL.

#### **4.4.2.2 Losses in the Pipe, Section 2 to Section 3.**

The continuity of the EGL upstream of the manhole depends on the friction losses through the sewer pipe. The flow in the sewer pipe can be one condition or a combination of open channel flow, full flow, or pressurized (surcharge) flow.

When a free surface exists through the pipe length, the open channel hydraulics apply to the backwater surface profile computations. The friction losses through the sewer pipe are the primary head losses for the type of water surface profile in the sewer. For instance, the sewer pipe carrying a subcritical flow may have an M-1 water surface profile if the water depth at the downstream manhole is greater than normal depth in the sewer or an M-2 water surface profile if the water depth in the downstream manhole is lower than normal depth. Under an alternate condition, the pipe carrying a supercritical flow may have an S-2 water surface profile if the pipe entering the downstream manhole is not submerged; otherwise, a hydraulic jump is possible within the sewer.

When the downstream sewer crown is submerged to a degree that the entire sewer pipe is under the HGL, the head loss for this full flow condition is estimated by pressure flow hydraulics.

When the downstream sewer crown is slightly submerged, the downstream end of the sewer pipe is surcharged, but the upstream end of the sewer pipe can have open channel flow. The head loss through a surcharge flow depends on the flow regime. For a subcritical flow, the head loss is the sum of the friction losses for the full flow condition and for the open channel flow condition. For a supercritical flow, the head loss may involve a hydraulic jump. To resolve which condition governs, culvert hydraulic principles can be used under both inlet and outlet control conditions and the governing condition is the one that produces the highest HGL at the upstream manhole.

Having identified the type of flow in the sewer pipe, the computation of friction losses begins with the determination of friction slope. The friction loss and energy balance are calculated as:

$$h_f = LS_f \quad (\text{ST-41})$$

$$E_3 = E_2 + \sum h_f \quad (\text{ST-42})$$

in which:

$h_f$  = friction loss

$L$  = length in feet of sewer pipe

$S_f$  = friction slope in the pipe in ft/ft

$E_3$  = EGL at the upstream end of sewer pipe

#### 4.4.2.3 Losses at the Upstream Manhole, Section 3 to Section 4

Additional losses may be introduced at the sewer entrance. The general formula to estimate the entrance loss is:

$$h_E = K_E \frac{V^2}{2g} \quad (\text{ST-43})$$

in which:

$h_E$  = entrance loss in feet

$V$  = pipe-full velocity in feet per second in the incoming sewer

$K_E$  = entrance loss coefficient between 0.2 to 0.5

In the modeling of sewer flow, the sewer entrance coefficients can be assumed to be part of the bend loss coefficient.

The energy principle between Sections 3 and 4 is determined by:

$$E_4 = E_3 + h_E \quad (\text{ST-44})$$

in which  $E_4$  = EGL at Section 4.

#### 4.4.2.4 Juncture and Bend Losses at the Upstream Manhole, Section 4 to Section 1

The analysis from Section 4 of the downstream *sewer-manhole* unit to Section 1 of the upstream *sewer-manhole* unit consists only of juncture losses through the manhole. To maintain the conservation of energy through the manhole, the outgoing energy plus the energy losses at the manhole have to equal the incoming energy. Often a manhole is installed for the purpose of maintenance, deflection of the sewer line, change of the pipe size, and as a juncture for incoming laterals. Although there are different causes for juncture losses, they are often, rightly or wrongly, considered as a minor loss in the computation of the EGL. These juncture losses in the sewer system are determined solely by the local configuration and geometry and not by the length of flow in the manhole.

##### 4.4.2.4.1 Bend/Deflection Losses

The angle between the incoming sewer line and the centerline of the exiting main sewer line introduces a bend loss to the incoming sewer. Bend loss is estimated by:

$$h_b = K_b \frac{V^2}{2g} \quad (\text{ST-45})$$

in which:

$h_b$  = bend loss in feet

$V$  = full flow velocity in feet per second in the incoming sewer

$K_b$  = bend loss coefficient

As shown in [Figure ST-8](#) and Table ST-9, the value of  $K_b$  depends on the angle between the exiting sewer line and the existence of manhole bottom shaping. A shaped manhole bottom or a deflector guides the flow and reduces bend loss. [Figure ST-9](#) illustrates four cross-section options for the shaping of a manhole bottom. Only sections “c. Half” and “d. Full” can be considered for the purpose of using the bend loss coefficient for the curve on [Figure ST-9](#) labeled as “Bend at Manhole, Curved or Shaped.”

Because a manhole may have multiple incoming sewers, Equation ST-45 shall be applied to each incoming sewer based on its incoming angle, and then the energy principle between Sections 4 and 1 is calculated as:

$$E_1 = E_4 + h_b \quad (\text{ST-46})$$

#### 4.4.2.4.2 Lateral Juncture Losses

In addition to the bend loss, the lateral juncture loss is also introduced because of the added turbulence and eddies from the lateral incoming flows. The lateral juncture loss is estimated as:

$$h_j = \frac{V_o^2}{2g} - K_j \frac{V_i^2}{2g} \quad (\text{ST-47})$$

in which:

$h_j$  = lateral loss in feet

$V_o$  = full flow velocity in feet per second in the outgoing sewer

$K_j$  = lateral loss coefficient

$V_i$  = full flow velocity in feet per second in the incoming sewer

In modeling, a manhole can have multiple incoming sewers, one of which is the main (i.e., trunk) line, and one outgoing sewer. As shown in Table ST-9, the value of  $K_j$  is determined by the angle between the lateral incoming sewer line and the outgoing sewer line.

**Table ST-9—Bend Loss and Lateral Loss Coefficients (FHWA 1996)**

<b>Angle in Degree</b>	<b>Bend Loss Coefficient for Curved Deflector in the Manhole</b>	<b>Bend Loss Coefficient for Non-shaping Manhole</b>	<b>Lateral Loss Coefficient on Main Line Sewer</b>
Straight Through	0.05	0.05	Not Applicable
22.50	0.10	0.13	0.75
45.00	0.28	0.38	0.50
60.00	0.48	0.63	0.35
90.00	1.01	1.32	0.25

At a manhole, the engineer needs to identify the main incoming sewer line (the one that has the largest inflow rate) and determine the value of  $K_j$  for each lateral incoming sewer line. To be conservative, the smallest  $K_j$  is recommended for Equation ST-44, and the lateral loss is to be added to the outfall of the incoming main line sewer as:

$$E_1 = E_4 + h_b + h_j \quad (h_j \text{ is applied to main sewer line only}) \quad (\text{ST-48})$$

The difference between the EGL and the HGL is the flow velocity head. The HGL at a manhole is calculated by:

$$H_1 = E_1 - \frac{V_o^2}{2g} \quad (\text{ST-49})$$

The energy loss between two manholes is defined as:

$$\Delta E = (E_1)_{upstream} - (E_1)_{downstream} \quad (\text{ST-50})$$

in which  $\Delta E$  = energy loss between two manholes. It is noted that  $\Delta E$  includes the friction loss, juncture loss, bend loss, and manhole drop.

#### 4.4.2.5 Transitions

In addition to *sewer-manhole* unit losses, head losses in a storm sewer can occur due to a transition in the pipe itself, namely, gradual pipe expansion. Transition loss,  $h_{LE}$ , in feet, can be determined using:

$$h_{LE} = K_e \left( \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \quad (\text{ST-51})$$

in which  $K_e$  is the expansion coefficient and subscripts 1 and 2 refer to upstream and downstream of the transition, respectively. The value of the expansion coefficient,  $K_e$ , may be taken from Table ST-10 for

free surface flow conditions in which the angle of cone refers to the angle between the sides of the tapering section (see [Figure ST-10](#)).

**Table ST-10—Head Loss Expansion Coefficients in Non-Pressure Flow (FHWA 1996)**

$D_2/D_1$	Angle of Cone						
	10°	20°	30°	40°	50°	60°	70°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	.86	1.02	1.06	1.04	1.00

Head losses due to gradual pipe contraction,  $h_{LC}$ , in feet, are determined using:

$$h_{LC} = K_c \left( \frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \quad (\text{ST-52})$$

in which  $K_c$  = contraction coefficient. Typically,  $K_c = 0.5$  provides reasonable results.

This *Manual* does not recommend pipe contractions for storm sewers.

#### 4.4.2.6 Curved Sewers

Head losses due to curved sewers (sometimes called radius pipe),  $h_{Lr}$ , in feet, can be determined using:

$$h_{Lr} = K_r \frac{V^2}{2g} \quad (\text{ST-53})$$

in which  $K_r$  = curved sewer coefficient from [Figure ST-8](#).

#### 4.4.2.7 Losses at Storm Sewer Exit

Head losses at storm sewer outlets,  $h_{LO}$ , are determined using:

$$h_{LO} = \frac{V_o^2}{2g} - \frac{V_d^2}{2g} \quad (\text{ST-54})$$

in which  $V_o$  is the velocity in the outlet pipe, and  $V_d$  is the velocity in the downstream channel. When the storm sewer discharges into a reservoir or into air because there is no downstream channel,  $V_d = 0$  and one full velocity head is lost at the exit.

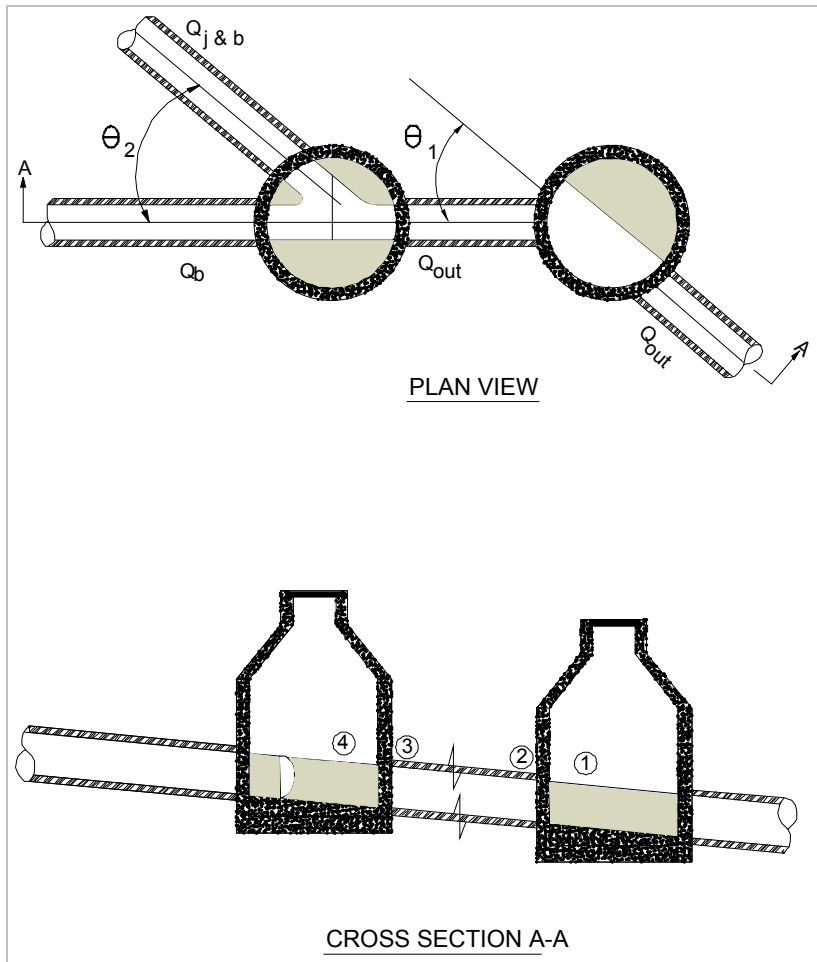


Figure ST-6—A Manhole-Sewer Unit

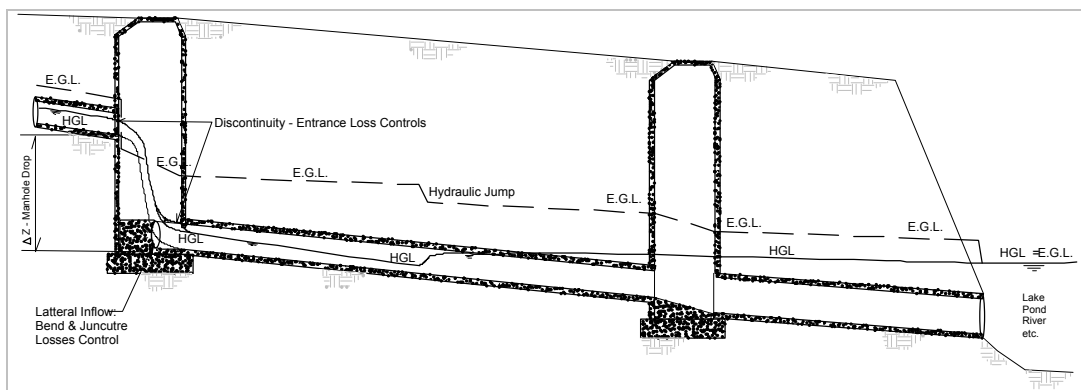


Figure ST-7—Hydraulic and Energy Grade Lines

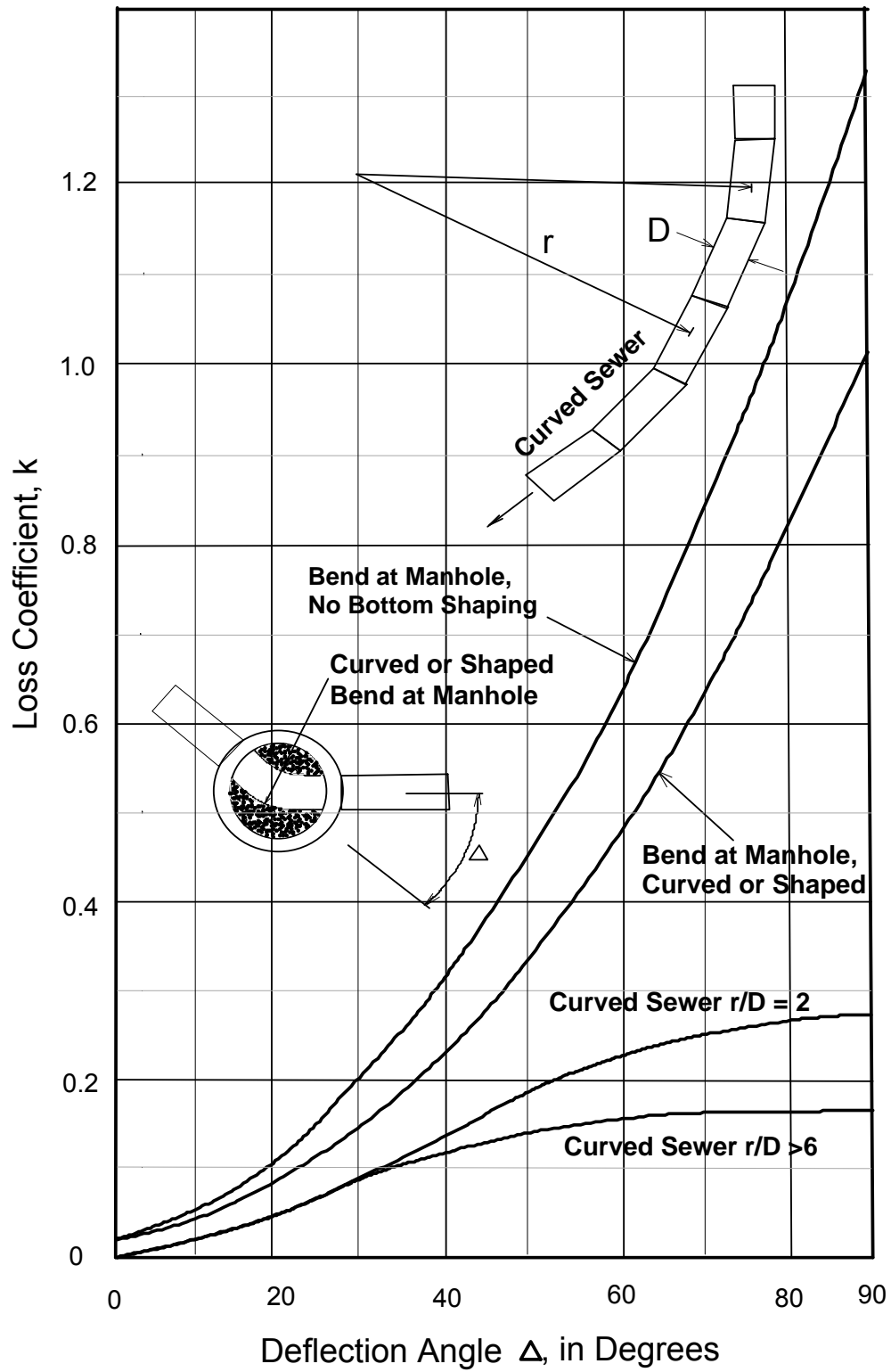


Figure ST-8—Bend Loss Coefficients

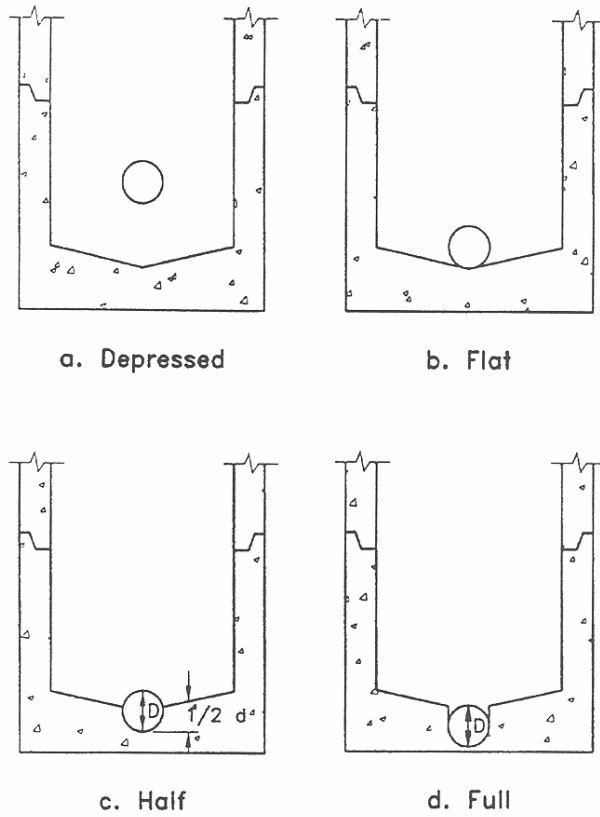


Figure ST-9—Access Hole Benching Methods

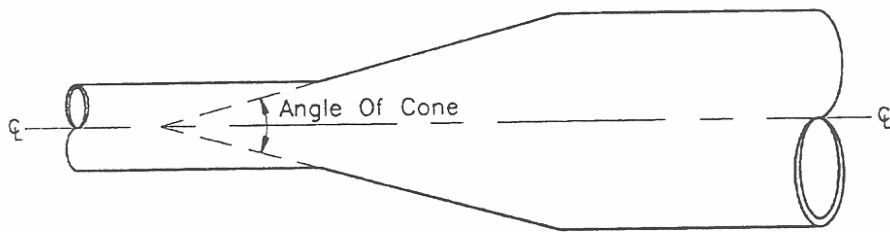


Figure ST-10—Angle of Cone for Pipe Diameter Changes



## 5.0 SPREADSHEETS

The [UD-Inlet Spreadsheet](#) provides quick solutions for many of the computations described in this chapter. A brief summary of each worksheet of the spreadsheet is provided below. Please note that some of the symbols and nomenclature in the worksheet do not correspond exactly with the nomenclature of the text. The text and the spreadsheets are computationally equivalent.

1. The **Q-Major Worksheet** calculates the gutter capacity for major storm events.
2. The **Q-Minor Worksheet** calculates the gutter capacity for minor storm events.
3. The **Flow Worksheet** provides Rational method hydrologic computations for streets and inlets.
4. The **Street Hy Worksheet** calculates gutter conveyance capacity ***and must be used in conjunction with any of the inlet capacity worksheets.***
5. The **Grate-G Worksheet** calculates the capacity of grate inlets on a grade.
6. The **Curb-G Worksheet** calculates the capacity of curb opening inlets on a grade.
7. The **Slot-G Worksheet** calculates the capacity of slotted inlets on a grade.
8. The **Grate G Worksheet** calculates the capacity of grate inlets in a sump.
9. The **Curb-G Worksheet** calculates the capacity of curb opening inlets in a sump.
10. The **Slot-G Worksheet** calculates the capacity of slotted inlets in a sump.

## 6.0 EXAMPLES

### 6.1 Example—Triangular Gutter Capacity

A triangular gutter has a longitudinal slope of  $S_L = 0.01$ , cross slope of  $S_x = 0.02$ , and a curb depth of 6 inches. Determine the flow rate and flow depth if the spread is limited to 9 feet.

Using Equation ST-1,

$$Q = [(0.56)(0.02)^{5/3}(0.01)^{1/2}(9.0)^{8/3}]/(0.016) = 1.81 \text{ cfs}$$

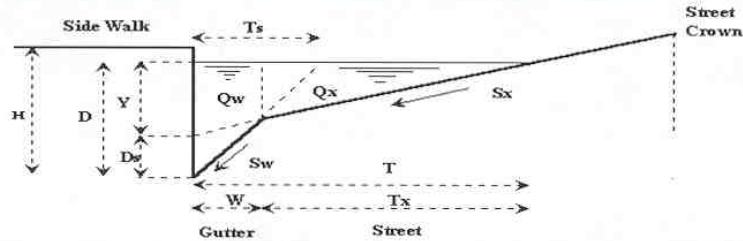
This is the theoretical flow rate. Then by using Equation ST-2,

$$y = (9.0)(0.02) = 0.18 \text{ ft}$$

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet). If it was not, the spread and associated flow rate would need to be reduced. A solution of this example using the **Q-Minor Worksheet** of the [UD-Inlet Spreadsheet](#) is included below.

**Gutter Storm Water Conveyance Capacity for Minor Event**

Project: **Example 6.1 - Triangular Gutter Capacity**  
 Street ID: \_\_\_\_\_



<b>Gutter Geometry</b>	
Curb Height	H = 6.00 inches
Gutter Width	W = 2.00 ft
Gutter Depression	Ds = 0.00 inches
Street Transverse Slope	Sx = 0.0200 ft/ft
Street Longitudinal Slope	So = 0.0100 ft/ft
Manning's Roughness	N = 0.016
Maximum Allowable Water Spread for Minor Event	T = 9.00 ft
<b>Gutter Conveyance Capacity Based On Maximum Water Spread</b>	
Gutter Cross Slope:	Sw = 0.0200 ft/ft
Water Depth without Gutter Depression	Y = 0.18 ft
Water Depth with a Gutter Depression	D = 0.18 ft
Spread for Side Flow on the Street	Tx = 7.00 ft
Spread for Gutter Flow along Gutter Slope	Ts = 9.00 ft
Flow Rate Carried by Width Ts	Qws = 1.8 cfs
Flow Rate Carried by Width (Ts - W)	Qww = 0.9 cfs
Gutter Flow	Qw = 0.9 cfs
Side Flow	Qx = 0.9 cfs
<b>Maximum Spread Capacity</b>	<b>Q-Tm = 1.8 cfs</b>
<b>Gutter Full Conveyance Capacity Based on Curb Height</b>	
Spread for Side Flow on the Street	Tx = 25.00 ft
Spread for Gutter Flow along Gutter Slope	Ts = 25.00 ft
Flow Rate Carried by Width Ts	Qws = 27.5 cfs
Flow Rate Carried by Width (Ts - W)	Qww = 22.0 cfs
Gutter Flow	Qw = 5.5 cfs
Side Flow	Qx = 27.5 cfs
<b>Gutter Full Capacity</b>	<b>Q-full = 33.0 cfs</b>
<b>Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)</b>	
Reduction Factor for Minor Event	R-min = 1.00
<b>Gutter Design Conveyance Capacity for Minor Event</b>	<b>Q-min = 1.8 cfs</b>

## 6.2 Example—Composite Gutter Capacity

Determine the discharge in a composite gutter section if the allowable spread is 9.0 feet, the gutter width,  $W$ , is 2 feet, and the gutter depression is 1.5 inches. The street's longitudinal slope is 0.01, the cross slope is 0.02, and the curb height is 6 inches.

Equation ST-8 yields the cross slope of the depressed gutter as:

$$S_w = 0.02 + (1.5/12)/2 = 0.083$$

Using [Figure ST-1a](#),  $W = 2$  feet,  $T_s = 7$  feet. Equation ST-1 can now be used to find the flow in the street section.

$$Q_s = [(0.56)(0.02)^{5/3}(0.01)^{1/2}(7.0)^{8/3}]/(0.016) = 0.92 \text{ cfs}$$

Now with  $S_w/S_x = 0.083/0.02 = 4.1$ ,  $T/W = 9.0/2.0 = 4.5$ , and  $T/W - 1 = 3.5$ , by using Equation ST-7,

$$E_o = \frac{1}{\left\{ 1 + \frac{4.1}{\left[ 1 + \frac{4.1}{3.5} \right]^{8/3} - 1.0} \right\}} = 0.63$$

Now the theoretical flow rate can be found using Equation ST-6 as:

$$Q = [(0.92)/(1 - 0.63)] = 2.49 \text{ cfs}$$

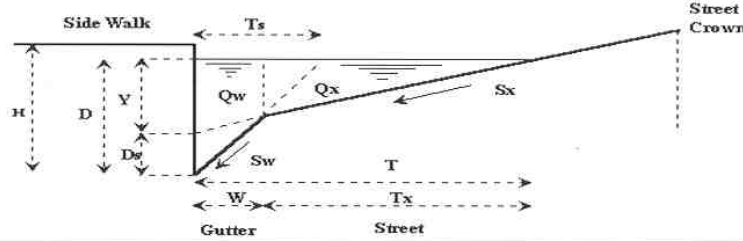
Then by using Equation ST-9,

$$y = 2/12 + (9.0)(0.02) = 0.35 \text{ feet}$$

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet). Also note that this is the same gutter section as Example 6.1 except for the depressed gutter section. This change has increased the gutter capacity by 38% and almost doubled the depth of flow. A spreadsheet solution of this example problem using the **Q-Minor Worksheet** of the [UD-Inlet Spreadsheet](#) is included below.

**Gutter Storm Water Conveyance Capacity for Minor Event**

Project: **Example 6.2 - Composite Gutter Capacity**  
 Street ID: \_\_\_\_\_



<u>Gutter Geometry</u>	
Curb Height	H = 6.00 inches
Gutter Width	W = 2.00 ft
Gutter Depression	Ds = 1.50 inches
Street Transverse Slope	Sx = 0.0200 ft/ft
Street Longitudinal Slope	So = 0.0100 ft/ft
Manning's Roughness	N = 0.016
Maximum Allowable Water Spread for Minor Event	T = 9.00 ft
<u>Gutter Conveyance Capacity Based On Maximum Water Spread</u>	
Gutter Cross Slope:	Sw = 0.0825 ft/ft
Water Depth without Gutter Depression	Y = 0.18 ft
Water Depth with a Gutter Depression	D = 0.31 ft
Spread for Side Flow on the Street	Tx = 7.00 ft
Spread for Gutter Flow along Gutter Slope	Ts = 3.70 ft
Flow Rate Carried by Width Ts	Qws = 1.8 cfs
Flow Rate Carried by Width (Ts - W)	Qww = 0.2 cfs
Gutter Flow	Qw = 1.6 cfs
Side Flow	Qx = 0.9 cfs
<b>Maximum Spread Capacity</b>	<b>Q-Tm = 2.5 cfs</b>
<u>Gutter Full Conveyance Capacity Based on Curb Height</u>	
Spread for Side Flow on the Street	Tx = 18.75 ft
Spread for Gutter Flow along Gutter Slope	Ts = 6.06 ft
Flow Rate Carried by Width Ts	Qws = 6.7 cfs
Flow Rate Carried by Width (Ts - W)	Qww = 2.3 cfs
Gutter Flow	Qw = 4.4 cfs
Side Flow	Qx = 12.8 cfs
<b>Gutter Full Capacity</b>	<b>Q-full = 17.1 cfs</b>
<u>Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)</u>	
Reduction Factor for Minor Event	R-min = 1.00
<b>Gutter Design Conveyance Capacity for Minor Event</b>	<b>Q-min = 2.5 cfs</b>

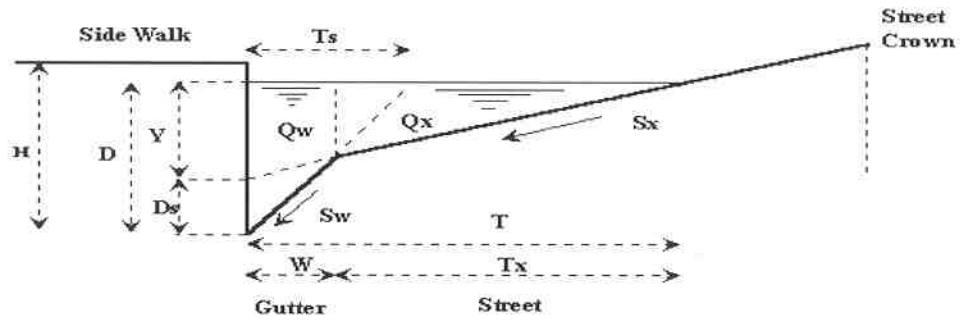
### **6.3 Example—Composite Gutter Spread**

A composite gutter section has  $S_x = 0.02$ ,  $S_L = 0.01$ ,  $a = 2$  inches,  $n = 0.016$  and  $W = 2$  feet. Determine the spread,  $T$ , at  $Q = 2.5$  cfs (Akan and Houghtalen 2002).

Solving this problem by using Equations ST-6 and ST-7 requires a trial-and-error procedure since the equations are implicit in  $T$ . In the trial-and-error procedure, the value of  $T$  is guessed, and  $Q$  is calculated using Equations ST-6 and ST-7. If the calculated  $Q$  is the same as the given  $Q$ , the guessed value of  $T$  is correct. Otherwise, the procedure is repeated using another guess for  $T$ . In this case, a guessed value of a spread equal to 8.5 feet yields the correct flow of 2.5 cfs. A direct solution is possible by using the **Street Hy Worksheet** of the [UD-Inlet Spreadsheet](#).

## GUTTER CONVEYANCE CAPACITY

Project = Example 6.3 - Composite Gutter Spread  
 Street ID = \_\_\_\_\_



Street Geometry (Input)	
Design Discharge in the Gutter	Qo = 2.5 cfs
Curb Height	H = 6.00 inches
Gutter Width	W = 2.00 ft
Gutter Depression	Ds = 2.00 inches
Street Transverse Slope	Sx = 0.0200 ft/ft
Street Longitudinal Slope	So = 0.0100 ft/ft
Manning's Roughness	N = 0.016
Gutter Conveyance Capacity	
Gutter Cross Slope	Sw = 0.10 ft/ft
Water Spread Width	T = 8.58 ft
Water Depth without Gutter Depression	Y = 0.17 ft
Water Depth with a Gutter Depression	D = 0.34 ft
Spread for Side Flow on the Street	Tx = 6.58 ft
Spread for Gutter Flow along Gutter Slope	Ts = 3.27 ft
Flowrate Carried by Width Ts	Qws = 1.9 cfs
Flowrate Carried by Width (Ts - W)	Qww = 0.2 cfs
Gutter Flow	Qw = 1.7 cfs
Side Flow	Qx = 0.8 cfs
<b>Total Flow (Check against Qo)</b>	<b>Qs = 2.5 cfs</b>
Gutter Flow to Design Flow Ratio	Eo = 0.69
Equivalent Slope for the Street	Se = 0.08 ft/ft
Flow Area	As = 0.90 sq ft
Flow Velocity	Vs = 2.77 fps
<b>VsD product</b>	<b>VsD = 0.94 ft<sup>2</sup>/s</b>

#### **6.4 Example—V-Shaped Swale Capacity**

Determine the maximum discharge and depth of flow in a V-shaped, roadside swale with the following characteristics:  $S_{x1} = 0.08$ ,  $S_{x2} = 0.06$ ,  $n = 0.016$ ,  $S_L = 0.02$ , and  $T = 6$  feet.

Equations ST-13, ST-1, and ST-3 are used to determine the adjusted slope, the flow, and the flow depth.

$$S_x = (0.08)(0.06)/(0.08 + 0.06) = 0.034$$

$$Q = [(0.56)(0.034)^{5/3}(0.02)^{1/2}(6.0)^{8/3}]/(0.016) = 2.09 \text{ cfs}$$

$$y = (0.034)(6.0) = 0.20 \text{ feet}$$

#### **6.5 Example—V-Shaped Swale Design**

Design a V-shaped swale to convey a flow of 1.8 cfs. The available swale top width is 8 feet, the longitudinal slope is 0.01, and the Manning's roughness factor is 0.016. Determine the cross slopes and the depth of the swale.

Solving Equation ST-1 for  $S_x$  (i.e., average side slope) yields:

$$S_x = [(1.8)(0.016)/(0.56)(0.01)^{1/2}(8.0)^{8/3}]^{3/5} = 0.024$$

Now Equation ST-13 is used to solve for the actual cross slope if  $S_{x1} = S_{x2}$ . Then,

$$0.024 = (S_{x1})^2/2S_{x1} = S_{x1}/2, \text{ and } S_{x1} = 0.048$$

Then using Equation ST-2 yields

$$y = (0.024)(8.0) = 0.19 \text{ ft}$$

The swale is 8-feet wide with right and left side slopes of 0.048 ft/ft.

#### **6.6 Example—Major Storm Street Capacity**

Determine the flow capacity of an arterial street during the major storm if the street is 60-foot wide (gutter to gutter) with a cross slope of 0.025 ft/ft, a curb height of 6 inches, and a longitudinal slope of 0.03. A 12-foot-wide sidewalk is adjacent to the curb. Flow capacity beyond the sidewalk cannot be relied upon because buildings often abut the sidewalk in this commercial district.

[Table ST-3](#) shows the limitations on the stormwater depth during the major storm (100-year) event. The depth cannot exceed the crown elevation, nor can it exceed 12 inches over the gutter flow line. If the flow depth was at the street crown elevation, the corresponding depth of flow at the curb would be  $(0.025)(30) = 0.75$  feet. Therefore, assume that the crown elevation controls the flood depth (i.e., the entry level into



the buildings will assumed to be high enough not to control the flood depth).

Since the street cross section is symmetric, determine the capacity on one side of the street crown and multiply by 2 to get the total capacity and break the flow section up into prismatic shapes. Flow occurs in a triangular section in the street and a rectangular section above the sidewalk (at a depth of  $0.75 - 0.5 = 0.25$  ft). The street section has a Manning's value of 0.016, and the sidewalk has a value of 0.013. The triangular flow area of the street is  $(1/2)(30)(0.75) = 11.25$  ft<sup>2</sup> and a wetted perimeter of approximately  $30 + 0.5 = 30.5$  feet (assuming the slope length is roughly equal to the width plus the curb height). The sidewalk section has a flow area of  $(12)(0.25) = 3.00$  ft<sup>2</sup> and a wetted perimeter of 12 feet (ignoring the vertical sides of buildings). Thus, Equation ST-14 yields

$$Q = (1.49/0.016)(11.25)(11.25/30.5)^{2/3}(0.03)^{1/2} = 93.3 \text{ cfs (street section)}$$

$$Q = (1.49/0.013)(3.0)(3.0/12.0)^{2/3}(0.03)^{1/2} = 23.6 \text{ cfs (sidewalk section)}$$

$$Q = 2(93.3 + 23.6) = 234 \text{ cfs (total flow capacity of the section)}$$

Oftentimes, the 100-year flow rate will be available and the flow depth will need to be determined, or the flow cross section will not be prismatic. Fortunately, proprietary software is available to perform normal depth computations (i.e., Manning's depth) for irregular cross sections, rendering these problems trivial.

### **6.7 Example—Grate Inlet Capacity**

Determine the efficiency of a curved vane grate ( $W = 2$  feet and  $L = 2$  feet) when placed in a composite gutter with the following characteristics:  $S_x = 0.02$ ,  $S_L = 0.01$ ,  $a = 0.167$  feet, and  $n = 0.016$ . The flow rate in the gutter is 2.5 cfs with a spread of 8.5 feet. Note: The depressed section of the composite gutter has a width equal to the width of the grate (Akan and Houghtalen 2002).

Find the gutter slope using Equation ST-8:

$$S_w = 0.02 + \frac{0.167}{2} = 0.1033$$

By using Equation ST-7:

$$E_o = \frac{1}{1 + \left[ \frac{0.1033/0.02}{\left[ 1 + \frac{(0.1033/0.02)}{(8.5/2) - 1} \right]^{2.67}} - 1 \right]} = 0.69$$

The side flow  $Q_s$  is calculated using Equation ST-6:

$$Q_s = 2.5(1-0.69) = 0.77 \text{ cfs}$$

The frontal flow  $Q_w$  is calculated using Equation ST-5:

$$Q_w = 2.5 - 0.77 = 1.73 \text{ cfs}$$

Next, find the flow area using Equation ST-10 and velocity using the continuity equation  $V = Q/A$ .

$$A = \frac{1}{2}(0.02)(8.5)^2 + \frac{1}{2}(0.167)(2) = 0.89 \text{ ft}^2$$

$$V = Q/A = 2.5/0.89 = 2.81 \text{ ft/sec}$$

The splash-over velocity  $V_o$  is determined from Equation ST-20:

$$V_o = 0.30 + 4.85(2) - 1.31(2)^2 + 0.15(2)^3 = 5.96 \text{ ft/sec}$$

Because  $V_o > V$ ,  $R_f = 1.0$  from Equation ST-20.

Using Equation ST-21, the side-flow capture efficiency is calculated as:

$$R_s = \frac{1}{1 + \frac{0.15(2.81)^{1.8}}{0.02(2)^{2.3}}} = 0.093$$

Finally, the overall capture efficiency is calculated using Equation ST-22:

$$E = 1.0 \left( \frac{1.73}{2.5} \right) + 0.093 \left( \frac{0.77}{2.5} \right) = 0.72$$

$$= 72\%$$

Alternatively, the **Grate-G Worksheet** of the [UD-Inlet Spreadsheet](#) also performs the calculations and calculates a capture percentage of 71.94%.

### **6.8 Example—Curb-Opening Inlet Capacity**

Determine the amount of flow that will be captured by a 6-foot-long curb-opening inlet placed in the composite gutter described in Example Problem 6.2. The composite gutter in that example had the following characteristics:  $T = 9.0$  ft.,  $W = 2.0$  ft,  $S_L = 0.01$ ,  $a = 1.5$  inches,  $S_x = 0.02$  and a Manning's roughness factor of  $n = 0.016$ . In Example Problem 6.2, it was determined that the frontal to total flow ratio was  $E_o = 0.63$  and the total gutter discharge was  $Q = 2.49$  cfs.

Equations ST-25 and ST-26 are used to determine the equivalent slope and the length of inlet required to capture 100% of the gutter flow.

$$S_e = 0.02 + [(1.5/12)/2]0.63 = 0.059$$

$$L_T = 0.60(2.49)^{0.42} (0.01)^{0.3} \left[ \frac{1.0}{(0.016)(0.059)} \right]^{0.6} = 14.4 \text{ ft}$$

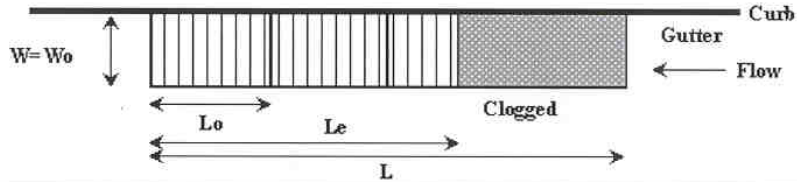
Then, by using Equation ST-23,

$$E = 1.0 \left( 1.0 - \frac{6.0}{14.4} \right)^{1.8} = 0.62$$

Therefore,  $Q_i = EQ = (0.62)(2.49) = 1.54$  cfs will be intercepted by the curb-opening inlet. Note that this problem was performed using the theoretical gutter capacity from Example Problem 6.2. The **Curb-G Worksheet** of the [UD-Inlet Spreadsheet](#) also performs these calculations.

**GRATE INLET ON A GRADE**

Project: **Example 6.8 - Grate Inlet Capacity**  
 Inlet ID: \_\_\_\_\_



Design Information (Input)	
Design Discharge on the Street (from <i>Street Hy</i> )	Qo = 2.5 cfs
Type of Grate	Type = Vane Grate
Length of a Unit Grate	Lo = 2.00 ft
Width of a Unit Grate	Wo = 2.00 ft
Clogging Factor for a Unit Grate	Co = 0.00
Water Depth for Design Condition	Yd = _____ inches
Number of Grates	No = 1
Analysis (Calculated)	
Total Length of Grate Inlet	L = 2.00 ft
Ratio of Gutter Flow to Design Flow Eo (from <i>Street Hy</i> )	Eo = 0.69
Equivalent Slope Se (from <i>Street Hy</i> )	Se = 0.0800 ft/ft
Flow Velocity Vs (from <i>Street Hy</i> )	Vs = 2.77 fps
Spash-over Velocity	Vo = 5.96 fps
Under No-Clogging Condition	
Interception Rate of Gutter Flow	Rf = 1.00
Effective Length of Grate Inlet	L = 2.00 ft
Interception Rate of Side Flow Rx (from <i>Street Hy</i> )	Rx = 0.09
Interception Capacity	Qi = 1.8 cfs
Under Clogging Condition	
Interception Rate of Gutter Flow	Rf = 1.00
Clogging Coefficient for Multiple-unit Grate Inlet	Coef = 0.00
Clogging Factor for Multiple-unit Grate Inlet	Clog = 0.00
Effective (unclogged) Length of Multiple-unit Grate Inlet	Le = 2.00 ft
Interception Rate of Side Flow Rx (from <i>Street Hy</i> )	Rx = 0.09
<b>Actual Interception Capacity</b>	<b>Qa = 1.8 cfs</b>
Carry-Over Flow = Qo-Qa =	Q-co = 0.7 cfs
Capture Percentage = Qa/Qo =	C% = 71.94 %

### **6.9 Example—Curb-Opening Inlet Capacity**

Determine the amount of flow that will be captured by the 6-foot-long curb-opening inlet of Example Problem 6.8 if the gutter did not have a depressed curb section.

Since the cross slope is given ( $S_x = 0.02$ ), an equivalent slope does not have to be determined. Equation ST-24 is used to determine the length of inlet required to capture 100% of the gutter flow.

$$L_T = 0.60(2.49)^{0.42} (0.01)^{0.3} \left[ \frac{1.0}{(0.016)(0.02)} \right]^{0.6} = 27.6 \text{ ft}$$

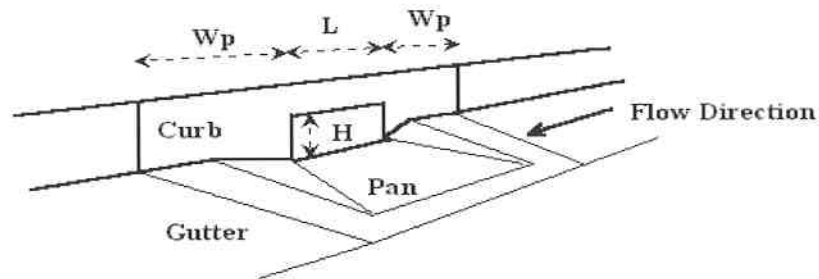
Then, by using Equation ST-23,

$$E = 1.0 - \left( 1.0 - \frac{6.0}{27.6} \right)^{1.8} = 0.36$$

Therefore,  $Q_i = EQ = (0.36)(2.49) = 0.90$  cfs will be intercepted by the curb-opening inlet. Note that the curb-opening inlet is far less effective without a depressed curb section. The **Curb-G Worksheet** of the [UD-Inlet Spreadsheet](#) also performs these calculations.

**CURB OPENING INLET ON A GRADE**

Project: **Example 6.9 - Curb Opening Inlet Capacity**  
 Inlet ID: \_\_\_\_\_



<b>Design Information (Input)</b>	
Design Discharge on the Street (from <i>Street Hy</i> )	Qo = 2.5 cfs
Gutter Flow to Design Flow Ratio (from <i>Street Hy</i> )	Eo = 0.63
Length of a Single Inlet Unit	Lu = 6.00 ft
Clogging Factor for a Single Unit Inlet	Co = 0.00
Number of Inlet Units in Curb Opening	No = 1
<b>Analysis (Calculated)</b>	
Total Length of Curb Opening Inlet	L = 6.00 ft
Equivalent Slope Se (from <i>Street Hy</i> )	Se = 0.0600 ft/ft
Required Length Lo to Have 100% Interception	Lo = 14.32 ft
Clogging Coefficient	C-coeff = 1.00
Clogging Factor for Multiple-unit Curb Opening Inlet	Clog = 0.00
Effective (Unclogged) Length	Le = 6.00 ft
<b>Under No-Clogging Condition</b>	
Effective Length of Curb Opening Inlet (must be ≤ Lo)	L = 6.00 ft
Interception Capacity	Qi = 1.6 cfs
<b>Under Clogging Condition</b>	
Effective Length of Curb Opening Inlet (must be ≤ Lo)	Le = 6.00 ft
Interception Capacity	Qa = 1.6 cfs
Carryover flow = Qo - Qa =	Q-co = 0.9 cfs
Capture Percentage for this Inlet = Qa / Qo =	C% = 62.37 %

**6.10 Example—Combination Inlet Capacity**

A combination inlet is installed in a triangular gutter carrying a discharge of 7 cfs. The gutter is characterized by  $S_L = 0.01$ ,  $S_x = 0.025$ , and  $n = 0.016$ . The curb opening is 10 feet long and the grate is a 2-foot by 2-foot reticuline grate. An 8-foot-long portion of the curb opening is upstream of the grate. Determine the flow intercepted by this combination inlet (Akan and Houghtalen 2002).

First consider the upstream curb-opening portion of the combination inlet. By using Equations ST-24 and ST-23, respectively,

$$L_T = (0.6)(7.0)^{0.42} (0.01)^{0.3} \left[ \frac{1.0}{(0.016)(0.025)} \right]^{0.6} = 37 \text{ ft}$$

$$E = 1.0 - \left[ 1.0 - \frac{8.0}{37} \right]^{1.8} = 0.36$$

Thus, the 8-foot-long portion of the curb opening intercepts  $(0.36)(7.0) = 2.5$  cfs. The remaining flow is  $7.0 - 2.5 = 4.5$  cfs. The spread corresponding to this discharge is calculated using Equation ST-1 as:

$$T = \left[ \frac{(4.5)(0.016)}{(0.56)(0.025)^{1.67} (0.01)^{0.5}} \right]^{3/8} = 11 \text{ ft}$$

Now the flow intercepted by the grate can be computed. By using Equation ST-17,

$$Q_w = 4.5 \left[ 1 - \left( 1 - \frac{2.0}{11} \right)^{2.67} \right] = 1.9 \text{ cfs}$$

and  $Q_s = Q - Q_w = 4.5 - 1.9 = 2.6$  cfs. The splash-over velocity for the grate (Equation ST-20) is  $0.28 + 2.28(2) - 0.18(2)^2 + 0.01(2)^3 = 4.2$  ft/sec. Also, by using Equation ST-4, the flow area just upstream from the grate is  $A = (0.5)(0.025)(11)^2 = 1.5$  ft<sup>2</sup>. Likewise,  $V = Q/A = 4.5/1.5 = 3.0$  ft/sec. Because  $V < V_o$ ,  $R_f = 1.0$  by using Equation ST-19. Next, by using Equation ST-21,

$$R_s = \frac{1}{1 + \frac{(0.15)(3.0)^{1.8}}{(0.025)(2.0)^{2.3}}} = 0.10$$

Then by using Equation ST-22, the efficiency of the grate is:

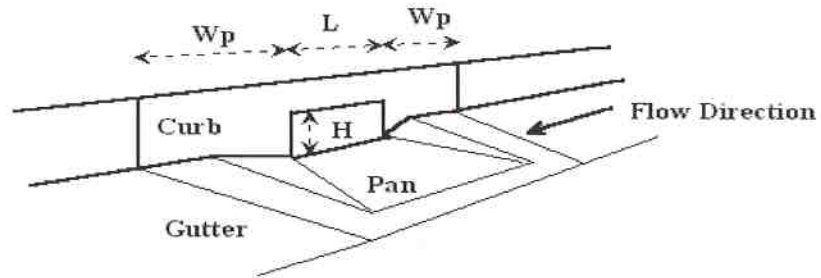
$$E = 1.0 \left( \frac{1.9}{4.5} \right) + 0.10 \left( \frac{2.6}{4.5} \right) = 0.48$$

The flow intercepted by the grate becomes  $(0.48)(4.5) = 2.2$  cfs. The total flow intercepted by the combination inlet is then  $2.5 + 2.2 = 4.7$  cfs. The overall efficiency is  $4.7 / 7.0 = 0.67$  and the bypass flow is  $7.0 - 4.7 = 2.3$  cfs.



**CURB OPENING INLET ON A GRADE**

Project: **Example 6.10 - Curb Opening Inlet Capacity**  
 Inlet ID: \_\_\_\_\_



<b>Design Information (Input)</b>	
Design Discharge on the Street (from <i>Street Hy</i> )	Qo = 2.5 cfs
Gutter Flow to Design Flow Ratio (from <i>Street Hy</i> )	Eo = 0.44
Length of a Single Inlet Unit	Lu = 6.00 ft
Clogging Factor for a Single Unit Inlet	Co = 0.00
Number of Inlet Units in Curb Opening	No = 1
<b>Analysis (Calculated)</b>	
Total Length of Curb Opening Inlet	L = 6.00 ft
Equivalent Slope Se (from <i>Street Hy</i> )	Se = 0.0200 ft/ft
Required Length Lo to Have 100% Interception	Lo = 27.68 ft
Clogging Coefficient	C-coeff = 1.00
Clogging Factor for Multiple-unit Curb Opening Inlet	Clog = 0.00
Effective (Unclogged) Length	Le = 6.00 ft
<b>Under No-Clogging Condition</b>	
Effective Length of Curb Opening Inlet (must be ≤ Lo)	L = 6.00 ft
Interception Capacity	Qi = 0.9 cfs
<b>Under Clogging Condition</b>	
Effective Length of Curb Opening Inlet (must be ≤ Lo)	Le = 6.00 ft
Interception Capacity	Qa = 0.9 cfs
Carryover flow = Qo - Qa =	Q-co = 1.6 cfs
Capture Percentage for this Inlet = Qa / Qo =	C% = 35.58 %

### **6.11 Example—Curb-Opening Inlet in a Sump Condition**

Determine the flow depth and spread at a curb-opening inlet placed in a sump given the following conditions:  $L = 6$  ft,  $h = 0.3$  ft,  $S_x = 0.025$ , and  $Q_i = 5.8$  cfs. Assume there is no clogging.

The flow condition must be assumed and then verified. Assuming orifice flow, Equation ST-28 yields

$$Q_i = C_o A_o (2gd)^{0.5}$$

Now, based on [Table ST-7](#),

$$Q_i = 0.67(h)(L)[(2g)(d_i - h/2)]^{0.5}$$

and by substituting known values,

$$5.8 = (0.67)(0.3)(6)[(2)(32.2)(d_i - 0.3/2)]^{0.5}$$

which yields:

$$d_i = 0.51 \text{ ft}$$

Since  $d_i > 1.4h$ , the orifice equation is appropriate. Equation ST-2 yields  $T = 0.51/0.025 = 20.4$  ft.

The **Curb-S Worksheet** performs these calculations.

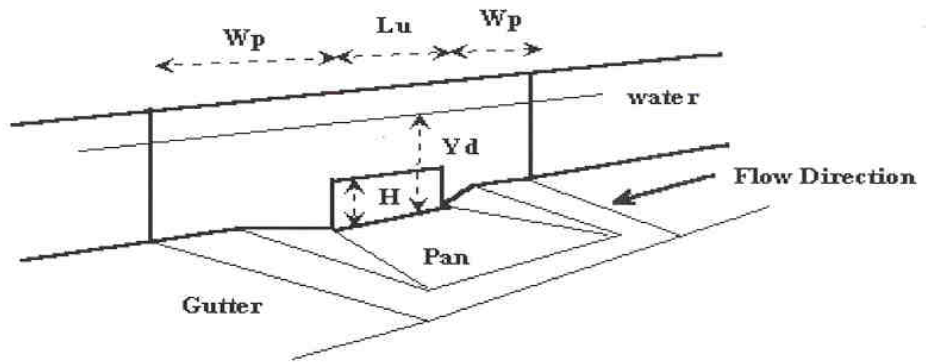
### **6.12 Example—Storm Sewer Hydraulics (Akan and Houghtalen 2002)**

Determine the depth of flow,  $y$ , flow area, and flow velocity in a storm sewer ( $D = 2.75$  ft,  $n = 0.013$ , and  $S_o = 0.003$ ) for a flow rate of 26.5 cfs.

Just full flow conditions are computed first. From Equations ST-34, ST-37 and ST-38,  $A_f = 5.94$  ft<sup>2</sup>,  $R_f = 0.69$  ft, and  $Q_f = 29.1$  cfs. Then,  $V_f = 29.1/5.94 = 4.90$  ft/sec. Now, by using [Figure ST-6](#) with  $Q/Q_f = 26.5/29.1 = 0.91$ , it is determined that  $y/D = 0.73$ ,  $A/A_f = 0.79$ , and  $V/V_f = 1.13$ . Therefore,  $y = (0.73)(2.75) = 2.0$  ft,  $A = (0.79)(5.94) = 4.69$  ft<sup>2</sup>, and  $V = (1.13)(4.90) = 5.54$  ft/sec.

**CURB OPENING INLET IN A SUMP**

Project = **Example 6.12 - Curb Opening Inlet in a Sump Condition**  
 Inlet ID = \_\_\_\_\_



Design Information (Input)	
Design Discharge on the Street (from <i>Street Hy</i> )	Qo = 2.5 cfs
Length of a Unit Inlet	Lu = 6.00 ft
Side Width for Depression Pan	Wp = 3.00 ft
Clogging Factor for a Single Unit	Co = 0.00
Height of Curb Opening in Inches	H = 3.60 inches
Orifice Coefficient	Cd = 0.67
Weir Coefficient	Cw = 3.00
Water Depth for the Design Condition	Yd = 0.51 ft
Angle of Throat (see USDCM Chapter 6, Figure ST-5)	Theta = 63.0 degrees
Number of Curb Opening Inlets	No = 1

Curb Opening Inlet Capacity in a Sump	
<b>As a Weir</b>	
Total Length of Curb Opening Inlet	L = 6.00 ft
Capacity as a Weir without Clogging	Qwi = 12.5 cfs
Clogging Coefficient for Multiple Units	Clog-Coeff = 1.00
Clogging Factor for Multiple Units	Clog = 0.00
Capacity as a Weir with Clogging	Qwa = 12.5 cfs
<b>As an Orifice</b>	
Capacity as an Orifice without Clogging	Qoi = 5.9 cfs
Capacity as an Orifice with Clogging	Qoa = 5.9 cfs
<b>Capacity for Design with Clogging</b>	Qa = 5.9 cfs
<b>Capture Percentage for this Inlet = <math>Qa / Qo =</math></b>	<b>C% = 100.00 %</b>

Note: Unless additional ponding depth or spilling over the curb is acceptable, a capture percentage of less than 100% in a sump may indicate the need for additional inlet units.

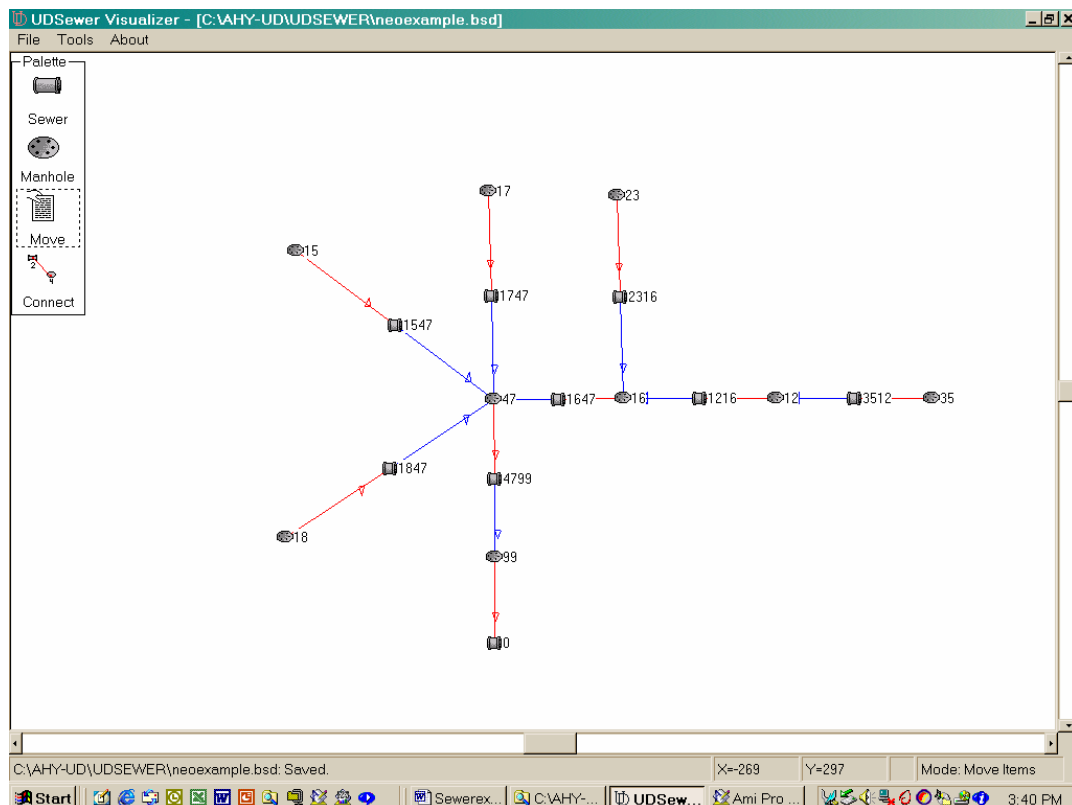
### 6.13 Example—Storm Sewer Hydrology

This example storm sewer system is based on the hydrology for the Denver, Colorado area. It is developed here to illustrate the solution using the NeoUDSEWER computer software. The storm sewer system is to be designed to fully convey the five-year runoff event. The following formula, taken from the Rainfall Chapter of this Manual, describes design rainfall intensity as a function of storm duration:

$$i = \frac{38.5}{(10 + T_d)^{0.768}}$$

in which  $i$  = rainfall intensity in inches per hour and  $T_d$  = rainstorm duration in minutes.

The illustration below depicts a layout of the storm sewer system. It is a copy of the input screen from the NeoUDSEWER software. An ID number is assigned to each manhole and to each sewer segment. The ID numbers have to be unique among the manholes in a system and cannot be duplicated, as is the case for sewer ID numbers among the sewers. At a manhole, NeoUDSEWER can accommodate one outgoing sewer and up to four incoming sewers.



*Example Storm Sewer System Using Computer Model: NeoUDSEWER*

NeoUDSEWER is a storm sewer system sizing and analysis software package. It calculates rainfall and runoff using the Rational Formula method and then sizes circular sewers using Manning's equation. It

has a graphical interface for data entry and editing. NeoUDSEWER can handle a storm sewer system having up to 100 manholes and up to 100 sewers.

Data entry includes project title, rainfall statistics, manhole information, basin hydrology, and sewer network information. Rainfall IDF information can be entered as a table or calculated using the equation given above by entering only a value for the 1-hour depth, P1. The user needs to check all of the default design constraints and criteria and make all necessary changes to these values as needed.

The input parameters for each manhole include the manhole identification number, ground elevation, and incoming and outgoing sewer identification numbers. The hydrologic parameters for the tributary area at a manhole include tributary area, runoff coefficients, overland flow length and slope, local tributary gutter flow length, and gutter flow velocity.

When the local runoff flow rate at a manhole is known, it may be entered (along with non-zero values for local tributary area and local runoff coefficient) to override the flows calculated by the Rational Equation for the local area. NeoUDSEWER will combine the local flow with the upstream flow to calculate the design discharge at the manhole. When the design discharge at a manhole is known for the entire upstream area, the user must enter this discharge (along with total tributary area) and the weighted runoff coefficient to have the program then analyze the EGL and HGL for the system.

A storm sewer is described by its length, slope, upstream crown elevation, Manning's roughness coefficient, shape, bend loss coefficient, and lateral loss coefficient. An existing sewer is identified by the user-defined size and shape. Use of noncircular sewers such as box sewers and arch pipes can be achieved by prescribing their dimensions. However, all new sewers are sized using circular pipes. The program provides suggested commercial sewer sizes for both new and existing sewers. Sewers with flat or negative slope may be analyzed as existing sewers with user-defined sizes provided to the program, along with user-defined tailwater surface elevation at the outlet end. NeoUDSEWER applies open channel hydraulics, culvert hydraulics, and pressure flow hydraulics to calculate the EGL and HGL along the predefined sewer system.

For this example, Table ST-11 provides the watershed hydrologic parameters for the determination of peak design flow rates at the manholes in the system. The design flow can be changed only at a manhole. Sewers 3512, 1216, 1647, and 1547 are treated as existing sewer and their sizes are given in Table ST-12. Other sewer segments are new and will be sized by NeoUDSEWER using circular pipes. In a case that a box conduit is preferred, the sewer may be treated as an existing sewer with a known width. NeoUDSEWER will calculate the water depth and recommend the height for a box sewer. All manholes must have an outgoing pipe except the system outfall pipe (i.e., Manhole 99 in this example) whose outgoing sewer has a pre-assigned ID of zero. For this example, the global Manning's roughness coefficient  $n = 0.013$  was used, and the tailwater surface elevation was set at an elevation of 87 feet.

Table ST-11—Hydrologic Parameters at Manholes

Manhole ID Number	Ground Elevation Feet	Tributary Area acres	Runoff Coeff.	Overland Slope percent	Overland Length Feet	Gutter Slope percent	Gutter Length Feet
35.00	111.00	3.00	0.90	0.15	250.00	0.49	150.00
12.00	109.00	6.45	0.85	0.25	180.00	1.00	450.00
23.00	110.00	5.00	0.90	1.00	275.00	1.00	450.00
16.00	101.50	0.00	0.00	0.00	0.00	0.00	0.00
15.00	104.00	5.00	0.85	0.50	285.00	2.25	450.00
47.00	99.00	3.00	0.80	0.40	250.00	1.56	255.00
99.00	97.50	0.00	0.00	0.00	0.00	0.00	0.00
17.00	99.90	1.00	0.65	0.10	200.00	0.36	300.00
18.00	99.75	1.20	0.45	0.40	300.00	0.00	0.00

Table ST-12—Vertical Profile Information of Sewers

Sewer ID	Length (feet)	Slope (percent)	Upstream Crown Elevation (feet)	Diameter (inches)	Height or Rise (inches)	Width or Span (inches)	Bend Loss Coef.	Lateral Loss Coef.
3512 (round)	450.00	0.50	104.50	24			0.05	
1216 (arch)	360.00	0.80	97.05		20.00	28.00	0.05	0.25
2316	460.00	1.20	105.50				1.00	
1647 (round)	380.00	- 0.10	94.25	27			0.05	0.25
1547 (round)	295.00	1.50	101.10	18			0.40	
4799 (box)	410.00	0.25	93.32		48.00	48.00	0.05	
1747	200.00	2.00	96.80				1.00	
1847	350.00	0.75	94.00				1.00	

For the input parameters in Tables St-11 and ST-12, Neo-UDSEWER produced the following outputs:

**NeoUDS Results Summary**

**Project Title:** CASE STUDY : EXAMPLE ONE  
**Project Description:** STORM SEWER SYSTEM DESIGN: NEW SEWERS WITH EXISTING SEWERS  
**Output Created On:** 8/2/2002 at 9:08:16 AM  
**Using NeoUDSewer Version 1.1.**  
**Rainfall Intensity Formula Used.**  
**Return Period of Flood is 5 Years.**

**A. Sub Basin Information**

Manhole ID #	Basin Area * C	Time of Concentration				Peak Flow (CFS)
		Overland (Minutes)	Gutter (Minutes)	Basin (Minutes)	Rain I (Inch/Hour)	
35	2.70	12.2	0.0	0.0	3.36	9.1
12	3.83	5.0	0.0	0.0	6.33	24.2
23	4.50	13.0	0.0	0.0	3.28	14.7
16	0.00	0.0	0.0	0.0	0.00	35.6
15	4.25	14.1	0.0	0.0	3.16	13.4
47	2.40	19.1	0.0	0.0	2.72	6.5
99	1.70	17.2	0.0	0.0	2.87	4.9
17	0.65	12.8	0.0	0.0	3.30	2.1
18	0.54	11.7	0.0	0.0	3.43	1.9

The shortest design rainfall duration is 5 minutes.  
 For rural areas, the catchment time of concentration is always => 10 minutes.  
 For urban areas, the catchment time of concentration is always => 5 minutes.  
 At the first design point, the time constant is <= (10+Total Length/180) in minutes.  
 When the weighted runoff coefficient => 0.2, then the basin is considered to be urbanized.  
 When the Overland Tc plus the Gutter Tc does not equal the catchment Tc, the above criteria supercedes the calculated values.

**B. Summary of Manhole Hydraulics**

Manhole ID #	Contributing Area * C	Rainfall Duration (Minutes)	Rainfall Intensity (Inch/Hour)	Design Peak Flow (CFS)	Ground Elevation (Feet)	Water Elevation (Feet)	Comments
35	2.7	12.2	3.36	9.1	111.00	106.60	
12	6.52	9.6	3.71	24.2	109.00	105.08	
23	4.5	13.0	3.28	14.7	110.00	105.17	
16	11.02	13.4	3.23	35.6	101.50	99.61	
15	4.25	14.1	3.16	13.4	104.00	101.46	
47	18.86	15.6	3.00	56.7	99.00	91.66	
99	0	0.0	0.00	0.0	97.50	87.00	
17	0.65	12.8	3.30	2.1	99.90	95.88	
18	0.54	11.7	3.43	1.9	99.75	93.03	

**C. Summary of Sewer Hydraulics**

Note: The given depth to flow ratio is 1.

Sewer ID #	Manhole ID Number		Sewer Shape	Calculated	Suggested	Existing	
	Upstream	Downstream		Diameter (Rise) (Inches) (FT)	Diameter (Rise) (Inches) (FT)	Diameter (Rise) (Inches) (FT)	Width (FT)
3512	35	12	Round	19.4	21	24	N/A
1216	12	16	Arch	25.7	27	20	28
2316	23	16	Round	19.8	21	21	N/A
1647	16	47	Round	27.0	27	27	N/A
1547	15	47	Round	18.3	21	18	N/A
4799	47	99	Box	2.3	2	4	4
1747	17	47	Round	8.7	18	18	N/A
1847	18	47	Round	9.9	18	18	N/A

Round and arch sewers are measured in inches.

Box sewers are measured in feet.

Calculated diameter was determined by sewer hydraulic capacity.

Suggested diameter was rounded up to the nearest commercially available size

All hydraulics were calculated using the existing parameters.

If sewer was sized mathematically, the suggested diameter was used for hydraulic calculations.



Sewer ID	Design Flow (CFS)	Full Flow (CFS)	Normal Depth (Feet)	Normal Velocity (FPS)	Critical Depth (Feet)	Critical Velocity (FPS)	Full Velocity (FPS)	Froude Number	Comment
3512	9.1	16.0	1.08	5.3	1.08	5.2	2.9	1	
1216	24.2	20.3	2.00	7.7	1.73	8.4	7.7	N/A	
2316	14.7	17.4	1.24	8.1	1.42	7.0	6.1	1.34	
1647	35.6	35.6	2.25	8.9	2.00	9.5	8.9	N/A	
1547	13.4	12.9	1.50	7.6	1.35	8.0	7.6	N/A	
4799	56.7	91.7	2.34	6.0	1.84	7.7	3.5	0.7	
1747	2.1	14.9	0.38	6.0	0.58	3.4	1.2	2.02	
1847	1.9	9.1	0.46	4.0	0.53	3.3	1.0	1.24	

A Froude number = 0 indicated that a pressured flow occurs.

**D. Summary of Sewer Design Information**

Sewer ID	Slope %	Invert Elevation		Buried Depth		Comment
		Upstream (Feet)	Downstream (Feet)	Upstream (Feet)	Downstream (Feet)	
3512	0.50	102.50	100.25	6.50	6.75	
1216	0.80	95.37	92.49	11.96	7.34	
2316	1.20	103.75	98.23	4.50	1.52	Sewer Too Shallow
1647	-0.10	92.00	92.38	7.25	4.37	
1547	1.50	99.60	95.17	2.90	2.33	
4799	0.25	89.32	88.29	5.68	5.21	
1747	2.00	95.30	91.30	3.10	6.20	
1847	0.75	92.50	89.88	5.75	7.62	

**E. Summary of Hydraulic Grade Line**

Sewer ID #	Sewer Length (Feet)	Surcharged Length (Feet)	Invert Elevation		Water Elevation		Condition
			Upstream (Feet)	Downstream (Feet)	Upstream (Feet)	Downstream (Feet)	
3512	450	450	102.50	100.25	106.60	105.08	Pressured
1216	360	360	95.37	92.49	105.08	99.61	Pressured
2316	460	256.58	103.75	98.23	105.17	99.61	Jump
1647	380	380	92.00	92.38	99.61	91.66	Pressured
1547	295	295	99.60	95.17	101.46	91.66	Pressured
4799	410	0	89.32	88.29	91.66	87.00	Subcritical
1747	200	0	95.30	91.30	95.88	91.66	Jump
1847	350	118.29	92.50	89.88	93.03	91.66	Jump

**F. Summary of Energy Grade Line**

Sewer ID #	Upstream Manhole		Sewer Friction (Feet)	Juncture Losses				Downstream Manhole	
	Manhole ID #	Energy Elevation (Feet)		Bend K Coefficient	Bend Loss (Feet)	Lateral K Coefficient	Lateral Loss (Feet)	Manhole ID #	Energy Elevation (Feet)
3512	35	106.73	0.72	0.05	0.01	0.00	0.00	12	106.00
1216	12	106.00	4.10	0.05	0.05	0.25	1.01	16	100.85
2316	23	105.94	4.51	1.00	0.58	0.00	0.00	16	100.85
1647	16	100.85	8.51	0.05	0.06	0.25	0.05	47	92.23
1547	15	102.35	9.77	0.40	0.36	0.00	0.00	47	92.23
4799	47	92.23	5.23	0.05	0.00	0.00	0.00	99	87.00
1747	17	96.06	3.81	1.00	0.02	0.00	0.00	47	92.23
1847	18	93.20	0.96	1.00	0.02	0.00	0.00	47	92.23

Bend loss = Bend K \* Flowing full vhead in sewer.

Lateral loss = Outflow full vhead - Junction Loss K \* Inflow full vhead.

A friction loss of 0 means it was negligible or possible error due to jump.

Friction loss includes sewer invert drop at manhole.

Notice: Vhead denotes the velocity head of the full flow condition.

A minimum junction loss of 0.05 Feet would be introduced unless Lateral K is 0.

Friction loss was estimated by backwater curve computations.

**G. Summary of Earth Excavation Volume for Cost Estimate**

The user given trench side slope is 1.

Manhole ID #	Rim Elevation (Feet)	Invert Elevation (Feet)	Manhole Height (Feet)
35	111.00	102.50	8.50
12	109.00	95.37	13.63
23	110.00	103.75	6.25
16	101.50	92.00	9.50
15	104.00	99.60	4.40
47	99.00	89.32	9.68
99	97.50	88.29	9.21
17	99.90	95.30	4.60
18	99.75	92.50	7.25

Sewer ID #	Upstream Trench Width		Downstream Trench Width		Trench Length (Feet)	Wall Thickness (Inches)	Earth Volume (Cubic Yards)
	On Ground (Feet)	At Invert (Feet)	On Ground (Feet)	At Invert (Feet)			
3512	16.5	4.5	17.0	4.5	450	3.00	1347
1216	27.8	4.8	18.5	4.8	360	3.00	1981
2316	12.3	4.2	6.3	4.2	460	2.75	562
1647	18.2	4.8	12.4	4.8	380	3.25	1031
1547	8.9	3.9	7.7	3.9	295	2.50	272
4799	16.4	6.9	15.5	6.9	410	5.51	1409
1747	9.3	3.9	15.5	3.9	200	2.50	358
1847	14.6	3.9	18.3	3.9	350	2.50	988

Total earth volume for sewer trenches = 7947.8 Cubic Yards. The earth volume was estimated to have a bottom width equal to the diameter (or width) of the sewer plus two times either 1 foot for diameters less than 48 inches or 2 feet for pipes larger than 48 inches.

If the bottom width is less than the minimum width, the minimum width was used.

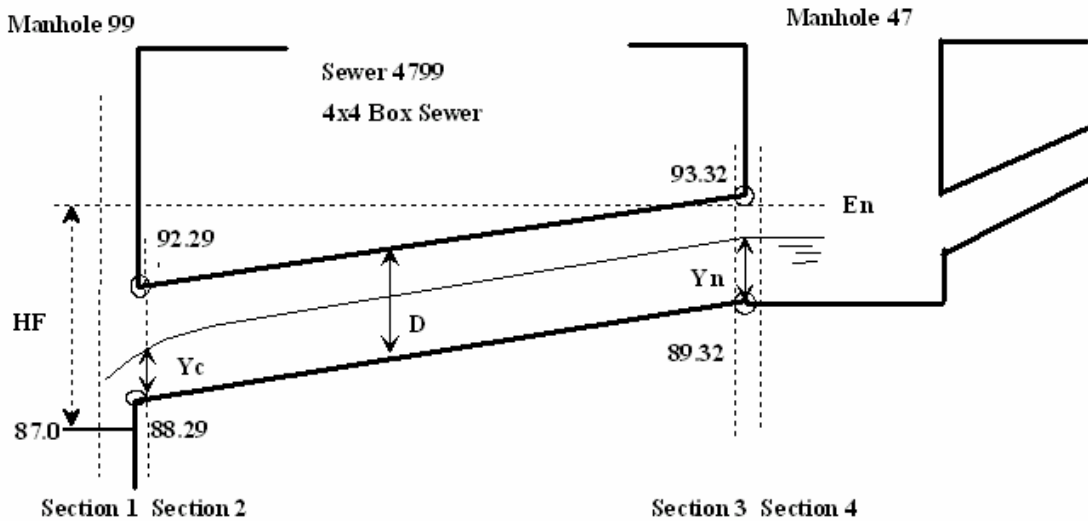
The backfill depth under the sewer was assumed to be 1 foot.

The sewer wall thickness is equal to:  $(\text{equivalent diameter in inches}/12)+1$

The following two cases illustrate how the HGL and EGL were calculated by NeoUDSEWER:

**Case 1. Energy Grade Line Calculation for Sewer 4799 in Example 6.13**

The profile for Sewer 4799 is shown below:



Calculation of an EGL requires the knowledge of flow hydraulics in the sewer and in the downstream manhole. The following parameters are extracted from the NeoUDSEWER output:

Q	Yn	Vn	Ss	Yc	Vc	Sc	N	Fr	Ls
cfs	ft	fps	ft/ft	ft	fps	ft/ft			ft
56.70	2.34	6.04	0.25%	1.84	7.71	0.48%	0.013	0.70	410

in which:

- Q = design flow
- N = Manning's roughness coefficient
- Fr = Froude number for normal flow
- Ss = sewer slope
- Ls = length of sewer
- S = energy slope
- V = flow velocity
- Y = flow depth

The subscript of *n* represents the normal flow condition and *c* represents the critical flow condition.

The calculations of energy balance for this example include three ports: (A) juncture loss at the

downstream manhole, (B) friction losses along Sewer 4799, and (C) energy balance between upstream and downstream manholes. They are conducted separately as follows:

**A. *Juncture Loss at Manhole 99***

Manhole 99 is the system exit. There is no bend loss and lateral loss at Manhole 99. As a result, the known tailwater surface elevation of 87 feet serves for both the EGL and HGL at Manhole 99.

**B. *Along Sewer 4799***

Sewer 4799 carries a discharge of 56.70 cfs. The water surface profile in Sewer 4799 is an M-2 curve produced by a subcritical flow with a Froude number of 0.70.

Section 1

With EGL = HGL = 87 feet at Manhole 99, the EGL and HGL at Section 1 are:

$$E_1 = 87 \text{ feet and } W_1 = 87 \text{ feet}$$

Section 2

With an unsubmerged condition at the sewer exit, an M-2 water surface profile is expected. Therefore, the EGL at Section 2 is dictated by the critical flow condition. Let  $Y_2 = Y_c$ ,  $V_2 = V_c$ . According to Equation ST-40, the EGL at Section 2 is:

$$E_2 = \text{Max}\left(\frac{7.71^2}{2 * 32.2} + 1.84 + 88.29, 87.0\right) = 91.05 \text{ feet}$$

and the HGL at Section 2 is:

$$W_2 = 1.82 + 88.29 = 90.13 \text{ feet}$$

Section 3

The determination of the EGL from Section 2 to Section 3 is essentially the backwater profile calculation using the direct step method. Assuming that the flow depth at Section 3 is the normal flow depth, the energy equation is written as:

$$E_c = E_n + h_f$$

in which:

$E_c = 92.23$  feet which is the EGL of the critical flow at Section 2

$E_n = 91.05$  feet which is the EGL of the normal flow at Section 3

$h_f$  = friction loss which is related to the critical energy slope,  $S_c$

$$S_c = 0.0048$$

$S_s = 0.0025$  which is the normal flow energy slope

Both energy slopes can be calculated by Manning's equation. Using the direct step method, the length of the M-2 water surface profile,  $X$ , between the critical flow section and normal flow section is calculated as:

$$X = \frac{E_n - E_c}{0.5(S_n + S_c)} = \frac{92.23 - 91.05}{0.5(0.0025 + 0.0048)} = 322.45 \text{ feet}$$

Because the length of the M-2 curve is shorter than the length of Sewer 4799, the assumption of normal flow at Section 3 is acceptable. Therefore, the EGL and HGL at Section 3 are:

$$E_3 = 92.23 \text{ feet (normal flow condition)}$$

$$W_3 = 2.34 + 89.23 = 91.66 \text{ feet}$$

#### Section 4

Assuming that the loss at the entrance of Sewer 4799 is negligible, the EGL and HGL at Section 4 are the same as those at Section 3, namely:

$$E_4 = 92.23 \text{ feet}$$

$$W_4 = 91.66 \text{ feet}$$

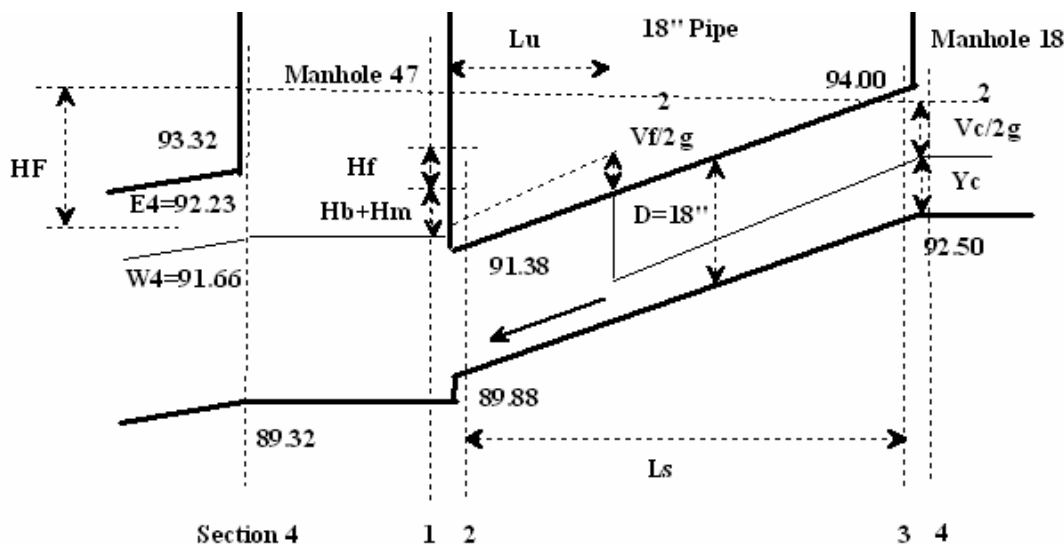
#### **C. Energy Balance Between Manholes.**

The calculations of the EGL along Sewer 4799 and across Manhole 99 do not include manhole drop and possible losses due to hydraulic jump. Therefore, it is necessary to perform energy balancing between Manholes 47 and 99 as:

$$92.23 = 87.0 + H_b + H_m + H_f$$

$$H_f = 92.23 - 87.0 - 0 - 0 = 5.23 \text{ feet}$$

**Case 2. Energy Grade Line for Sewer 1847 in Example 6.13**



The flow parameters along Sewer 1847 and at Manhole 47 can be found in the NeoUDSEWER output file. They are summarized as follows:

Q	Yn	Vn	Yc	Vc	V <sub>f</sub>	N	Fr	Ss	Ls	Kb
cfs	ft	fps	ft	fp	ft			ft/ft	ft	
1.85	0.46	4.05	0.53	3.33	1.05	0.01	1.05	.75%	350	1.00

in which  $V_f$  = full-flow velocity and the definitions of other flow parameters can be found in Example 12-1.

**A. Juncture Loss at Manhole 47**

Sewer 1847 carries a discharge of 1.85 cfs, which is a supercritical flow with a Froude number of 1.05. At Manhole 47, the EGL and HGL have been calculated as  $E_4 = 92.23$  and  $W_4 = 91.66$  feet.

To cross Manhole 47 (i.e., from Section 4 to Section 1) the bend loss is:

$$H_b = K_b \frac{V_f^2}{2g} = 1.0 \frac{1.05^2}{2 * 32.2} = 0.017 \text{ feet}$$

Because Sewer 1847 is not on the main line, it does not have a lateral loss (i.e.,  $K_m = 0.0$ ). Between Sections 4 and 1, the energy principle is written as:

$$E_1 = E_4 + H_b + H_m = 92.23 + 0.017 + 0 = 92.25 \text{ feet}$$

$$H_1 = E_1 - \frac{V_f^2}{2g} = 92.25 - \frac{1.05^2}{2 * 32.2} = 92.23 \text{ feet}$$

**B. Friction Losses through Sewer 1847**Section 1

With EGL = 92.25 feet and HGL = 92.23 feet at Manhole 99, the EGL and HGL at Section 1 are:

$$E_1 = 92.25 \text{ feet and } W_1 = 92.23 \text{ feet}$$

The downstream end of Sewer 1847 is submerged.

Section 2

With a submerged exit, the EGL for the full-flow condition is:

$$E_F = \frac{1.05^2}{2 * 32.2} + 1.5 + 89.88 = 91.39 \text{ feet}$$

The EGL at Section 2 is chosen as the higher one between the one for the full-flow condition and the EGL at Section 1, thus:

$$E_2 = \text{Max}(E_F, E_1) = 92.25 \text{ feet}$$

and the resulting HGL at Section 2 is:

$$W_2 = 92.23 \text{ feet}$$

Section 3

The lower portion of Sewer 1847 is surcharged because of the exit submergence. According to Manning's equation, the friction slope for the full flow condition in Sewer 1847 is 0.003 ft/ft. According to the direct step method, the surcharge length near the downstream end of Sewer 1847 can be approximated by  $W_2$  and the sewer crown elevation, *Crown*, as:

$$L_u = \frac{W_2 - \text{Crown}}{(S_s - S_f)} = \frac{92.23 - 91.38}{(0.0075 - 0.003)} = 118.1 \text{ feet}$$

The friction loss through the surcharged length is:

$$H_f = S_f * L_u = 0.003 * 118.1 = 0.35 \text{ feet}$$

The EGL at Section 3 is controlled by either the friction loss through the surcharged length or the critical flow condition at the entrance. Considering the friction loss, we have:

$$E_{31} = E_1 + H_f = 92.25 + 0.35 = 92.6 \text{ feet}$$



Considering the critical flow condition at the entrance, we have:

$$E_{32} = \frac{V_c^2}{2g} + Y_c + 92.50 = 93.20 \text{ feet}$$

In comparison, the EGL at Section 3 is determined as:

$$E_3 = \text{Max}(E_{31}, E_{32}) = 93.20 \text{ feet}$$

This process is similar to the culvert hydraulics under a possible hydraulic jump. The headwater depth at the entrance of Sewer 1847 shall consider both inlet and outlet controls; whichever is higher dictates the answer. As a result, the HGL at Section 3 is:

$$W_3 = E_3 - \frac{V_c^2}{2g} = 93.03 \text{ feet}$$

#### Section 4

Considering that the entrance loss is negligible for Sewer 1847, we have:

$$E_4 = E_3 \text{ and } W_4 = W_3$$

#### **C. Between Manholes 47 and 18**

The energy balance between Manhole 18 and Manhole 47 is:

$$93.20 = 92.23 + H_b + H_m + H_f$$

$$H_f = 93.20 - 92.23 - 0.017 - 0 = 0.96 \text{ feet}$$

## 7.0 REFERENCES

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