Chapter 9 - Storm Drain Conduits

Within the highway drainage system, a storm drain receives surface water through inlets and conveys the water through conduits to an outfall. Its components include different lengths and sizes of pipe or conduit connected by appurtenant structures. Designers term a section of conduit connecting one inlet or appurtenant structure to another a “segment” or “run.” Typically, designers use circular pipe for storm drain conduits, but they also use a box or other enclosed conduit shapes. Appurtenant structures include inlet structures (excluding the actual inlet opening), access holes, junction chambers, and other miscellaneous structures. Chapter 8 presents generalized design considerations for these structures. This chapter includes information on the computation of energy losses through these structures.

9.1 Hydraulics of Storm Drainage Systems

The design of storm drainage systems depends on an understanding of the basic concepts of hydrology and fluid flow. Chapter 4 discussed hydrologic concepts. Important hydraulic principles include open channel and pressure flow, classification of open channel flow, conservation of mass, conservation of momentum, and conservation of energy. Chapter 6 introduced some of these elements. Many textbooks and references, including HDS-4 (FHWA 2008) and Chow (1959), also discuss these topics. The following sections assume that the designer has a basic understanding of these topics.

9.1.1 Flow Type Assumptions

The design procedures presented here assume each storm drain segment has a **steady** and **uniform** flow. Steady means that the discharge does not change with time; uniform means that flow depth in each segment does not change with distance along the conduit. Because of these assumptions, and since storm drain conduits have regular, prismatic shapes, designers consider the average velocity throughout a segment to be constant.

In actual storm drainage systems under operating conditions, the flow at each inlet varies in time, so actual flow conditions are not truly steady or uniform. However, since the usual hydrologic methods for storm drain design estimate the peak flow at the beginning of each run, using the steady uniform flow assumption represents a “conservative” design practice.

9.1.2 Open Channel and Pressure Flow

Two primary philosophies exist for the design of storm drains under the steady uniform flow assumption. Designers refer to the first as “open channel” design because the pipes flow partially full. To maintain open channel flow, designers size the segment to maintain a free water surface within the conduit. The pressure above the surface remains at atmospheric pressure. For open channel flow, flow energy comes from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). To maintain the water surface throughout the conduit at atmospheric pressure, the designer keeps flow depth at less than the height of the conduit.

Pressure flow design, the other major method, assumes that the flow in the conduit will be at a pressure greater than atmospheric. Under this condition, no exposed flow surface exists within the conduit. In pressure flow, energy again comes from the flow velocity, depth, and elevation. The significant difference here is that the total energy head will be above the top of the conduit and be greater than the depth of flow in the conduit. In this case, the hydraulic grade line represents the pressure head level (see Section 9.4 for a discussion of the hydraulic grade line). The designer should remember that the pressure condition is at the design discharge; during the rising and falling of discharge, flow in the conduit will go from zero, through open channel conditions, to pressure flow, then back through open channel conditions. In actual performance, only parts of a system may operate in pressure flow at any given time.

For decades, highway agencies have considered the relative merits of using open channel or pressure flow to control design. For a given flow rate, open channel flow designs involve larger conduit sizes than do pressure flow designs. While the materials may cost more for storm drainage systems based on open channel flow, this approach provides a margin of safety by adding capacity in the conduit to accommodate larger discharges than the design discharge. Designers often want to include this margin of safety given the inexact nature of runoff estimation methods and the technical and financial difficulty of replacing existing storm drains. When designers add the costs of excavation, trench protection, pipe bedding, trench backfill, compaction, and other associated storm drain construction expenses, they typically find only minor cost savings from using the smaller conduit allowed by design for pressure flow. The most expensive decision designers make regarding storm drains is choosing to install them at all. Having made that decision, designers will wish to maximize associated benefits.

However, some situations may call for pressure flow design. For example, designers may choose to use an existing system that only accommodates the increased flow rates when placed under pressure flow. In such instances, the designer may make a hydraulic and economic analysis of a storm drain using both design methods before final selection.

Most ordinary conditions call for sizing storm drains based on open channel flow at or less than flow full. Designing for full flow is a conservative assumption since the peak flow capacity actually occurs at 93 percent of the full flow depth of a circular pipe. When using pressure flow, designers will want to ensure the joints can withstand the pressure to avoid exfiltration. However, the pressures encountered are usually moderate.

9.1.3 Hydraulic Capacity

Storm drain size, shape, slope, and friction resistance control its hydraulic capacity. Several flow friction formulas describe the relationship between flow capacity and these parameters. Engineers most often use Manning's equation for designing storm drains.

Chapter 5 introduced Manning's equation for computing the capacity for roadside and median channels. For circular storm drains flowing full, Manning's equation becomes:

$$V = \left(\frac{K_V}{n}\right) D^{0.67} S_o^{0.5} \tag{9.1}$$

$$Q = \left(\frac{K_Q}{n}\right) D^{2.67} S_o^{0.5} \tag{9.2}$$

where:

V	=	Mean velocity, ft/s (m/s)
Q	=	Rate of flow, ft^3/s (m^3/s)
K_V	=	Unit conversion constant, 0.59 in CU (0.397 in SI)
K_Q	=	Unit conversion constant, 0.46 in CU (0.312 in SI)
n	=	Manning's roughness coefficient
D	=	Storm drain diameter, ft (m)
S_o	=	Slope of the energy grade line, ft/ft (m/m)

Table 9.1 provides representative values of the Manning's roughness coefficient for various storm drain materials. Figure 9.1 illustrates storm drain conduit capacity sensitivity to the parameters in

Manning's equation. For example, doubling the diameter of a circular storm drain conduit increases its capacity by a factor of 6.35; doubling the slope increases capacity by a factor of 1.4; but doubling the roughness reduces pipe capacity by 50 percent.

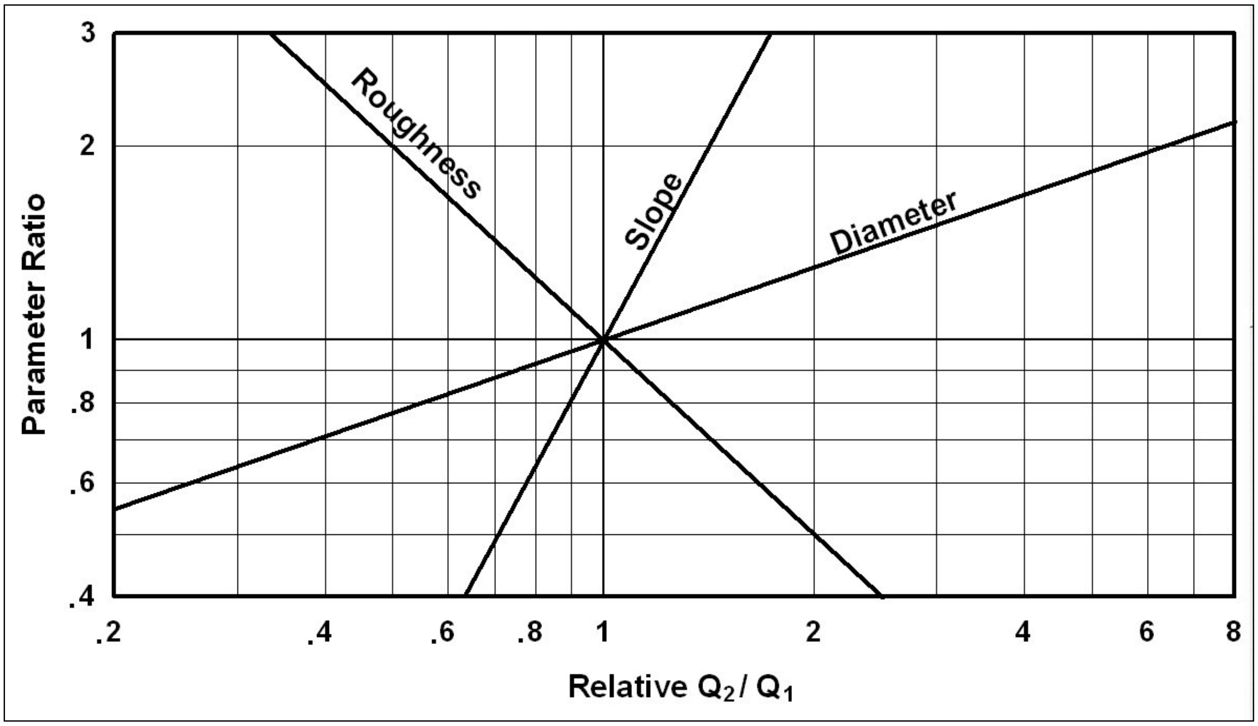

Figure 9.1. Storm drain capacity sensitivity.

As shown in Figure 9.1, slope and Manning's roughness coefficient represent continuous variables, while storm drain diameter comes in discrete sizes. For example, circular conduit only comes in increments such as 3 inches or 6 inches. To limit inventory size, vendors typically stock circular conduit on 6-inch increments (e.g., 12, 18, 24, and 30 inches) making it readily available, and thus less expensive, than sizes on 3-inch increments not divisible by 6. For example, 24-inch pipe usually costs less than 21-inch pipe. While manufacturers often list 21-inch (along with 15-, 27-, and 33-inch), designs rarely use those sizes.

Economies in Pipe Quantities

Pipe sizes involve some "economy of scale." Often, ordering small quantities of many different pipe sizes costs more than specifying a more frequently used but larger pipe size for a project. Within reason, the greater the total length of a given size pipe on a project, the less that size will cost per unit length. In addition, larger pipe sizes can provide added hydraulic capacity.

For circular conduits:

- Peak flow occurs at 93 percent of the height of the circular pipe. Therefore, a design using a circular pipe for full flow will be slightly conservative.
- Velocity in a pipe flowing half-full equals that for full flow.
- Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
- As the depth of flow falls below half-full, the flow velocity drops off rapidly.

Table 9.1 Manning's roughness coefficients for storm drain conduits.

Type of Culvert	Roughness or Corrugation	Manning's n *
Concrete Pipe	Smooth	0.010-0.011
Concrete Boxes	Smooth	0.012-0.015
Spiral Rip Metal Pipe	Smooth	0.012-0.013
Corrugated Metal Pipe, Pipe-Arch and Box (Manning's n varies with barrel size)	2-2/3 by 1/2 inch (annular)	0.022-0.027
	2-2/3 by 1/2 inch (helical)	0.011-0.023
	6 by 1 inch (helical)	0.022-0.025
	5 by 1 inch	0.025-0.026
	3 by 1 inch	0.027-0.028
	6 by 2 inch (structural plate)	0.033-0.035
	9 by 2-1/2 inch (structural plate)	0.033-0.037
Corrugated Polyethylene	Smooth	0.009-0.015
	Corrugated	0.018-0.025
Polyvinyl chloride (PVC)	Smooth	0.009-0.011

*HDS-5 (FHWA 2012a) documents laboratory-derived Manning's n values. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

The shape of a storm drain conduit also influences its capacity. Designers most commonly use circular storm drain conduits; using an alternate shape sometimes increases capacity. Table 9.2 lists the increase in capacity obtained by using alternate conduit shapes with the same height as the original circular shape but with a different cross-sectional area. Although these alternate shapes generally cost more than circular shapes, specific project area conditions sometimes warrant their use. For example, limited headroom (vertical clearance) may warrant use of elliptical, pipe-arch, and box shapes. Standard practice orients elliptical and box shapes with the longer dimension horizontal. In case of limited horizontal clearance, orienting elliptical pipe vertically may enhance performance over circular pipe.

Some shapes are not available from suppliers in all locations. Designers may wish to consult industry suppliers in their State before specifying a particular shape to ensure its availability for their project. Commonly, either pipe-arch or elliptical shapes are available, but not both.

Table 9.2. Increase in capacity of alternate shapes based on a circular pipe with the same height.

Shape	Area (Percent Increase)	Conveyance (Percent Increase)
Circular	--	--
Oval	63	87
Arch	57	78
Box (B = D)	27	27

Example 9.1: Pipe size alternative and capacity.

Objective: Estimate the pipe diameter to convey the design flow. Consider use of both concrete and helical corrugated metal pipes. Estimate the full flow capacity of the selected pipes.

Given:

Q = 17.6 ft^3/s (0.50 m^3/s)

S_o = 0.015 ft/ft (m/m)

n = 0.013 for concrete, 0.017 for corrugated metal

Step 1. Estimate the concrete circular pipe diameter.

Using equation 9.2, calculate the reinforced concrete pipe (RCP) diameter.

$D = [(Q\ n)/(K_Q\ S_o^{0.5})]^{0.375} = [(17.6)(0.013)/\{(0.46)(0.015)^{0.5}\}]^{0.375} = 1.69$ ft (20.3 inches)

Use D = 21-inch diameter standard pipe size.

Step 2. Estimate helical corrugated metal pipe (CMP) diameter.

Using equation 9.2, calculate the CMP diameter.

$D = [(Q\ n)/(K_Q\ S_o^{0.5})]^{0.375} = [(17.6)(0.017)/\{(0.46)(0.015)^{0.5}\}]^{0.375} = 1.87$ ft (22.4 inches)

Use D = 24-inch diameter standard size. Note that the n value of 0.017 corresponds to the value for a 24-inch CMP, as shown in Table 9.1.

Step 3. Compute the full flow capacity for the concrete pipe.

$Q = (K_Q/n)\ D^{2.67}\ S_o^{0.5} = (0.46)/(0.013)\ (1.75)^{2.67}\ (0.015)^{0.5} = 19.3$ ft^3/s

$V = (K_V/n)\ D^{0.67}\ S_o^{0.5} = (0.59)/(0.013)\ (1.75)^{0.67}\ (0.015)^{0.5} = 8.0$ ft/s

Step 2. Compute the full flow capacity for the helical CMP.

$Q = (K_Q/n)\ D^{2.67}\ S_o^{0.5} = (0.46)/(0.017)\ (2.0)^{2.67}\ (0.015)^{0.5} = 21.1$ ft^3/s

$V = (K_V/n)\ D^{0.67}\ S_o^{0.5} = (0.59)/(0.017)\ (2.0)^{0.67}\ (0.015)^{0.5} = 6.8$ ft/s

Solution: The RCP and CMP have design diameters of 21 inches (530 mm) and 24 inches (610 mm), respectively. A rougher surface produces more friction, resulting in a larger diameter. The concrete pipe has a full flow capacity and velocity of 19.3 ft^3/s (0.55 m^3/s) and 8.0 ft/s (2.4 m/s). The metal pipe has a full flow capacity and velocity of 21.1 ft^3/s (0.60 m^3/s) and 6.8 ft/s (2.1 m/s).

9.1.4 Energy Grade Line/Hydraulic Grade Line

The "energy grade line" (EGL) refers to a conceptual line longitudinally connecting points of total energy along a channel or conduit carrying water. Total energy includes elevation (potential) head, velocity head, and pressure head. Calculating the EGL for the full length of the system represents a critical step in storm drain evaluation. Designers develop the EGL by calculating all losses through the system. The principle of conservation of energy states that the energy head at any cross-section equals that at any other downstream section, plus the losses occurring between. Designers typically describe the intervening losses as either friction losses or form (minor) losses. Understanding and estimating the hydraulic grade line (HGL) elevation depends on knowing the location of the EGL and the velocity at each cross-section.

The HGL in an open channel is the surface level of flowing water at any point along the channel. In closed conduits flowing under pressure, the HGL is a conceptual line like the EGL, but without the velocity component. It describes the level to which water would rise in any connecting system open to the atmosphere (often represented as a vertical tube) at any point along the pipe. When determining the acceptability of a proposed storm drainage system, designers use HGL by establishing the elevation to which water will rise under design conditions.

To determine the elevation of the HGL at any point, subtract the velocity head ($V^2/2g$) from the EGL. Energy concepts introduced in Chapter 6 apply to pipe flow as well as open channel flow. Figure 9.2 illustrates the EGLs and HGLs for open channel and pressure flow in pipes.

When water flows through the pipe and a space of air exists between the top of the water and the inside of the pipe, designers consider the flow to be open channel flow, with the HGL at the water surface. When the pipe flows full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, the energy condition lies in an unstable condition between open channel flow and pressure flow (with flow often oscillating between open channel and pressure conditions). At this condition, the resistance of the total pipe circumference influences the flow. Under gravity full flow, the HGL coincides with the crown of the pipe.

If the HGL exceeds the top elevation of a storm drain feature such as a junction box, access hole, or inlet, surcharging will occur, and water can flow out of the structure. If not secured, the structure lid can also be displaced, a condition known as a "blowout."

Engineers carefully plan designs based on open channel conditions as well, including evaluating the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As designers perform hydraulic calculations, they frequently verify the existence of the desired flow condition. Under actual operating conditions, storm drainage systems may alternate between pressure and open channel flow conditions from one section to another.

Section 9.1.6 presents methods for determining energy losses in a storm drain. Section 9.4 presents a suggested detailed procedure for evaluating the EGL and the HGL for storm drainage systems.

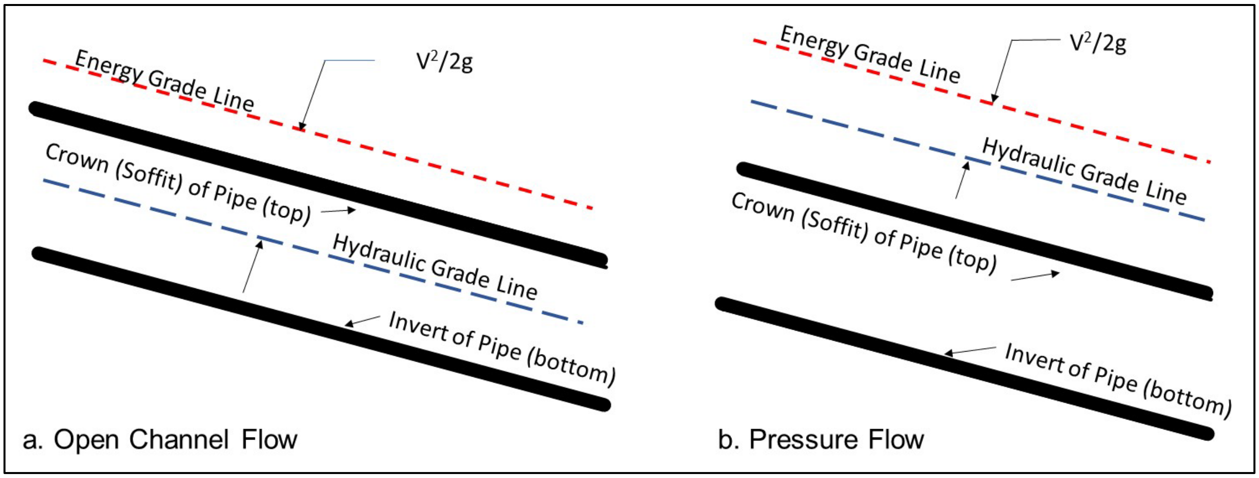

Figure 9.2. Hydraulic and energy grade lines in pipe flow.

9.1.5 Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system discharges into a surface conveyance. The discharge point can be a natural river or stream, an existing stormwater conveyance system, or an existing or proposed channel that conveys stormwater away from the highway. Designers refer to the water surface elevation of the receiving water as the **tailwater elevation**. Outfall design involves consideration of the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, energy dissipation needs, and the outlet structure orientation.

HGL and EGL computations start at the **tailwater depth or elevation** at the storm drain outfall and proceed upstream. If the tailwater elevation is below the invert elevation of the outlet conduit, the tailwater has no influence on the HGL. If above the invert elevation but below critical depth in the outfall pipe, the tailwater also has no influence on the HGL. Higher tailwater elevations, especially those above the crown of the outlet conduit, create backwater conditions the designer considers in the HGL computations. In most cases, designers use the greater of the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(y_c + D)/2$, as the starting point for HGL evaluation.

If the outfall channel is a river or stream, to adequately determine the elevation of the tailwater in the receiving stream, the designer can consider the joint or coincidental probability of two hydrologic events. Designers can evaluate the relative independence of the storm drainage system discharge by comparing the drainage area of the receiving stream to the area of the storm drainage system. The FHWA's HEC-19 (FHWA 2022b) includes information on the occurrence of coincident flows that are applicable to storm sewers and the receiving streams.

Designers should consider the potential for an excessive tailwater to cause flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, the designer may isolate the storm drain from the outfall with a pump station (see Chapter 12).

Protection of the storm drain outlet may also depend on **energy dissipation**. Designers typically use such protection at the outlet to prevent erosion of the outfall bed and banks. As the HEC-14 (FHWA 2006a) manual for designing an appropriate dissipator describes, engineers expecting high velocities should provide riprap aprons or energy dissipators.

The **orientation of the outfall** is another important design consideration. Where practical, designers position the outlet of the storm drain in the outfall channel to orient its flow in a downstream direction relative to the receiving stream. This will reduce turbulence and the potential for excessive erosion. If designers cannot orient the outfall structure in a downstream direction, they will want to consider the potential for outlet scour. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, erosion may occur on the opposite channel bank. If erosion potential exists, designers may consider a channel bank lining of riprap or other suitable material on the bank. Alternatively, they could use an energy dissipator structure at the storm drain outlet.

9.1.6 Energy Losses

To compute the HGL and EGL, designers estimate all energy losses in pipe runs and junctions. In addition to the principal energy lost to friction in each conduit run, turbulence causes energy (or head) losses at outlets, inlets, bends, transitions, junctions, and access holes.

9.1.6.1 Pipe Friction Losses

Friction or boundary shear loss represents the major loss in a storm drainage system. The head loss due to friction in a pipe is computed as follows:

$$h_f = S_f L \qquad (9.3)$$

where:

h_f	=	Friction loss, ft (m)
S_f	=	Friction slope, ft/ft (m/m)
L	=	Length of pipe, ft (m)

The friction slope is also the slope of the hydraulic gradient for a particular pipe run. Assuming steady uniform flow (see Section 9.1.1), the friction slope is the same as the pipe slope for partially full flow. Pipe friction loss for full flow in a circular pipe is:

$$S_f = \left(\frac{h_f}{L}\right) = \left(\frac{Qn}{K_Q D^{2.67}}\right)^2 \qquad (9.4)$$

where:

K_Q	=	Unit conversion constant, 0.46 in CU (0.312 in SI)

9.1.6.2 Exit Losses

The exit loss from a storm drain outlet is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as at an end wall, the exit loss equals:

$$H_0 = 1.0\left[\frac{V_o^2}{2g} - \frac{V_d^2}{2g}\right] \qquad (9.5)$$

where:

V_o	=	Average outlet velocity
V_d	=	Channel velocity downstream of outlet in the direction of the pipe flow
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

Note that when $V_d = 0$, as in a reservoir, the exit loss equals one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, consider the exit loss as virtually zero.

9.1.6.3 Bend Losses

Estimate the bend loss coefficient (H_b) for storm drain design (for bends in the pipe run, not in an access hole structure) using the following formula (AASHTO 2014):

$$H_b = 0.0033(\Delta)\frac{V^2}{2g} \tag{9.6}$$

where:

Δ	=	Angle of bend, degrees

9.1.6.4 Transition Losses

Figure 9.3 shows an expansion transition. Typically, designers use access holes when pipe size increases; however, in rare cases, expansions without an access hole may be appropriate.

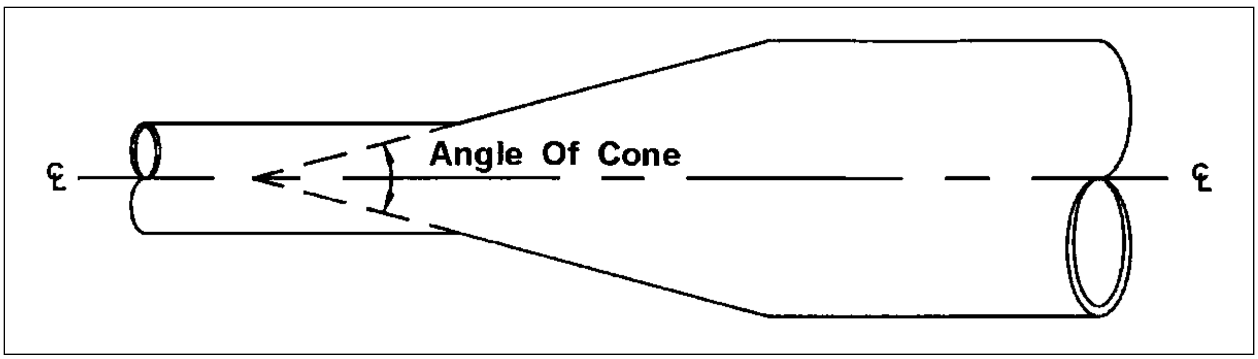

Figure 9.3. Angle of cone for pipe diameter changes.

Designers can express energy losses in expansions or contractions in open channel flow in terms of the kinetic energy at the two ends:

$$H_e = K_e\left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right] \tag{9.7}$$

where:

K_e	=	Expansion coefficient
V_1	=	Velocity upstream of transition, ft/s (m/s)
V_2	=	Velocity downstream of transition, ft/s (m/s)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

Table 9.3 presents typical values of K_e for gradual expansions.

Designers do not use contractions in storm drains because of the potential for clogging and safety hazards when transitioning to a smaller pipe size. However, contractions have an analogous energy loss relationship:

$$H_c = K_c \left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right] \tag{9.8}$$

where:

K_c	=	Contraction coefficient
V_1	=	Velocity upstream of transition, ft/s (m/s)
V_2	=	Velocity downstream of transition, ft/s (m/s)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

Table 9.3. Typical values for K_e for gradual enlargement of pipes in open channel flow.

D_2/D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
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

Debris and Clogging

Storm drains invariably transport debris (potentially including litter, vegetative materials, and many other items) off the watershed and into receiving waters. Much of this debris could clog the system, accumulate with other debris, and obstruct the flow of stormwater. Cardboard boxes, branches, and other items with at least one dimension greater than the diameter of the conduits are common.

Although uncommon, persons (particularly children), pets, or livestock can be swept into storm drains. Wild animals often inhabit them during dry periods.

To facilitate passage of debris through the system, and to reduce the risk of trapping people or animals, designers typically ensure the maximum dimension (e.g., diameter) of conduits does not decrease in the downstream direction.

9.1.6.5 Junction Losses

Figure 9.4 shows a pipe junction connecting a lateral pipe to a larger trunk pipe without using an access hole structure. Underground pipe junctions represent both potential debris clogging hazard points and access challenges for maintenance staff in the case of clogging. For these reasons, designers use junction boxes with access as a preferable alternative where conduits join. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j = \frac{(Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta_j)}{0.5 g (A_o + A_i)} + h_i - h_o \tag{9.9}$$

where:

H_j	=	Junction loss, ft (m)
Q_o	=	Outlet flow, ft^3/s (m^3/s)
Q_i	=	Inlet flow, ft^3/s (m^3/s)
Q_l	=	Lateral flow, ft^3/s (m^3/s)

V_o = Outlet velocity, ft/s (m/s)
V_i = Inlet velocity, ft/s (m/s)
V_l = Lateral velocity, ft/s (m/s)
h_o = Outlet velocity head, ft (m)
h_i = Inlet velocity head, ft (m)
A_o = Outlet cross-sectional area, ft^2 (m^2)
A_i = Inlet cross-sectional area, ft^2 (m^2)
θ_j = Angle between the inflow trunk pipe and inflow lateral pipe

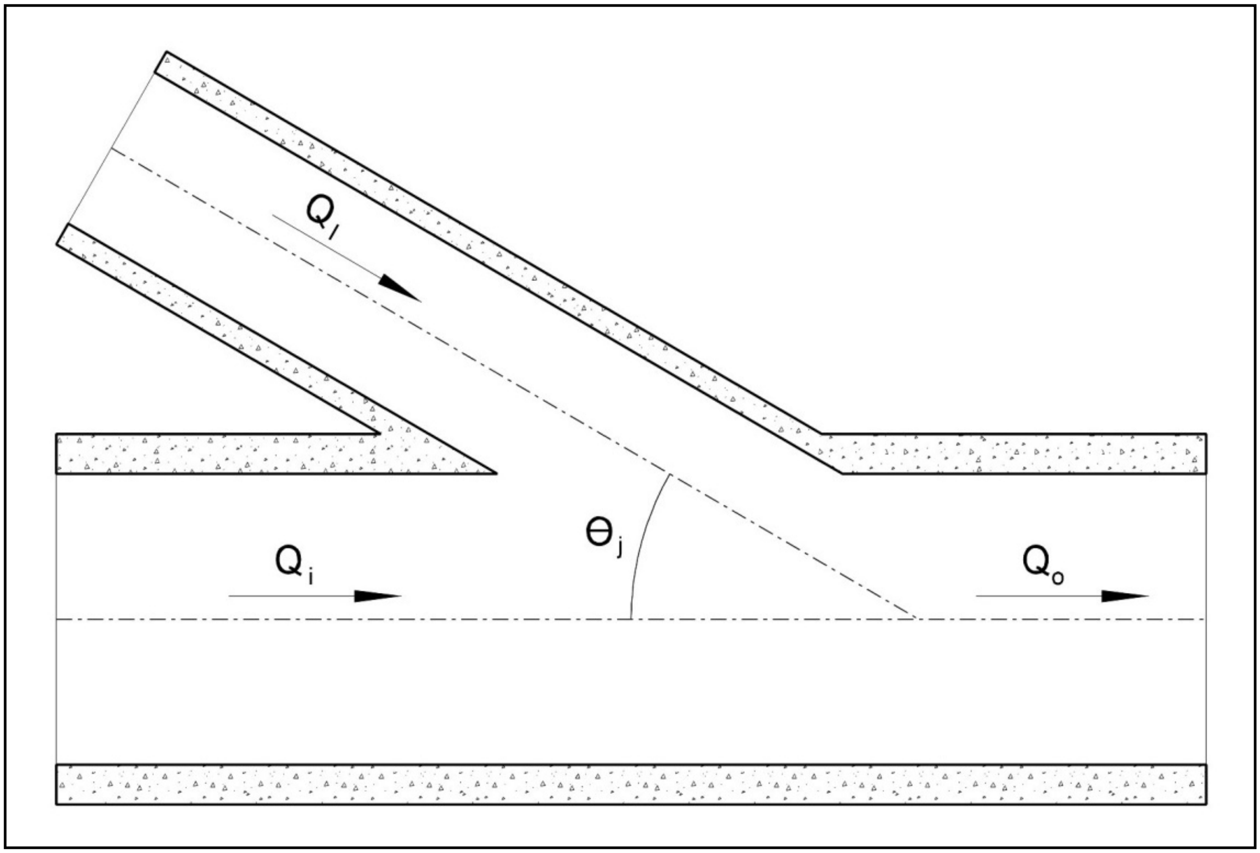

Figure 9.4. Interior angle definition for pipe junctions.

9.1.6.6 Approximate Method for Inlet and Access Hole Energy Loss

Estimating inlet and access hole energy losses at the junction between inflow and outflow pipes presents more complexity than estimating junction losses. The **Approximate Method** is the simplest and appropriate only for preliminary design estimates. The method recognizes that initial layout of a storm drain system begins at its upstream end. The designer estimates sizes and establishes preliminary elevations as the design progresses downstream. The Approximate Method estimates losses across an access hole by multiplying the velocity head of the outflow pipe by a coefficient:

$$H_{ah} = K_{ah}\frac{V_o^2}{2g} \tag{9.10}$$

where:

H_{ah}	=	Head loss across an access hole, ft (m)
K_{ah}	=	Head loss coefficient
V_o	=	Outlet pipe velocity, ft/s (m/s)

Table 9.4 presents applicable coefficients (K_{ah}) and Figure 9.5 describes the angle of connection for the coefficients. With the estimated head loss, the designer estimates the initial pipe crown drop across an access hole (or inlet) structure to offset energy losses at the structure. The designer then uses the crown drop to establish the appropriate pipe invert elevations.

However, access hole and inlet energy losses are more complex than a simple proportional relationship to outlet velocity head and interior angle. Therefore, this represents a preliminary estimate only and ***does not apply*** to EGL calculations.

Table 9.4. Head loss coefficients.

Inlet/Access Hole	Structure Configuration	K_{ah}	Source
Inlet	Straight run, square edge	0.50	FHWA (2012a)
	Angled through 90°	1.50	--
Access Hole	Straight run	min ~ 0.15	ASCE (1992)
	90° angle	1.00	UDFCD (2001)
	120° angle	0.85	UDFCD (2001)
	135° angle	0.75	UDFCD (2001)
	157.5° angle	0.45	UDFCD (2001)

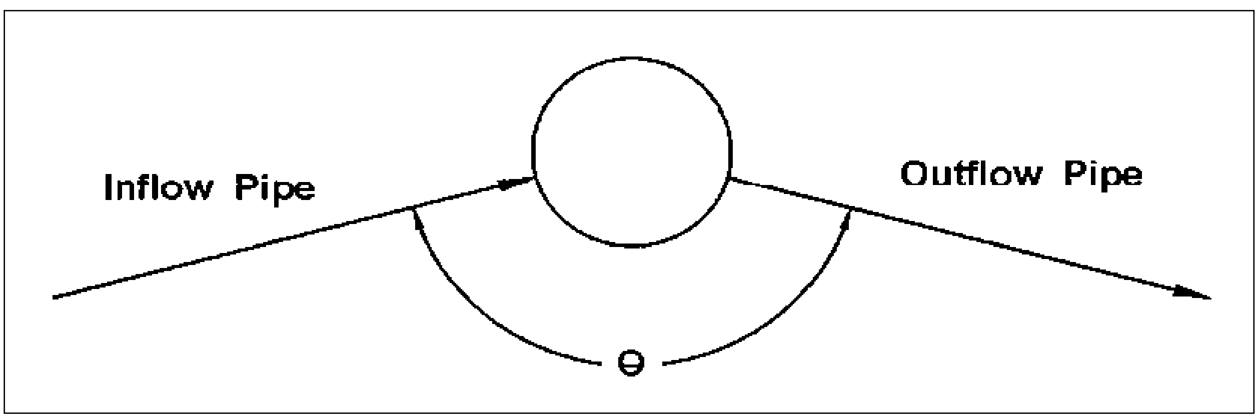

Figure 9.5. Interior angle for access holes.

9.1.6.7 FHWA Inlet and Access Hole Energy Loss

Other FHWA research has produced other approaches for estimating inlet and access hole energy losses to improve on the Approximate Method (Chang and Kilgore 1989, Chang et al. 1994, Kilgore 2006, Kerenyi et al. 2006). These also have limitations, some shared with the Approximate Method, including:

- Limited representation of very different hydraulic conditions within access holes when using or developing a single coefficient multiplied by an outlet velocity head.
- Difficulties producing reasonable results on some surcharged systems and systems with supercritical flows leaving the access hole.
- Under-prediction of calculated versus observed access hole flow depths.
- Dependence on relatively complex iterative methods.
- Development of problematic solutions in some situations resulting from limitations in these methods.

To address these issues, the FHWA developed a more comprehensive method for estimating losses in access holes and inlets. The resulting approach classifies access holes and their hydraulic conditions in a manner analogous to inlet control and full flow for culverts (Kilgore 2005, Kilgore 2006, Kerenyi et al. 2006). The method then applies equations in appropriate forms for the given classification. In addition to avoiding the limitations described above, this method has the following benefits:

- Uses hydraulically sound fundamentals for key computations (inlet control and full flow analogies) as a foundation for extrapolating the method beyond laboratory data.
- Incorporates an approach to handling surcharged systems with the full flow component of the method.
- Avoids problems associated with supercritical flows in outlet pipe by using a culvert inlet control analogy.
- Provides equivalent or better performance in predicting access hole water depth and inflow EGL on the extensive FHWA laboratory dataset.
- Presents a direct (non-iterative), simple, and manually verifiable computational procedure.

The method includes three fundamental activities (with terms described in Figure 9.6):

- Determines an initial access hole energy level (E_{ai}) based on inlet control (weir and orifice) or outlet control (partially full and full flow) equations.
- Adjusts the initial access hole energy level based on benching, inflow angle(s), and plunging flows to compute the final calculated energy level (E_a).
- Calculates the exit loss from each inflow pipe and estimating the energy grade line (EGL_o), which will then be used to continue calculations upstream.

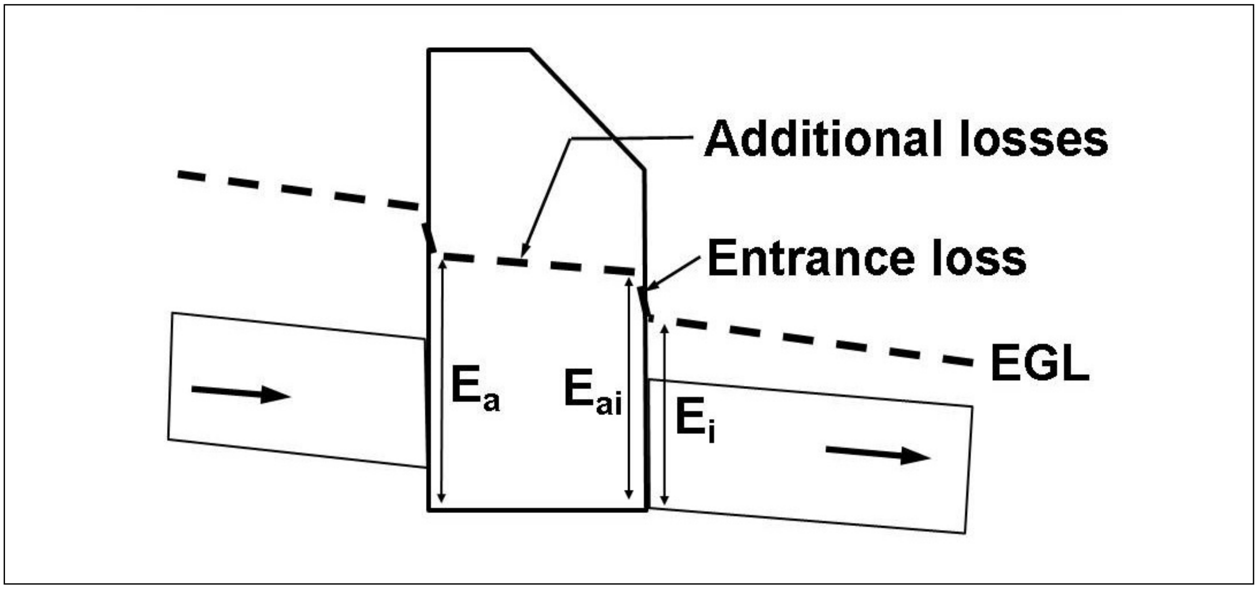

Figure 9.6. Access hole energy level definitions.

9.1.6.7.1 Determining Initial Access Hole Energy Level

Calculate the initial energy level in the access hole structure (E_{ai}) as the maximum of three possible conditions; these determine the hydraulic regime within the structure. The three conditions considered for the outlet pipe are:

- Outlet control condition: 1) full flow condition—a common occurrence with a surcharged storm drain system or if pipe capacity limits flow in the pipe; and 2) partially full flow condition—considered for an outlet pipe flowing partially full and in subcritical flow.
- Inlet control (submerged) condition: considered to possibly occur if the opening in the access hole structure to the outlet pipe is limiting and the resulting water depth in the access hole is sufficiently high that flow through the opening is treated as an orifice.
- Inlet control (unsubmerged) condition: considered to possibly occur if the flow control is also limited by the opening, but the resulting water level in the access hole involves treating the opening as a weir.

The method addresses one of the weaknesses of other methodologies: the large reliance on outflow pipe velocity. The full flow computation uses velocity head, but full flow only applies when the outflow pipe flows full. The two inlet control estimates depend only on discharge and pipe diameter. This improvement recognizes that velocity is not a reliable parameter because:

- In cases where supercritical flow occurs in the outflow pipe, the upstream condition at the access hole, not the velocity head, determines flow in the outflow pipe and the corresponding velocity head.
- In the laboratory setting used to derive most coefficients and methods, researchers do not directly measured velocity. They calculate it from depth and the continuity relationship, so. small errors in depth measurement can cause large variations in velocity head.
- Velocities produced in laboratory experiments result from localized hydraulic conditions, which do not necessarily represent the velocities calculated based on equilibrium pipe hydraulics in storm drain computations.

Seeking to obtain values for other elements of total outflow pipe energy head (E_i), such as outflow pipe depth (potential head) and pressure head, may exacerbate this issue. Consider E_i as the sum of the potential, pressure, and velocity head components:

$$E_i = y + \left(\frac{P}{\gamma}\right) + \left(\frac{V^2}{2g}\right) \tag{9.11}$$

where:

E_i	=	Outflow pipe energy head, ft (m)
y	=	Outflow pipe depth (potential head), ft (m)
(P / γ)	=	Outflow pipe pressure head, ft (m)
$(V^2 / 2g)$	=	Outflow pipe velocity head, ft (m)

Solving for equation 9.11 may cause a problem for certain conditions (e.g., where P cannot be assumed to equal atmospheric pressure). Designers can also determine E_i by subtracting the outflow pipe invert elevation (Z_i) from the outflow pipe energy grade line (EGL_i) (both known values) at that location:

$$E_i = EGL_i - Z_i \tag{9.12}$$

Knowing E_i serves as a check on the method. In circumstances with very low flow, the computations may result in access hole energy levels less than the outflow pipe energy head. In such cases, the designer sets the access energy level equal to the outflow pipe energy head.

To determine the initial estimate of the access hole energy level, the designer takes the maximum of the three values:

$$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu}) \tag{9.13}$$

where:

E_{aio}	=	Estimated access hole energy level for outlet control (full and partially full flow), ft (m)
E_{ais}	=	Estimated access hole energy level for inlet control (submerged), ft (m)
E_{aiu}	=	Estimated access hole energy level for inlet control (unsubmerged), ft (m)

In the **outlet control condition**, the downstream storm drain system limits discharge out of the access hole such that the outflow pipe either flows full or partially full in subcritical flow. The initial structure energy level (E_{aio}) estimate is:

$$E_{aio} = E_i + H_i \tag{9.14}$$

where:

H_i	=	Entrance loss assuming outlet control, calculated using equation 9.16, ft (m)

$$H_i = K_i \frac{V^2}{2g} \tag{9.15}$$

where:

K_i	=	Entrance loss coefficient = 0.2 (Kerenyi et al. 2006)

As described earlier, using the concept of outflow pipe energy head (E_i) and equation 9.13 allows the designer to estimate energy level directly without considering the water surface within the access hole. Defining a one-dimensional velocity head in a location with highly turbulent multi-directional flow presents a challenge.

Inlet control calculations employ a dimensionless ratio (the discharge intensity) adapted from the analysis of culverts. The discharge intensity (DI) parameter, or the ratio of discharge to pipe dimensions, describes the discharge intensity:

$$DI = \frac{Q}{A(gD_o)^{0.5}} \tag{9.16}$$

where:

A	=	Area of outflow pipe, ft^2 (m^2)
D_o	=	Diameter of outflow pipe, ft (m)

The **submerged inlet control condition** uses an orifice analogy to estimate the energy level (E_{ais}). Researchers derived the equation using data with discharge intensities less than or equal to 1.6 resulting in:

$$E_{ais} = D_o(DI)^2 \tag{9.17}$$

Laboratory analyses (Kilgore 2005, Kilgore 2006) revealed that **unsubmerged inlet control conditions** involve DIs in a 0.0 to 0.5 range (though the equation is not limited to this range). The unsubmerged inlet control condition uses a weir analogy to estimate the energy level (E_{aiu}):

$$E_{aiu} = 1.6D_o(DI)^{0.67} \tag{9.18}$$

9.1.6.7.2 Adjusting for Benching, Angled Inflow, and Plunging Inflow

Use the initial structure energy level 1 as a basis for estimating additional losses for: 1) discharges entering the structure at angles other than 180 degrees; 2) benching configurations; and 3) plunging flows entering the structure at elevations above the water depth in the access hole (treating flows entering a structure from an inlet or elevated incoming pipe as plunging flows).

Use the principle of superposition to estimate the effects of these conditions and apply them to the initial access hole energy level. This additive approach avoids a problem experienced in other methods where extreme values of energy losses are obtained when a single multiplicative coefficient takes on an extreme value.

The revised access hole energy level (E_a) equals the initial estimate (E_{ai}) modified by each of the three factors covered in this section:

$$E_a = E_{ai} + H_B + H_\theta + H_P \tag{9.19}$$

where:

H_B	=	Additional energy loss for benching (floor configuration), ft (m)
H_θ	=	Additional energy loss for angled inflows other than 180 degrees, ft (m)
H_P	=	Additional energy loss for plunging flows, ft (m)

E_a represents the level of the EGL in the access hole. However, if calculations result in an E_a less than the outflow pipe energy head (E_i), then set E_a equal to E_i.

Designers may also wish to know the water depth in the access hole (y_a). A conservative approach would use E_a as y_a for design purposes.

Benching tends to direct flow through the access hole, resulting in a reduction in energy losses. Figure 9.7 illustrates some typical bench configurations. Generally, from Figure 9.7 (a) to (e), the energy losses tend to decrease.

For access hole benching, the additional benching energy loss is:

$$H_B = C_B(E_{ai} - E_i) \tag{9.20}$$

where:

C_B = Energy loss coefficient for benching.

Table 9.5 summarizes benching coefficients. A negative value indicates water depth will be reduced rather than increased.

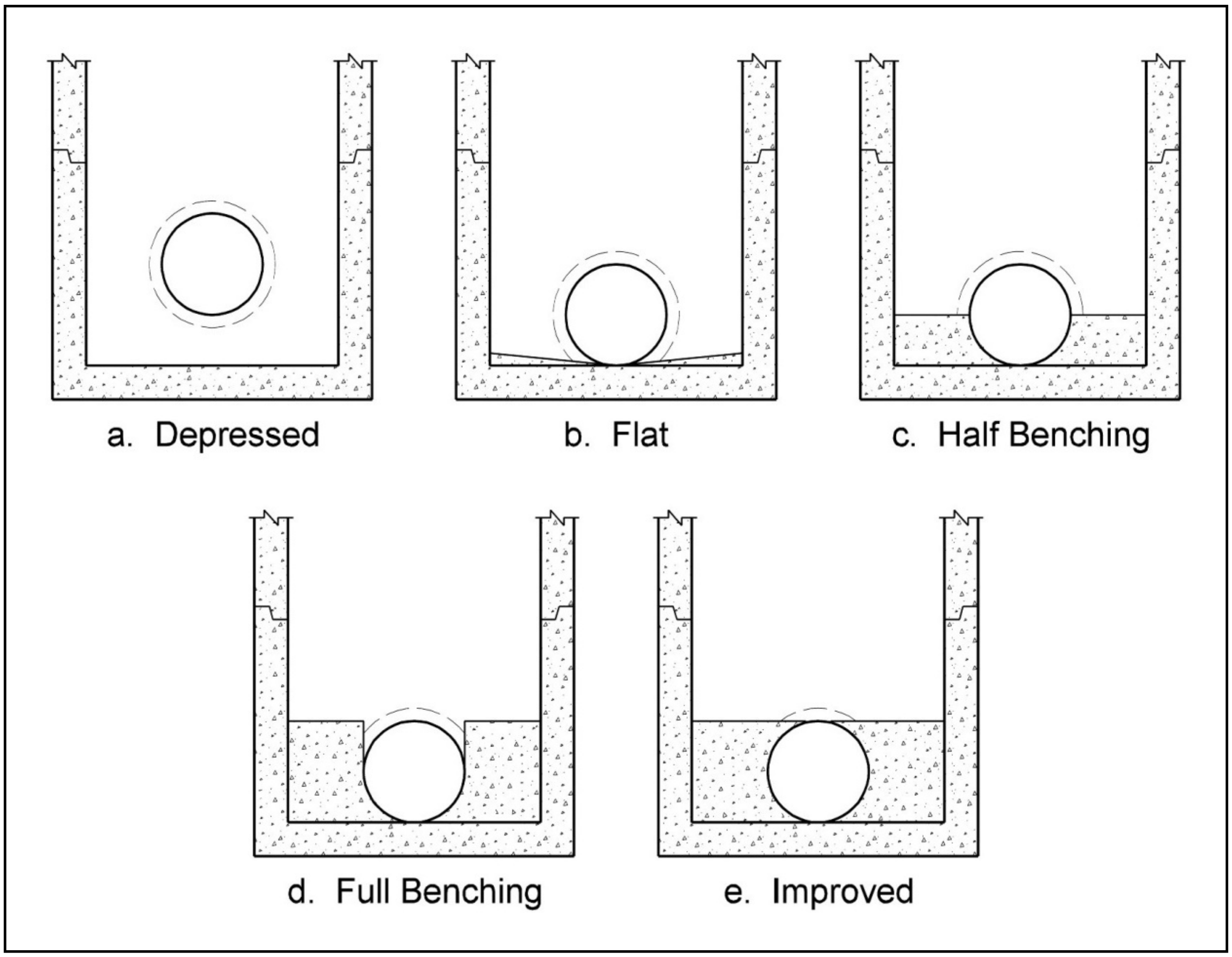

Figure 9.7. Access hole benching methods.

Table 9.5. Values for the coefficient, C_B.

Floor Configuration	Bench Submerged *	Bench Unsubmerged *
Flat (level)	-0.05	-0.05
Depressed	0.0	0.0
Half Benched	-0.05	-0.85
Full Benched	-0.25	-0.93
Improved	-0.60	-0.98

*A bench submerged condition has the properties of $(E_{ai}/D_o)>2.5$ and bench unsubmerged condition has the properties of $(E_{ai}/D_o)<1.0$. Use linear interpolation between the two values for intermediate values.

Address the effect of **skewed inflows** entering the structure by considering momentum vectors. To maintain simplicity, the contribution of all inflows contributing to structure and with a hydraulic connection (i.e., not plunging) resolves into a single flow weighted angle (θ_w):

$$\theta_w = \frac{\Sigma(Q_j \theta_j)}{\Sigma Q_j} \tag{9.21}$$

where:

Q_J = Contributing flow from inflow pipe, ft³/s (m³/s)

θ_J = Angle measured from the outlet pipe (180 degrees is a straight pipe)

Figure 9.8 illustrates the orientation of the pipe inflow angle measurement. The angle for each of the non-plunging inflow pipes references to the outlet pipe, so that the angle is not greater than 180 degrees. A straight pipe angle is 180 degrees. The summation only includes non-plunging flows as indicated by the subscript j. If all flows are plunging, set θ_w to 180 degrees.

Then, calculate an angled inflow coefficient (C_θ) as follows:

$$C_\theta = 4.5 \frac{\Sigma Q_j}{Q_0} \cos\left(\frac{\theta_w}{2}\right) \tag{9.22}$$

where:

Q_o = Flow in outflow pipe, ft³/s (m³/s)

The angled inflow coefficient approaches zero as θ_w approaches 180 degrees and as the relative inflow approaches zero. The additional angle inflow energy loss is:

$$H_\theta = C_\theta (E_{ai} - E_i) \tag{9.23}$$

Plunging inflow describes inflow (pipe or inlet) where the invert of the pipe (z_k) is greater than the estimated structure water depth (approximated by E_{ai}). The value of z_k represents the difference between the access hole invert elevation and the inflow pipe invert elevation.

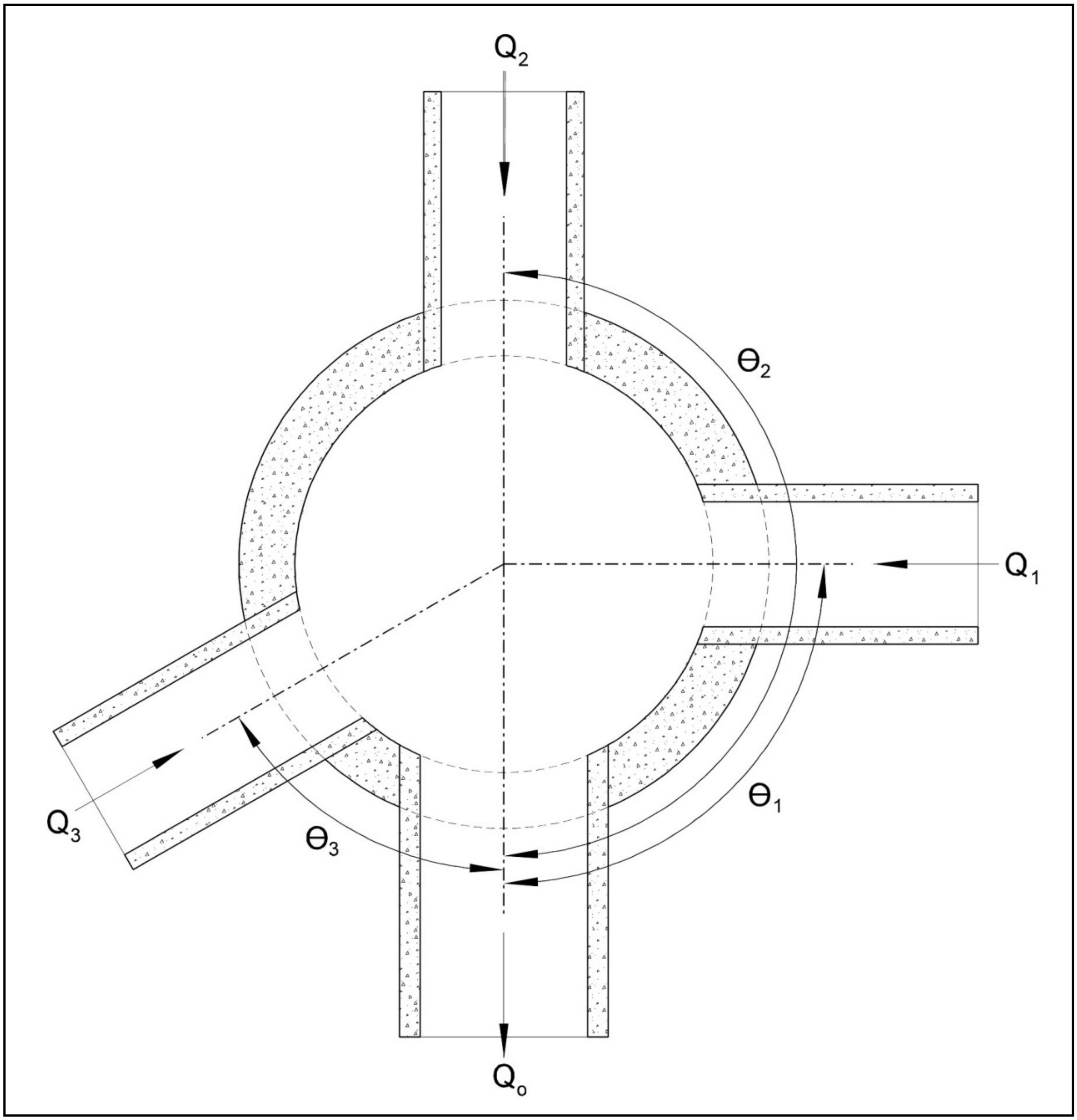

Figure 9.8. Access hole angled inflow definition.

The method determines a relative plunge height (h_k) for a plunging pipe (denoted by the subscript k) as:

$$h_k = \frac{(z_k - E_{ai})}{D_o} \tag{9.24}$$

This relative plunge height allows determination of the plunging flow coefficient (C_P):

$$C_P = \frac{\Sigma(Q_k h_k)}{Q_o} \tag{9.25}$$

As the proportion of plunging flows approaches zero, C_P also approaches zero. Equations 9.24 and 9.25 only apply to conditions where $z_k < 10D_o$. If $z_k > 10D_o$ set it to $10D_o$.

The additional plunging inflow energy loss is given by:

$$H_P = C_P(E_{ai} - E_i) \tag{9.26}$$

The method allows determination of the incremental benching (H_B), inflow angle (H_θ), and plunging energy (H_P) terms. However, incremental losses can be small, possibly even small enough to be "lost in the rounding." Alternatively, compute H_a by algebraically rearranging the benching, inflow angle, and plunging equations to yield:

$$H_a = (C_B + C_q + C_P)(E_{ai} - E_i) \tag{9.27}$$

Note that the value of H_a should always be positive. If the calculation yields a negative value, the designer sets H_a equal to zero. The revised access hole energy level is:

$$E_a = E_{ai} + H_a \tag{9.28}$$

If the computed estimate of E_a is less than the outlet pipe energy (E_i), use the higher of the two values for E_a.

Knowing the access hole energy level (E_a) and assuming the access hole invert (z_a) has the same elevation as the outflow pipe invert (z_i) allows determination of the access hole energy grade line (EGL_a):

$$EGL_a = E_a + Z_a \tag{9.29}$$

The potentially highly turbulent nature of flow within the access hole makes determination of water depth problematic. However, designers can reasonably use EGL_a as a comparison elevation to check for potential surcharging of the system. Research has shown the difficulty of determining velocity head within the access hole, even in controlled laboratory conditions (Kerenyi et al. 2006).

9.1.6.7.3 Calculating Inflow Pipe Exit Losses

In the final step, the designer calculates the EGL into each inflow pipe. **Non-plunging inflow pipes** are pipes with a hydraulic connection to the water in the access hole. The designer identifies inflow pipes operating under this condition when the revised access hole energy grade line (E_a) is greater than the inflow pipe invert elevation (Z_o). In this case, the inflow pipe energy head equals:

$$EGL_o = E_a + H_o \tag{9.30}$$

where:

EGL_o	=	Inflow pipe energy head, ft (m)
E_a	=	Revised access hole energy grade line, ft (m)
H_o	=	Inflow pipe exit loss, ft (m)

The subscript "o" is used for the inlet pipe because the equation represents losses at the outlet end of the inlet pipe. Calculate exit loss in the traditional manner using the inflow pipe velocity head since a condition of supercritical flow is not a concern on the inflow pipe. The equation is:

$$H_o = K_o \frac{V^2}{2g} \tag{9.31}$$

where:

K_o = Exit loss coefficient = 0.4, dimensionless (Kerenyi et al. 2006)

Water discharging from **plunging inflow pipes** freely falls into the access hole. For plunging pipes, take the inflow pipe energy grade line (EGL_o) as the energy grade line calculated from the inflow pipe hydraulics. In this case, EGL_o does not depend on access hole water depth and losses. Determining the EGL for the outlet of a pipe has already been described in Section 9.1.5.

For both the non-plunging and plunging cases, use the resulting EGL to continue computations upstream to the next access hole. At each access hole, repeat the three-step procedure of estimating: 1) entrance losses from entering the outlet pipe; 2) additional benching, angled inflow, and plunging losses within the access hole, and 3) exit losses leaving the inlet pipe and entering the access hole.

9.2 Design Considerations

Design criteria and other design considerations influence design. The following sections discuss several of these considerations, including design and check storm frequency, time of concentration and discharge determination, allowable high water at inlets and access holes, minimum flow velocities, minimum pipe grades, and alignment.

9.2.1 Design Storm Frequency

The storm drain conduit represents one of the most expensive and permanent elements within storm drainage systems. Storm drains typically remain in use longer than any other system elements. Once installed, DOTs face a high cost to increase capacity or repair the line. Consequently, designers carefully select the design flood frequency for projected hydrologic conditions to meet the need of the proposed facility both now and well into the future.

Most State highway agencies consider a 0.1 AEP frequency storm as a minimum for the design of storm drains on major highways in urban areas. Critical considerations for storm frequency selection include traffic volume, type and use of roadway, speed limit, flood damage potential, and the needs of the local community. DOTs typically design interstate highways subject to pluvial flooding for no less than the 0.02 AEP event. Section 5.1.1 and Table 5.1 discuss design frequencies in more detail.

Design of storm drains that drain sag points where runoff can only be removed through the storm drainage system uses a minimum 0.02 AEP frequency storm. Designers size the inlet at the sag point as well as the storm drain pipe leading from the sag point to accommodate this additional runoff. Designers accomplish this by computing the bypass occurring at each inlet during a 0.02 AEP rainfall and accumulating it at the sag point.

To minimize the bypass to the sag point, an alternative approach involves designing the upstream system for a 0.02 AEP design. Designers evaluate each case on its own merits, assessing the impacts and risk of flooding a sag point.

Following the initial design of a storm drainage system, prudence recommends evaluating the system using a higher check storm. A 0.01 AEP frequency storm is typically recommended for the check storm. Designers use the check storm to evaluate the performance of the storm drainage system and determine if the major drainage system is adequate to handle the flooding from a storm of this magnitude.

Maximum high water describes the allowable elevation of the water surface (HGL) during the design storm at any given point within a storm drain system including inlets, access holes, or other connections to the ground surface. Before initiating hydraulic evaluation, designers establish the maximum high water at any point to avoid impairment of the intended function of these locations and surface flooding.

9.2.2 Time of Concentration and Discharge

Designers most often use the Rational Method to determine design discharges for storm drain design. As discussed in Section 4.2.2.3, the time of concentration strongly influences the determination of the design discharge using the Rational Method.

Designers size each inlet and conduit using the drainage area and time of concentration applicable to that location. Because each component serves potentially unique drainage conditions, the resulting design flows are not additive moving downstream in a storm drainage system. Because of timing, design flows attenuate.

The time of concentration for each inlet reflects the unique drainage area contributing only to that inlet. Typically, this equals the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. Common design practice uses a minimum time of concentration, such as five minutes, when the total travel time to the inlet is less than the minimum.

Similarly, the time of concentration for each pipe reflects the unique drainage area contributing only to that pipe. Typically, this time consists of two components: 1) the time for overland and gutter flow to reach the first inlet, and 2) the time to flow through the storm drainage system to the upstream end of the pipe.

Design Conditions

Engineers conceptualize the conditions for which they design storm drains (discharge, depth and velocity of flow, time of contribution, etc.) to result in the most severe conditions for each element (conduit run, node, etc.) in a storm drain. The designer is well served to bear in mind that that set of conditions is highly unlikely to exist simultaneously. For example, the design depths of flow will likely never exist in the system at the same instant in time. In a system designed for pressure flow, that condition is unlikely to exist along the entire length of a system at any instant in time. Also, a system designed for pressure flow will transition from no flow at all, through open channel flow, to pressure flow, and back through open channel flow, and return to zero flow.

For each inlet or pipe, the longest time of concentration usually establishes the duration used in selecting the rainfall intensity value for the Rational Method. Designers watch for exceptions to the general application of the Rational Method when a sub-area (highly impervious) of a larger area under consideration could dominate the design flow. When this scenario occurs, the designer makes two sets of calculations and uses the larger value for design.

1. Calculate the design flow from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration. This is the general application of the Rational Method.
2. Estimate the time of concentration and associated rainfall intensity for the highly impervious sub-area alone. Calculate the design flow from the total area using this shorter time of concentration but also reducing the total area to reflect parts of the larger area that cannot reach the design point in the shorter time.

For the second calculation, the designer estimates the portion of the area relevant to the shorter time of concentration using:

$$A_c = A\,\frac{t_{c1}}{t_{c2}} \tag{9.32}$$

where:

A_c	=	Part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area, ac (ha)
A	=	Area of the larger primary area, ac (ha)
t_{c1}	=	Time of concentration of the smaller, less pervious, area, min
t_{c2}	=	Time of concentration associated with the larger primary area as is used in the first calculation, min

The second calculation uses the weighted C value that results from combining C values of the smaller less pervious tributary area and the area A_c.

9.2.3 Minimum Velocity and Grades

Maintaining a self-cleaning velocity in the storm drain system prevents deposition of sediments and subsequent loss of capacity. For this reason, designers typically develop storm drains to maintain full flow pipe velocities of 3 ft/s or greater. The minimum slope to achieve a design velocity can be computed from Manning's equation:

$$S = K_u\left[\frac{nV}{D^{0.67}}\right]^2 \tag{9.33}$$

where:

K_u	=	Unit conversion constant, 2.87 in CU (6.35 in SI)
D	=	Diameter, ft (m)

Conduit Slopes and Velocity

Ideally, the velocity in storm drain conduits will increase slightly with each conduit run, and when flowing as an open channel, flow will be subcritical. Drops in velocity in the downstream direction can result in settlement of sediment and debris both in conduits and in box appurtenances.

High velocities inside the conduits promote impact and abrasion damage from the sand, gravel, cobbles, and debris that invariably make their way into the system. This can shorten the lifespan of the system. The relative cost of conduit of incrementally different sizes is usually not an effective way to save costs.

Since the decision has been made to expend funds to install storm drain, savings on trenching and backfill are minimal when compared to the total cost of the system, and close attention to anticipated hydraulic qualities across the range of operating discharge pays dividends in both lifespan and maintenance effort.

9.2.4 Cover, Location, and Alignment

Designers consider both minimum and maximum cover limits in the design of storm drainage systems. Establishing minimum cover limits ensure the conduits have structural stability under live and impact loads. With increasing fill heights, dead load becomes the controlling factor.

Depth of Cover Under the Traveled Way

Before beginning the design of a storm drain system, designers will benefit from consulting with pavement engineers on the anticipated pavement structure depth, including subgrade compaction. When minimum cover standards allow storm drains to be relatively shallow underneath pavement sections and areas subject to compaction, a designer may choose to place the storm drain sufficiently deep to avoid damage from pavement installation and future maintenance.

For highway applications, designers typically maintain minimum cover depth of 3.0 ft where possible or follow applicable guidelines. Where designs cannot meet this criterion, designers evaluate the storm drains to determine if they are structurally capable of supporting imposed loads.

The *Handbook of Steel Drainage and Highway Construction Products* (AISI 1983) and the *Concrete Pipe Design Manual* (ACPA 1978) outline suggested procedures for analyzing loads on buried structures. Other materials (e.g., aluminum, plastics) have similar industry standard recommendations in other publications – although recommendations may vary according to the relevant material.

Fill and other dead loads control maximum cover limits. State highway agencies typically provide height of cover tables. These procedures (AISI 1983, ACPA 1978) apply to evaluating special fill or loading conditions.

Most local or state highway agencies maintain standards for storm drain location, typically placing them a short distance behind the curb or in the roadway near the curb. Designers typically prefer to locate storm drains on public property, although they may occasionally place storm drains within private property easements. Using private property easements adds cost to a project.

Where possible, designers place storm drains straight between access holes. However, designers may use curved storm drains where necessary to conform to street layout or avoid obstructions. Designers should not develop curved storm drains in pipe sizes smaller than 4.0 ft. For larger diameter storm drains deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Many suppliers have long radius bends; this represents the preferable means of changing direction in pipes 4.0 ft in diameter and larger. The radius of curvature specified should coincide with standard curves available for the type of material being used.

Vertical alignment of storm drains represents a key feature in the overall design. Elevation at the outfall restricts vertical storm drain alignment. Other governing factors include energy and hydraulic grade line management, utilities, and other obstructions. Gravity flow utilities such as sanitary sewers present the greatest challenge, but large water distribution lines, gas lines, underground electrical or communication lines, or any other preexisting utility may present an obstacle. Because utility adjustments are often costly and time-consuming, the storm drain design seeks opportunities to minimize disruption of existing utilities.

9.2.5 Maintenance

Design, construction, and maintenance closely relate to each other. Storm drain maintenance represents a critical consideration during both design and construction. Common maintenance problems associated with storm drains include debris accumulation, sedimentation, erosion, scour, piping, roadway and embankment settlement, and conduit structural damage.

Debris and sediment frequently accumulate in storm drains, particularly during construction. Designs for a minimum full flow velocity as discussed in Section 9.2.3 reduce the likelihood of sedimentation. Providing access hole spacing in accordance with the criteria presented in Chapter 8 ensures adequate access for cleaning.

DOTs also frequently report the maintenance issue of scour at storm drain outlets. Riprap aprons or energy dissipators at storm drain outlets can minimize scour.

Following appropriate design and installation specifications avoids piping, roadway and embankment settlement, and conduit structural failure. These problems, when they occur, usually relate to poor construction. Tight specifications along with thorough construction inspections can help reduce these problems.

Even in a properly designed and constructed storm drainage system, proper functioning depends on a comprehensive program for storm drain maintenance. Regular inspections detail long-term changes and will indicate appropriate maintenance to ensure safe and continued operation of the system. An appropriate maintenance program includes both periodic inspections and supplemental inspections following storm events. Since storm drains exist almost entirely underground, inspection of the system is more difficult than surface facilities. The *Culvert Inspection Manual* (FHWA 1986) provides information for inspecting storm drains or culverts.

9.3 Preliminary Design

Designers create preliminary storm drain plans following a multi-step procedure. This procedure assumes that each conduit will be initially designed to flow full under gravity conditions using the Rational Method. For final design, the design analyzes the HGL and EGL as described in Section 9.4. Designers usually implement these procedures using widely available software computational tools.

Step 1. Prepare a working plan for the layout and profile of the storm drain system.

The designer compiles the following initial design information:

- Location of storm drains.
- Flow direction.
- Location of access holes and other structures.
- Number or label assignments to each structure.
- Location of all existing utilities (water, sanitary sewer, gas, underground cables, etc.).

Step 2. Determine the hydrologic parameters to each inlet.

Collect the hydrologic parameters for the drainage areas tributary to each inlet needed for the Rational Method to estimate the design discharge starting with the upstream most storm drain run.

- Run length.
- Incremental drainage area to the inlet at the upstream end of the storm drain run under consideration.
- The runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases, a composite runoff coefficient will need to be computed.

- Inlet time of concentration.

Step 3. Size and locate the pipes.

Using the information from step 3, compute the following for each pipe starting with the upstream most pipe:

- Total area draining to the pipe adding the incremental area to the upstream areas that drain to the pipe.
- Cumulative time of concentration to the upstream end of the pipe. System time of concentration. For the upstream most storm drain run this value will be the same as the value for the inlet. For all other pipe runs this value is computed comparing the cumulative (system) time of concentration to the local inlet time of concentration. The cumulative time of concentration is estimated by adding the previous cumulative time of concentration with the upstream pipe travel time. (See Section 9.2.2 for a general discussion of times of concentration.)
- Select the larger of the inlet time of concentration (step 2) and the cumulative time of concentration.
- Rainfall intensity from an intensity-duration-frequency (IDF) curve for the longer time of concentration.
- Design discharge, Q, using the Rational Method.
- Initial pipe slope. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.
- Pipe size. Size the pipe using relationships presented in Section 9.1.3 to convey the discharge by varying the slope and pipe size as necessary. Initially, designers size the storm drain so it flows as close as possible to full flow, or a desirable d/D ratio, but not under pressure. Nominal sizes, usually in 6” increments, will be used. Pipe on 3” increments is shown in catalogs, but sizes other than 6” increments (e.g., 15” or 21”) is usually more costly than the next larger size 6” increment. The designer decides whether to go to the next larger size and have part full flow or whether to go to the next smaller size and have pressure flow.
- Full flow capacity for the pipe. Compute the full flow capacity of the selected pipe using equation 9.2.
- Full and design velocity. Compute the full flow and design flow velocities (if different) in the conduit. If the pipe is flowing full, the velocities can be determined from V = Q/A. If the pipe is not flowing full, the velocity can be determined from calculations.
- Conduit travel time. Calculate the travel time in the pipe section by dividing the pipe length by the design flow velocity.
- Crown drop. Calculate an approximate addition drop or vertical offset beyond the initial pipe slope at the structure to off-set potential structure energy losses using equation 9.10 introduced in Section 9.1.6.6.
- Invert elevations. Compute the pipe inverts at the upstream and downstream ends of the pipe.

This process results in a preliminary layout of the storm drain system. Section 9.4 describes the methodology for analyzing the system for the HGL and EGL.

9.4 Hydraulic and Energy Grade Line Evaluation

This section presents a step-by-step procedure for calculation of the EGL and the HGL using the energy loss method. For most storm drainage systems, computer methods are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process so that they can better interpret the output from computer generated storm drain designs. Figure 9.9 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following procedure can be used as a template to construct a spreadsheet to compute the EGL and HGL.

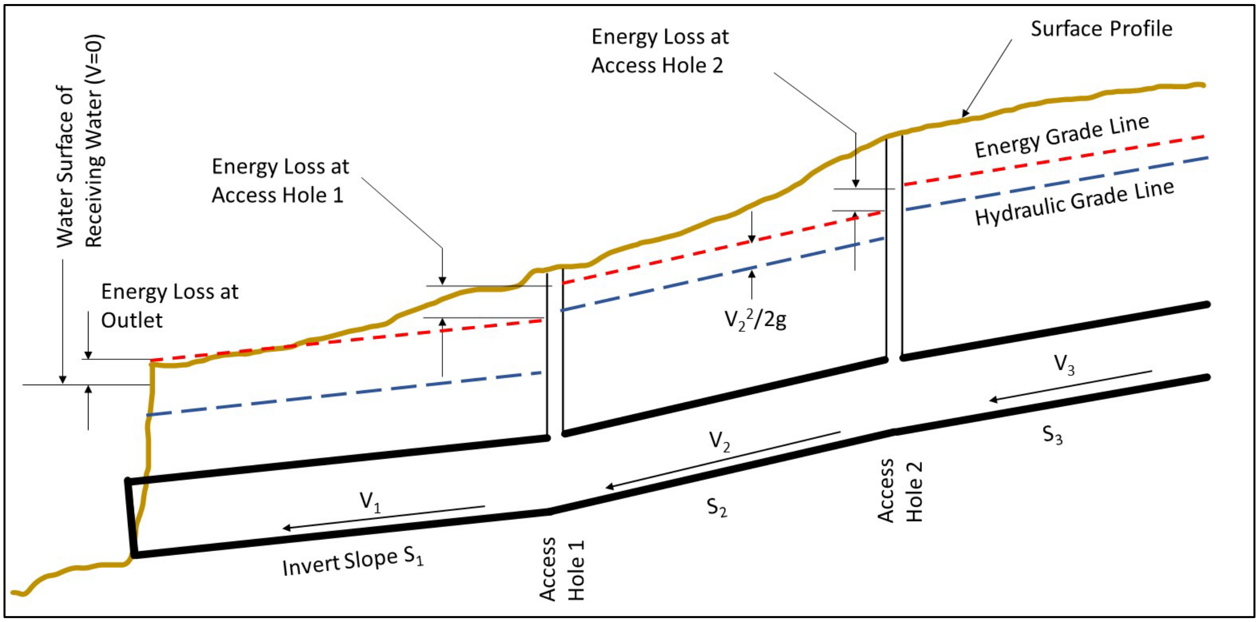

Figure 9.9. Energy and hydraulic grade line illustration.

EGL computations begin at the outfall and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer advances to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime. The steps continue from the preliminary design in Section 9.3 with step 4.

Step 4. Determine the tailwater elevation at the storm drain outfall.

The designer estimates the water surface elevation at the discharge point of the storm drain system. This may be at a larger body of water such as a pond, lake, or tidally influenced receiving point. In these cases, the receiving water is assumed to have no velocity. The tailwater elevation may also be determined by a stream or the critical depth in the discharge pipe if the receiving elevation is lower than the pipe invert elevation. Estimating the tailwater elevation may involve coincident flow calculations, tidal condition assumptions, reservoir/detention pond routing assumptions, or other, unrelated calculations.

Estimate critical depth in the conduit for the design discharge and calculate the elevation of critical depth at the outfall. If the water surface elevation in the receiving water is below that elevation, critical depth will dominate outfall conditions. If it is above critical depth, the receiving water will exert some influence on the energy state at the outfall.

From the initial elevation and the corresponding area of flow and discharge, compute the total energy at the outfall. The HGL elevation is the initial elevation, the EGL is that, plus the energy head.

Step 5. Estimate the HGL and EGL at the downstream end of the next pipe.

Beginning with the outfall pipe, estimate the HGL and EGL at the downstream end of the pipe, but just inside the pipe. The designer determines this from the tailwater elevation, physical pipe characteristics and design flow. This step is repeated for each pipe in the storm drain system. For subsequent pipes the designer uses the energy state in the downstream structure as the tailwater condition.

The designer estimates the EGL at the downstream end based on one of the conditions comparing the pipe with the downstream EGL shown in Table 9.6. All cases apply for mildly sloped pipes. Cases A and E also apply to steep pipes. For the outfall pipe discharging to a receiving water, the tailwater elevation is substituted for EGL_a.

Table 9.6. Case conditions and downstream EGL estimates.

Case	Situation	EGL_o Estimate
A	$EGL_a \geq TOC_o$ (submerged)	EGL_a + exit loss
B	$TOC_o \geq EGL_a > BOC_o + y_n$	BOC_o + velocity head + exit loss
C	$BOC_o + y_n \geq EGL_a > BOC_o + y_c$	BOC_o + max(EGL_a + exit loss or normal depth + velocity head)
D	$BOC_o + y_c \geq EGL_a > BOC_o$	BOC_o + normal depth + velocity head
E	$BOC_o \geq EGL_a$ (plunging pipe)	BOC_o + normal depth + velocity head

Step 6. Estimate the HGL and EGL at the upstream end of the outfall pipe.

Compute the elevation of the HGL at the upstream end of the conduit run (next structure upstream). The HGL is the depth of flow in the conduit plus change in elevation of the conduit (slope times length).

Table 9.7 summarizes potential flow conditions within the conduit that determine calculation of the upstream pipe end EGL and HGL. For conditions A, B, and C, the HGL and EGL are computed using pipe friction and minor losses. For condition D, the designer computes the velocity head at normal depth. Added to the HGL elevation, that is the EGL elevation at the upper end of the conduit.

Table 9.7. Flow conditions at the upstream end of a conduit.

Condition	Situation	Flow Condition
A	$HGL_i \geq TOC_i$	Full flow (surcharge)
B	$TOC_i \geq HGL_i > BOC_i + y_n$ and $TOC_i \geq HGL_i > BOC_i + y_c$	Conduit not full but conditions are downstream-controlled
C	$BOC_i + y_n \geq HGL_i > BOC_i + y_c$	Subcritical partial flow conditions
D	$BOC_i + y_c \geq HGL_i$	Supercritical partial flow conditions

Step 7. Calculate EGL and HGL in a structure.

Most structures include one outlet pipe and one or more inlet pipes, except at the most upstream inlet structures. At each structure, using the methods shown in Section 9.1.6, calculate the energy losses through the structure. The HGL at the structure will be the HGL at the upstream end of the outflow conduit (that was just calculated) and the losses. The HGL is calculated for the outflow discharge.

Consider each lateral or trunk line entering the structure. The HGL and EGL at the structure will be estimated according to method in Section 9.1.6.7.

Iterative Solvers in Modern Spreadsheets

The spreadsheet programs currently in most common use exhibit built-in iterative solver functions. They can vary from simple univariate solvers to very complex multivariate solvers with the ability to set conditions on the solution. Judicious use of these features greatly reduces the work required to perform many of the calculations shown herein.

In addition, macro programming languages could be of great benefit to the user who takes the time to learn how to use them.

Step 8. Repeat steps 5, 6, and 7 for all pipes and structures in the storm drain system.

Continuing to work upstream, compute the HGL and EGL at the downstream end of the next pipe (step 5) and the upstream end of each pipe based on the HGL and EGL of the downstream end (step 6). Calculate the HGL and EGL in the next upstream structure (step 7). For a terminal structure, e.g., upstream inlet, no additional computations are needed for that storm drain branch. The designer continues with additional branches, if any.

Both trunk lines and laterals often involve hydraulic drops into a junction as energy controls or to control the depth of trenching. It is desirable to avoid conduit runs that exhibit supercritical flow. Hydraulic jumps inside of conduits are difficult to predict and can result in “blowouts” or damage to structures. Hydraulic drops are preferable, and do not result in the shortening of travel time and the associated increase in rainfall intensity of high conduit velocities.

Step 9. Examine the results.

Compare EGL elevations to topographic elevations. Be sure that the ground surface is above the EGL. Also check that the exterior of the top of conduits is below the pavement structure, adjacent roadside features, and other appurtenances.

The designer now has a complete set of calculations to validate against applicable design criteria. If the designer believes that the system can be improved by changes in conduit diameters, invert elevations, etc., the designer makes the appropriate changes and repeats the process.

Example 9.2:	Storm drain design.
Objective:	Determine appropriate pipe sizes and inverts for a storm drain system and evaluate the HGL for the system configuration.

Given: Table 9.8 summarizes the rainfall data.

The pipes are RCP with a Manning’s n value of 0.013.

Minimum design pipe diameter is 18 inches (460 mm) for maintenance purposes.

Minimum cover is 3 ft (0.91 m).

Table 9.8. Intensity-duration data for example.

Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/h)	7.1	5.9	5.1	4.5	3.5	3.0	2.6	2.4	1.4

Step 1. Prepare a working plan for the layout and profile of the storm drain system.

Figure 9.10 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference.

Step 2. Determine the hydrologic parameters for each inlet.

Table 9.9 provides the drainage area information to compute the inlet design flows. Use the Rational Method to compute inlet design flows.

Table 9.9. Drainage area information for example.

Structure Number	Structure Type	Drainage Area (ac)	Runoff Coefficient	Time of Concentration (min)
40	Inlet	0.64	0.73	3
41	Inlet	0.35	0.73	2
42	Inlet	0.32	0.73	2
43	Access hole	n/a	n/a	n/a
44	Outlet	n/a	n/a	n/a

Step 3. Size and locate the pipes.

Figure 9.11 shows the roadway profile grade and ground elevations used as a starting point for the vertical pipe alignment. Size the pipes assuming 100 percent capture by the inlets.

Step 3a. Size and locate the upstream most pipe (pipe 40-41).

Estimate the design flow for pipe connecting structures 40 and 41. From Table 9.9 for inlet number 40:

A = 0.64 ac

C = 0.73

Inlet time of concentration = 3 min (use minimum time of 5 min)

From Table 9.8:

I = 7.1 in/h

Q = CIA = (0.73)(7.1)(0.64) = 3.3 ft^3/s

Pipe slope = 0.03 ft/ft (initially calculated from topography).

Estimate pipe diameter assuming full flow using equation 9.2:

$D = [(Q\,n)/(K_Q\,S_o^{0.5})]^{0.375} = [(3.3)(0.013)/\{(0.46)(0.03)^{0.5}\}]^{0.375} = 0.8$ ft (9.6 inches)

However, minimum pipe diameter is 18 inches. Use 18-inch pipe.

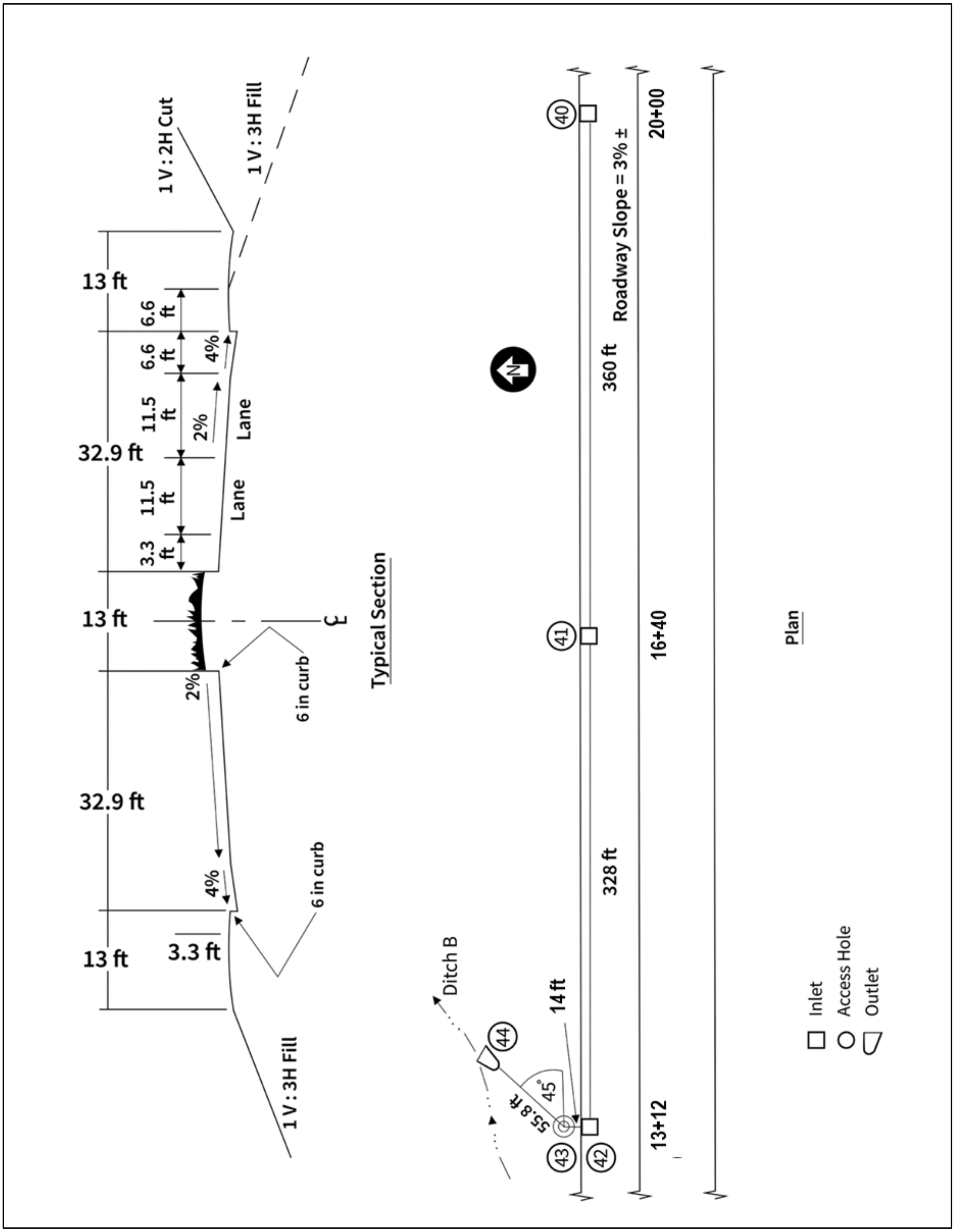

Figure 9.10. Roadway plan and section for example.

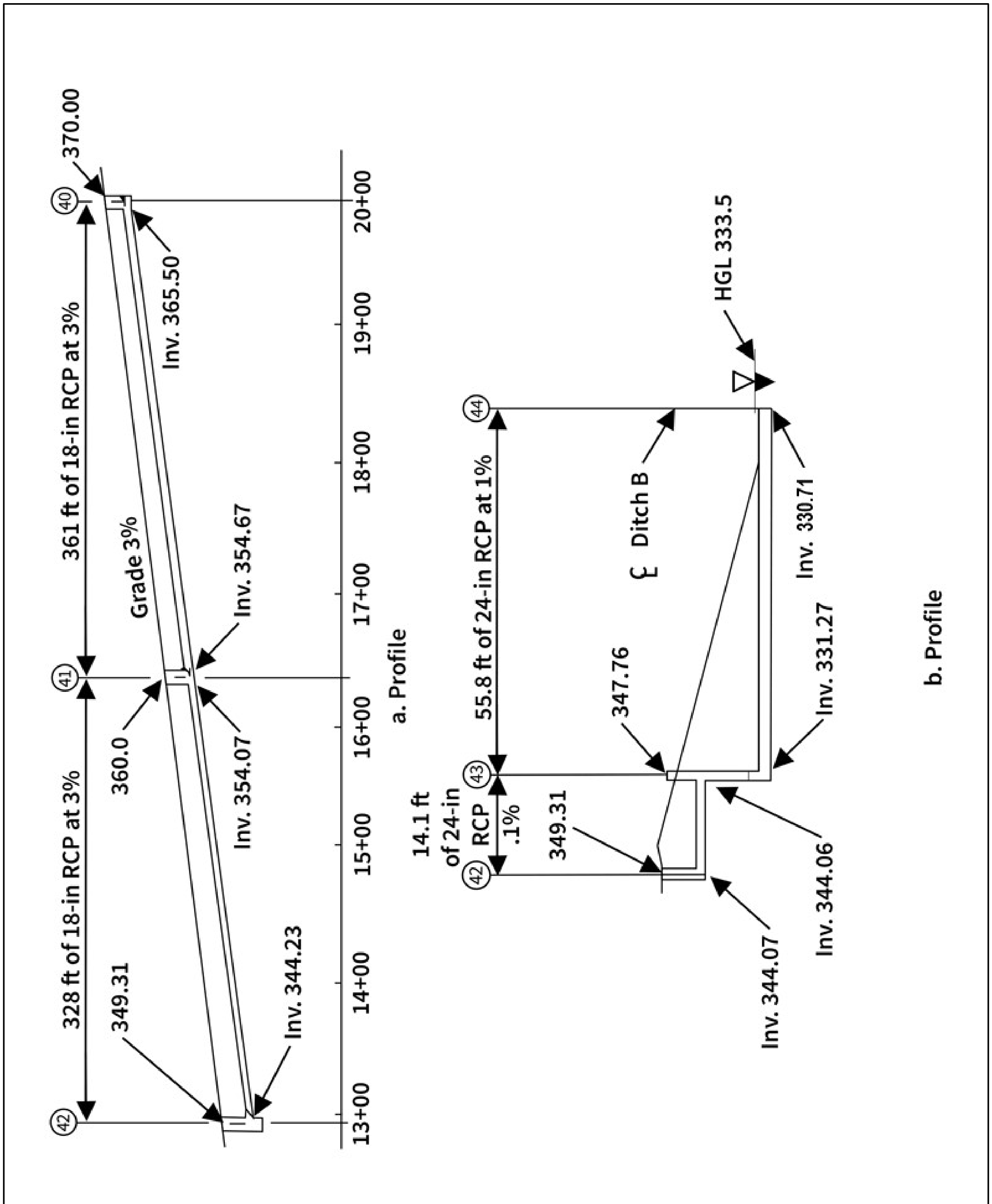

Figure 9.11. Storm drain profiles for example.

Calculate full pipe capacity using equation 9.2:

$Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5} = 18.1$ ft^3/s

Calculate full pipe velocity using equation 9.1:

$V_f = (0.59/0.013)(1.5)^{0.67}(0.03)^{0.5} = 10.3$ ft/s

Calculate the design velocity for a pipe flowing partially full. For this example, use the following approximation (software will calculate V_f from Manning's equation for a partially full circular section):

0.75 V_f if Q is less than or equal to 0.5 Q_f

1.1V_f if Q is greater than 0.5 Q_f

Q < 0.5 Q_f, therefore, V = (0.75)(10.3) = 7.73 ft/s

Calculate the travel time in the pipe to estimate the cumulative time of concentration to downstream pipes:

t_s = L/V = 361 / 7.73 / 60 = 0.8 min; use 1 min

Since inlet 40 is the most upstream structure no crown drop is needed, therefore, the crown drop is 0 ft.

Calculate the upstream invert elevation:

$Z_{u/s}$ = ground elevation - minimum cover - pipe diameter = 370.0 – 3.0 – 1.5 = 365.5 ft

Calculate the downstream invert elevation:

$Z_{d/s} = Z_{u/s} – (S_0)(L)$ = 365.5 – (0.03)(361.0) = 354.67 ft

Validate that pipe has adequate cover at the downstream end:

From Figure 9.11, the ground elevation is 360.0 ft.

Invert elevation + minimum cover + pipe diameter = 354.67 + 3.0 + 1.5 = 359.17 ft; less than ground elevation – OK

Pipe Wall Thickness and Cover

Designers select pipe invert elevations to be no higher than the ground elevation less the sum of the minimum cover, pipe wall thickness, and pipe diameter. This example assumes that the pipe wall thickness is implicitly included in the minimum cover. For jurisdictions where this is not the case, the wall thickness is explicitly added to the invert elevation computation.

Step 3b. Size and locate the next pipe (pipe 41-42).

Estimate the design flow for pipe connecting structures 41 and 42 considering all upstream contributing areas. From Table 9.9:

A = 0.64 + 0.35 = 0.99 ac

C = 0.73

Inlet time of concentration = 2 min

System time of concentration = 3 + 1 = 4 min (use minimum time of 5 min)

From Table 9.8:

I = 7.1 in/h

Q = CIA = (0.73)(7.1)(0.99) = 5.1 ft^3/s

$D = D_{min}$ = 1.5 ft

$V = 8.7$ ft/s

Travel time = 0.6 min, use 1 min

Estimate crown drop using equation 9.10, loss coefficient from Table 9.4 equal to 0.5 for an inlet – straight run:

$H_{ah} = (K_{ah})(V^2 / 2g) = (0.5)(8.7^2 / 64.4) = 0.6$ ft

Calculate the upstream invert:

$Z_{u/s} = 354.67 – 0.6 = 354.07$ ft

Calculate the downstream invert:

$Z_{d/s} = Z_{u/s} – (S_0)(L) = 354.07 – (0.03)(328) = 344.23$ ft

Step 3c. Size and locate the next pipe (pipe 42 – 43).

Estimate the design flow for pipe connecting structures 42 and 43 considering all upstream contributing areas. From Table 9.9:

A = 0.64 + 0.35 + 0.32 = 1.31 ac

C = 0.73

Inlet time of concentration = 2 min

System time of concentration = 3 + 1 + 1 = 5 min (use minimum time of 5 min)

From Table 9.8:

I = 7.1 in/h

Q = CIA = (0.73)(7.1)(1.31) = 6.75 ft^3/s

D = 1.96 ft = use 2.0 ft

V = 2.6 ft/s

Travel time = 0.09 min, use 0 min

Estimate crown drop using equation 9.10, loss coefficient from Table 9.4 equal to 1.5 for inlet – 90° angle:

$H_{ah} = (K_{ah})(V^2 / 2g) = (1.5)(2.6^2 / 64.4) = 0.16$ ft

Calculate the upstream invert:

$Z_{u/s} = 344.23 – 0.16 = 344.07$ ft

Calculate the downstream invert:

$Z_{d/s} = Z_{u/s} – (S_0)(L) = 344.07 – (0.001)(14) = 344.06$ ft

Step 3d. Size and locate the next pipe (pipe 43-44).

Q = 6.75 ft^3/s (no additional CA accumulated and no addition to the system time of concentration)

D = 1.27 ft = use 2.0 ft (to prevent possible clogging, do not reduce conduit size)

V = 6.1 ft/s

Travel time = 0.15 min, use 0 min

$H_{ah} = (K_{ah})(V^2 / 2g) = (1.5)(6.1^2 / 64.4) = 0.87$ ft

Since this is the downstream most pipe, set the downstream invert:

$Z_{d/s}$ =330.71 ft (outfall elevation for the pipe)

Calculate the upstream invert:

$Z_{u/s} = Z_{d/s} + (S_0)(L) = 330.71 + (0.01)(55.8) = 331.27$ ft

This invert results in a crown drop = 344.06 – 331.27 = 12.79 ft (which is greater than 0.87 ft)

Figure 9.11 summarizes the preliminary pipe lengths, diameters, and inverts for the example.

Step 4. Determine the tailwater elevation at the storm drain outfall.

For the EGL and HGL computations, start at the downstream end at the outfall (structure 44). Computations proceed in the upstream direction. From Figure 9.11:

HGL_{TW} = downstream pool water surface elevation = 333.5 ft

EGL_{TW} = 333.5 ft (assume no velocity head in the pool)

Step 5. Estimate the HGL and EGL at the downstream end of pipe 43-44 (the outfall pipe).

BOC_o = 330.71 ft

TOC_o = 330.71 + 2.0 ft = 332.71 ft

Determine the applicable case from Table 9.6:

Is the downstream end of the pipe submerged ($EGL_{TW} > TOC_o$)?

333.5 ft > 332.71 ft is true. Pipe is submerged (case A). Use full pipe flow.

Estimate the energy loss exiting the outfall pipe:

V = Q / A = 6.75 ft^3/s / 3.14 ft^2 (area of a 2 ft diameter circle) = 2.15 ft/s

$V^2 / 2g = (2.15^2)\ (64.4) = 0.07$ ft

Exit loss for an outfall:

$H_o = (1.0)V^2/2g = (1.0)\ (0.07) = 0.07$ ft

$EGL_o = TW + H_o$ = 333.5 ft + 0.07 ft = 333.57 ft

$HGL_o = EGL_o - V^2/2g$ = 333.57 ft – 0.07 ft = 335.50 ft

Step 6. Estimate the HGL and EGL at the upstream end of pipe 43-44.

At structure 43 for the pipe from structure 43 to 44 (pipe 43-44):

Pipe length, L = 55.8 ft

BOC_i = 331.27 ft

$TOC_i = BOC_i + D$ = 331.27 ft + 2.0 ft = 333.27 ft

$S_f = [(Qn)/(KQD^{2.67})]^2 = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^{\,2} = 0.00090$ ft/ft

$H_f = S_f\, L = (0.00090)\ (55.8) = 0.05$ ft

No other losses in conduit: $H_b, H_c, H_e, H_j = 0.0$

$EGL_i = EGL_o$ + pipe loss = 333.57 ft + 0.05 ft = 333.62 ft

$HGL_i = EGL_i - V^2/2g$ = 333.62 ft - 0.07 ft = 333.55 ft

$E_i = EGL_i - BOC_i$ = 333.62 ft – 331.27 ft = 2.35 ft

Determine the flow condition at the upstream end of the outflow pipe from Table 9.7:

HGL_i (333.55 ft) > TOC_i (333.27 ft), therefore, pipe is flowing full (surcharge) (condition A), and losses are carried upstream as computed.

Step 7. Calculate EGL and HGL at structure 43.

$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu})$

Compute E_{aio} using equations 9.14 and 9.15:

$H_i = (K_o)(V_o^2/2g) = (0.2)(0.07) = 0.014$ ft

$E_{aio} = E_i + H_i = 2.35$ ft + 0.014 ft = 2.364 ft

Discharge intensity for outflow pipe (pipe 43-44):

$DI = Q / [A(Dg)^{0.5}] = 6.75 / [((\pi/4)(2.0)^2)((2.0)(32.2))^{0.5}] = 0.268$

Compute E_{ais} using equation 9.17:

$E_{ais} = (1.0)(DI)\ 2\ (D) = (0.268)\ 2\ (2.0) = 0.14$ ft

Compute E_{aiu} using equation 9.18:

$E_{aiu} = (1.6)(DI)\ 0.67\ (D) = 1.6(0.268)\ 0.67\ (2.0) = 1.32$ ft

Select maximum (to two decimal places): $E_{ai} = 2.36$ ft

Compute E_a:

For benching:

$C_B = -0.05$, $E_{ai} / D < 1$, therefore, assume flat bench, unsubmerged access hole

For angled inflow:

$C_\theta = 0.0$ because the flow is plunging and $\theta_w = 180$ even though there is a 45-degree bend

For plunging flow:

$z_k = 334.06$ ft - 331.27 ft = 12.79 ft

$h_k = (z_k - E_{ai}) / D_o = (12.79 – 2.29)/(2.0) = 5.25$

$C_p = (\Sigma(Q_k h_k))/Q_o = ((6.75)(5.25))/(6.75) = 5.25$ (only one inflow)

Check to ensure net energy. Is $E_{ai} > E_i$? Since 2.36 > 2.35 OK

$H_a = (E_{ai} - E_i)(C_B + C_\theta + C_P) = (2.36 – 2.35)(-0.05 + 0.0 + 5.25) = 0.05$ ft (greater than 0)

$E_a = E_{ai} + H_a = 2.36$ ft + 0.05 ft = 2.41 ft

$EGL_a = E_a + BOC_i = 2.41$ ft + 331.27 ft = 333.68 ft

Ground elevation = 347.76 ft. Since 347.76 ft > 333.68 ft HGL is OK

Step 8. Repeat steps 5, 6, and 7 for all pipes and structures in the storm drain system.

Continue upstream with all pipes entering the structure analyzed in the previous step. In this example, only one pipe enters structure 43. Go to step 5 for this pipe.

Step 5 (for next pipe). Estimate the HGL and EGL at the downstream end of pipe 42-43.

One pipe enters structure 43 from structure 42 (pipe 42-43):

D = 2.0 ft

$Q = 6.75\ ft^3/s$

$L = 14.1\ ft$

$BOC_o = 344.06\ ft$

$TOC_o = BOC_o + D = 344.06\ ft + 2.0\ ft = 346.06\ ft$

Determine the applicable case from Table 9.6:

Is the downstream end of the pipe submerged ($EGL_a > TOC_o$)?

EGL_a (333.68 ft) > TOC_o (346.06 ft) is not true. Pipe outlet is not submerged (case A).

Is the pipe plunging ($EGL_a < BOC_o$)?

EGL_a (333.68 ft) < BOC_o (344.06 ft) is true. Pipe is not plunging (case E)

$V = 2.6\ ft/s$ (part full flow)

$Q/Q_f = 6.75 / 7.12 = 0.95$

$y_n = 1.56\ ft$

$V^2/2g = (2.6)^2/(2)(32.2)= 0.10\ ft$

$y_c = 0.80\ ft$

$H_o = 0.0$

$EGL_o = (BOC_o + y_n) + V^2/2g = 344.06\ ft + 1.56\ ft + 0.10\ ft = 345.72\ ft$

$HGL_o = EGL_o – V^2/2g = 345.72 – 0.10 = 345.62\ ft$

Step 6. Estimate the HGL and EGL at the upstream end of pipe 42-43.

$BOC_i = 344.07\ ft$

$TOC_i = BOC_i + D = 344.07\ ft + 2.0\ ft = 346.07\ ft$

Pipe not full, so S_f = pipe slope. However, recall that the D/S conduit invert was dropped 2-3 inches, changing the original design slope.

$S_f = (344.07 - 344.06) / 14.1 = 0.0007\ ft/ft$

$H_f = S_f\ L = (0.0007)(14.1) = 0.01\ ft$

No other losses in conduit: H_b, H_c, H_e, $H_j = 0$

Total pipe loss = 0.01 ft

$EGL_i = EGL_o$ + total pipe loss = 345.72 ft + 0.01 ft = 345.73 ft

$HGL_i = EGL_i - V^2/2g = 345.73\ ft - 0.10\ ft = 345.63\ ft$

$E_i = EGL_i - BOC_i = 345.73\ ft - 344.07 ft = 1.66\ ft$

Determine the flow condition at the upstream end of the outflow pipe from Table 9.7:

HGL_i (345.63 ft) < TOC_i (346.07 ft), therefore not full flow (surcharged) (condition A).

$y_n = 1.56\ ft$

$y_c = 0.8\ ft$

$BOC_i + y_n$ (344.07+1.56 = 345.63 ft) ≥ HGL_i (345.63 ft) > $BOC_i + y_c$ (344.07+0.8 = 344.87 ft), therefore subcritical partial flow conditions (condition C). Therefore, losses are carried upstream as computed.

Step 7. Calculate EGL and HGL at structure 42.

$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu})$

Compute E_{aio} using equations 9.14 and 9.15:

$H_i = (K_o)(V_o^2/2g) = (0.2)(0.10) = 0.02$ ft

$E_{aio} = E_i + H_i = 1.66$ ft $+ 0.02$ ft $= 1.68$ ft

Discharge intensity for the outflow pipe (pipe 42-43):

$DI = Q / [A(Dg)^{0.5}] = 6.75/[((\pi/4)(2.0)^2)((2.0)(32.2))^{0.5}] = 0.268$

Compute E_{ais} using equation 9.17:

$E_{ais} = (1.0)(DI)\ 2\ (D) = (0.268)\ 2\ (2.0) = 0.14$ ft

Compute E_{aiu} using equation 9.18:

$E_{aiu} = (1.6)(DI)\ 0.67\ (D) = 1.6(0.268)\ 0.67\ (2.0) = 1.32$ ft

Select maximum (to two decimal places): $E_{ai} = 1.68$ ft

$C_B = -0.05$, $E_{ai} / D < 1$, so assume flat bench, unsubmerged access hole

$C_\theta = 4.5\ (\Sigma\ Q_j /Q_o) \cos(\theta_w/2) = 4.5\ (5.1/6.75) \cos(90/2) = 2.40$

Plunging flow:

$z_k = 349.31$ ft $- 344.07$ ft $= 5.24$

$H_k = (z_k – E_{ai})/D_o = (5.24 – 1.68) / 2 = 1.78$

$C_p = Q_k H_k / Q_o = 1.65\ (1.78) / 6.75 = 0.44$

Check to ensure net energy:

$E_{ai} > E_i$? 1.68 ft > 1.66 ft therefore, OK

$H_a = (E_{ai} - E_i)(C_B + C_\theta + C_P) = (1.68 - 1.66)(-0.05 + 2.40 + 0.44) = 0.06$ ft

$E_a = E_{ai} + H_a = 1.68$ ft $+ 0.06$ ft $= 1.74$ ft

$EGL_a = E_a + BOC_i = 1.74$ ft $+ 344.07$ ft $= 345.81$ ft

Surf. Elev. = 349.31 ft, 349.31 ft > 345.82 ft, therefore, surface elev. exceeds HGL. OK

Step 8. Repeat steps 5, 6, and 7 for all pipes and structures in the storm drain system.

Continue upstream with all pipes entering the structure analyzed in the previous step. In this example, only one pipe enters structure 42. Go to step 5 for this pipe.

Step 5 (for next pipe). Estimate the HGL and EGL at the downstream end of pipe 41-42.

$D = 1.5$ ft

$Q = 5.1$ ft^3/s

$L = 328$ ft

$BOC_o = 344.23$ ft

$TOC_o = BOC_o + D = 344.23$ ft $+ 1.5$ ft $= 345.73$ ft

Determine the applicable case from Table 9.6:

Is the downstream end of the pipe submerged ($EGL_a > TOC_o$)?

EGL_a (345.81 ft) > TOC_o (345.73 ft) is true. Pipe outlet is submerged (case A).

With submerged outlet: $V = Q/A = 5.1/[(\pi/4)(1.5)^2] = 2.9$ ft/s

$V^2/2g = (2.9)^2/(2)(32.2) = 0.13$ ft

$H_o = (0.4)V^2/2g = 0.05$ ft

$EGL_o = EGL_a + H_o = 345.81 \text{ ft} + 0.05 \text{ ft} = 345.86$ ft

$HGL_o = EGL_o – V^2/2g = 345.86 \text{ ft} – 0.13 \text{ ft} = 345.73$ ft

Step 6. Estimate the HGL and EGL at the upstream end of pipe 41-42.

$BOC_i = 354.07$ ft

$TOC_i = BOC_i + D = 354.07 \text{ ft} + 1.5 \text{ ft} = 355.57$ ft

$S_f = [(Qn)/(KQD^{2.67})]^2 = [(5.1)(0.013)/(0.46)(1.5)^{2.67}]^2 = 0.0024$ ft/ft

$H_f = S_f L = (0.0024)(328.0) = 0.78$ ft

No other conduit losses: H_b, H_c, H_e, $H_j = 0.0$

Total pipe loss = 0.78 ft

$EGL_i = EGL_o$ + pipe loss = 345.86 ft + 0.78 ft = 346.64 ft

$HGL_i = EGL_i - V^2/2g = 346.64 \text{ ft} - 0.13 \text{ ft} = 346.51$ ft

Determine the flow condition at the upstream end of the outflow pipe from Table 9.7:

HGL_i (346.51 ft) < TOC_i (355.57 ft), therefore, pipe is not flowing full (surcharge) (condition A).

Estimate y_n and y_c:

$Q/Q_f = 5.1 / 18.1 = 0.28$

$V/Vf = 0.86$

$V = (0.86)(10.3) = 8.86$ ft/s

$y_n / D = 0.37$

$y_n = (0.37)(1.5) = 0.56$ ft

$V^2/2g = (8.86)^2/(2)(32.2) = 1.22$ ft

$y_c = 0.87$ ft

$BOC_i + y_c$ (354.07 + 0.87 = 354.94 ft) ≥ HGL_i (346.51 ft), therefore, supercritical partial flow conditions (condition D). Pipe losses not carried upstream. Recompute HGL_i and EGL_i.

$HGL_i = y_n + BOC_i = 0.56 \text{ ft} + 354.07 \text{ ft} = 354.63$ ft

$EGL_i = HGL_i + V^2/2g = 354.63 \text{ ft} + 1.22 \text{ ft} = 355.85$ ft

In this conduit, the flow is in a supercritical regime. Given that the D/S portion of the conduit is nearly submerged by the access hole, there would likely be a hydraulic jump somewhere within the barrel. Important observations are that the energy should not decrease when moving up the conduit and assumptions of full flow could yield erroneous results.

Step 7. Calculate EGL and HGL at structure 41.

$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu})$

Since condition D was identified in the previous step, $E_{aio} = 0.00$ ft

Discharge intensity for the outflow pipe (pipe 41-42):

$DI = Q / [A(Dg)^{0.5}] = 5.1/[((\pi/4)(1.5)2)((1.5)(32.2))0.5] = 0.415$

Compute E_{ais} using equation 9.17:

$E_{ais} = (1.0)(DI)\ 2\ (D) = (0.415)\ 2\ (1.5) = 0.26$ ft

Compute E_{aiu} using equation 9.18:

$E_{aiu} = (1.6)(DI)\ 0.67\ (D) = 1.6(0.415)\ 0.67\ (1.5) = 1.33$ ft

Select maximum (to two decimal places): $E_{ai} = 1.33$ ft

$C_B = -0.05$, $E_{ai} / D < 1$, so assume flat bench, unsubmerged access hole

$C_\theta = 0.0$ since $\theta_w = 180$

$z_k = 360.00$ ft - 354.07 ft = 5.93

$H_k = (z_k – E_{ai}) / Do = (5.93 – 1.33) / 1.5 = 3.06$

$C_p = Q_k H_k / Q_o = 1.8 (3.08) / 5.1 = 1.08$

Check to ensure net energy:

Is $E_{ai} > E_i$?

1.33 ft > 1.78 ft is not true. Since E_{ai} is less than E_i, use E_i as the net energy.

$E_a = E_i = 1.78$ ft

$EGL_a = E_a + BOC_i = 1.78$ ft + 354.07 ft = 355.85 ft

Surface elevation (360.0 ft) > EGL_a (355.85 ft) therefore HGL OK.

Step 8. Repeat steps 5, 6, and 7 for all pipes and structures in the storm drain system.

Continue upstream with all pipes entering the structure analyzed in the previous step. In this example, only one pipe enters structure 41. Go to step 5 for this pipe.

Step 5 (for next pipe). Estimate the HGL and EGL at the downstream end of pipe 40-41.

D = 1.5 ft

Q = 3.3 ft^3/s

L = 361.0 ft

$BOC_o = 354.67$ ft

$TOC_o = BOC_o + D = 354.67$ ft + 1.5 ft = 356.17 ft

Determine the applicable case from Table 9.6:

Is the downstream end of the pipe submerged ($EGL_a > TOC_o$)?

355.85 ft > 356.17 ft is not true. Pipe is not submerged (case A).

Is the pipe plunging ($EGL_a < BOC_o$)?

355.85 ft < 354.67 ft is not true. Pipe is not plunging (case E).

Need to determine normal and critical depth to make case determination:

$Q/Q_f = 3.3 / 18.1 = 0.18$

$V/V_f = 0.74$

$V = (0.74)(10.3) = 7.62$ ft/s

$V^2/2g = (7.62)2/(2)(32.2) = 0.90$ ft

$y_n/d_f = 0.30$

$y_n = (0.30)(1.5) = 0.45$ ft

$y_c = 0.67$ ft

TOC_o (356.17 ft) > EGL_a (355.85 ft) > $BOC_o + y_n$ (355.34 ft) Therefore, condition is case B.

D/S face depth = $EGL_a - BOC_o$ = 355.85 ft - 354.67 ft = 1.18 ft

Ratio of face depth to diameter: $d_f / D = 1.18 / 1.5 = 0.79$

$A_f / A = 0.84$

$A_f = 0.84(1.77) = 1.49$ ft^2

$V_f = Q / A_f = 3.3 / 1.49 = 2.21$ ft/s

$V_f^2/2g = (2.21)^2/(2)(32.2) = 0.08$ ft

$H_o = (0.4)V_f^2/2g = (0.4)(0.08) = 0.03$ ft

$EGL_o = EGL_a + H_o$ = 355.85 ft + 0.03 ft = 355.88 ft

$HGL_o = EGL_o – V^2/2g$ = 355.88 ft – 0.08 = 355.80 ft

Step 6. Estimate the HGL and EGL at the upstream end of pipe 40-41.

BOC_i = 365.50 ft

$TOC_i = BOC_i + D$ = 365.50 ft + 1.5 ft = 367.00 ft

Since y_n (0.45 ft) < y_c (0.67 ft) pipe flow is supercritical. The flow condition (Table 9.7) is supercritical partial flow (condition D). Pipe energy losses are not carried upstream.

$HGL_i = y_n + BOC_i$ = 0.45 ft + 365.50 ft = 365.95 ft

$EGL_i = HGL_i + V^2/2g$ = 365.95 ft + 0.90 ft = 366.85 ft

Given that the downstream portion of the conduit is partially submerged by the access hole, there would likely be a hydraulic jump somewhere within the barrel.

Step 7. Calculate EGL and HGL at structure 40.

Structure 40 is a terminal inlet with no incoming pipe.

$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu})$

Since condition D was identified in the previous step, E_{aio} = 0.00 ft

Discharge intensity for the outflow pipe (pipe 40-41):

$DI = Q / [A(Dg)^{0.5}] = 3.3 / [((\pi/4)(1.5)^2)((1.5)(32.2))^{0.5}] = 0.269$

Compute E_{ais} using equation 9.17:

$E_{ais} = (1.0)(DI)\ 2\ (D) = (0.269)\ 2\ (1.5) = 0.11$ ft

Compute E_{aiu} using equation 9.18:

$E_{aiu} = (1.6)(DI)\ 0.67\ (D) = 1.6(0.269)\ 0.67\ (1.5) = 1.00$ ft

Select maximum (to two decimal places): E_{ai} = 1.00 ft

C_B = 0.0 (No inflow pipes, benching not a factor)

C_θ = 0.0 (No inflow pipes, angled inflow not a factor)

Plunging flow from the inlet:

z_k = 370.0 ft - 365.50 ft = 4.50 ft

$h_k = (z_k - E_{ai}) / D_o$ = (4.50 – 1.00)/(1.5) = 2.34 ft

$C_p = ((\Sigma Q_k)(h_k))/Q_o$ = ((3.3)(2.34))/(3.3) = 2.34

Check to ensure net energy:

Is $E_{ai} > E_i$?

1.00 ft > 1.35 ft, not true. Since E_{ai} is less than E_i, use E_i as the net energy.

$E_a = E_i$ = 1.35 ft

$EGL_a = E_a + BOC_i$ = 1.35 ft + 365.50 ft = 366.85 ft

Surf. Elev. = 370.0 ft, 370.0 ft > 366.85 ft, therefore, surface elevation exceeds HGL, OK

Step 8. Repeat steps 5, 6, and 7 for all pipes and structures in the storm drain system.

Continue upstream with all pipes entering the structure analyzed in the previous step. In this example, structure 40 is a terminal structure with no incoming pipes. Go to step 9.

Step 9. Examine the results.

The designer compares the EGL and HGL elevations with ground elevations and confirms minimum cover satisfied. The designer also confirms that the pipe sizes and inverts are reasonable and considers ways to improve the design.

Solution:	The example illustrates many of the situations that may occur in designing a storm drain system. Many software tools are available to perform these computations but not all use the full energy loss method illustrated here. The designer may also develop spreadsheets to perform the computations.