

CHAPTER 7

INLET AND ENCLOSED STORM DRAINAGE SYSTEMS



Note: For trunkline projects, direct questions and comments to the Hydraulic Unit Supervisor, Environmental Services Section.

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If you require assistance accessing this information or require it in an alternative format, contact the Michigan Department of Transportation's (MDOT) Americans with disabilities Act (ADA) coordinator at Michigan.gov/MDOT-ADA.

7.1 INTRODUCTION/PURPOSE

This chapter provides guidance on storm sewer design and analysis for roadways. Various design aspects, such as system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, and hydraulic grade line (HGL) calculations are presented. The methods described apply to new systems and retrofitting or fully replacing an existing system.

The design of a drainage system is required to address the needs of the traveling public as well as the local community. The drainage system for roadways in urbanized regions can be more complex than in rural areas. This complexity is often due to:

- Wide roadway sections, limited right-of-way (ROW), flat grades in longitudinal and transverse directions, shallow watercourses, and absence of side channels.
- Potentially costly property damage may occur from ponding of water or from flow of water through developed areas.
- Competing interests between conveying traffic and drainage flows within the same roadway section. Unless proper precautions are taken, the flow of water along the roadway may interfere with the passage of traffic.
- Water quality concerns on receiving waterbodies may impact storm sewer design. Refer to the [MDOT Stormwater Treatment Manual](#) for additional information.

Impacts of a roadway drainage system that does not meet the design discharge are:

- Damage to surrounding or adjacent property resulting from water overflowing the roadway curbs and entering adjacent property
- Risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway
- Weakening of base and subgrade due to saturation from frequent ponding of long duration

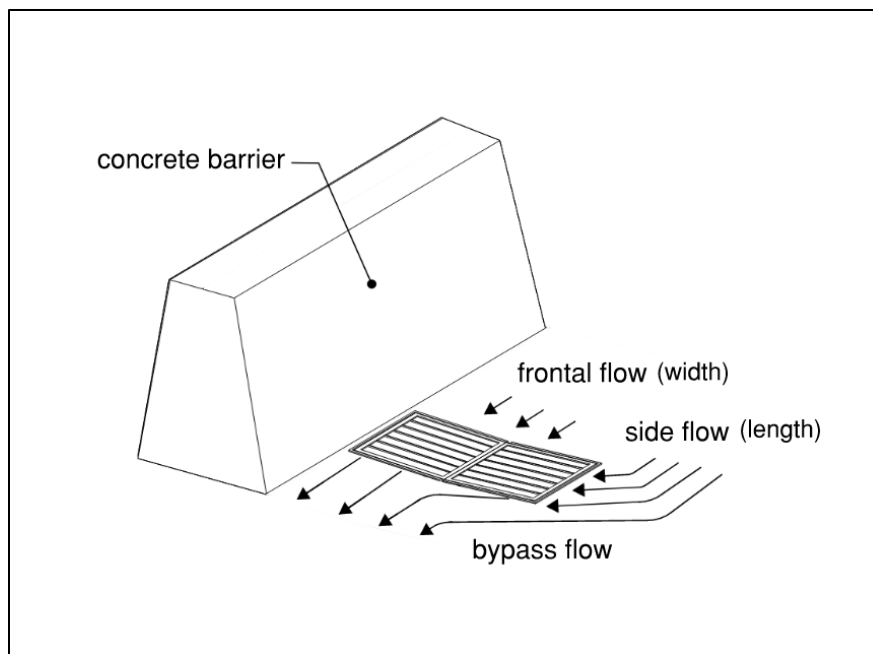
7.2 DEFINITIONS

Annual Exceedance Probability (AEP) - The probability that the magnitude of the random variable (e.g., annual maximum water level) will be equaled or exceeded each year, often expressed as a percentage.

Annual Recurrence Interval (ARI) - The average time interval between occurrences of an event of a given or greater magnitude, usually expressed in years. The ARI is the reciprocal of the AEP of an event.

Bypass - Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Refer to **Figure 7-1**.

Figure 7-1. Grate Intake Flow Definitions



Catch Basin - A structure for intercepting stormwater from a gutter or median and discharging the water through a conduit. In common usage, it is a grated inlet, a curb opening inlet, or a combination inlet with a sump.

Cover - Distance from the outside top of the pipe or structure to the final grade of the ground surface or pavement.

Combination Inlet - A drainage inlet that uses a curb opening inlet and a grate inlet.

Combined Sewer - A combined sewer is a pipe that carries stormwater and sanitary sewage.

Critical Depth (d_c) - In hydraulic analysis, the depth when flow has a Froude number of 1.0. The depth associated with the minimum total energy to pass a given flow through a given cross section.

Crown - The highest interior elevation of a culvert, sewer, drain pipe, or stormwater tunnel. This may also be referred to as underclearance or low chord.

Culvert - A structure that is usually designed hydraulically to convey flow through an embankment. The span length is less than 20 feet measured along the road centerline.

Curb Opening Inlet - A drainage inlet consisting of an opening in the roadway curb.

Depressed Roadway - A roadway that is approximately 16 feet or more below the surface of the adjacent streets.

Design Life - The average usable life of a pipe or structure.

Drainage Structure - Refers to manholes, catch basins, leaching basins, inlets, and drop inlets detailed in the MDOT Road Standard Plans or contract plans.

Drop Inlet - An inlet directly connected to a vertical or steeply sloping pipe to drop water to a lower pipe.

Energy Grade Line (EGL) - The plotted line that represents the total energy of flow at a location. It is the sum of the elevation head, pressure head, and velocity head. Refer to **Figure 7-9**.

Frontal Flow - The portion of flow that passes over the upstream side of a grate. Refer to **Figure 7-1**.

Grate Inlet - A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale, or channel.

Grate Perimeter - Perimeter of the grate disregarding the side against the curb. Used in weir flow computations.

Gutter - The portion of roadway section adjacent to the curb that is utilized to convey stormwater runoff.

Harmful Interference - Causing an increased stage or change in direction of flow of a river or stream that causes, or is likely to cause, any of the following: i) Damage to property, ii) A threat to life, iii) A threat of personal injury, iv) Pollution, impairment, or destruction of water or other natural resources. MCL 324.3103, R 323.1311g

Hydraulic Grade Line (HGL) - A plotted line coinciding with the level of flowing water in an open channel or conduit. In a closed conduit flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. It is equal to the EGL minus the velocity head ($V^2/2g$). Refer to **Figure 7-9**.

Hydrodynamic Separators - Hydrodynamic separators are flow-through structures with a separation or settling unit to remove sediments, debris, and other pollutants.

Inlet - A structure, without a sump, for capturing concentrated surface flow. May be located along the roadway, or in a gutter, the highway median, or a field.

Inlet Efficiency - The ratio of flow intercepted by an inlet to total design flow in the gutter.

Invert - The lowest interior elevation of a culvert, sewer, or stormwater tunnel.

Lateral Sewer Line - A sewer line that connects inlet structures to the trunk sewer.

Leaching Basin - A drainage structure with a porous sump bottom, as opposed to the concrete sump of the catch basin. Leaching basins may only be used where soils will allow infiltration and where potential groundwater contaminants will not be introduced through stormwater.

Manhole - A drainage structure that provides access to a closed drainage system.

Normal Depth (d_n) - The depth of flow in a pipe when the flow is uniform and steady and the water surface and pipe slope are the same. It occurs when the gravitational force of the water is equal to the friction drag along the pipe, and there is no acceleration of flow.

Outfall - The location at which a point source discharge first enters a surface water of the state.

Outlet - The location at which a point source discharge leaves the MDOT ROW and discharges to something other than a water of the state.

Peak Shave - Reduction in hydrograph peak flow to a downstream system. Generally achieved using a diversion chamber with high overflow weir and a restrictive outlet.

Point of Discharge - The location of a point source discharge where stormwater is discharged directly into a separate storm sewer system.

Point Source Discharge - A discharge from any discrete conveyance, including but not limited to any pipe, ditch, channel, stormwater tunnel, or pipe.

Pressure Flow - Flow not exposed to atmospheric pressure.

Pressure Head - The height of a column of water that would exert a unit pressure equal to the pressure of the water. This head is measured from the HGL down to a point in the water column such as the invert.

Sag Point - The lowest point in a vertical road or gutter profile.

Sanitary Sewer - A conduit or pipe system that carries household and/or industrial waste. Sanitary sewers do not convey stormwater.

Scupper - A small opening (usually vertical) in a bridge deck, curb, or barrier through which water can flow.

Seiche - A standing wave oscillating in a body of water that continues, in pendulum fashion, after cessation of the originating force. Seiches are caused when strong winds and rapid changes in atmospheric pressure push water from one end of a body of water to the other.

Side Flow - Flow which is intercepted along the side of a grate inlet, as opposed to the front. Refer to **Figure 7-1**.

Slotted Drain Inlets - A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect, and transport the flow.

Spillway - A structure used to provide the controlled release of water to a downstream channel.

Splash-Over - The portion of frontal flow at a grate which skips or splashes over the grate and is not intercepted.

Spread - A measure of the transverse distance from the curb or barrier face to the limit of the water flowing on the roadway.

Storm Sewer - A pipe that conveys stormwater that has been collected by inlets to an outfall. It generally consists of laterals or leads, and trunk lines or mains. Culverts connected to the storm sewer system are considered part of the storm sewer.

Stormwater Tunnel - In this chapter, stormwater tunnel refers to the main component of a stormwater storage system. These stormwater tunnels are not part of the National Tunnel Inventory Program that focuses on enclosed roadways for vehicle traffic.

Tailwater - The depth of flow directly downstream of a storm sewer system or culvert, measured from the invert at the storm sewer system or culvert outlet.

Time of Concentration, T_c - The time for water to flow from the hydraulically most distant point in the watershed to the outlet or discharge point.

Trunk Line - The main storm sewer line. Lateral sewer lines may be connected to the trunk line at inlet structures or manholes. A trunk line is sometimes referred to as a "main."

Velocity Head - A quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.

Symbols and acronyms used in this chapter are listed in **Appendix 7-A**.

7.3 POLICY AND DESIGN CRITERIA

7.3.1 Introduction

Highway storm sewer facilities collect stormwater runoff and convey it through the roadway ROW draining the roadway and minimizing the potential for flooding and erosion to properties adjacent to the ROW. Storm sewer facilities consist of curbs, gutters, storm sewers, channels, and culverts. During the design process, storm sewer placement and hydraulic capacity should consider damage to adjacent property and reduce the risk of traffic interruption caused by flooding. Roadway classification, design requirements, and available project budget are considerations.

Refer to Chapter 2, Policy and Procedures for Federal and State policies. For a general discussion of policies and guidelines for storm sewer drainage systems, refer to *AASHTO A Policy on Geometric Design of Highways and Streets* (2011). For more design and engineering guidance, refer to *AASHTO Drainage Manual* (2014), and FHWA HEC-22 (2024).

7.3.2 Hydrology

Design pavement drainage using the Rational Method (refer to Chapter 3, Hydrology). Include all contributing drainage area in the analysis. The design peak flows are calculated for roadways using the following time of concentration, T_c :

- 15 minutes for non-depressed roadways
- 10 minutes for depressed roadways or roadways discharging to a pump station
- 10 minutes for sag locations

For bridge deck drainage, calculate the T_c using FHWA HEC-21 (1993). The minimum T_c for bridge decks is 5 minutes.

7.3.3 Selection of Design Storm Frequency and Spread

Roadway classification is a major consideration when selecting a design frequency and spread and reflects public expectations for finding water on the pavement surface on interstates. Highway design speed is another major consideration. At speeds greater than or equal to 45 miles per hour, a shallow depth of water on the pavement can cause hydroplaning. Minimize ponding on the traffic lanes of high-speed, high-volume roadways.

Other considerations include inconvenience, hazards, and nuisances to pedestrian traffic and buildings located within the splash zone. These considerations are important, especially in commercial areas.

Inlet storm frequency and spread criteria for interstates, freeways, and roadways with shoulders are presented in **Table 7-1**. The allowable spread in **Table 7-1** is based on a standard 12-foot lane width. For example, if a principal arterial with a 40 mph design

speed has an 11-foot lane and no shoulder, the allowable spread is 5 feet from the curb face instead of 6 feet.

Table 7-1. Allowable Spread Design Criteria – AEP (ARI)

Roadway Classification	Design Speed	Design Storm	Allowable Spread
MDOT: Interstate, Freeways (Urban/Rural)	All design speeds	10% (10-year)	Shoulder
	Sag (all design speeds)	2% (50-year)	Shoulder
MDOT: Depressed Interstate, Freeway or Discharge to MDOT Pump Station	All design speeds	2% (50-year)	Shoulder
MDOT: Principal Arterial, Minor Arterial, or Roadway Tapers to Turn Lanes	<45 mph	10% (10-year)	Shoulder + 6 ft
	≥45 mph	10% (10-year)	Shoulder + 3 ft
	Sag (<45 mph)	2% (50-year)	Shoulder + 6 ft
	Sag (≥45 mph)	2% (50-year)	Shoulder + 3 ft
MDOT: Multilane Principal Arterial, Minor Arterial, or Roadway Tapers to Turn Lanes	<45 mph	10% (10-year)	Shoulder + 6 ft
	≥45 mph	10% (10-year)	Shoulder + 3 ft
	Sag (all design speeds)	2% (50-year)	Shoulder + 6 ft
MDOT: Right Turn Lanes	<45 mph	10% (10-year)	Shoulder + 6 ft
	≥45 mph	10% (10-year)	Shoulder + 4 ft
	Sag (all design speeds)	2% (50-year)	Shoulder + 6 ft
MDOT: Left Turn Lanes	All design speeds	10% (10-year)	Shoulder + 4 ft
	Sag (all design speeds)	2% (50-year)	Shoulder + 6 ft
MDOT: Roundabouts	<45 mph	10% (10-year)	Shoulder + 6 ft
Local: Minor Arterial System	<45 mph	10% (10-year)	Shoulder + 6 ft
	≥45 mph	10% (10-year)	Shoulder + 3 ft
	Sag (all design speeds)	2% (50-year)	Shoulder + 6 ft
Local: Collector System	<45 mph	10% (10-year)	Shoulder + 6 ft
	≥45 mph	10% (10-year)	Shoulder + 3 ft
	Sag (all design speeds)	10% (10-year)	Shoulder + 6 ft
Local Road System	All design speeds	10% (10-year)	Shoulder + 6 ft

7.3.4 Roadways with Enclosed Drainage

Compute runoff for a roadways with enclosed drainage based on the criteria in **Table 7-2** for the design storm and harmful interference. For the design storm, size the sewer to flow full with a HGL at or near the crown of the pipe. For non-depressed interstates, freeways limit the HGL elevation to a minimum of 1 foot below the gutter grade for the 2% AEP (50-year) storm.

The sewer may surcharge due to high downstream boundary conditions (also referred to as tailwater). Limit the surcharge elevation to a minimum of 1 foot below the gutter grade of the roadway. If possible, maintain design velocities in the surcharged portion of the sewer and provide a hydraulic slope that brings the upstream HGL below the crown of the sewer.

Design criteria for roadways with roadside and median ditches is provided in Chapter 4, Open Channels and Roadside Ditches, Section 4.3.3.

Table 7-2. Storm Sewer Minimum Design Criteria AEP (ARI)

Roadway Classification	Design Storm	Harmful Interference
MDOT: Interstate, Freeways (Urban/Rural)	10% (10-year)	1% (100-year)
MDOT: Depressed Interstate, Freeway or Discharge to MDOT Pump Station	2% (50-year)	1% (100-year)
MDOT: Principal or Minor Arterial	10% (10-year)	1% (100-year)
Local: Minor Arterial	10% (10-year)	1% (100-year)
Local: Collector	10% (10-year)	1% (100-year)
Local Road	10% (10-year)	1% (100-year)

7.3.5 Depressed Roadways

Compute runoff for a depressed roadway with enclosed drainage based on the criteria in **Table 7-2**. For the design storm, size the sewer with an HGL meeting the following requirements:

- New storm sewer installation (outlet tailwater at or below crown) - flow full at or near the crown of the pipe
- Existing storm sewer system (outlet tailwater at or below crown) - limit to 1 foot below the gutter grade line of the roadway
- Existing and new sewer systems (outlet tailwater above the crown) - limit to 1 foot below the gutter grade of the roadway

Provide a trunk sewer and a properly designed independent outlet for MDOT projects. An existing municipal sewer may only be used by agreement. Contact the Hydraulic Unit Supervisor for assistance on trunkline projects.

For a depressed roadway, a pump station is required when a gravity drainage system cannot be used to drain the low points. For additional information, refer to Chapter 10, Pump Stations, or contact the Hydraulic Unit Supervisor for assistance.

7.3.6 Drainage Structure Location and Spacing

Locate new sewers on freeway projects outside the travel lanes. Utilize manholes to provide entry to underground storm sewers for inspection and cleanout. Inlets can be used as manholes when entry to the system is provided at the inlet. The benefit of additional stormwater interception can be achieved with minimal cost.

Avoid locating manholes in traffic lanes. If this is not possible, bolt the manhole lid down and locate it out of the vehicle wheel path. Use bicycle safe grate inlets on roadways that allow bicycle travel.

Lay out inlets to accommodate the design spread and limit spacing to a maximum of 300 feet apart or 300 feet from the roadway crest. On grade grate inlets assume a 30% clogging factor.

Combination inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities. If grate inlets are used at sag locations, assume the grate is half clogged with debris and confirm spread is not exceeded in the design event. Provide flanking inlets at major sag points. Major sag points include, but are not limited to, interstates, freeways, underpasses, and sag vertical curves that have no outlet except through the drainage system. When adding flanking inlets, perform an additional resiliency check calculation assuming the grate at the sag is fully clogged to confirm spread is not exceeded. Refer to **Section 7.4.6.6** for more information regarding flanking inlets.

For temporary drainage during construction on trunkline projects, a waiver may be obtained from the Hydraulic Unit Supervisor to exceed the 300-foot maximum spacing. No clogging factor is used for temporary drainage calculations. To obtain a waiver, submit spread calculations to demonstrate that the design spread criteria in **Table 7-1** is met.

7.3.7 Minimum Pipe Sizes

Use a minimum diameter of 12 inches for storm sewers, including laterals.

7.3.8 Peak Flow Storage

Peak flows can be reduced by storing runoff in basins, oversized storm sewer pipes, swales with outlet control devices, or other storage facilities. Evaluate storage needs upstream of pump stations (refer to Chapter 10, Pump Stations). Refer to **Section 7.8** for additional analysis requirements for linear underground storage facilities that may be at risk of hydraulic surge. A restrictive outlet is required for each of these storage options, which releases stormwater to the downstream conveyance facility at a reduced flow rate. Match pre- and post-project release rates at locations where existing downstream conveyance facilities are inadequate to handle a range of flows up to and including the 1% AEP (100-year), 24-hour peak flow rates from highway storm sewer facilities including the 50% AEP (2-year), 24-hour design storm to meet Municipal Separate Storm Sewer System (MS4) requirements. In many locations, highway

agencies or developers are not permitted to increase runoff rates over existing conditions and require storage facilities. Reducing the peak flow may allow for a reduction in downstream pipe sizes. For more information, refer to [MDOT Stormwater Treatment Manual](#), Chapter 8, Stormwater Storage Facilities for Flood Control, and Chapter 2, Policy and Procedures.

7.3.9 System Planning

System planning prior to designing a storm sewer system is essential. Basic requirements are discussed in **Section 7.6** and include the general design approach, type of data required, information on initiating a cooperative agreement with a municipality, the importance of a preliminary sketch, and special considerations (refer to Chapter 2, Policy and Procedures, Section 2.4.2.)

Cooperative storm sewer projects with cities and municipalities may be beneficial when a mutual economic benefit and demonstrated need exists. Early coordination with the governmental entity involved is necessary to determine the project scope and stakeholder interests/design requirements. Each cooperative project may be initiated by a resolution adopted by the governing body of the municipality either by (1) requesting the improvements or indicating a willingness to share the cost of a State project; or (2) indicating the municipality's intention to make certain improvements and requesting State cost participation in the municipal project.

7.4 DESIGN GUIDANCE AND PROCEDURE FOR INLET SPACING

7.4.1 Pavement and Bridge Deck Drainage

Roadway features considered during gutter, inlet, and pavement drainage calculations include:

- Roadway geometrics (longitudinal and cross slope)
- Bridge decks
- Curb and gutter sections
- Roadside and median ditches

The overland flow, pavement width, cross slope, and profile control the time it takes for stormwater to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow that is carried in the gutter section.

7.4.1.1 Roadway Geometrics (Longitudinal Slope and Cross Slope)

Gutter flow calculations are necessary to relate the quantity of flow to the spread of water on the shoulder, parking lane, or pavement section.

For curbed pavements, the desirable minimum longitudinal gutter grades are 0.5% with a minimum of 0.3%.

The minimum pavement cross slope is 2% for New Construction/Reconstruction (4R) projects or 1.5% for Construction on Existing Roads (3R) projects. It is desirable to provide a break in cross slope at two lanes, with three lanes as the upper limit. Do not drain median areas across traveled lanes. Refer to Chapter 6 of the MDOT Road Design Manual and Standard Plan R-107 Series for more information.

7.4.1.2 Superelevated Horizontal Curves

Superelevation is the rotation of the pavement on the approach to and through a horizontal curve. Superelevation transitions present several challenges for drainage design:

- Contributing pavement runoff increases when all lanes drain to the same side. In addition, the curvature of the roadway causes the cross-roadway flow to converge from wide at the high, outer side, to narrow at the low, inner side, increasing the depth of surface flow.
- The flat cross slope during the transition is important for design because ponding can occur and the available head for inlets to remove water is zero. Avoid flat cross slopes at sag locations. Flow in the roadway is then governed by longitudinal grade. Avoid conditions where concentrated flow is directed across the roadway from the outer side of the curve to the inner side.
- The roadway profile grade and gutter profile grade do not change at the same rate in a superelevation transition. Relative to the roadway profile grade, the

gutter profile grade may decrease, increase, or even change direction. Depth of flow available at the curb may decrease and spread may increase (depending on flow direction).

Coordination between drainage and roadway geometry designers is important to minimize problem areas created by superelevated curves and transitions. Review the profile grade line, gutter grades, and the rotation of the roadway (centerline, inner edge, or outer edge) in the superelevation transition to avoid flatter areas where water builds up and drains slowly from the roadway and to avoid areas where water is directed from one side of the road to the other.

7.4.1.3 Bridge Decks

Avoid zero gradients, sag vertical curves, and superelevation transitions with flat pavement sections to prevent ponding on bridges. The minimum longitudinal slope for bridge decks is 0.5%. Many bridges do not require any drainage facilities. Chapter 6, Bridge Hydraulic Design, discusses the maximum length of deck permitted without drainage facilities. Avoid direct discharges to waterbodies. Maintain the quantity and quality of runoff as required by applicable local stormwater regulations. Refer to Chapter 6, Bridge Hydraulic Design, and the [MDOT Stormwater Treatment Manual](#) for additional information.

7.4.1.4 Curbs, Inlets, and Spillways

Curbs, inlets, and spillways are used where runoff from the pavement would erode fill slopes and/or to reduce ROW needed for shoulders, channels, etc. Where storm sewers are necessary, pavement sections are usually curbed.

7.4.1.5 Median Barriers

Inlets and storm sewers collect stormwater that accumulates against a median barrier, particularly on horizontal curves with superelevation. Openings under the median barrier to pass water from one side to the other are not allowed.

7.4.1.6 Roadside and Median Ditches

Roadside channels and median channels are part of the storm sewer system and are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas that drain toward the highway. Due to ROW limitations, roadside channels cannot always be used on urban arterials. They can be used in cut sections, depressed sections, and other locations where sufficient ROW is available and driveways or intersections are infrequent.

Where practical, intercept the flow from adjacent areas draining toward curbed highway pavements in the ditch. Intercepting the flow minimizes the deposition of sediment and other debris on the roadway and reduces the amount of water carried in the gutter section. Shallow swale sections at the edge of the roadway pavement or shoulder offer advantages over curbed sections where curbs are not needed for traffic control. These

advantages include a lesser hazard to traffic as compared to a near-vertical curb and hydraulic capacity that is not dependent on spread on the pavement. Swale sections without curbs are particularly appropriate where curbs have historically been used to prevent water from eroding fill slopes.

Slope median areas and inside shoulders to a center swale to prevent drainage from the median area from running across the pavement. This center swale is important for facilities that are high-speed or have more than two lanes of traffic in each direction. Additional information on channels is included in Chapter 4, Open Channels and Roadside Ditches. Refer to **Table 7-3** for catch basin drainage covers used in ditches.

7.4.2 Curb and Gutter

Curbing at the outside edge of pavements is a normal practice for low-speed, urban highway facilities. Curbs contain the surface runoff within the roadway, preventing erosion and providing pavement delineation. Refer to MDOT Standard Road Plans and Special Detail Numbers: R-30-Series, R-31-Series, R-32-Series, and R-33-Series for curb and gutter standards.

A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility conveying runoff of a lesser magnitude than the design flow without interrupting traffic. When a design storm flow occurs, the water surface spread widens to include the gutter width and possibly parking lanes, shoulders, or portions of the traveled lanes. This width of flow, or spread, is an important design criterion and is limited during design.

The curb and gutter directs flow through an inlet to an enclosed storm drainage system. Depending on the configuration of the curb and gutter specific drainage covers are used. Drainage cover selection is described in **Table 7-3**.

Table 7-3. Drainage Covers

Standard Plan Series	Cover	Cover Type/ Hydraulic Toolbox Grate	Guidance
R-7	B	Storm Manhole	For use on storm manholes
R-18	Q	Sanitary Manhole	For use on manholes or sanitary sewers where vent holes are not desired
R-8	C	Combination/ Reticuline	For use with Concrete Curb & Gutter, Detail D (bicycle safe, R-30), HMA sloped curb
R-8X	CX	Combination/ Reticuline	For use with Concrete Curb & Gutter, Detail D (freeways only, R-30)
R-9	D	Grate/ Reticuline	For use with Straight Curb and Curb & Gutter (non-freeway valley gutter, bicycle safe, R-30), curbed driveway openings, or bridge approach curb & gutter (R-27)
R-9X	DX	Grate/ Reticuline	For use with Concrete Valley Gutter (freeways only, R-33)
R-10	E	Grate/P-1-7/8	For use on structures in ditches where not subject to traffic (outside clear zone, non-mountable)
R-12	G	Grate/P-1-7/8	For use in ditches and areas subject to minor traffic (inside clear zone, mountable)
R-14	J	Combination/ Reticuline	For use with Concrete Curb & Gutter, Detail B (bicycle safe, R-30)
R-15	K	Combination/ Reticuline	For use with Concrete Curb & Gutter, Details C, E & F (bicycle safe, R-30) or bridge approach curb & gutter (R-32)
R-20	R	Grate/ Reticuline	For use with Integral Curb and Integral Curb and Gutter (bicycle safe, R-31) and Concrete Valley Gutter and Urban Freeway Curb, Detail G (bicycle safe, R-33)
R-20X	RX	Grate/ Reticuline	For use with Concrete Valley Gutter and Urban Freeway Curb, Detail G (freeways only, R-33)
R-22	V	Grate/ Curved Vane	For use on flat grades or multi-lanes; double grate
R-23	W	Grate/ Curved Vane	For use with Concrete Valley Gutter and Urban Freeway Curb, Detail G (freeways only, R-33)
R-24	VG	Grate/ Curved Vane	For use with Concrete Valley Gutter (R-33)

Shoulder gutters and/or curbs are recommended for the following locations:

- Fill slopes higher than 20 feet with 1V:2H slopes.
- Fill slopes higher than 10 feet with slopes between 1V:6H and 1V:3H and roadway longitudinal grades greater than 2%.
- In areas where permanent vegetation cannot be established, shoulder curb and gutter are recommended on fill slopes higher than 10 feet regardless of the grade. Inspect the existing and proposed site conditions and contact maintenance and construction personnel to determine if vegetation will survive.
- Shoulder gutters are not required on the high side of superelevated sections or adjacent to barrier walls on high fills.

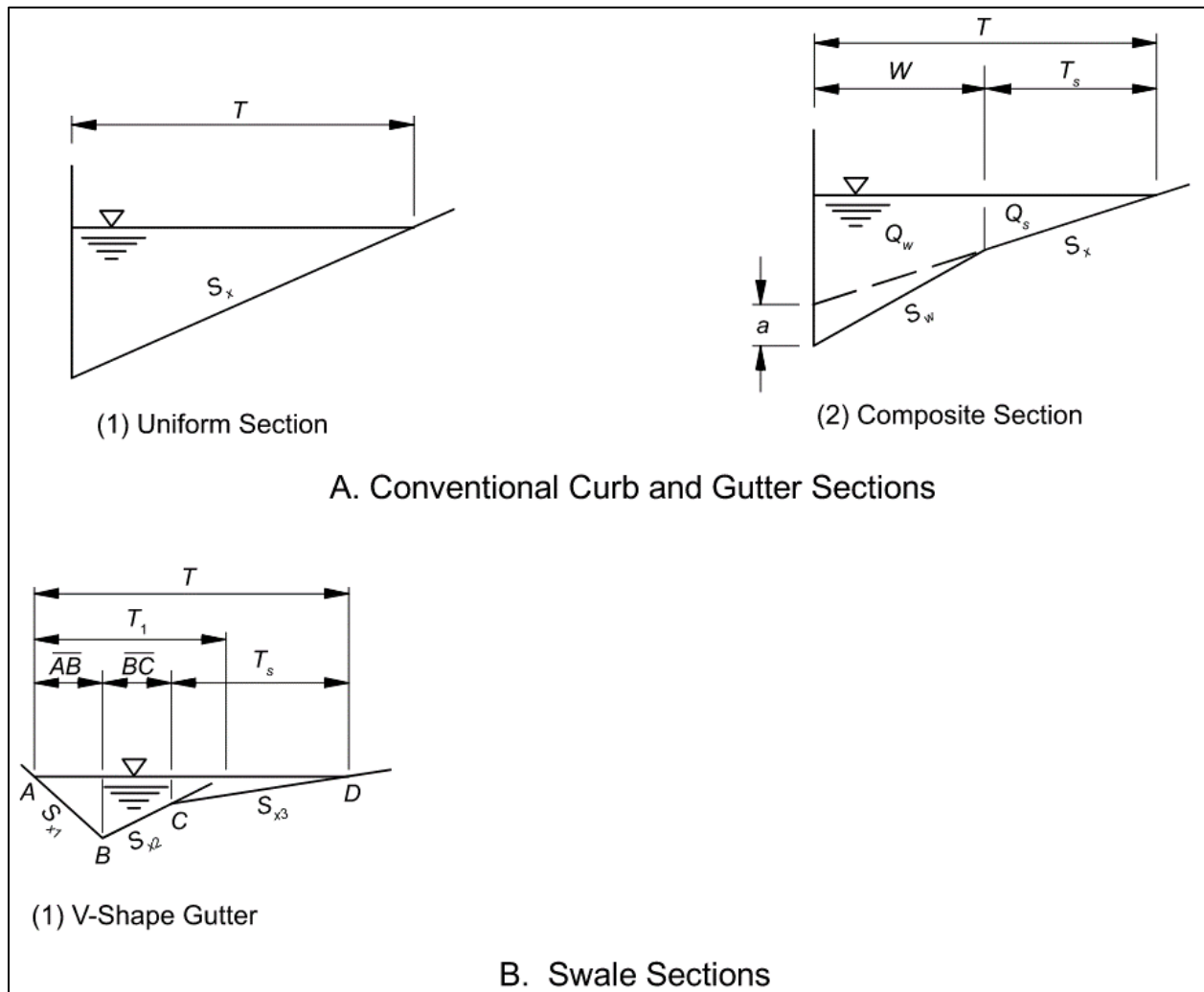
7.4.3 Gutter Flow Calculations

7.4.3.1 Introduction

Gutter flow calculations estimate the spread of water on the shoulder, parking lane, or pavement section. Several typical gutter sections are described in the FHWA HEC-22 (2024). FHWA HEC-22 (2009) provides nomograph figures that can be used to calculate spread for a range of gutter sections.

The MDOT Drainage Manual focuses on the three most common gutter sections shown in **Figure 7-2** and provides design guidance and example design calculations using the FHWA Hydraulic Toolbox. Hydraulic Toolbox is available for download here: [FHWA Hydraulic Toolbox](#).

Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are preferred. Example problems for each gutter type are provided in the following sections.

Figure 7-2. Typical Gutter Sections (AASHTO Drainage Manual, 2014)**7.4.3.2 Capacity Relationship**

Gutter flow calculations estimate the spread of water on the shoulder, parking lane, or pavement section. Use the modified Manning's equation for gutter flow to determine the flow rate along the curb:

$$Q = \frac{K_u}{n} S_x^{1.67} S_L^{0.5} T^{2.67} \quad (7.1)$$

Where:

- Q = Flow rate, cfs
- K_u = Unit conversion constant, 0.56
- n = Manning's coefficient
- T = Width of spread, ft
- S_x = Cross slope, ft/ft
- S_L = Longitudinal grade, ft/ft

Spread on the pavement and flow depth at the curb are criteria used for inlet spacing. **Equation 7.1** can be solved in terms of T to estimate the spread with a given flow rate:

$$T = \left[\frac{Q n}{K_u S_x^{1.67} S_L^{0.5}} \right]^{0.375} \quad (7.2)$$

Standard practice is to use a Manning's n of 0.020 to account for debris build-up, with 0.016 as a minimum where street sweeping occurs at least twice a year.

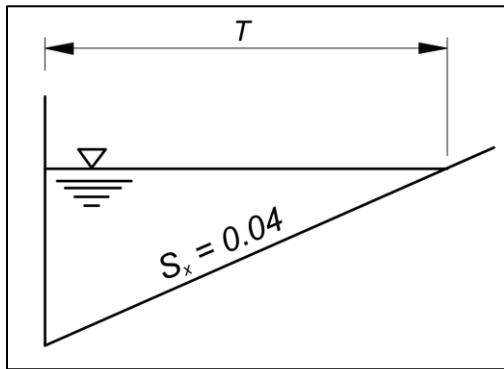
7.4.3.3 Uniform Cross Slope Procedure

Uniform gutter sections have a cross slope, S_x , equal to the cross slope of the shoulder or travel lane adjacent to the gutter. **Equations 7.1** and **7.2** calculate the gutter flow characteristics in a uniform triangular gutter as illustrated in the following example.

Example: Estimate the spread for a given flow rate of 1.5 cfs and find the gutter flow for a given spread of 5.9 ft.

Given: The uniform triangular gutter section has the following parameters:

$$\begin{aligned} S_L &= 0.01 \text{ ft/ft} \\ S_x &= 0.04 \text{ ft/ft} \\ \text{Manning's } n &= 0.02 \end{aligned}$$



Step 1 – Compute spread, T , using **Equation 7.2**.

$$T = \left[\frac{(1.5)(0.02)}{(0.56)(0.04)^{1.67}(0.01)^{0.5}} \right]^{0.375} = 5.9 \text{ ft}$$

Step 2 – Compute flow, Q , using **Equation 7.1**.

$$Q = \left(\frac{0.56}{0.02} \right) (0.04)^{1.67} (0.01)^{0.5} (5.9)^{2.67} = 1.5 \text{ cfs}$$

Alternatively, use the Hydraulic Toolbox:

Step 1 – Compute spread, T. Input the design parameters using the Curb and Gutter tool. Keep the check box for “Define Cross-slope of Gutter” blank and select the “Design Flow” box. Press the “Compute unknown” button. The result of this example calculates a “Width of Spread”, T, of 5.9 ft.

Curb and Gutter Analysis

Gutter

Longitudinal Slope of Road: 0.010 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☐ Define Cross-slope of Gutter 0.040 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 2.000 (ft)

Enter one of the following:

☒ Design Flow: 1.500 (cfs)

☐ Width of Spread: 5.916 (ft)

Compute unknown

Gutter Depression: 0.000 (in)

Area of Flow: 0.700 (ft²)

E_o (Gutter Flow to Total Flow): 0.668

Depth at Curb: 2.840 (in)

Inlet

Inlet Location: Inlet on grade

Percent Clogging: 0.000 (%)

Inlet Types: Grate

Grate Types: Reticuline

Grate Width: 0.000 (ft)

Grate Length: 0.000 (ft)

Length of Inlet: 0.000 (ft)

Curb Opening Height: 0.000 (in)

Local Depression: 0.000 (in)

Compute Inlet Data

Parameter	Value	Unit
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OK Cancel

Step 2 – Compute flow, Q. Using the same design parameters, update the “Width of Spread” with the known 5.9 ft and press the “Compute unknown” button. The result of this example calculates a “Design Flow”, Q, of 1.5 cfs.

7.4.3.4 Composite Gutter Section Procedure

Gutters with composite sections are depressed in relation to the adjacent pavement slope. Because the cross-section is no longer a simple triangle, the total flow is conceptually divided between the flow in the depressed section and the flow in the side section. Based on the composite channel geometry, there is no direct computational solution for estimating spread. Use the FHWA Hydraulic Toolbox to calculate spread for a composite section. Alternatively, spread for a composite section can be calculated using an iterative approach presented in FHWA HEC-22 (2024).

Example: Estimate the spread for a given flow rate of 2.0 cfs and find the gutter flow for a given spread of 6.1 ft. Use FHWA Hydraulic Toolbox.

Given: The composite section has the following characteristics:

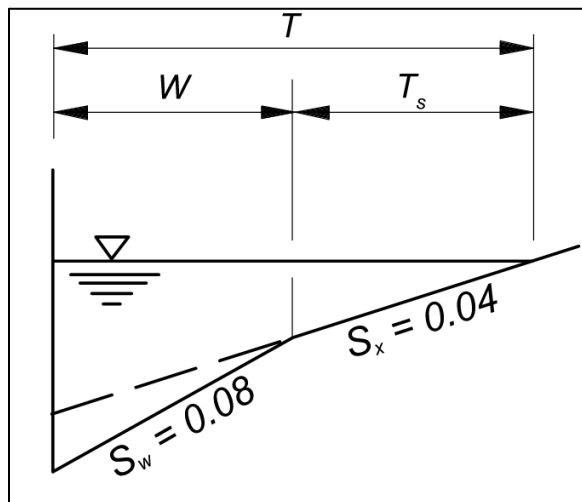
$$S_L = 0.01 \text{ ft/ft}$$

$$S_x = 0.04 \text{ ft/ft}$$

$$S_w = 0.08 \text{ ft/ft}$$

$$W = 2.0 \text{ ft}$$

$$\text{Manning's } n = 0.02$$



Step 1 – Compute spread, T . Input the design parameters using the Curb and Gutter tool. Select the “Define Cross-slope of Gutter” to set the gutter slope. Press the “Compute unknown” button. The result of this example calculates a “Width of Spread”, T , of 6.1 ft.

Step 2 – Compute flow, Q. Using the same design parameters, update the “Width of Spread” with the known 6.1 ft and press the “Compute unknown” button. The result of this example calculates a “Design Flow”, Q, of 2.0 cfs.

7.4.3.5 V-Type Gutter Section Procedure

A V-shape channel section is often adjacent to a barrier or curb section (i.e. valley gutter). To estimate the cross slope, S_x , of a V-shape gutter use **Equation 7.3**:

$$S_x = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \quad (7.3)$$

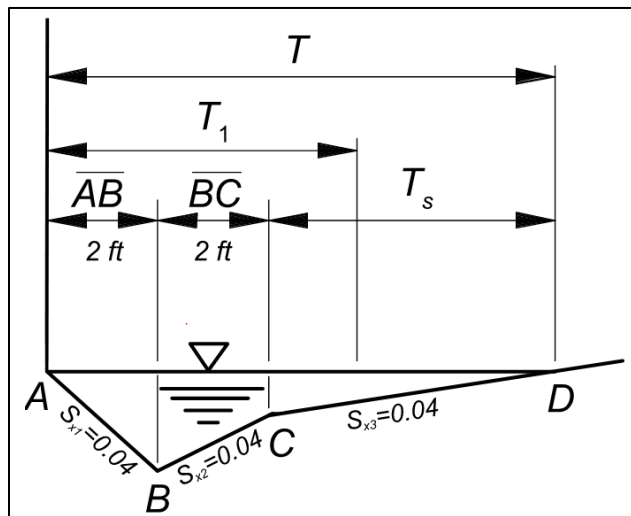
The flow conditions are estimated similar to a composite gutter section. For calculation purposes, S_x is equivalent to S_w and the pavement cross slope S_{x3} is equivalent to S_x in

the composite gutter calculations. Use the FHWA Hydraulic Toolbox to calculate spread for a V-shape gutter section.

Example: Estimate the spread for a given flow rate of 2.0 cfs and find the gutter flow for a given spread of 7.4 ft. Use FHWA Hydraulic Toolbox.

Given: The V-shape gutter section has the following characteristics:

$$\begin{aligned} S_L &= 0.01 \text{ ft/ft} \\ S_{x1} &= 0.04 \text{ ft/ft} \\ S_{x2} &= 0.04 \text{ ft/ft} \\ S_{x3} &= 0.04 \text{ ft/ft} \\ \text{Manning's } n &= 0.02 \end{aligned}$$



Step 1 – Compute spread, T . Calculate S_x , using **Equation 7.3**.

$$S_x = \frac{(0.04)(0.04)}{(0.04 + 0.04)} = 0.02 \frac{\text{ft}}{\text{ft}}$$

Step 2 – Input the design parameters using the Curb and Gutter tool. Select the “Define Cross-slope of Gutter” to set the gutter slope. Press the “Compute unknown” button. The result of this example calculates a “Width of Spread”, T , of 7.4 ft.

Step 3 – Compute flow, Q . Using the same design parameters, update the “Width of Spread” with the known 7.4 ft and press the “Compute unknown” button. The result of this example calculates a “Design Flow”, Q , of 2.0 cfs.

Curb and Gutter Analysis

Gutter

Longitudinal Slope of Road: 0.010 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☒ Define Cross-slope of Gutter: 0.020 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 4.000 (ft)

Enter one of the following:

☒ Design Flow: 2.000 (cfs)

☐ Width of Spread: 7.390 (ft)

Compute unknown

Gutter Depression: -0.960 (in)

Area of Flow: 0.932 (ft²)

E_o (Gutter Flow to Total Flow): 0.830

Depth at Curb: 2.587 (in)

Inlet

Inlet Location: Inlet on grade

Percent Clogging: 0.000 (%)

Inlet Types: Grate

Grate Types: Curved vane

Grate Width: 0.000 (ft)

Grate Length: 0.000 (ft)

Length of Inlet: 0.000 (ft)

Curb Opening Height: 0.000 (in)

Local Depression: 0.000 (in)

Compute Inlet Data

Parameter	Value	Unit
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OK Cancel

7.4.4 Inlet Types

Various types of inlets are used in combination with curb and gutter systems that drain to storm sewer systems. Each inlet type is described as follows with recommended guidelines for use.

Inlets used for the drainage of highway surfaces are divided into four major classes including: grate, curb opening, combination, and slotted or trench drains. **Figure 7-3** shows standard drainage inlet types.

7.4.4.1 Grate Inlets

Grate inlets consist of an opening in the gutter covered by one or more grates. They are most effective for use on continuous grades. Since they are susceptible to clogging with

debris, using standard grate inlets at sag points is limited to minor sag point locations without debris potential. Special design (oversize) grate inlets can be used at major sag points if sufficient capacity is provided for clogging. Provide flanking inlets at major sag points. Use bicycle safe grates where bike traffic is anticipated.

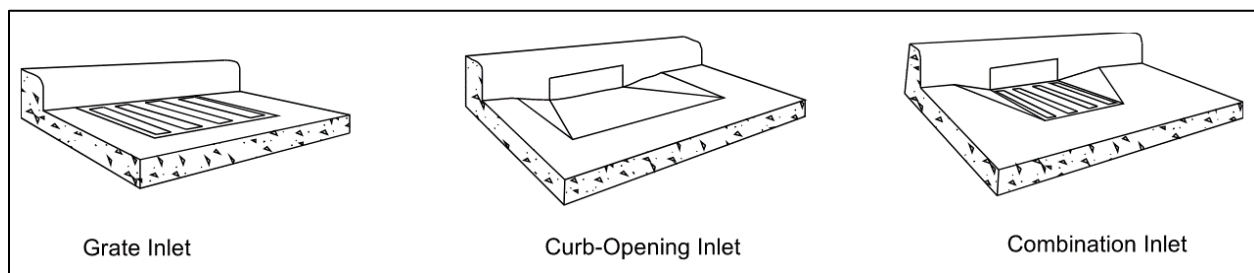
7.4.4.2 Curb Opening Inlets

Curb opening inlets are vertical openings in the curb covered by a top slab. They are most effective for use at sag points since they can convey large quantities of water and debris and on grades less than 3%. They are a viable alternative to grates in many locations where grates would be hazardous for pedestrians or bicyclists. MDOT does not use curb opening inlets.

7.4.4.3 Combination Inlets

Combination inlets that consist of a curb opening and grate inlet in a side-by-side configuration are common. The gutter grade, cross slope, and proximity of the inlets to each other are significant factors when selecting this inlet type. Combination inlets are desirable in sags because they provide additional capacity in the event of clogging.

Figure 7-3. Inlet Types (AASHTO Drainage Manual, 2014)



7.4.4.4 Slotted Drain and Trench Drain Inlets

MDOT does not recommend the use of slotted drain inlets or trench drain inlets for permanent drainage. Slotted and trench drains require frequent maintenance due to clogging. They are used where slopes are very flat and during maintenance of traffic for temporary crossovers. Slotted drains are best used in areas to intercept sheet flow before flow crosses onto a section of roadway. Slotted drains intercept flow over a wide section and reduce ponding depth and spread at grate inlets. Slotted drains consist of a slotted opening with bars perpendicular to the opening for internal support. The slotted opening connects with a round conduit cut along the longitudinal access, allowing large volumes of stormwater to be conveyed downstream. Slotted drains function as weirs with flow entering from the side and can be used to intercept sheet flow, collect gutter flow with or without curbs, and modify existing systems to accommodate roadway widening or increased runoff. Slotted drain inlets are also used in combination with grates, located either longitudinally upstream of the grate or transversely adjacent to the grate, which provide cleanouts and improved access for maintenance. Engineering judgment is necessary to determine if the total capacity of the inlet is the sum of the individual components or a portion of each.

Trench drains are another type of linear drain solution. Trench drains have a wider opening covered with a removable grate resulting in a higher capture rate and less surface clogging. However, trench drains have limited channel body sizes with low conveyance rates. In addition, these systems are not recommended in direct traffic flow due to grate shifting.

7.4.5 Inlet Locations

Inlets are required at locations to collect runoff meeting design criteria (**Section 7.3.3** and **7.3.6**). In addition, many locations require inlets regardless of contributing drainage area. These locations are marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or bypass. Examples of these locations include:

- Place inlets at all low points in gutter grades.
- Place flanking inlets at points on either side of the low point, as required per **Section 7.4.6.6** or a maximum of 75 feet.
- Place inlets upstream of bridges to prevent pavement drainage from flowing onto bridge decks.
- Place inlets downstream of bridges to intercept bridge deck drainage per **Section 7.4.2** to limit erosion.
- Place inlets at locations to prevent water sheeting across the roadway in areas such as the transition to superelevated sections.
- Place inlets on intersecting streets that drain toward the roadway. For concrete pavements, do not place inlets at spring points of street intersections because of interference with construction of the expansion joint. Place at 10 feet on either side of the spring point or at the low spot along the arc as shown on the detailed grades.
- Place inlets upslope of pedestrian cross walks. Do not locate drainage structures in line with a sidewalk ramp. Where existing structures are being used, the ramp location takes precedence over drainage structure locations.
- Place inlets upslope of driveways and intersecting roadways. Do not carry water across intersections.
- Place catch basins behind shoulders or behind sidewalks to drain low spots.

7.4.6 Inlet Spacing

7.4.6.1 General

Plot inlets that are required to collect runoff at locations regardless of contributing drainage area on the plan first. Then, locate inlets from the crest and work downgrade to the sag point. The location of the first inlet from the crest is found by determining the length of pavement and the area behind the curb sloping toward the roadway generating the design runoff. The design runoff is computed as the maximum allowable flow in the gutter, which meets the design criteria as specified in **Section 7.3.3**. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet is calculated using **Equation 7.4**:

$$L = \frac{43,560 Q_t}{C i W} \quad (7.4)$$

Where:

- L = Distance from the crest, ft
- Q_t = Maximum allowable flow, cfs
- C = Composite runoff coefficient for contributing drainage area
- W = Width of contributing drainage area, ft
- i = Rainfall intensity for design frequency, in/hr

This process is also used for bridge deck drainage inlet spacing. Refer to FHWA HEC-21 (1993) for more information.

If the drainage area contributing to the first inlet from the crest is irregular in shape, trial and error is necessary to match a design flow with the maximum allowable flow.

To space successive downgrade inlets, compute the amount of flow that is intercepted by the inlet (Q_i) and subtract it from the total gutter flow to compute the bypass. Add the bypass from the first inlet to the computed flow to the second inlet for a total that cannot exceed the maximum flow meeting the allowable spread. Use **Worksheet 7-1** for inlet spacing computation and to document the spacing calculations.

FHWA has investigated the inlet interception capacity of many types of grate inlets, slotted drain inlets, and combination inlets. FHWA HEC-22 (2024) or FHWA Hydraulic Toolbox may be used to analyze the flow in gutters and the interception capacity of many types of inlets on continuous grades and sags. Both uniform and composite slopes can be analyzed. Calculate inlet capacity to determine spread.

7.4.6.2 Interception Capacity of Grate Inlets on Grade

The capacity of a grate inlet depends on its geometry, cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities with mild slopes, water flowing in the section of gutter

occupied by the grate, frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, side flow, is intercepted. On steep slopes with high velocities, a portion of the frontal flow may splash over the end of the grate. Parallel bar grates are the most efficient grates on steep slopes but are not bicycle safe. For combination inlets where the curb opening and the grate length are equal, the grate is the primary design feature with the curb opening considered as a factor of safety only.

The ratio of frontal flow to total gutter flow, E_o , for a straight cross slope is calculated using **Equation 7.5**:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \quad (7.5)$$

Where:

- E_o = Frontal flow ratio
- Q_w = Flow in width, W, cfs
- Q = Total gutter flow, cfs
- W = Width of depressed gutter or grate, ft
- T = Total spread of water in the gutter, ft

The ratio of side flow, Q_s , to total gutter flow is calculated using **Equation 7.6**:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \quad (7.6)$$

The ratio of frontal flow intercepted to total frontal flow, R_f , is calculated using **Equation 7.7**:

$$R_f = 1 - 0.09 (V - V_o) \quad (7.7)$$

Where:

- V = Velocity of flow in the gutter, fps
- V_o = Gutter velocity where splash-over first occurs, fps

This ratio is equivalent to the frontal flow interception efficiency.

The ratio of side flow intercepted to total side flow, R_s , or side flow interception efficiency, is calculated using **Equation 7.8**:

$$R_s = \frac{1}{1 + \frac{0.15 V^{1.8}}{S_x L^{2.3}}} \quad (7.8)$$

Where:

V = Velocity of flow in gutter, fps

S_x = Cross slope, ft/ft

L = Length of the grate, ft

The efficiency, E , of a grate is calculated using by **Equation 7.9**:

$$E = R_f E_o + R_s (1 - E_o) \quad (7.9)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow as shown in **Equation 7.10**:

$$Q_i = E Q = Q [R_f E_o + R_s (1 - E_o)] \quad (7.10)$$

Example: Estimate the interception capacity of a Cover K grate (1.55 ft wide x 1.85 ft long) on grade using 30% clogging. Use FHWA Hydraulic Toolbox.

Given: The composite gutter section has the following characteristics:

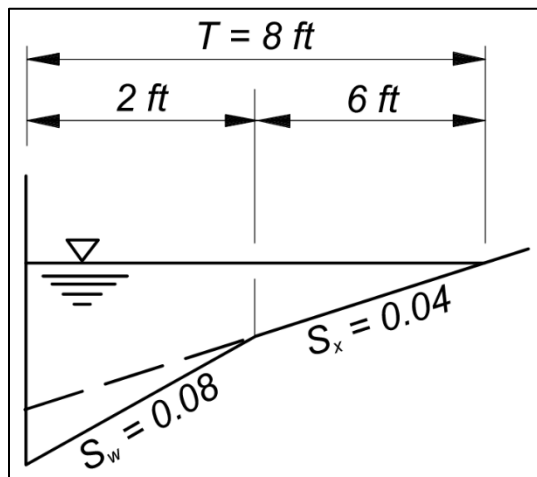
$T = 8.0$ ft

$S_L = 0.01$ ft/ft

$S_x = 0.04$ ft/ft

$S_w = 0.08$ ft/ft

Manning's $n = 0.020$



Curb and Gutter Analysis

Gutter

Longitudinal Slope of Road: 0.010 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☒ Define Cross-slope of Gutter: 0.080 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 2.000 (ft)

Enter one of the following:

☐ Design Flow: 3.819 (cfs)

☒ Width of Spread: 8.000 (ft)

Compute unknown

Gutter Depression: 0.960 (in)

Area of Flow: 1.360 (ft²)

E_o (Gutter Flow to Total Flow): 0.593

Depth at Curb: 4.800 (in)

Inlet

Inlet Location: Inlet on grade

Percent Clogging: 0.000 (%)

Inlet Types: Grate

Grate Types: Reticuline

Grate Width: 1.090 (ft)

Grate Length: 1.850 (ft)

Length of Inlet: 0.000 (ft)

Curb Opening Height: 0.000 (in)

Local Depression: 0.000 (in)

Compute Inlet Data

Parameter	Value	Units
E _o	0.360	
Intercepted Flow	1.731	cfs
Bypass Flow	2.087	cfs
Approach Velocity	2.808	fps
Splash-over Velocity	3.946	fps
Efficiency	0.453	

OK Cancel

Step 1 – Input the design parameters using the Curb and Gutter tool. To represent 30% clogging, reduce the grate width by 30% or 1.09 ft for this example. Press the “Compute Inlet Data” button. The result of this example calculates “Design Flow”, $Q = 3.82$ cfs, “Intercepted Flow”, $Q_i = 1.73$ cfs, and “Efficiency”, $E = 0.45$.

7.4.6.3 Interception Capacity of Grate Inlets in Sag

Combination inlets are preferred to grate inlets at a sag where debris is likely, such as on a city street. However, grate inlets may be used for minor sag points where debris potential is limited. One example of a minor sag point is where a ramp joins a mainline.

Provide flanking inlets at major sag points. Major sag points include, but are not limited to, interstates, freeways, or underpasses and sag vertical curves that have no outlet

except through the drainage system. Refer to **Section 7.4.6.6** for more information regarding flanking inlets.

A grate inlet in a sag location operates as a weir at the perimeter to depths dependent on the size of the grate and as an orifice over the clear opening area at greater depths. Between these depths, a transition from weir to orifice flow occurs. Calculate depths for the weir and orifice equations and use the highest value for design. The capacity of a grate inlet operating as a weir is calculated using **Equation 7.11**:

$$Q_i = C_w \sqrt{2g} P d^{1.5} \quad (7.11)$$

Where:

Q_i = Intercepted flow, cfs

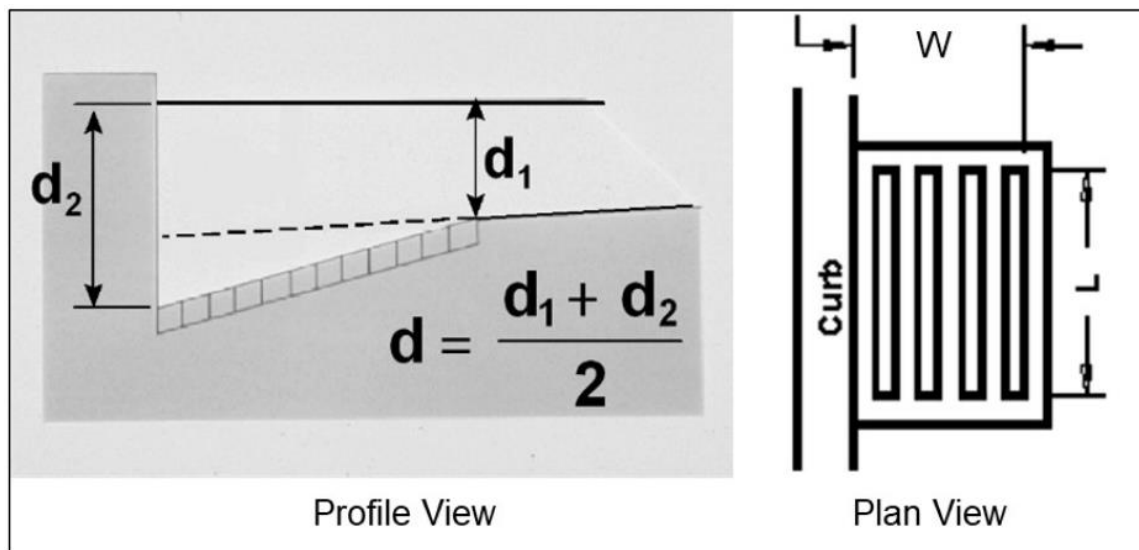
C_w = 0.37, weir coefficient

P = Perimeter of grate excluding side against curb ($P = 2W + L$), ft

d = Average depth across the grate $(d_1 + d_2)/2$, ft (refer to **Figure 7-4**)

g = Gravitational acceleration, 32.2 ft/s²

Figure 7-4. Plan and Profile Views of a Grate Inlet (FHWA HEC-22, 2024)



The capacity of a grate inlet operating as an orifice is calculated using **Equation 7.12**:

$$Q_i = C_o A_g (2 g d)^{0.5} \quad (7.12)$$

Where:

Q_i = Intercepted flow, cfs

C_o = 0.6 orifice coefficient for all other covers

A_g = Clear opening area of the grate, sft

g = Gravitational acceleration, 32.2 ft/s²

Example: Find the capacity of a Cover K grate (1.55 ft wide x 1.85 ft long) in a sag using 50% clogging. Use FHWA Hydraulic Toolbox.

Given: The composite gutter section has the following characteristics

T (allowable) = 8.0 ft

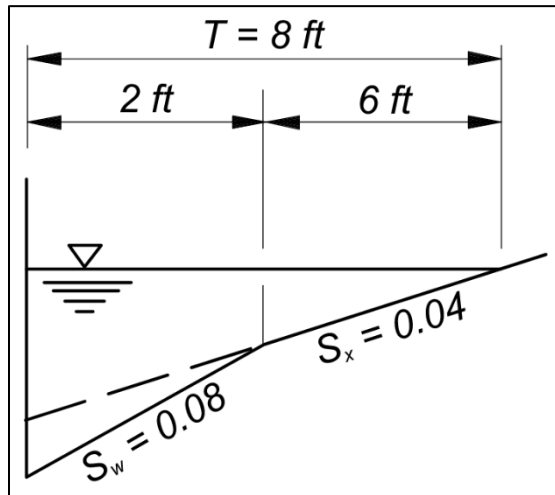
Q = 2.5 cfs

S_L = 0.010 ft/ft

S_x = 0.04 ft/ft

S_w = 0.08 ft/ft (continuous gutter depression, a = 0.96 in)

Manning's n = 0.020



Step 1 – Input the design parameters using the Curb and Gutter tool. Once all parameters have been entered, press the “Compute Inlet Data” button. The result indicates the “Computed Width of Spread at Sag” = 9.09 ft. This spread exceeds the allowable spread of 8.0 ft. Try two grates.

Curb and Gutter Analysis

Gutter

Longitudinal Slope of Road: 0.010 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☒ Define Cross-slope of Gutter: 0.080 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 2.000 (ft)

Enter one of the following:

☒ Design Flow: 1.750 (cfs)

☐ Width of Spread: 5.731 (ft)

Gutter Depression: 0.960 (in)

Area of Flow: 0.737 (ft²)

E_o (Gutter Flow to Total Flow): 0.750

Depth at Curb: 3.711 (in)

Inlet

Inlet Location: Inlet in sag

Percent Clogging: 50.000 (%)

Inlet Types: Grate

Grate Types: Reticuline

Grate Width: 1.550 (ft)

Grate Length: 1.850 (ft)

Length of Inlet: 0.000 (ft)

Curb Opening Height: 0.000 (in)

Local Depression: 0.000 (in)

Parameter	Value	Units
Perimeter	4.950	ft
Effective Perimeter	2.475	ft
Area	2.294	ft ²
Effective Area	1.147	ft ²
Depth at center of grate	0.382	ft
Computed Width of Spread at Sag	9.089	ft
Flow type	Weir Flow	
Efficiency	1.000	

Step 2 – Update the grate length to 3.7 ft to represent the length of two grates and press the “Compute Inlet Data” button. The result indicates the “Computed Width of Spread at Sag” = 7.26 ft. This spread meets the allowable spread of 8.0 ft.

Curb and Gutter Analysis
✕

Gutter

Longitudinal Slope of Road: (ft/ft)

Cross-slope of Pavement: (ft/ft)

☒ Define Cross-slope of Gutter: (ft/ft)

Manning's Roughness:

Gutter Width: (ft)

Enter one of the following:

☒ Design Flow: (cfs)

☐ Width of Spread: (ft)

Gutter Depression: (in)

Area of Flow: (ft²)

E_o (Gutter Flow to Total Flow):

Depth at Curb: (in)

Inlet

Inlet Location:

Percent Clogging: (%)

Inlet Types:

Grate Types:

Grate Width: (ft)

Grate Length: (ft)

Length of Inlet: (ft)

Curb Opening Height: (in)

Local Depression: (in)

Parameter	Value	Units
Perimeter	6.800	ft
Effective Perimeter	3.400	ft
Area	4.588	ft ²
Effective Area	2.294	ft ²
Depth at center of grate	0.309	ft
Computed Width of Spread at Sag	7.269	ft
Flow type	Weir Flow	
Efficiency	1.000	

7.4.6.4 Interception Capacity of Curb Opening Inlets or Slotted Inlets on Grade

Curb opening inlets are effective in the drainage of highway pavements when the flow depth at the curb is sufficient for the inlet to perform efficiently. Curb opening inlets are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists. Slotted inlets use the same procedures presented in this section. Design spillways (R-32 and R-35 Series) as curb opening inlets.

The length of curb opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is calculated using **Equation 7.13**:

$$L_T = K_u Q^{0.42} S_L^{0.3} \left[\frac{1}{n S_x} \right]^{0.6} \quad (7.13)$$

Where:

- L_T = Curb opening length required to intercept 100% of the gutter flow, ft.
- K_u = 0.6
- S_L = Longitudinal slope, ft/ft
- S_x = Cross slope, ft/ft
- Q = Gutter Flow, cfs

The length of curb opening in **Equation 7.13** is based on FHWA HEC-22 (2024). Current software, such as FHWA Hydraulic Toolbox and OpenRoads Designer Drainage and Utilities, also use HEC-22 equations. However, FHWA recently researched this subject and developed a more conservative equation documented in *Curb-Opening Inlet Interception On Grade* (FHWA-HRT-22-061, 2022).

The efficiency of curb opening inlets shorter than the length required for total interception is calculated using **Equation 7.14**:

$$E = 1 - \left(1 - \frac{L}{L_T} \right)^{1.8} \quad (7.14)$$

Where:

- L = Curb opening length, ft

The length of inlet required for total interception by depressed curb openings in depressed gutter sections is found by using an equivalent cross slope, S_e , in **Equation 7.13**.

$$S_e = S_x + S'_w E_o \quad (7.15)$$

Where:

- S'_w = Cross slope of the gutter measured from the pavement cross slope, computed as $a/(12W)$, ft/ft
- a = Gutter depression, in
- W = Gutter width, ft
- E_o = Frontal flow ratio for the depressed section determined by the gutter configuration upstream of the inlet
- Note: S_e is used to calculate the length of curb opening by substituting S_e for S_x in **Equation 7.13**.

Example: Find the interception capacity of an intermediate curb opening in the curb and gutter (spillway) that is 12 ft long: A) without a depressed gutter section, and B) with a depressed gutter section. Use FHWA Hydraulic Toolbox for B.

Given: $S_L = 0.01$ ft/ft
 $S_x = 0.04$ ft/ft
 $n = 0.020$
 $Q = 3.0$ cfs
 $L = 12$ ft

For the gutter depression:

$a = 1$ in
 $W = 2$ ft

Step A1 - Determine the length of the curb opening required for total interception of gutter flow without gutter depression using **Equation 7.13**.

$$L_T = 0.6 (3.0)^{0.42} (0.01)^{0.3} \left[\frac{1}{(0.020)(0.04)} \right]^{0.6} = 17.2 \text{ ft}$$

Step A2 - Compute the curb opening efficiency without gutter depression using **Equation 7.14**.

$$E = 1 - \left(1 - \frac{12}{17.2} \right)^{1.8} = 0.88$$

Step A3 - Compute the interception capacity without gutter depression.

$$Q_i = (0.88)(2.0) = 2.64 \text{ cfs}$$

Alternatively, Hydraulic Toolbox input and results:

Curb and Gutter Analysis [X]

Gutter

Longitudinal Slope of Road: 0.010 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☐ Define Cross-slope of Gutter: 0.040 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 2.000 (ft)

Enter one of the following:

☒ Design Flow: 3.000 (cfs)

☐ Width of Spread: 7.673 (ft)

Compute unknown

Gutter Depression: 0.000 (in)

Area of Flow: 1.177 (ft²)

E_o (Gutter Flow to Total Flow): 0.554

Depth at Curb: 3.683 (in)

Inlet

Inlet Location: Inlet on grade

Percent Clogging: 0.000 (%)

Inlet Types: Curb opening

Grate Types: Reticuline

Grate Width: 0.000 (ft)

Grate Length: 0.000 (ft)

Length of Inlet: 12.000 (ft)

Curb Opening Height: 0.000 (in)

Local Depression: 0.000 (in)

Compute Inlet Data

Parameter	Value	Units
Intercepted Flow	2.648	cfs
Bypass Flow	0.352	cfs
Approach Velocity	2.548	fps
Efficiency	0.883	

OK Cancel

Step B1 – Compute curb opening interception using Hydraulic Toolbox due to a composite section requiring an iterative solution. Update the local depression to 1 inch and press the “Compute Inlet Data” button. The result indicates the “Intercepted Flow” is 2.96 cfs.

Curb and Gutter Analysis

Gutter

Longitudinal Slope of Road: 0.010 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☐ Define Cross-slope of Gutter: 0.040 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 2.000 (ft)

Enter one of the following:

☒ Design Flow: 3.000 (cfs)

☐ Width of Spread: 7.673 (ft)

Compute unknown

Gutter Depression: 0.000 (in)

Area of Flow: 1.177 (ft²)

E_o (Gutter Flow to Total Flow): 0.554

Depth at Curb: 3.683 (in)

Inlet

Inlet Location: Inlet on grade

Percent Clogging: 0.000 (%)

Inlet Types: Curb opening

Grate Types: Reticuline

Grate Width: 0.000 (ft)

Grate Length: 0.000 (ft)

Length of Inlet: 12.000 (ft)

Curb Opening Height: 0.000 (in)

Local Depression: 1.000 (in)

Compute Inlet Data

Parameter	Value	Units
Intercepted Flow	2.964	cfs
Bypass Flow	0.036	cfs
Approach Velocity	2.548	fps
Efficiency	0.988	

OK Cancel

7.4.6.5 Interception Capacity of Curb Opening Inlets in Sag

The capacity of a curb opening inlet in a sag depends on the water depth at the curb, curb opening length, and height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage. Design spillways (R-32 and R-35 Series) as curb opening inlets.

The weir equation for curb opening inlets without depression is calculated using **Equation 7.16**:

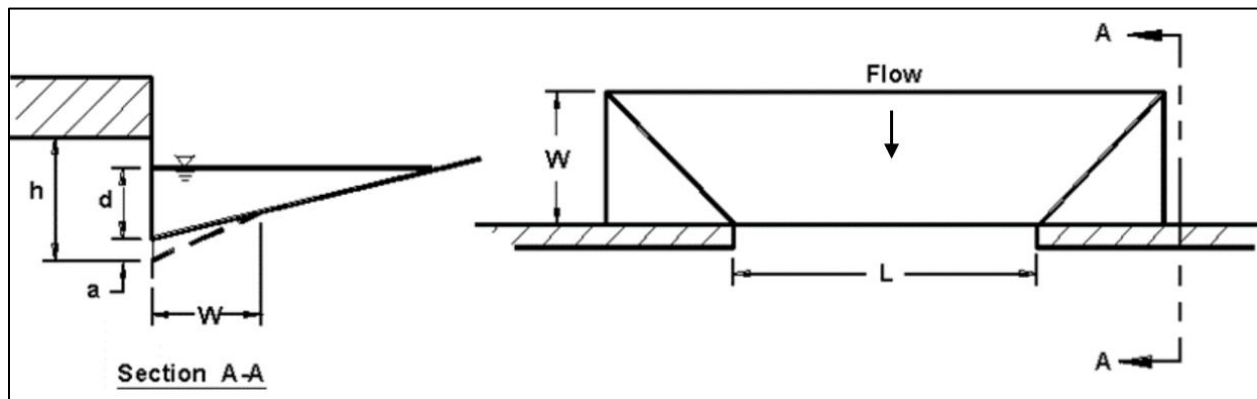
$$Q_i = C_w \sqrt{2g} L d^{1.5} \quad (7.16)$$

Where:

- C_w = 0.37, weir coefficient
- L = Length of curb opening, ft
- D = Depth at curb, ft
- g = Gravitational acceleration, 32.2 ft/s²

For curb opening inlets without depression, the curb opening operates as a weir up to the curb opening height, h . The weir location is at the lip of the curb opening and the curb opening length equals the inlet length as shown in **Figure 7-5**. Use this figure for spillway design.

Figure 7-5. Curb Opening Inlet Definition Sketch (FHWA HEC-22, 2024)



For depressed curb opening inlets with a combination gutter, the weir location is at the edge of the gutter and the effective weir length depends on the width of the depressed gutter and length of the curb opening. The interception capacity of a depressed curb opening inlet operating as a weir is calculated using **Equation 7.17**:

$$Q_i = C_w \sqrt{2g} (L + 1.8 W) d^{1.5} \quad (7.17)$$

Where:

- C_w = 0.29, weir coefficient
- L = Length of curb opening, ft
- W = Width of depression, ft
- d = Depth of water at curb measured from the pavement cross slope (S_x), ft
- g = Gravitational acceleration, 32.2 ft/s²

The weir equation applies to depths at the curb. For weir flow at a depressed curb inlet is calculated using **Equation 7.18**, the depth at the curb is required to be less than the curb opening height:

$$d_i \leq h = d + a = T S_x + \frac{a}{12} \quad (7.18)$$

Where:

- d_i = Depth at the curb with local depression, ft
- h = Curb opening height, ft
- d = Depth of water at curb measured from the pavement cross slope (S_x), ft
- a = Depth of depression, ft

Although laboratory experiments for FHWA did not include curb opening inlets with a continuously depressed gutter, FHWA expects that the effective weir length described in **Equation 7.17** is greater than or equal to the length of a continuously depressed gutter. Therefore, **Equation 7.17** provides conservative estimates of the interception capacity in this situation.

For curb opening lengths greater than 12 ft, **Equation 7.16** for a non-depressed inlet produces intercepted flows which exceed the values for depressed inlets computed using **Equation 7.17**. Since depressed inlets perform as well as non-depressed inlets of the same length, **Equation 7.16** is accurate for all curb opening inlets longer than 12 ft.

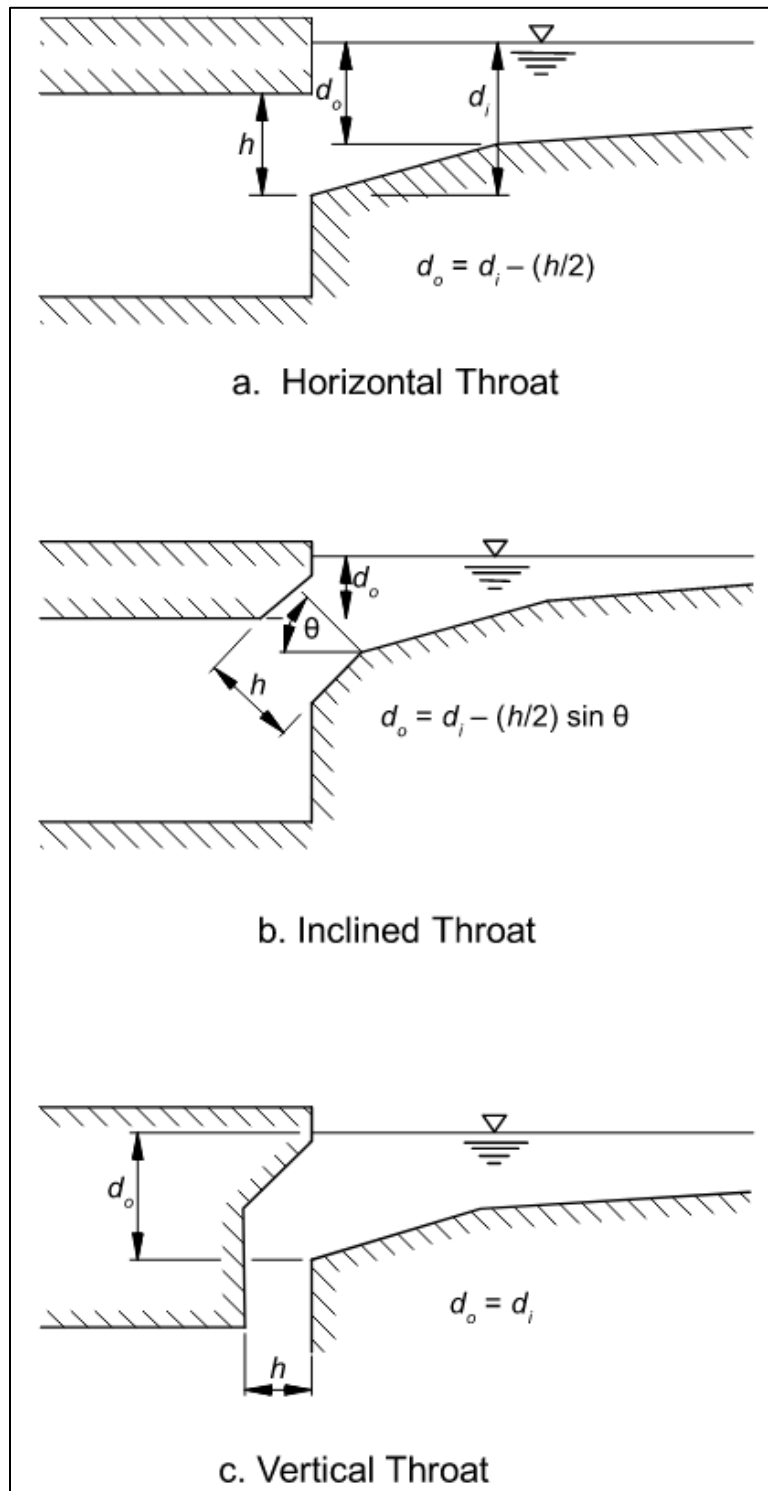
Curb opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity is calculated using **Equation 7.19**:

$$Q_i = C_o A_o (2 g d_o)^{0.5} \quad (7.19)$$

Where:

- C_o = 0.67, orifice coefficient
- A_o = Clear area of opening, sft
- d_o = Effective head on the center of the orifice throat, ft (refer to **Figure 7.6**)

Equation 7.19 is applicable to depressed and undepressed curb opening inlets. The depth at the inlet includes any gutter depression.

Figure 7-6. Curb Opening Inlets (AASHTO Drainage Manual, 2014)

Example: Find the spread of an opening in the curb and gutter (spillway) that is 14 ft long located in a sag: A) without a depressed gutter section, and B) with a depressed gutter section.

Given: $S_x = 0.04$ ft/ft
 $n = 0.020$
 $Q_i = 3.0$ cfs
 $L = 14$ ft
 $h = 4$ in (0.33 ft)
 $T = 8$ ft (allowable)

For local depression:

$a = 1$ in
 $W = 2$ ft

Step A1 - Determine depth at curb for undepressed inlet using **Equation 7.16**.

$$3.0 \text{ cfs} = (0.37) (2 \times 32.2 \text{ ft/s}^2)^{0.5} (14 \text{ ft}) (d)^{1.5}$$

$$d = (3.0 \text{ cfs} / ((0.37) (2 \times 32.2 \text{ ft/s}^2)^{0.5} (14 \text{ ft})))^{(1/1.5)}$$

$$d = 0.17 \text{ ft}$$

Check if weir flow controls:

$$0.17 \text{ ft} < 0.33 \text{ ft weir flow controls}$$

Step A2 - Determine spread for undepressed inlet.

$$T = d / S_x = (0.17 \text{ ft}) / (0.04 \text{ ft/ft}) = 4.25 \text{ ft}$$

The computed spread of 4.25 ft meets the allowable spread of 8 ft at the sag.

Alternatively, Hydraulic Toolbox input and results:

Curb and Gutter Analysis

Gutter

Longitudinal Slope of Road: 0.003 (ft/ft)

Cross-slope of Pavement: 0.040 (ft/ft)

☐ Define Cross-slope of Gutter: 0.040 (ft/ft)

Manning's Roughness: 0.020

Gutter Width: 2.000 (ft)

Enter one of the following:

☒ Design Flow: 3.000 (cfs)

☐ Width of Spread: 9.616 (ft)

Compute unknown

Gutter Depression: 0.000 (in)

Area of Flow: 1.849 (ft²)

E_o (Gutter Flow to Total Flow): 0.463

Depth at Curb: 4.616 (in)

Inlet

Inlet Location: Inlet in sag

Percent Clogging: 0.000 (%)

Inlet Types: Curb opening

Grate Types: Reticuline

Grate Width: 0.000 (ft)

Grate Length: 0.000 (ft)

Length of Inlet: 14.000 (ft)

Curb Opening Height: 4.000 (in)

Local Depression: 0.000 (in)

Compute Inlet Data

Parameter	Value	Units
Perimeter	14.000	ft
Effective Perimeter	14.000	ft
Area	4.667	ft ²
Effective Area	4.667	ft ²
Depth at curb face	0.172	ft
Computed Width of Spread at Sag	4.304	ft
Flow type	Weir Flow	
Efficiency	1.000	

OK Cancel

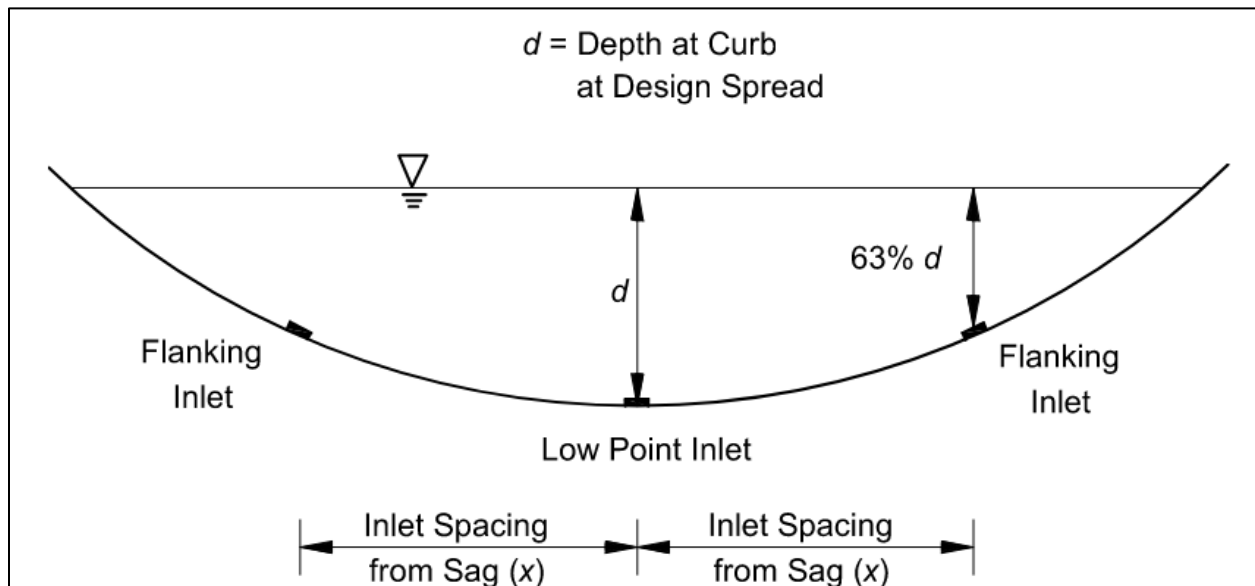
Step B1 – Determine depth at the curb for depressed inlet. For curb opening lengths greater than 12 ft, **Equation 7.16** for non-depressed inlets produce intercepted flows that would exceed **Equation 7.17**. Since the depressed inlet will perform as well as the non-depressed inlets of the same length, use **Equation 7.16**. The result matches Part A of the example problem.

7.4.6.6 Flanking Inlets

Inlets are required at low points in the gutter profile to provide adequate drainage. At major sag points where there is a high risk of the sag inlet clogging, provide flanking inlets on each side of the sag inlet. **Figure 7-7** illustrates the location of flanking inlets that relieve flow when the sag inlet is clogged. If the flanking inlets are the same

dimension as the sag inlet, each inlet intercepts half of the flow when they are located at 63% of the ponding depth at the low point.

Figure 7-7. Location of Flanking and Sag Inlets (AASHTO Drainage Manual, 2014)



For major sags, locate and size the flanking inlets to receive the design flow when the primary inlet at the bottom of the sag is clogged without exceeding the allowable spread. Perform spread calculations following standard inlet spacing design procedures described in **Section 7.4.6.7**. Spread calculation requirements for sag locations are as follows:

- Perform the initial spread calculations using the design year for sags according to **Table 7-1** and assume the sag grate is half clogged. Do not include flanking inlets in the initial spread calculations.
- If flanking inlets are required, perform an additional resiliency check calculation for the sag spread calculations. Include the flanking inlets in the spread calculations with the sag grate fully clogged for the sag design year. Confirm allowable spread is not exceeded at the flanking basins.

The spacing required for the allowable depth at a vertical sag curb is calculated using **Equation 7.20**:

$$X = (74 dK)^{0.5} \quad (7.20)$$

Where:

- X = Maximum distance from bottom of sag to the flanking inlet, ft
- d = Depth of water over the inlet in the sag, ft
- K = Vertical curve constant, ft/percent

The vertical curve constant is calculated using **Equation 7.21**:

$$K = \frac{L}{(G_2 - G_1)} \quad (7.21)$$

Where:

L = Horizontal length of the vertical curve, ft
 G_1, G_2 = Approach grades, percent

The AASHTO policy on geometrics specifies maximum K values for various design speeds and a maximum K of 167 considering drainage.

Example: Determine the location of flanking inlets relative to the inlet at the low point.

Given: A sag vertical curve at an underpass on a 4-lane divided highway with an allowable spread of 10 feet. Assume all inlets have the same dimensions.

$G_1 = -2.5\%$
 $G_2 = +2.5\%$
 $L = 500 \text{ ft}$
 $S_x = 0.02 \text{ ft/ft}$

Step 1 - Find the vertical curve constant, K.

$$K = L / (G_2 - G_1) = 500 / (2.5 - (-2.5)) = 100 \text{ ft/percent}$$

Step 2 - Determine the depth at the curb for design spread.

$$D = S_x T = (0.02) (10) = 0.2 \text{ ft}$$

Step 3 - Determine the flanking inlet locations.

$$X = (74 d K)^{0.5} = ((74) (0.2)(100))^{0.5} = 38.5 \text{ ft}$$

Flanking inlets are located 38.5 ft from the sag meet the allowable spread.

7.4.6.7 Design Procedure for Inlet Spacing

Use OpenRoads Designer Drainage and Utilities (ORD D&U) for spread analysis except for valley gutter spread calculations. ORD D&U does not properly calculate the valley gutter spread. Refer to the Drainage and Utilities Guidance software guide on the MDOT Development Guide SharePoint Site. Refer to **Appendix 7-C** for ORD Drainage and Utilities Hydraulic Settings.

Spread calculations, including valley gutter, can also be completed manually using a spreadsheet or **Worksheet 7-1**. A step-by-step procedure follows:

- Step 1** Complete the worksheet header with the date, route, job number, control section, designer initials, checker initials, design frequency, and allowable spread.
- Step 2** On the plan, mark the location of inlets that are necessary regardless of any specific drainage area. Refer to **Section 7.4.5** for additional information.
- Step 3** Start at one high point and work towards the low point. Then space inlets from the other high point back to the same low point.
- Step 4** Select a trial drainage area approximately 300 ft below the high point and outline the area including any area that drains over the curb. Delineate areas on a drainage area map. Where practical, intercept large drainage areas behind the curb before it reaches the highway.
- Step 5** Describe the location of the proposed inlet by number and station in Col. 1 and Col. 2. Identify the curb and gutter type in Col. 19, Remarks. Sketch the cross section with cross slopes and dimensions in the open area of the computation sheet.
- Step 6** Compute the drainage area in acres and enter in Col. 3.
- Step 7** Select a C value from the table in Chapter 3, Hydrology, or compute a weighted C value based on area and cover type and enter in Col. 4.
- Step 8** Use the T_c from **Section 7.3.2**, typically 10 or 15 minutes. Enter value in Col. 5.
- Step 9** Using the T_c , determine the intensity. Refer to Chapter 3, Hydrology, Appendix 3-B to obtain an intensity at the T_c for the design frequency. Enter in Col. 6.
- Step 10** Calculate Q by multiplying Col. 3 by Col. 4 by Col. 6. Enter in Col. 7.
- Step 11** Determine the gutter longitudinal slope, S_L , at the inlet from the roadway profile; check effect of superelevation. Enter in Col. 8.
- Step 12** Enter cross slope, S_x , adjacent to inlet in Col. 9 and gutter width, W, in Col. 13.
- Step 13** For the first inlet in a series (high point to low point), enter Col. 7 in Col. 11 since no previous bypass has occurred and enter 0 into Col. 10.
- Step 14** Determine the spread, T, and enter in Col. 14. Calculate the depth, d, at the curb by multiplying T by the cross slope(s) and enter in Col. 12. Compare the calculated spread with the allowable spread and the calculated depth of curb with the actual curb height noted in Col. 19. If the calculated spread is near the allowable spread and the depth, d, at the curb is less than the actual curb

height, continue to Step 15. Otherwise, revise the drainage area to meet the criteria and repeat Steps 5 through 14 until Col. 14 meets the allowable spread before proceeding to Step 15.

- Step 15** Calculate W/T and enter in Col. 15.
- Step 16** Select the inlet type and dimensions and enter in Col. 16.
- Step 17** Calculate the Q intercepted, Q_i , by the inlet and enter in Col. 17. Use equations or Hydraulic Toolbox to define gutter flow and intercepted flow by the inlet.
- Step 18** Calculate the bypass flow, Q_b , by subtracting Col. 17 from Col. 11 and enter in Col. 18.
- Step 19** Proceed to the next inlet downgrade. Place an inlet approximately 300 feet below the first inlet as a trial. Repeat Steps 5 through 7 considering only the area between the inlets.
- Step 20** Use the T_c from **Section 7.3.2** for the second inlet downgrade.
- Step 21** Determine the intensity based on the T_c determined in Step 20 and enter it in Col. 6.
- Step 22** Calculate Q from this area by multiplying Col. 3 by Col. 4 by Col. 6. Enter in Col. 7.
- Step 23** Record the value from Col. 18 of the previous line in Col 10 of the current line. Determine total gutter flow by adding Col. 7 and Col. 10, and enter in Col. 11.
- Step 24** Determine spread, T , and the depth at the curb, d , as outlined in Step 14. Repeat Steps 18 through 24 until Col. 14 meets the allowable spread.
- Step 25** Select inlet type and dimensions and enter in Col. 16.
- Step 26** Determine Q_i and enter in Col. 17 in accordance with Step 17.
- Step 27** Calculate the bypass by subtracting Col. 17 from Col. 11, and enter in Col. 18. This completes the spacing design for this inlet.
- Step 28** Go back to Step 19 and repeat Step 19 through Step 27 for each subsequent inlet down to the low point.

Worksheet 7-1. Inlet Spacing Calculations

INLET COMPUTATION SHEET DATE _____ ROUTE _____ JN _____ CONTROL SECTION _____ COMPUTED BY _____ SHEET ____ OF ____ CHECKED BY _____ 																		
LOCATION		GUTTER DISCHARGE Design Frequency _____					GUTTER DISCHARGE Allowable Spread _____								INLET DISCHARGE			REMARKS
Inlet No.	Sta.	Drain Area "A" (acres)	Runoff Coif "C"	Time of Conc. "Tc" (min)	Rain Intensity "I" (in/hr)	Q = CiA (cfs)	Grade "SL" (ft/ft)	Cross Slope "Sx" (ft/ft)	Prev Bypass (cfs)	Total Gutter Flow (cfs)	Depth "d" (ft)	Gutter Width "W" (ft)	Spread "T" (ft)	W/T	Inlet TYPE	Intercept "Qi" (cfs)	Bypass "Qb" (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

7.5 DRAINAGE STRUCTURES

7.5.1 Manhole Location and Spacing

Locate manholes where:

- Two or more pipes converge
- Pipe size changes
- Horizontal alignment changes
- Vertical alignment changes
- Pipe material changes (i.e., CMP to RCP).
- Additional policy on manhole locations is provided in **Section 7.3.6**.

In addition, locate manholes at intermediate points along tangent sections to facilitate maintenance in accordance with the following criteria:

- Space manholes no more than 300 feet apart for trunk sewers 36-inch diameter and under.
- Space manholes no more than 100 pipe diameters apart for trunk sewers 42-inch diameter and over with a maximum spacing of 1,200 feet.

7.5.2 Drainage Structure Types

Manholes to access storm sewers are detailed on the following MDOT Standard Plans:

- Standard Plan R-1-Series, Drainage Structures – manhole structure for new or existing sewers 48 inches and under.
- Standard Plan R-2-Series, Manhole Base Type 1 – intermediate manhole between junctures on new or existing sewers 48-inch through 108-inch. Manhole Base Type 1 may be substituted for Precast Manhole Tee.
- Standard Plan R-3-Series, Precast Manhole Tees – tee for use on sewers 42-inch through 144-inch. Precast Manhole Tees may be used as an intermediate manhole on new sewers between sewer junctures.
- Standard Plan R-4-Series, Manhole Base Type 2 – manhole structure at sewer junctions of new or existing sewers 48-inch through 108-inch.

Catch basin and inlets are detailed on MDOT Standard Plan R-1 Series:

- Catch basins have a sump and inlets do not have a sump. Use catch basins unless site constraints do not have clearance for a sump (i.e. utility conflict, etc.). In the case of a site constraint, an inlet may be used.
- The diameter and sump shown on the standard plan provides the minimum dimensions. An inlet (no sump) does not contribute towards a project's stormwater credit. Refer to the [MDOT Stormwater Treatment Manual](#) for catch basin diameter and sump sizing to meet MS4 requirements.

7.5.3 Drainage Structure Sizing

7.5.3.1 Catch Basin Structure Diameter

Use 24-inch-diameter catch basins at upstream ends of 12-inch-diameter sewer runs where the pipe length is 65 feet or less and where the structure depth does not exceed 8 feet including any additional depth utilized for stormwater treatment. Additionally, do not tap underdrains to 24-inch catch basins. If the 24-inch diameter requirements are not met, use 48-inch-diameter or larger drainage structures for catch basins.

7.5.3.2 Manhole Structure Sizing Considerations

Size a manhole or manhole base using the maximum pipe size shown in **Table 7-4** when the deflection angle of inlet and outlet pipes is between 135 and 180 degrees.

If the deflection angle between the pipes is less than 135 degrees, the minimum manhole size is required to meet the following criteria:

- Manhole or inlet structure is required to be large enough to accommodate the maximum pipe as shown in **Table 7-4**.
- Maintain a minimum of 12 inches between the outside of each pipe (measured on the inside of the manhole) using **Equation 7.22**.

Equation 7.22 is used to confirm the minimum distance between the outside of each pipe is met.

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 14}{\Delta} \quad (7.22)$$

Where:

- K = Manhole sizing coefficient, in/degree
- R₁ = Interior radius of pipe no. 1, in
- T₁ = Wall thickness of pipe no. 1, in
- R₂ = Interior radius of pipe no. 2, in
- T₂ = Wall thickness of pipe no. 2, in
- Δ = Angle between the pipes (measured from the centerline of each pipe), degrees

In cases of complex geometry, lay out the manhole using CAD to ensure the minimum 12-inch distance between pipe openings is maintained.

Table 7-4. Manhole or Manhole Base Maximum Pipe Sizes

Manhole or Manhole Base Diameter (Inch)	K (Inch/Degree)	Maximum Pipe Size (Inch)
48	0.42	24
60	0.52	36
72	0.63	48
84	0.73	60
96	0.84	72
108	0.94	84
120	1.05	96

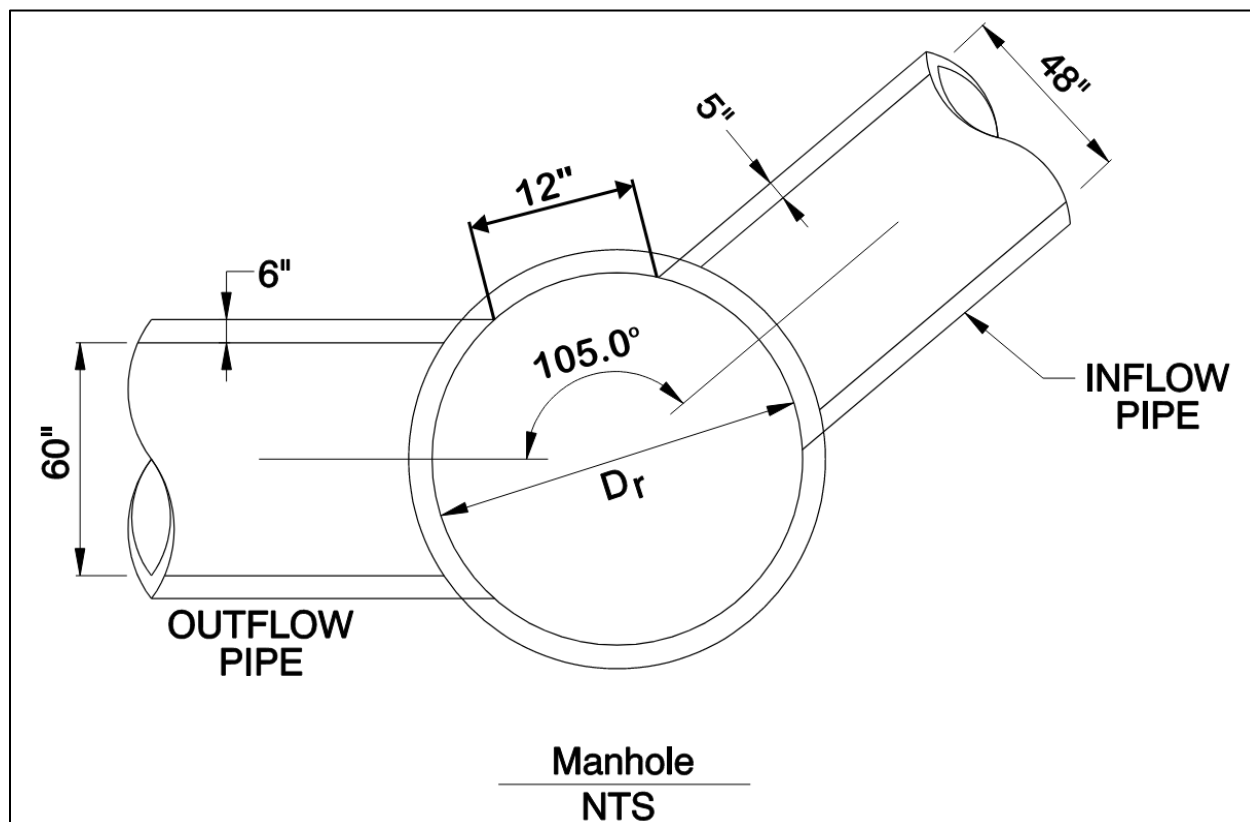
Example: Determine the required manhole diameter.

Given: Manhole configuration has the following design parameters.

Pipe No. 1 = 60 in

Pipe No. 2 = 48 in

Δ = 105 degrees



Step 1 - Determine if the angle between the two pipes is less than 135 degrees:

The angle is 105 degrees. Therefore, the manhole diameter required is based on either pipe size or maintaining the minimum distance between the outside of the pipes.

Step 2 - Check the manhole diameter required for the maximum pipe size at the manhole:

The maximum pipe size is 60 inches. Based on **Table 7-4**, an 84-inch manhole is required.

Step 3 - Determine the manhole diameter that meets the minimum distance of 12 inches between the outside of the pipes using **Equation 7.22**:

$$K = \frac{\frac{60}{2} + 6 + \frac{48}{2} + 5 + 14}{105} = 0.75 \text{ in/degree}$$

Based on **Table 7-4**, a 96-inch manhole is required for a $K = 0.75$.

Step 4 - Determine the manhole diameter that meets both criteria for pipes with an angle less than 135 degrees:

A 96-inch diameter manhole is required to meet both criteria.

7.5.3.3 Measurement and Payment

The current standard specifications classify access manholes as drainage structures paid for as each. Payment for additional structure depth is paid for in feet. The riser above a manhole tee or manhole base is paid for separately in feet.

Numbering drainage structures on projects is valuable for field personnel, especially on complex urban projects. Number drainage structures consecutively on plans, as practical. Show the corresponding number for each drainage structure on the profiles, drainage tables, and any other places on the plans where the drainage structure appears.

7.6 Design Guidance and Procedure for Storm Sewer Systems

7.6.1 Introduction

A storm sewer is defined as the portion of the storm drainage system that receives runoff from inlets and conveys the runoff to a point where it is discharged into a channel, water body, or piped system. A storm sewer may function under pressure flow, open-channel flow, or a combination of the two. Storm sewers are designed to convey runoff that enters the system for the required design flow rate. The design storm flow includes existing flow rates and future development flow rates with a 20-year outlook.

Design storm sewers to avoid existing utilities to the extent practical. Review each potential utility conflict and implement one of the following:

- Provide the minimum storm sewer clearance per Standard Plan R-83 Series, Trench Detail E
- Refer to the Great Lakes – Upper Mississippi River Board (GLUMRB) 10 State Standards for the recommended clearance between storm sewer and water mains. The clearances is measured from edge to edge of each pipe.
- Provide the storm sewer clearance required by a specific utility

When minimum clearances cannot be met, utility relocation is preferred to maintain minimum slopes and avoid inverted siphons. If relocation is required, contact the TSC Utility Coordinator.

Coordinate drainage system design with proposed staging of construction projects to maintain an outlet throughout the duration of a construction project. Additionally, provide temporary inlet spacing calculations for each stage per requirements in **Section 7.3.6**.

The placement and capacities of storm sewers are required to be consistent with local stormwater management plans. Place storm sewer pipe below the pavement section or with a minimum cover of 3 feet, whichever is greater. Storm sewer pipe is placed in the road subbase only when there are utility conflicts, constructability issues, or outlet elevation needs.

When evaluating a drainage system, confirm the contributing drainage area, including area outside MDOT's ROW, is captured in the analysis. Use the Rational Method to develop peak flows for all outside inputs into the system. Then determine the capacity of the downstream system. When draining to a shared system, use as-built plans or GIS to develop hydrology and perform a high-level analysis of the sewer trunk capacity in ORD D&U. For trunkline projects, contact the Hydraulic Unit Supervisor and the Drain Coordinator for guidance. Use the results of this analysis to confirm there are no existing downstream issues. This analysis occurs during the scoping phase of a project. This initial step establishes the tailwater condition at the point of connection from all inlets, connecting pipes, and outfalls. The next step is to compute the rate of discharge carried by each reach of the storm sewer and determine the size and gradient of pipe required to convey this discharge. Start at the upstream reach, calculate the discharge

and size the pipe, then proceed downstream, reach-by-reach, to the point where the storm sewer connects with other drains or the outfall. Match the crown of smaller pipe in a longitudinal trunk sewer to the crown of the adjacent downstream pipe, where practical. If matching the crown is not feasible in flat terrain, match the inverts.

The rate of discharge at any point in the storm sewer is not necessarily the sum of the inlet flow rates of all inlets above that section of storm sewer. Due to timing of peak flows and attenuation, the peak flow is generally less than this total. The time of concentration has a significant impact on the peak flow rate. For a given tributary area, as the time of concentration grows, the rainfall intensity used in the design decreases. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entire drainage area is not contributing. Identify unusual conditions and determine which time of concentration controls each pipe segment. Refer to Chapter 3, Hydrology, Section 3.4, for a discussion on time of concentration. For ordinary conditions, size storm sewers assuming they flow full, or practically full, under the design discharge. Unless under high tailwater conditions, avoid designing sewers for pressure head conditions.

7.6.1.1 Storm Sewer Design Considerations

Design considerations for storm sewers include:

- Contributing drainage area
- Slope of ground
- Type of soil
- Ground cover and land use as it relates to the determination of the runoff coefficient "C"
- Receiving water body boundary condition
- Water quality and channel protection treatment per NPDES MS4 permit
- Flow path channel type
- Risk associated with possible overload of sewer
- Elevation difference of the HGL and gutter line
- Future development and local ordinances
- Participation of municipality if sewer is to accommodate more than highway needs
- Location of catch basins and inlets
- Location of trunk sewer - horizontal and vertical
- Location of existing and new outfalls or points of discharge
- Location, flow line, and available capacity of existing sewers
- Flow lines and contributing area of connecting sewers
- Existing or proposed underground utilities
- Elevation, location, and adequacy of outlets
- Pipe velocity and outlet velocity
- Ease and economics of constructing sewer
- Pipe material

7.6.1.2 Outlets, Outfalls, and Points of Discharge

Outfalls and points of discharge are included in the MS4 permit. Outlets are not included in the MS4 permit. Refer to the following examples for acceptable outfall and points of discharge and unacceptable drainage outlets.

Examples of acceptable outfalls and points of discharge from the project include, but are not limited to:

- Inland lake or stream: New outfalls to waters of the state or navigable rivers may require EGLE/USACE permits.
- County drains: Plans that involve discharging stormwater to a county drain are required to be submitted to the County Drain Commissioner.
- Storm sewers or ditches owned by other public agencies: Connections to these systems are considered “points of discharge”. Approval to use these facilities is required from the owner. These connections are required to meet the owner’s MS4 permit requirements, as applicable. When considering a sewer owned by others, an agreement is required. Reference Chapter 2, Policy and Procedures. MDOT will evaluate operation and maintenance costs, outlet costs, and future capacity.

Examples of unacceptable drainage outlets include, but are not limited to:

- Natural depressions outside of MDOT ROW without obtaining a permanent easement
- Farm tiles unless with a recorded agreement
- Sanitary sewers
- Drainage course across private property unless drainage or flowage easements are obtained and recorded
- New connections to combined sewers unless existing facilities drain there by an agreement
- Infiltration near sources of contaminated soils or groundwater

7.6.1.3 Storm Sewer Outlet Guidance

Proper storm sewer outlet configuration is important to storm sewer design and permitting. There are many considerations when establishing a storm sewer outlet including, but not limited to, project outfalls, outlet invert placement, location, and water quality.

7.6.1.3.1 Project Outfalls (Refer to Section 7.6.1.2)

Outfalls are locations on a project where discharge first enters a regulated stream (water of the state). Projects are required to identify new and existing outfalls early in the design process. MDOT’s MS4 permit requires EGLE authorization for new outfalls to discharge stormwater. Refer to the [MDOT Stormwater Treatment Manual](#) for more information.

7.6.1.3.2 Outfall/Outlet Invert Placement

Avoid placing storm sewer outfall/outlet inverts on a roadway embankment or side slope. Inverts are generally placed near the toe of slope. Invert placement depends on the type of the receiving channel. Guidance for typical invert placement includes:

- Storm sewers with outlet to roadside ditch – match the invert of the storm sewer outlet to the flowline of the receiving ditch.
- Storm sewers with outfalls to regulated streams – place invert of the storm sewer outlet at or near the stream's OHWM.

7.6.1.3.3 Safety Grates or Culvert Safety Sloped End Sections

Install culvert safety sloped end sections (MDOT Standard Plan R-95 Series) on concrete or metal storm sewers within the clear zone. Circular pipes that are 15- to 60-inch in diameter can be fitted with culvert sloped end sections. Custom safety grates are used at locations to prevent people or animals from entering the storm sewer system.

Considerations:

- Limit use on regulated streams due to risk of clogging with debris
- Specify toe plates extension in the plans for R-95

7.6.1.3.4 Water Quality and Channel Protection

Refer to the [MDOT Stormwater Treatment Manual](#) for water quality and channel protection requirements for discharging stormwater to waters of the state.

7.6.2 Storm Sewer Pipe Classification and Usage Guidelines

Follow these procedures for storm sewer class and material selection:

- Select the appropriate class of sewer according to the current MDOT Standard Specifications for Construction. The design life for MDOT sewers is 70 years and for LAP sewers is a minimum of 50 years unless a local agency specifies a higher design life.
- A specific sewer pipe material may be required for unique field conditions or application. The required material is specified in the pay item. The prohibited material is identified by note on the plans.

Example: When a specific location has a corrugated metal pipe (CMP) that has failed prematurely due to corrosion, exclusion of that material may be warranted. If the environment is suspected to be corrosive for a CMP, collect pH and resistivity data that justifies excluding that pipe material.

7.6.2.1 Special Pipe Designs

Special pipe designs may be required for unique projects. Special designs include, but are not limited to, sewer installed by jacking, tunneling, pipes on a curved alignment, or depths of cover less than 1 foot and greater than 33 feet. Refer to the MDOT Road Design Manual, Section 4.02.20 for addition information on jacked-in-place sewers. The design and method of construction for special designs require a multidisciplinary approach. For trunkline projects, contact the Hydraulic Unit Supervisor to initiate coordination.

7.6.2.2 Storm Sewer Soil Borings

Plans and specifications do not automatically provide for the additional work required to install sewers through areas of unstable soils. Therefore, obtain soil borings and identify where remedial treatment is necessary on the plans. Corrective treatment usually includes undercutting and backfilling. In areas with a high water table, consider a well point system. Consult with the Region Soils Engineer or Geotechnical Service Section for trunkline projects.

For further discussion, refer to the MDOT Road Design Manual, Section 4.02.21, and the MDOT Geotechnical Manual, Section 6.3.4.1. and 9.3.6.

7.6.2.3 Storm Sewer Measurement and Payment

Sewers are paid per foot. Use the following examples as a guide when calling out sewers on plans:

- 100-foot Sewer, CI A, 18-inch, Tr Det A
- 100-foot Sewer, CI III, 18-inch, Tr Det A
- 100-foot Sewer, CI IV, 48-inch, Jacked-in-Place

Use the same sewer class between structures or structure to an outlet that meets the requirements of the most critical point of the section. However, the trench detail can change in the same run of pipe.

7.6.3 Hydraulic Capacity

The Manning's equation is used to determine the hydraulic capacity of storm sewers for gravity and pressure flows excluding upstream or downstream controls. The Manning's equation for a direct calculation of velocity is calculated using **Equation 7.23**:

$$V = \left(\frac{1.49}{n} \right) (R^{2/3} S^{1/2}) \quad (7.23)$$

Where:

V = Mean velocity of flow, fps

n = Manning's roughness coefficient

R = Hydraulic radius, ft (R = area of flow divided by the wetted perimeter (A/WP))

S = Slope of energy grade line, ft/ft

In terms of discharge, the equation becomes:

$$Q = V A = \left(\frac{1.49}{n} \right) (A R^{2/3} S^{1/2})$$

Where:

Q = Rate of flow, cfs

A = Cross-sectional area of flow, sf

Under full pipe flow conditions, the term $(1.49/n) (AR^{2/3})$ may be grouped into a constant for each section of pipe and the equation becomes:

$$Q = K S^{1/2} \quad (7.24)$$

Where:

$$K = \left(\frac{1.49}{n} \right) (A R^{2/3})$$

Use **Table 7-5** to determine the size of pipe necessary for each section based on the conveyance factor, K, and Manning's n for that reach. MDOT standards for Manning's n coefficient in storm sewers are:

n = 0.013 for vitrified clay, concrete, plastic, and spiral ribbed pipe

n = 0.024 for CMP

Table 7-5. Conveyance Factor (K) and Pipe Sizes

Pipe Diameter	Area in Square Ft (A)	Hydraulic Radius (R)	K for n = 0.013	K for n = 0.024
12-in	0.785	0.250	35.66	19.3
15-in	1.227	0.3125	64.7	35.01
18-in	1.767	0.375	105.1	56.9
24-in	3.142	0.500	226.2	N/A
30-in	4.909	0.625	410.1	N/A
36-in	7.069	0.750	666.6	N/A
42-in	9.621	0.875	1006	N/A
48-in	12.566	1.000	1436	N/A
54-in	15.904	1.125	1967	N/A
60-in	19.635	1.250	2604	N/A
66-in	23.758	1.375	3357	N/A
72-in	28.274	1.500	4234	N/A
78-in	33.183	1.625	5242	N/A
84-in	38.485	1.750	6388	N/A
90-in	44.179	1.875	7681	N/A
96-in	50.266	2.000	9119	N/A
102-in	56.745	2.125	10722	N/A
108-in	63.617	2.250	12486	N/A
114-in	70.822	2.375	14422	N/A
120-in	78.54	2.500	16537	N/A

The conveyance factor, K, represents the capacity of the pipe under full flow conditions. The actual design flow will not flow under these conditions and will result in a normal depth, d_n , less than the pipe diameter and a different calculated velocity.

The calculation of normal depth in a circular pipe using the Manning's equation is not a direct solution and requires an iterative solution using various estimates of depth until the desired flow rate is calculated. FHWA Hydraulic Toolbox can be used to calculate the normal depth in a circular pipe without iterations.

Example: Solve for normal depth using FHWA Hydraulic Toolbox.

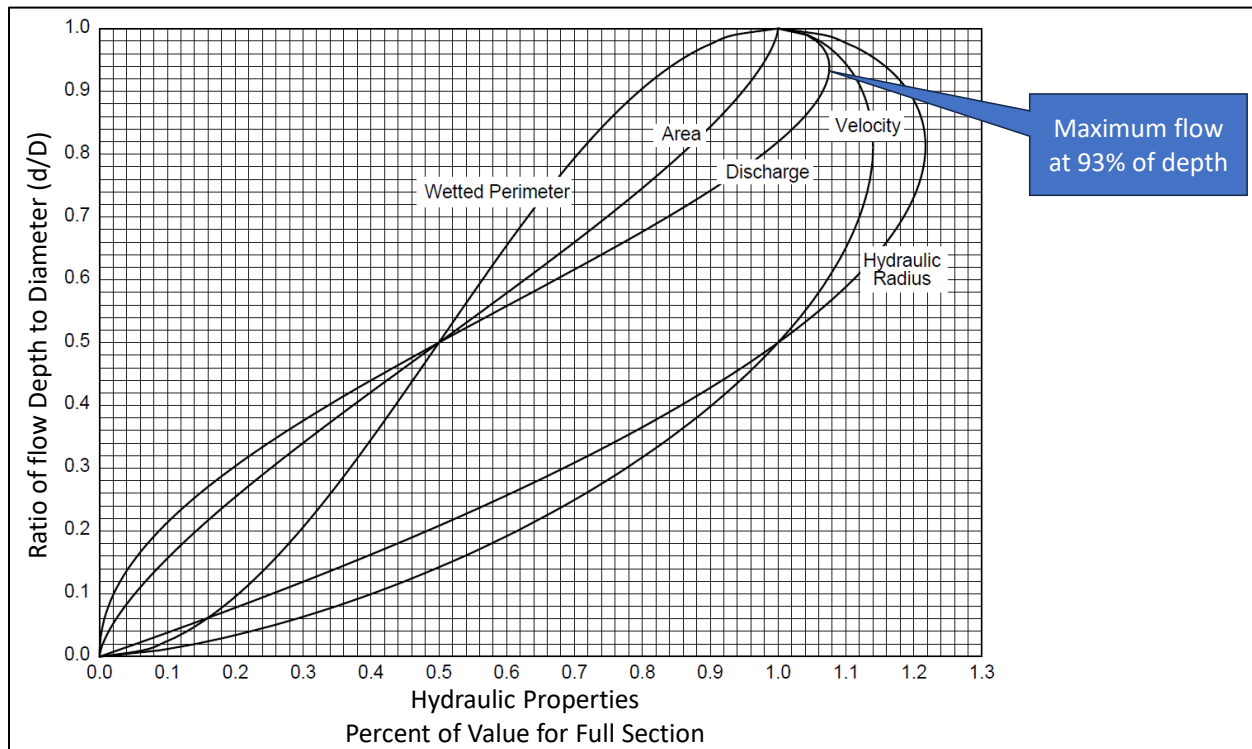
Given: Pipe diameter = 1.5 ft
 Longitudinal slope = 0.0028 ft/ft
 Manning's n = 0.024 (CMP)
 Design flow = 3.0 cfs

Step 1 – Input the design parameters using the channel analysis tool. Press the “Calculate” button. The result calculates “Depth” = 1.23 ft.

Parameter	Value	Units
Flow	3.000	cfs
Depth	1.225	ft
Area of Flow	1.545	sq ft
Wetted Perimeter	3.384	ft
Hydraulic Radius	0.456	ft
Average Velocity	1.942	fps
Top Width (T)	1.162	ft
Froude Number	0.297	
Critical Depth	0.658	ft
Critical Velocity	4.018	fps
Critical Slope	0.01750	ft/ft
Critical Top Width	1.489	ft
Max Shear Stress	0.214	lb/ft ²
Avg Shear Stress	0.080	lb/ft ²

The relationship between flow, velocity, area, wetted perimeter, and hydraulic radius for a circular pipe is shown in **Figure 7-8**, where the difference between maximum pipe capacity is 93% of full depth and full flow capacity is 100% of full flow depth.

Figure 7-8. Values of Hydraulic Elements of Circular Section for Various Depths of Flow



7.6.3.1 Minimum Grades

Storm sewers are required to have velocities greater than or equal to 3.0 fps at design flow. For very flat grades, the general practice is to design components that will increase flow velocities progressively throughout the length of the pipe system. Verify the storm sewer system has sufficient velocity in all drains to deter settling of particles. Minimum slopes for circular concrete, plastic, or spiral rib pipe with a 0.013 Manning's n value that provide velocities of 3.0 fps or greater are presented in **Table 7-6**.

Table 7-6. Minimum Sewer Slopes for Self-Cleaning

Pipe Size	Pipe Grade (%)
12-in	0.48
15-in	0.36
18-in	0.28
24-in	0.17
30-in	0.15
36-in	0.12
42-in	0.12
48-in	0.10
54-in	0.09
60-in	0.09
66-in	0.08
78-in	0.08
90-in	0.07
102-in	0.06
108-in	0.06
120-in	0.06

Sewers are designed with a maximum velocity of 12 fps. Avoid the use of hydraulically steep pipes to prevent supercritical flow conditions. If supercritical conditions cannot be avoided, size the downstream pipes large enough to prevent pressure flow conditions as the flow regime transitions from supercritical to subcritical in the form of a hydraulic jump.

7.6.3.2 Outlet Velocity and Erosion Protection

Compute outlet velocities for storm sewers with a free discharge to a ditch using critical depth at the outlet of the pipe. Outlet velocities from storm sewers into ditches greater than 6 fps require the design and installation of energy dissipation structures at the outlet of the storm sewer (refer to FHWA HEC-14, 2006). MDOT's commonly used methods include drop structures with a series of manholes, riprap basin (may require additional ROW), and stilling basins.

Place riprap or an equivalent soil erosion stabilization measure at the storm sewer outlet. Develop riprap details and sizing based on guidance found in FHWA HEC-14 (2006).

7.6.4 Hydraulic Grade Line and Energy Grade Line

7.6.4.1 Introduction

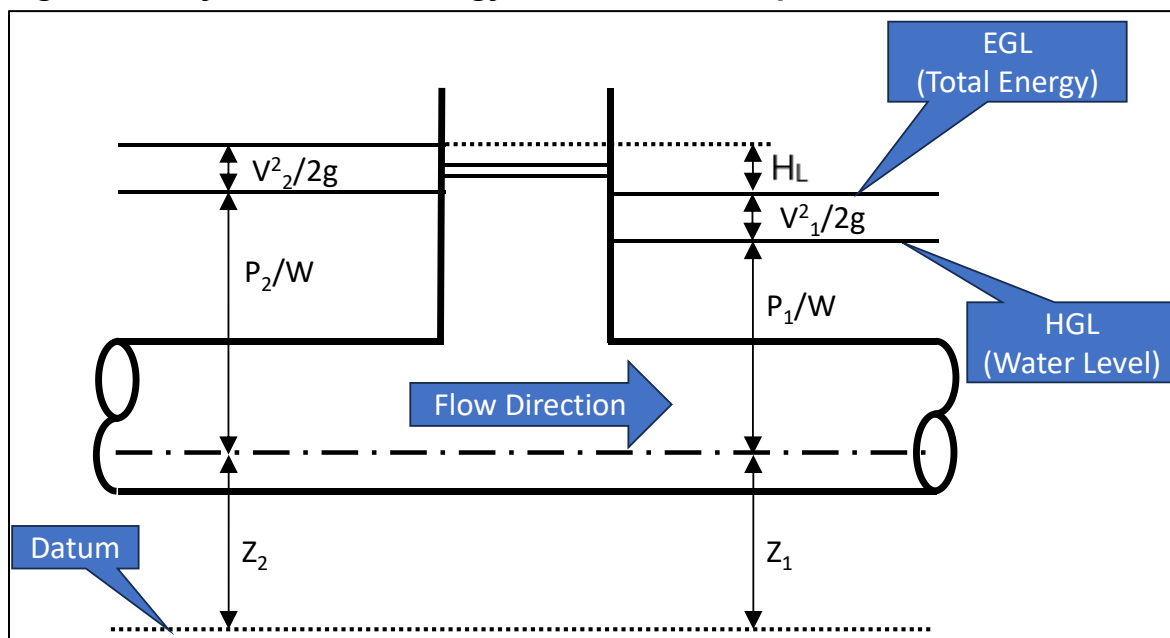
The hydraulic grade line (HGL) and energy grade line (EGL) are important components to understand when determining the water levels in storm sewers. The HGL is the water level in an open channel condition or the level the water would rise to within a riser pipe in a pressurized condition. The EGL is the total energy at a point in the system. The EGL is equal to the HGL plus the velocity head ($V^2/2g$). When the velocity is zero, the EGL and HGL are equal.

Because energy is conserved, the total energy of a system is calculated at a point in the system by adding the depth above datum, pressure head, and velocity head.

Starting at the downstream end of a mild sloping system, friction and minor losses are additive resulting in an increase in the total energy in the system in the upstream direction. To compute the HGL and EGL, 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, junctions, manholes, and combining flow. Use ORD D&U for determining HGL and EGL for trunkline projects. This software has several methods for calculating energy loss calculations at the manholes (nodes/junctions). The approved method is described in FHWA HEC-22, 3rd Edition.

An example of how EGL, HGL, and hydraulic losses are tracked through a system using the energy equation (simplified version of the Bernoulli equation) is shown in **Figure 7-9**. In this example, the system is operating under pressure (surcharge) conditions and the hydraulic loss is associated with a minor loss at a manhole.

Figure 7-9. Hydraulic and Energy Grade Lines in Pipes



In any system, energy is conserved which allows the energy at any point in the system to be calculated using the energy equation shown in **Equation 7.25**. This equation shows how total energy is balanced while accounting for head loss.

$$Z_1 + \frac{V_1^2}{2g} + \frac{P_1}{W} + H_L = Z_2 + \frac{V_2^2}{2g} + \frac{P_2}{W} \quad (7.25)$$

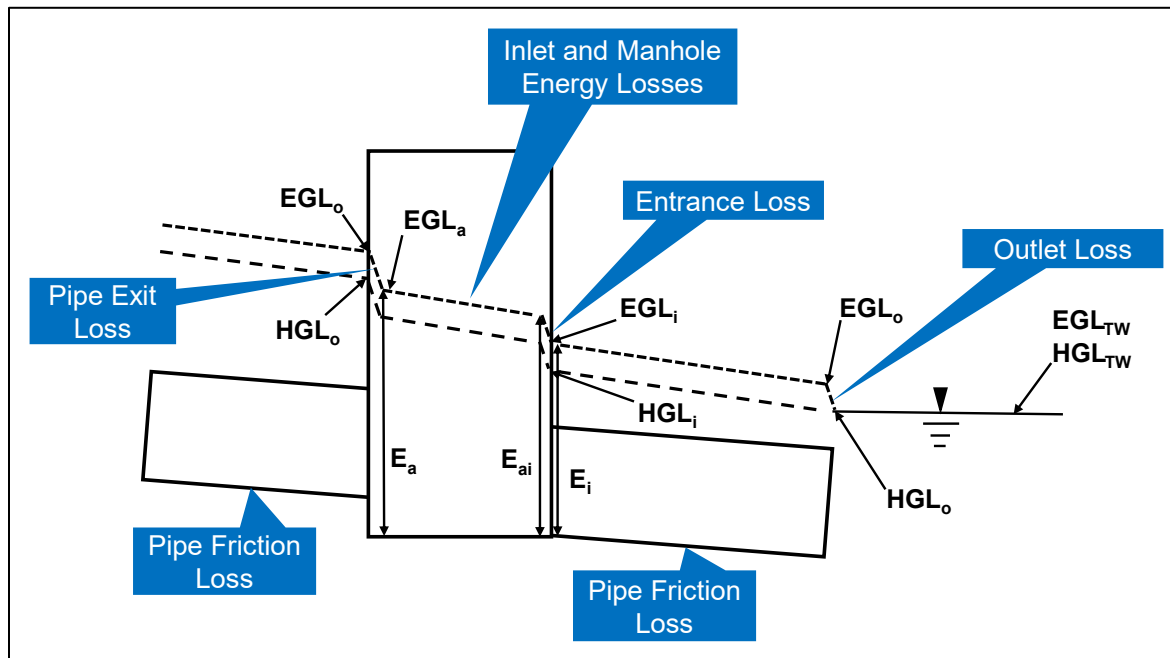
Where:

- g = Acceleration due to gravity, 32.2 ft/s²
- W = Unit weight of water, 62.4 lb/ft³
- V = Velocity of flow in ft/s
- H_L = Head loss
- Z = Distance above chosen datum, referred to as elevation head
- P = Pressure of fluid in lb/ft²

In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. A special concern with storm sewers designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the HGL rises above the ground surface. A design based on open channel conditions requires careful planning, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, verify the desired flow condition. Storm sewer systems can often alternate between pressure and open channel flow conditions from one section to another.

Although the HGL and its relation to the pipe crown is the focus of the hydraulic calculations, calculations are required to track the EGL. The HGL is determined at any point by subtracting the velocity head from the EGL. The detailed methodology employed in calculating the EGL through the system begins by establishing tailwater for the system.

The first step of an EGL calculation is establishing the tailwater. EGL calculations proceed upstream to compute head losses such as exit losses, friction losses, inlet and manhole energy losses, and entrance losses as appropriate. A graphical representation of these losses is shown in **Figure 7-10**.

Figure 7-10. Graphical Representation of System Losses**7.6.4.2 Tailwater**

An accurate representation of the tailwater for a system is required to determine the potential backwater impacts on a proposed system. Three basic scenarios are possible. Each scenario describes the level of the HGL. The EGL should be determined by adding the velocity head ($V^2/2g$) to the HGL.

1. Free discharge to a ditch – The normal depth at the outlet from the proposed system is above the water level in the ditch and is not impacted by backwater from the downstream system. Set the tailwater based on the critical depth (d_c) and diameter (D), using $(d_c + D)/2$.
2. Backwater from a ditch or water bodies – The level in the downstream system is above the normal depth of the proposed system. Set the tailwater equal to this level. If a storm sewer drains to a large water body or pond, set the tailwater to this level and assume the velocity to be zero. Therefore, HGL equals EGL. If downstream water bodies or hydraulic controls are present, refer to Chapter 4, Open Channels and Roadside Ditches, Section 4.4.3.10.
3. Discharge into an enclosed system – Calculate the downstream system water level up to the point of connection from the proposed system to establish a tailwater elevation. If the downstream system information is not available, use the crown of the pipe.

When estimating tailwater depth on the receiving stream, consider the coincidental frequency of two events occurring at the same time. In the case of a tributary stream or a storm sewer, its relative independence may be qualitatively evaluated by comparing

its drainage area with that of the receiving stream. A short duration storm that causes peak discharges on a small basin may not be critical for a larger basin. Assume that if the same storm causes peak discharges on both basins, the peaks will be out of phase. To aid in the evaluation of frequencies for coincidental occurrence, refer to **Table 7-7**.

Table 7-7. Frequencies for Coincidental Occurrence – AEP (ARI)

(a) Receiving System Frequency for a 10% AEP (10-Year) Contributing System Design

Receiving Area to Contributing Area Ratio	Receiving System Frequency
10,000 to 1	100% (1-year)
1,000 to 1	50% (2-year)
100 to 1	20% (5-year)
10 to 1	10% (10-year)
1 to 1	10% (10-year)

(b) Receiving System Frequency for a 4% AEP (25-Year) Contributing System Design (Estimated)

Receiving Area to Contributing Area Ratio	Receiving System Frequency
10,000 to 1	50% (2-year)
1,000 to 1	20% (5-year)
100 to 1	20% (5-year)
10 to 1	10% (10-year)
1 to 1	4% (25-year)

(c) Receiving System Frequency for a 2% AEP (50-Year) Contributing System Design

Receiving Area to Contributing Area Ratio	Receiving System Frequency
10,000 to 1	50% (2-year)
1,000 to 1	20% (5-year)
100 to 1	20% (5-year)
10 to 1	10% (10-year)
1 to 1	4% (25-year)

(d) Receiving System Frequency for a 1% AEP (100-Year) Contributing System Design

Receiving Area to Contributing Area Ratio	Receiving System Frequency
10,000 to 1	50% (2-year)
1,000 to 1	10% (10-year)
100 to 1	4% (25-year)
10 to 1	2% (50-year)
1 to 1	1% (100-year)

Example: Determine design storm for sizing a storm sewer system for a 10% AEP (10-year) performance.

Given: Storm Sewer Drainage Area = 15 acres
Watercourse Drainage Area = 1,500 acres

Step 1 – Determine the drainage area ratio.

1,500 / 15 or 100 to 1

Step 2 – Use **Table 7-7** to determine the receiving area design storm.

Receiving area design storm = 20% AEP (5-year)

Design the storm sewer for a 10% AEP (10-year) storm and the receiving water at a 20% (5-year) flood elevation for the tailwater condition.

7.6.4.3 Exit Loss

The exit loss (H_o) from a storm sewer drain outlet is a function of the change in velocity at the pipe outlet. For a sudden expansion, such as an end wall, the exit loss is calculated using **Equation 7.26**:

$$H_o = 1.0 \left[\frac{V_o^2}{2g} - \frac{V_d^2}{2g} \right] \quad (7.26)$$

Where:

V_o = Average outlet velocity, fps

V_d = Channel velocity downstream of outlet in the direction of the pipe flow, fps

g = Gravitational acceleration, 32.2 ft/s²

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

7.6.4.4 Bend, Transition, and Junction Losses

The bend loss coefficient (H_b) for storm sewer design (for bends in the pipe run, not in a manhole structure) is minor but can be calculated using **Equation 7.27**:

$$H_b = 0.0033 (\Delta) \frac{V^2}{2g} \quad (7.27)$$

Where:

Δ = Angle of bend in degrees

g = Gravitational acceleration, 32.2 ft/s²

Horizontal and vertical pipe bends without a manhole structure are not allowed. A vertical bend can occur in a downspout without a manhole.

Using expansion transitions where the diameter of storm sewer pipe increases without a manhole are not allowed.

Using junctions from one pipe to another pipe without a manhole (blind tap) are not allowed.

7.6.4.5 Pipe Friction Loss

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is calculated using **Equation 7.28**:

$$H_f = S_f L \quad (7.28)$$

Where:

- H_f = Friction loss, ft
- S_f = Friction slope, ft/ft
- L = Length of pipe, ft

The friction slope is the slope of the hydraulic gradient for a particular pipe run. Assuming steady uniform flow, 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 calculated using **Equation 7.29**.

$$S_f = \left(\frac{H_f}{L} \right) = \left(\frac{Q n}{K_Q D^{2.67}} \right)^2 \quad (7.29)$$

Where:

- Q = Rate of flow, cfs
- n = Manning's roughness coefficient
- K_Q = Unit conversion constant, 0.46
- D = Diameter of pipe, ft

7.6.4.6 Inlet and Manhole Energy Losses

Determining the losses in a manhole structure is essential to calculating the EGL in a storm sewer system. Calculate the initial energy level in the manhole structure (E_{ai}) as the maximum of three conditions that determine the hydraulic regime within the structure. The conditions considered for the outlet pipe are:

- Outlet control condition: 1) full flow condition – a common occurrence with a surcharged storm sewer 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 manhole structure to the outlet pipe is limiting and the resulting water depth in the manhole 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 limited by the opening, but the resulting water level in the manhole involves treating the opening as a weir.

This method addresses one of the weaknesses of other methodologies: a 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 manhole, 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 measure velocity. Velocity is calculated 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 that do not necessarily represent the velocities calculated based on equilibrium pipe hydraulics in storm sewer 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 is calculated using **Equation 7.30**:

$$E_i = y + \left(\frac{P}{\gamma} \right) + \left(\frac{V^2}{2g} \right) \quad (7.30)$$

Where:

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

Solving for **Equation 7.30** may cause a problem for certain conditions (e.g., where P cannot be assumed to equal atmospheric pressure). E_i is also determined by subtracting the outflow pipe invert elevation (Z_i) from the outflow pipe energy grade line (EGL_i) (both known values) at that location as shown in **Equation 7.31**:

$$E_i = EGL_i - Z_i \quad (7.31)$$

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

The initial estimate of the manhole energy level is the maximum of three values:

$$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu}) \quad (7.32)$$

Where:

E_{aio} = Estimated manhole energy level for outlet control (full and partially full flow), ft

E_{ais} = Estimated manhole energy level for inlet control (submerged), ft

E_{aiu} = Estimated manhole energy level for inlet control (unsubmerged), ft

In the outlet control condition, the downstream storm sewer system limits discharge out of the manhole and the outflow pipe either flows full or partially full in subcritical flow.

The initial structure energy level (E_{aio}) estimate is calculated using **Equation 7.33**:

$$E_{aio} = E_i + H_i \quad (7.33)$$

Where:

H_i = Entrance loss assuming outlet control, calculated using **Equation 7.34**:

$$H_i = K_i \frac{V^2}{2g} \quad (7.34)$$

Where:

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

Using the concept of outflow pipe energy head (E_i) and **Equation 7.35**, the energy level is estimated directly without considering the water surface within the manhole. 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:

$$DI = \frac{Q}{A (g D_o)^{0.5}} \quad (7.35)$$

Where:

A = Area of outflow pipe, ft²

D_o = Diameter of outflow pipe, ft

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:

$$E_{ais} = D_o (DI)^2 \quad (7.36)$$

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}):

$$E_{aiu} = 1.6 D_o (DI)^{0.67} \quad (7.37)$$

7.6.4.7 Adjusting for Benching, Angled Inflow, and Plunging Inflow

Use the initial structure energy level 1 as a basis for estimating additional losses for:

- Discharges entering the structure at angles other than 180 degrees
- Benching configurations
- Plunging flows entering the structure at elevations above the water depth in the manhole (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 manhole 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 manhole energy level (E_a) equals the initial estimate (E_{ai}) modified by each of the three factors covered in this section is calculated using **Equation 7.38**:

$$E_a = E_{ai} + H_B + H_\theta + H_P \quad (7.38)$$

Where:

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

E_a represents the level of the EGL in the manhole. However, if calculations result in an E_a less than the outflow pipe energy head (E_i), set E_a equal to E_i . Designers may also wish to know the water depth in the manhole (y_a). A conservative approach would use E_a as y_a for design purposes.

7.6.4.7.1 Additional Energy Loss: Benching

Benching tends to direct flow through the manhole, resulting in a reduction in energy losses. **Figure 7-11** illustrates typical bench configurations. Generally, from **Figure 7-11** (a) to (e), the energy losses tend to decrease. Standard practice is to assume the Half Benching configuration shown in **Figure 7-11** (c). Refer to Standard Plans to determine appropriate benching configuration.

For manhole benching, the additional benching energy loss is calculated using **Equation 7.39**:

$$H_B = C_B (E_{ai} - E_i) \quad (7.39)$$

Where:

C_B = Energy loss coefficient for benching (refer to **Table 7-8**)

Table 7-8 summarizes benching coefficients. A negative value indicates water depth will be reduced rather than increased.

Figure 7-11. Manhole Benching Methods (FHWA HEC-22, 2024)

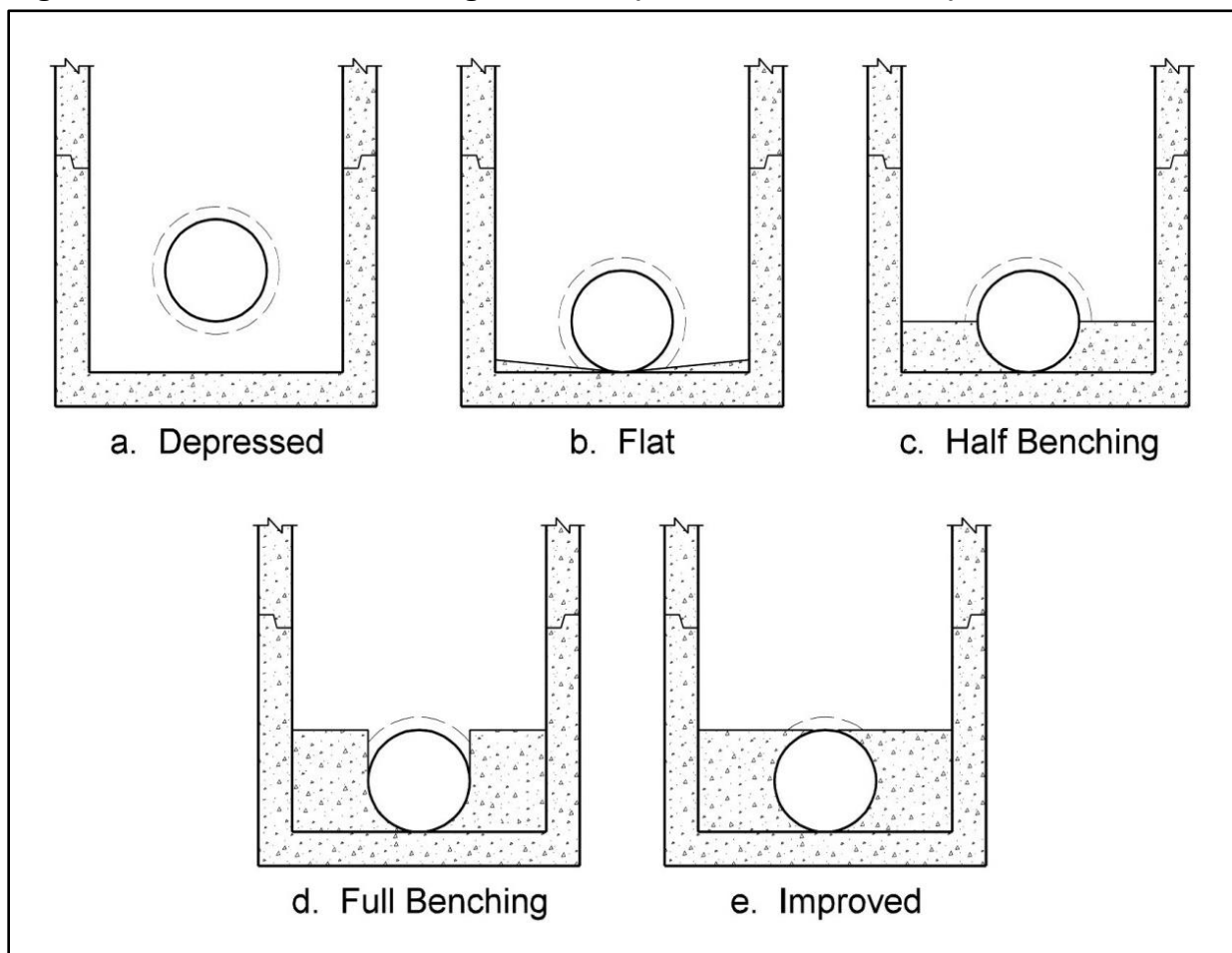


Table 7-8. Values for the Coefficient, C_B (FHWA HEC-22, 2024)

Floor Configuration	Bench Submerged	Bench Unsubmerged
Flat (level)	-0.05	-0.05
Depressed	0.00	0.00
Half Benched	-0.05	-0.85
Full Benched	-0.25	-0.93
Improved	-0.60	-0.98

For all benching floor configurations, a bench submerged condition has the properties of $(E_{ai} / D_o) > 2.5$ and a bench unsubmerged condition has the properties of $(E_{ai} / D_o) < 1.0$. Use linear interpolation between the two values for intermediate values.

7.6.4.7.2 Additional Energy Loss: Angled Inflow

Address the effect of skewed inflows entering the structure by considering momentum vectors. To maintain simplicity, all inflows contributing to a structure with a hydraulic connection (i.e., not plunging) resolves into a single flow weighted angle (Θ_w) and is calculated using **Equation 7.40**:

$$\theta_w = \frac{\sum(Q_j \theta_j)}{\sum Q_j} \quad (7.40)$$

Where:

Q_j = Contributing flow from the inflow pipe, cfs

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

Figure 7-12 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.

An angled inflow coefficient (C_θ) is calculated using **Equation 7.41**:

$$C_\theta = 4.5 \frac{\sum Q_j}{Q_o} \cos\left(\frac{\theta_w}{2}\right) \quad (7.41)$$

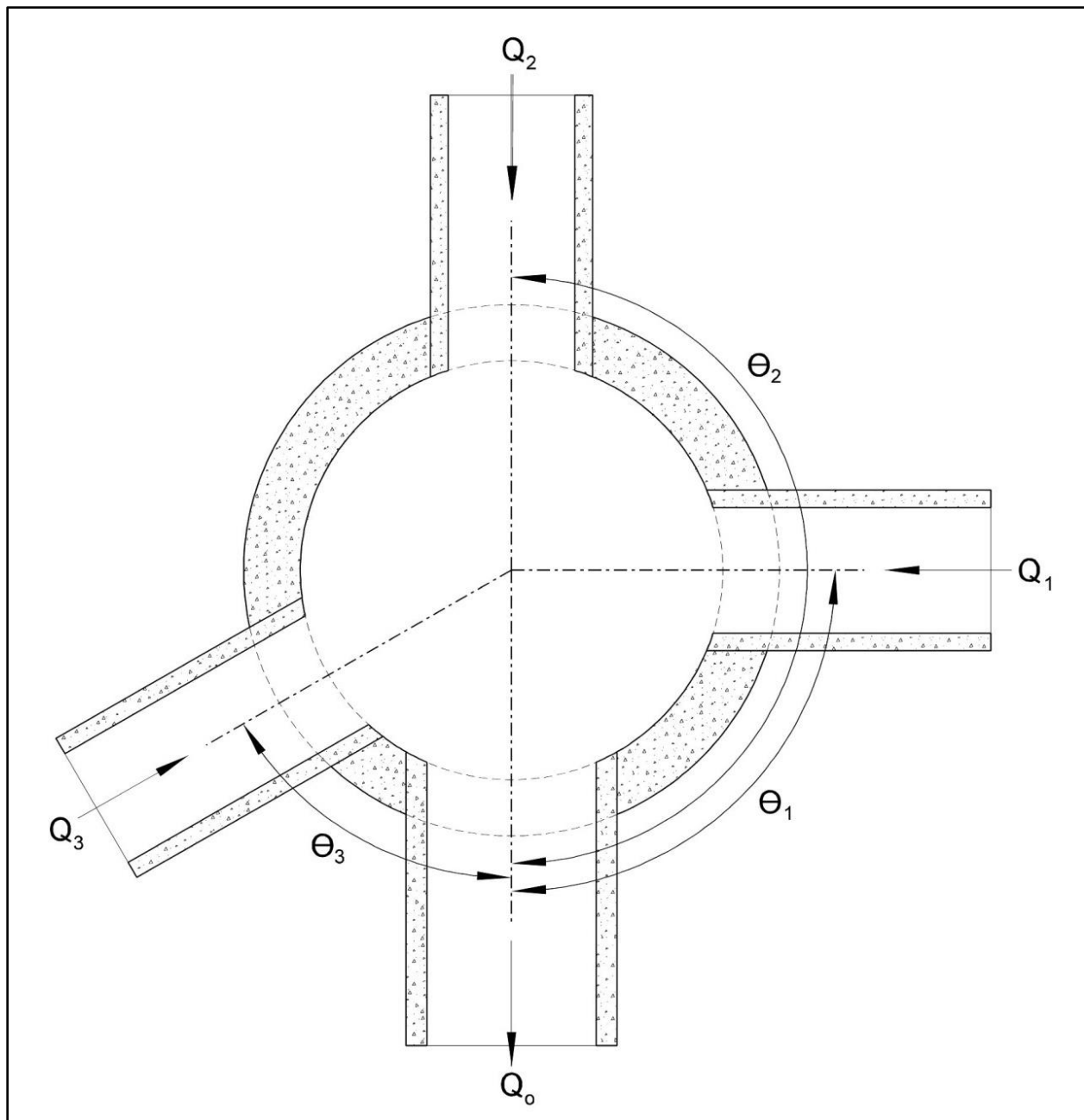
Where:

Q_o = Flow in the outflow pipe, cfs

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 calculated using **Equation 7.42**:

$$H_{\theta} = C_{\theta} (E_{ai} - E_i) \quad (7.42)$$

Figure 7-12. Manhole Angled Inflow Definition (FHWA HEC-22, 2024)



7.6.4.7.3 Additional Energy Loss: Plunging Inflow

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 manhole invert elevation and the inflow pipe invert elevation.

This method determines a relative plunge height (h_k) for a plunging pipe (denoted by the subscript k) and is calculated using **Equation 7.43**:

$$h_k = \frac{(z_k - E_{ai})}{D_o} \quad (7.43)$$

This relative plunge height is used to determine the plunging flow coefficient (C_P) and is calculated using **Equation 7.44**:

$$C_P = \frac{\sum(Q_k h_k)}{Q_o} \quad (7.44)$$

As the proportion of plunging flows approaches zero, C_P also approaches zero.

Equations 7.43 and **7.44** only apply to conditions where $z_k < 10D_o$. If $z_k > 10D_o$, C_P equals $10D_o$.

The additional plunging inflow energy loss is calculated using **Equation 7.45**:

$$H_P = C_P(E_{ai} - E_i) \quad (7.45)$$

This method determines 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 using **Equation 7.46**:

$$H_a = (C_B + C_\theta + C_P)(E_{ai} - E_i) \quad (7.46)$$

Note that the value of H_a is always positive. If the calculation yields a negative value, set H_a equal to zero. The revised manhole energy level is calculated using **Equation 7.47**:

$$E_a = E_{ai} + H_a \quad (7.47)$$

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 . Assuming the manhole invert (z_a) has the same elevation as the outflow pipe invert (z_i), the manhole energy grade line (EGL_a) is calculated using **Equation 7.48**:

$$EGL_a = E_a + Z_a \quad (7.48)$$

The potentially highly turbulent nature of flow within the manhole makes it difficult to determine water depth. However, the EGL_a is used as a comparison elevation to check for potential surcharging of the system. Research has shown the difficulty of determining velocity head within the manhole, even in controlled laboratory conditions (Kerenyi et al. 2006).

7.6.4.8 Calculating Inflow Pipe Exit Losses

In the final step, the EGL into each inflow pipe is calculated. Non-plunging inflow pipes are pipes with a hydraulic connection to the water in the manhole. Inflow pipes operating under this condition are identified when the revised manhole energy grade line (E_a) is greater than the inflow pipe invert elevation (Z_o). In this case, the inflow pipe energy head is calculated using **Equation 7.49**:

$$EGL_o = EGL_a + H_o \quad (7.49)$$

Where:

- EGL_o = Inflow pipe energy head, ft
- EGL_a = Revised manhole energy grade line, ft
- H_o = Inflow pipe exit loss, ft

The subscript “o” is used for the inlet pipe because the equation represents losses at the outlet end of the inlet pipe. Exit loss is calculated using the inflow pipe velocity head, since supercritical flow is not a concern on the inflow pipe. The inflow pipe exit loss is calculated using **Equation 7.50**:

$$H_o = K_o \frac{V^2}{2g} \quad (7.50)$$

Where:

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

Water discharging from plunging inflow pipes freely falls into the manhole. 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 manhole water depth and losses. Determining the EGL for the outlet of a pipe has already been described in **Section 7.6.4.2**.

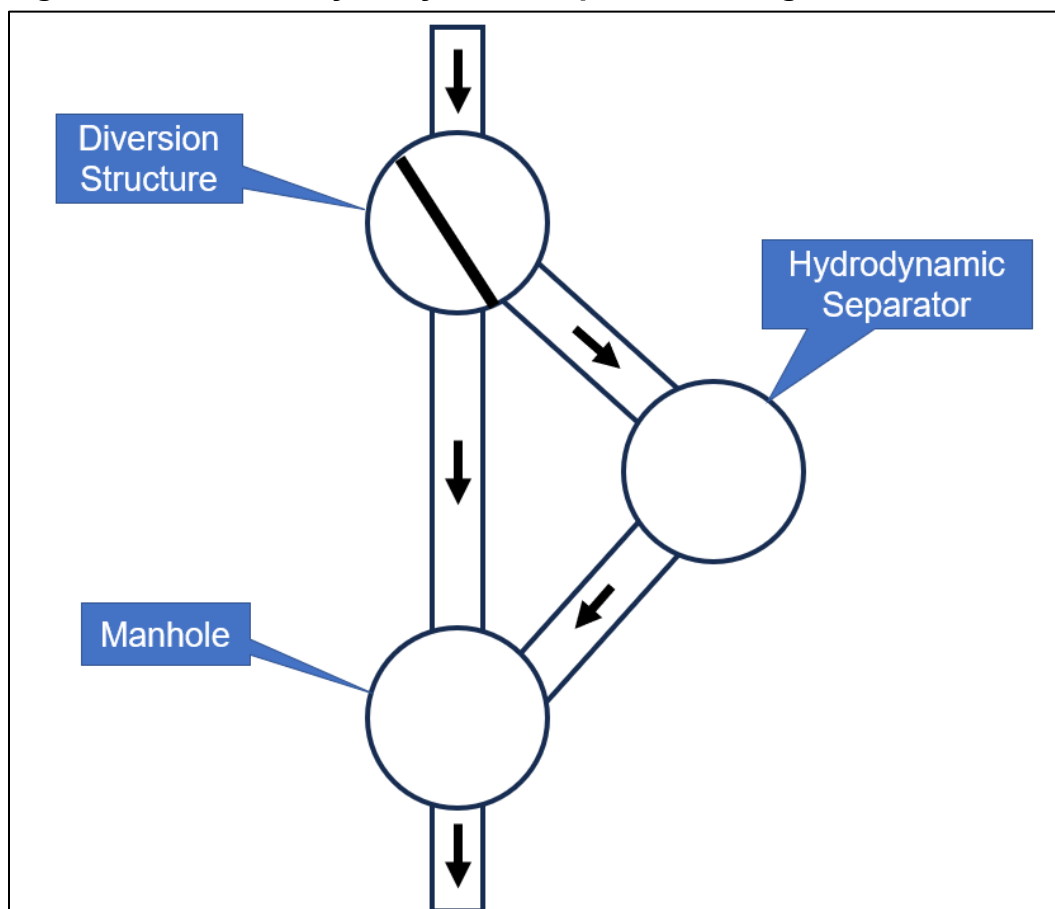
For the non-plunging and plunging cases, the resulting EGL is used to continue computations upstream to the next manhole. At each manhole, repeat the three-step procedure of estimating:

- Entrance losses from entering the outlet pipe
- Additional benching, angled inflow, and plunging losses within the manhole
- Exit losses leaving the inlet pipe and entering the manhole

7.6.4.9 Offline Hydrodynamic Separator Loss

Head loss coefficients used in HGL analysis are determined using the information provided in **Section 7.6.4** for all structures except the hydrodynamic separator. A hydrodynamic separator is used for water quality to remove sediment, debris, and other pollutants (refer to [MDOT Stormwater Treatment Manual](#)). A benching factor of 1.0 is applied to the flow diversion structure in lieu of standard benching loss calculations. Refer to **Figure 7-13**.

Figure 7-13. Offline Hydrodynamic Separator Configuration



Avoid use of inline hydrodynamic separators. Contact the Hydraulic Unit Supervisor for use on trunkline projects that may require inline systems for a variance of the offline

requirement. Head loss coefficients are obtained from the manufacturer with supporting documentation for inline systems.

7.6.4.10 Design Procedure for Storm Sewers

Use ORD D&U for storm sewer calculations. ORD D&U uses “Analysis Mode” for analyzing existing and proposed storm drainage systems. MDOT does not recommend using “Design Mode” for storm drainage layout. Refer to the Drainage and Utilities Guidance software guide on the MDOT Development Guide SharePoint Site. Refer to **Appendix 7-C** for ORD Drainage and Utilities Hydraulic Settings.

Other calculation methods include using **Worksheet 7-2** or spreadsheets. ORD D&U is not approved for use in culvert analysis, watershed analysis using the SCS Method, or when combining ditch (open) and storm sewer (closed) conduits in one network. A step-by-step design procedure using **Worksheet 7-2** is as follows:

- Step 1** Obtain topographic information for the area that is contributing runoff to the proposed sewer, i.e., contributing drainage area, from sources such as USGS quadrangle maps, photogrammetry 2-foot contours, aerial photographs, Digital Elevation Model (DEM), county drains maps, plan/profile sheets, etc. Include drainage areas outside of the ROW. For instructions to import 1 meter DEM into ORD, refer to **Appendix 7-B**.
- Step 2** Identify outlets for the storm sewer system to existing available conveyance systems, such as watercourses, public drains, and storm sewers (refer to Chapter 2, Policy and Procedures). Identify any flow restrictions of the receiving water body.
- Step 3** Prepare a working plan for the layout and profile of the storm sewer system. Locate the existing catch basins and inlets based on the information gathered in Step 1. Place preliminary locations for proposed catch basins and inlets based on topography, roadway geometry, and structure spacing requirements. Consider the capacity of each inlet/catch basin when placing these structures.
- Step 4** Locate the trunk sewer and manholes on the plan portion. Number the manholes sequentially beginning at the upper end of the line.
- Step 5** Develop a tentative grade on the profile for the trunk sewer. Note any conflicts with existing utilities. Refer to Step 17 regarding depth of cover.
- Step 6** Identify the boundaries of the areas contributing to each structure (catch basin or inlet, where applicable). Determine the pervious and impervious percentages within each area.
- Step 7** Compute total acreage within each area. Determine the runoff coefficient for the different percentages of imperviousness.

- Step 8** Determine a weighted runoff coefficient “C” value for the first inlet on the trunk sewer.
- Step 9** Determine proper geographic location, frequency, and the initial time of concentration using the rainfall intensity information referenced in Appendix 3-B.
- Step 10** Use $Q = CiA$ to compute “Q” for the first sewer run and enter it in **Worksheet 7-2**.
- Step 11** After computing “Q” and selecting a tentative sewer grade, calculate the conveyance factor “K” using the formula: $K = Q/S^{0.5}$.
- Knowing Manning’s n, the diameter of sewer pipe can be determined by using **Table 7-5**. For example, when the conveyance factor equals 55, either an 18-inch corrugated steel pipe (CSP) with an $n = 0.024$, or a 15-inch reinforced concrete pipe (RCP) with an $n = 0.013$ will be required.
- Step 12** Compute velocity “V” for this pipe. If the pipe is not full, use **Figure 7-8**. Confirm that the minimum velocity is exceeded. Enter “V” into the sewer tabulation chart.
- Step 13** Using the velocity in Step 12, compute the time required for the water to reach the next manhole. Add this time to the initial time to get the time of concentration for the second manhole.
- Step 14** Add the new areas of the second manhole to the areas of the first manhole according to their runoff coefficients and compute a new weighted “C” value.
- Step 15** Use the new time of concentration, area, intensity, and weighted “C” value to compute the runoff and pipe size.
- Step 16** Repeat this procedure throughout the length of the sewer.
- Step 17** After computing the sewer sizes, note the size and percent grade on the profile sheet along with preliminary pipe invert elevations at the upstream and downstream ends of the pipe. The sewer sizes and grades may be altered to give the most economical sewer in terms of pipe size and depth of sewer or because of conflicts with existing or proposed underground utilities.

After a preliminary sizing is completed, calculate the HGL and EGL through the system, starting at the downstream end. This is important to ensure the tailwater elevation and manhole losses do not cause surcharged conditions.

Job Name _____

Route _____
Control Section _____
Checked by _____

[illegible]

7.6.4.11 Hydraulic and Energy Grade Lines Design Procedure

This section presents a step-by step procedure for calculating the EGL and HGL using the energy loss method as presented in FHWA HEC-22 (2024). Storm sewer design software, such as ORD D&U, is an efficient tool to evaluate the EGL and HGL for most drainage systems. However, understanding the analysis process is important to interpret the output from computer-generated storm sewer designs. The equations and charts necessary to manually calculate the location of the HGL are included in this chapter. Document the outlined steps for each run of pipe in the system using **Worksheet 7-3**, *Storm Sewer Hydraulic and Energy Grade Lines calculations*, or using ORD D&U Flex Tables. Refer to the Drainage and Utilities Guidance software guide on the MDOT Development Guide SharePoint Site. Refer to **Appendix 7-C** for ORD Drainage and Utilities Hydraulic Settings.

Step 1 – Determine the tailwater elevation at the storm sewer outlet

Estimate the water surface elevation at the storm sewer outlet. Refer to the tailwater discussion regarding different scenarios such as large water bodies, existing storm sewers, etc. in **Section 7.6.4.2**. From the initial elevation and corresponding flow and velocity, compute the total energy at the outlet. The HGL is the initial tailwater elevation and the EGL is equal to the HGL plus the velocity head.

The information for the outlet is placed on the first line of **Worksheet 7-3**. Define the Structure ID as the stream, pool, existing storm sewer system, etc. Estimate the HGL_{TW} and EGL_{TW} elevations.

Step 2 – Estimate the HGL_o and EGL_o at the downstream end of the next pipe

Beginning with the outfall pipe, estimate the HGL_o and EGL_o at the downstream end of the pipe, but just inside the pipe. Determine this from the tailwater elevation, physical pipe characteristics, and design flow. Repeat this step for each pipe in the storm sewer system. For subsequent pipes, use the energy state (EGL_a) in the downstream structure as the tailwater condition.

Estimate the EGL_o at the downstream end based on one of the conditions, comparing the pipe with the downstream EGL_o shown in **Table 7-9**. 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 body, substitute the tailwater elevation for EGL_a .

Table 7-9. Case Conditions and Downstream EGL Estimates

Case	Situation (at outlet)	EGL_o Estimate
A	$EGL_a \geq \text{Crown}$ (submerged)	$EGL_a + \text{exit loss}$
B	$\text{Crown} \geq EGL_a > \text{Invert} + d_n$	$\text{Invert} + \text{velocity head} + \text{exit loss}$
C	$\text{Invert} + d_n \geq EGL_a > \text{Invert} + d_c$	$\text{Invert} + \max(EGL_a + \text{exit loss or } d_n + \text{velocity head})$
D	$\text{Invert} + d_c \geq EGL_a > \text{Invert}$	$\text{Invert} + d_n + \text{velocity head}$
E	$\text{Invert} \geq EGL_a$ (plunging pipe)	$\text{Invert} + d_n + \text{velocity head}$

Select the case from **Table 7-9** by determining the required information to evaluate the situation at the outlet of the pipe. The following information may be needed:

- EGL_a – use the energy state from the downstream structure or the EGL_{TW} if discharging to a receiving water body.
- Invert and crown elevations at the outlet end of the pipe.
- Normal depth (d_n) or critical depth (d_c).

After the case has been selected, the following information may be needed to calculate EGL_o :

- Exit loss (H_o) – calculate the exit loss of the system using **Equation 7.26**.
- Inflow pipe exit loss (H_o) – calculate the pipe exit loss using **Equation 7.50**.
- Velocity head ($V^2/2g$).
- Establish the velocity for calculations – If the pipe is in full flow conditions (submerged), use the Continuity Equation ($V=Q/A$). If the pipe is partially full, use the velocity from the storm sewer design calculations.

Calculate EGL_o based on the case. Calculate HGL_o by subtracting the velocity head from EGL_o .

Step 3 – Estimate the HGL_i and EGL_i at the upstream end of the outfall pipe

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

Table 7-10 summarizes potential flow conditions within the conduit that determine calculation of the upstream pipe end's EGL_i and HGL_i . For conditions A, B, and C, the HGL_i and EGL_i are computed using pipe friction. Determine the friction slope (S_f) of the outflow pipe. For condition D, the normal depth establishes the HGL_i . Calculate the EGL_i by computing the velocity head at normal depth and adding it to the HGL_i .

Table 7-10. Flow Conditions at the Upstream End of a Conduit

Condition	Situation (at inlet)	Flow Condition
A	$HGL_i \geq \text{Crown}$	Full flow (surcharge)
B	$\text{Crown} \geq HGL_i > \text{Invert} + d_n$ and $\text{Crown} \geq HGL_i > \text{Invert} + d_c$	Conduit not full but conditions are downstream-controlled
C	$\text{Invert} + d_n \geq HGL_i > \text{Invert} + d_c$	Subcritical partial flow conditions
D	$\text{Invert} + d_c \geq HGL_i$	Supercritical partial flow conditions

The following information is needed to calculate the EGL_i and HGL_i and determine the flow condition at the upstream end:

- Pipe length (L)
- Invert and crown elevations at the inlet end of the pipe
- Friction slope (S_f) – calculate friction slope for pipes flowing full at the outlet end using **Equation 7.29**, otherwise use the pipe slope for partially full flow
- Pipe loss (H_f) – calculate the pipe loss using **Equation 7.28**

Calculate the EGL_i by adding pipe loss to the EGL_o . Then calculate the HGL_i by subtracting the velocity head from the EGL_i .

Determine the flow condition at the upstream end using **Table 7-10** by checking the situation at the upstream end:

- Check condition A. If true, carry losses upstream as computed.
- If condition A is false, determine d_n and d_c and check for conditions B, C, or D.
 - Condition B – Pipe is partially full but downstream controlled. Confirm the H_f was calculated using **Equation 7.29** and carry losses upstream as computed.
 - Condition C – Pipe is partially full flow. Confirm the H_f was calculated using pipe slope and carry losses upstream as computed.
 - Condition D – Pipe is supercritical and partially full flow. Pipe losses are not carried upstream. Recompute the HGL_i and EGL_i . The HGL_i is the d_n plus the pipe invert. The EGL_i is the HGL_i plus the velocity head.

Compute the outflow pipe energy head (E_i) using **Equation 7.31**.

Step 4 – Calculate the EGL and HGL in the structure

Except at the most upstream inlet structures, structures include one outlet pipe and one or more inlet pipes. At each structure, use the methods shown in **Sections 7.6.4.6** and **7.6.4.7** to calculate the energy losses through the structure. The HGL at the structure is the HGL at the upstream end of the outflow conduit (that was just calculated) plus the losses. The HGL is calculated for the outflow discharge.

Consider each lateral or trunk line entering the structure. Calculate the HGL and EGL at the structure according to methods in **Sections 7.6.4.6** and **7.6.4.7**.

Initial Structure Energy Level (E_{ai})

Determine the E_{ai} as the maximum energy level of:

- Outlet control – partial or full flow (E_{aio}): If the outflow is in supercritical flow, the E_{aio} is zero. Otherwise, E_{aio} equals E_i plus the entrance loss using **Equations 7.33 and 7.34**.
- Inlet control – submerged (E_{ais}): Compute the E_{ais} using **Equation 7.36**.
- Inlet control – unsubmerged (E_{aiu}): Compute the E_{aiu} using **Equation 7.37**.

Revised Manhole Energy (E_a)

Calculate head loss through the structure:

- Determine the energy loss coefficient for benching (C_B) using **Table 7-8**. Half benching is typically used for trunkline projects.
- Determine the energy loss coefficient for angled flows (C_θ) using **Equation 7.41**.
- Determine the energy loss coefficient for plunging flows (C_P) using **Equation 7.44**.

Check to ensure net energy is positive, $E_{ai} > E_i$.

- If $E_{ai} < E_i$, the head loss (H_a) through the structure will be zero, $E_a = E_i$.
- If $E_{ai} \geq E_i$, calculate the H_a using **Equation 7.46**. The value of H_a is always positive. If the calculation yields a negative value, set the H_a equal to zero.

Calculate the revised manhole energy (E_a) using **Equation 7.47**.

Manhole Energy Grade Line (EGL_a)

Calculate the EGL_a using **Equation 7.48**.

Step 5 – Repeat steps 2 through 4 for all pipes and structures in the storm sewer system

Continuing to work upstream, compute the HGL and EGL at the downstream end of the next pipe (step 2) and the upstream end of each pipe based on the HGL and EGL of the downstream end (step 3). Calculate the HGL and EGL in the next upstream structure (step 4). For a terminal structure, e.g., upstream inlet, no additional computations are needed for that storm drain branch. Continue with additional branches, as required.

Trunk lines and laterals often involve hydraulic drops into a junction as energy controls or to control the depth of trenching. Avoid conduit runs that exhibit supercritical flow. Hydraulic jumps inside of conduits are difficult to predict and may result in “blowouts” or damage to structures. Hydraulic drops are used to avoid supercritical flow.

Job Name: _____

Date: _____ Route: _____
Job No.: _____ Control Section: _____
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Sheet: _____ of _____

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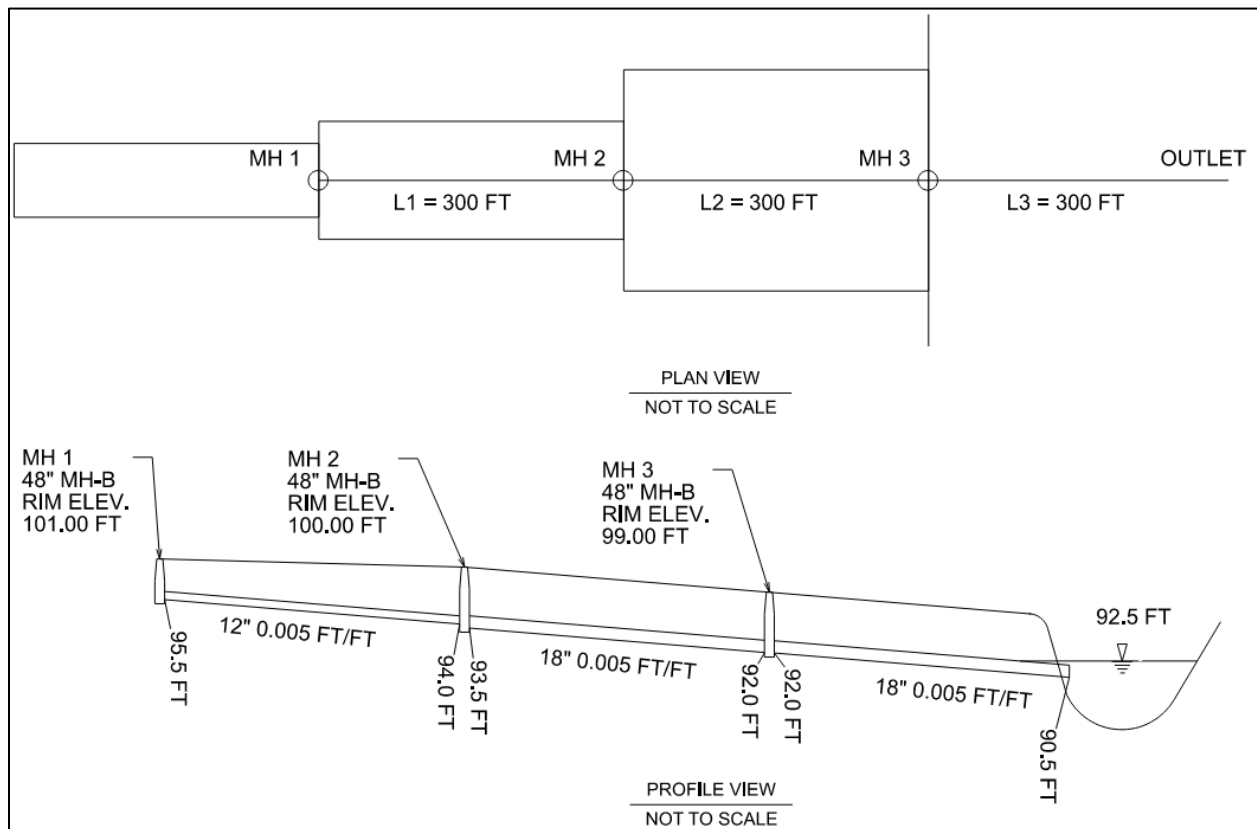
7.6.4.12 Example Problem

The example problem, shown in **Figure 7-14**, illustrates how to size a storm sewer using the Rational Method as discussed in **Section 7.3.2** and how to calculate the HGL and EGL for the system as described in **Section 7.6.4**.

Problem: For a project located in Lansing, size a storm sewer for a 10% AEP (10-year) storm event using the data provided. The storm sewer is constructed with RCP (Manning's $n = 0.013$). The receiving water body elevation is 92.5 ft.

Size the storm sewer following the procedures in **Section 7.6.4.10** for a 10% AEP (10-year) storm event. Calculate the EGL and HGL of the storm sewer system starting at the downstream end of the storm sewer. Follow **Section 7.6.4.11** for detailed step descriptions.

Figure 7-14. Example Problem for EGL Calculation



Where:

MH 1
Area = 0.5 acres
 $C = 0.9$
 $T_c = 15$ min

MH 2
Area = 0.8 acres
 $C = 0.7$
 $T_c = 15$ min

MH 3
Area = 1.5 acres
 $C = 0.5$
 $T_c = 15$ min

Storm Sewer Preliminary Layout Worksheet

Date _____
Job No. _____
Computed by _____

Sheet _____ of _____

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Step 1 – Determine the tailwater elevation at the storm sewer outlet

The storm sewer outlet drains to a pond with an elevation of 92.50 ft. The HGL is set at the initial tailwater elevation of 92.50 ft. The EGL is the HGL plus the velocity head ($V^2/2g$).

$$\text{HGL}_{\text{TW}} = \text{Downstream pond water surface elevation} = 92.50 \text{ ft}$$

$$\text{EGL}_{\text{TW}} = 92.50 \text{ ft (assume no velocity head in the pond)}$$

Step 2 – Estimate the HGL_o and EGL_o at the downstream end of the MH 3 pipe

General information to determine case:

$$D = 1.5 \text{ ft}$$

$$Q = 5.8 \text{ cfs}$$

$$\text{Invert} = 90.50 \text{ ft}$$

$$\text{Crown} = \text{Invert} + D = 90.50 \text{ ft} + 1.5 \text{ ft} = 92.00 \text{ ft}$$

$$\text{EGL}_a = 92.50 \text{ ft (for outlet pipe } \text{EGL}_{\text{TW}} \text{ is substituted for } \text{EGL}_a)$$

Determine case from **Table 7-9**:

Is the downstream end of the pipe submerged ($\text{EGL}_{\text{TW}} \geq \text{Crown}$)?

92.50 ft $\geq$ 92.00 ft is true. Pipe is submerged (case A). Use full pipe flow.

Estimate the energy loss exiting the pipe:

$$V = Q/A = 5.8 \text{ cfs} / (\pi (0.75 \text{ ft})^2) = 3.28 \text{ fps}$$

$$V^2/2g = (3.28 \text{ fps})^2 / (64.4 \text{ ft/s}^2) = 0.17 \text{ ft}$$

Exit loss for an outfall:

$$H_o = (1.0) V^2/2g = (1.0) (0.17 \text{ ft}) = 0.17 \text{ ft}$$

$$\text{EGL}_o = \text{EGL}_a + H_o = 92.50 \text{ ft} + 0.17 \text{ ft} = 92.67 \text{ ft}$$

$$\text{HGL}_o = \text{EGL}_o - V^2/2g = 92.67 \text{ ft} - 0.17 \text{ ft} = 92.50 \text{ ft}$$

Step 3 – Estimate the HGL_i and EGL_i at the upstream end of the MH 3 pipe

Required information to determine HGL_i and EGL_i :

$$\text{Pipe length, } L = 300 \text{ ft}$$

$$\text{Invert} = 92.00 \text{ ft}$$

$$\text{Crown} = 92.00 \text{ ft} + 1.5 \text{ ft} = 93.50 \text{ ft}$$

Estimate the pipe loss, H_f :

$$S_f = [(Q_n)/(K_Q D^{2.67})]^2 = [(5.80 \text{ cfs})(0.013)/(0.46)(1.5 \text{ ft})^{2.67}]^2 = 0.0031 \text{ ft/ft}$$

$$H_f = S_f L = (0.0031 \text{ ft/ft}) (300 \text{ ft}) = 0.93 \text{ ft}$$

$$\text{EGL}_i = \text{EGL}_o + \text{pipe loss} = 92.67 \text{ ft} + 0.93 \text{ ft} = 93.60 \text{ ft}$$

$$\text{HGL}_i = \text{EGL}_i - V^2/2g = 93.60 \text{ ft} - 0.17 \text{ ft} = 93.43 \text{ ft}$$

$$E_i = \text{EGL}_i - \text{Invert} = 93.60 \text{ ft} - 92.00 \text{ ft} = 1.60 \text{ ft}$$

Determine the flow condition at the upstream end of the pipe from **Table 7-10**:
 $HGL_i (93.43 \text{ ft}) \geq \text{Crown} (93.50 \text{ ft})$ is not true, therefore, pipe is not flowing full, not condition A.

Estimate d_n and d_c :

$$d_n = 1.00 \text{ ft}$$

$$d_c = 0.93 \text{ ft}$$

$\text{Crown} (93.50 \text{ ft}) \geq HGL_i (93.43 \text{ ft}) > \text{Invert} + d_n (92.00 \text{ ft} + 1.00 \text{ ft} = 93.00 \text{ ft})$ and $\text{Crown} (93.50 \text{ ft}) \geq HGL_i (93.43 \text{ ft}) > \text{Invert} + d_c (92.00 \text{ ft} + 0.93 \text{ ft} = 92.93 \text{ ft})$ is true; therefore, pipe is not full but conditions are downstream-controlled (condition B). Pipe losses are carried upstream as computed.

Step 4 – Calculate EGL and HGL in the MH 3 pipe

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

Compute E_{aio} :

$$H_i = (K_o)(V^2/2g) = (0.2)(0.17 \text{ ft}) = 0.03 \text{ ft}$$

$$E_{aio} = E_i + H_i = 1.60 \text{ ft} + 0.03 \text{ ft} = 1.63 \text{ ft}$$

Compute discharge intensity (DI) for the outflow pipe:

$$DI = Q/[A(Dg)^{0.5}] = 5.8 \text{ cfs} / [(\pi/4)(1.5 \text{ ft})^2((1.5 \text{ ft})(32.2 \text{ ft/s}^2))^{0.5}] = 0.47$$

Compute E_{ais} :

$$E_{ais} = (DI)^2(D) = (0.47)^2(1.5 \text{ ft}) = 0.33 \text{ ft}$$

Compute E_{aiu} :

$$E_{aiu} = 1.6 (D)(DI)^{0.67} = 1.6 (1.5 \text{ ft}) (0.47)^{0.67} = 1.45 \text{ ft}$$

Select maximum: $E_{ai} = 1.63 \text{ ft}$

Compute E_a :

For benching:

$C_B = -0.80$; $E_{ai} / D = 1.45 \text{ ft} / 1.5 \text{ ft} = 1.09 \text{ ft} > 1$, therefore, assume half benched, interpolate between submerged and unsubmerged values in **Table 7-8**.

For angled inflow:

$C_\Theta = 0.0$; because $\Theta_w = 180$ degrees.

For plunging flow:

$$z_k = 92.00 \text{ ft} - 92.00 \text{ ft} = 0.00 \text{ ft}$$

$$h_k = (z_k - E_{ai}) / D = (0.00 \text{ ft} - 1.63 \text{ ft}) / 1.5 \text{ ft} = -1.09$$

$$C_P = \Sigma Q_k h_k / Q = ((5.80 \text{ cfs})(-1.09)) / (5.80 \text{ cfs}) = 0, \text{ result not greater than } 0.$$

Check to ensure net energy. Is $E_{ai} > E_i$? Since $1.63 \text{ ft} > 1.60 \text{ ft}$ OK

$$H_a = (E_{ai} - E_i) (C_B + C_\Theta + C_P) = (1.63 \text{ ft} - 1.60 \text{ ft})(-0.80 + 0.0 + 0.0) = 0, \text{ result not greater than } 0.$$

$$E_a = E_{ai} + H_a = 1.63 \text{ ft} + 0.00 \text{ ft} = 1.63 \text{ ft}$$

Compute EGL_a:

$$\text{EGL}_a = E_a + \text{BOC}_i = 1.63 \text{ ft} + 92.00 \text{ ft} = 93.63 \text{ ft}$$

Ground elevation = 99.00 ft > 93.63 ft by more than 1 ft OK

Repeat Step 2 – Estimate the HGL_o and EGL_o at the downstream end of the MH 2 pipe

General information to determine case:

$$D = 1.5 \text{ ft}$$

$$Q = 3.5 \text{ cfs}$$

$$\text{Invert} = 92.00 \text{ ft}$$

$$\text{Crown} = \text{Invert} + D = 92.00 \text{ ft} + 1.5 \text{ ft} = 93.50 \text{ ft}$$

$$\text{EGL}_a = 93.63 \text{ ft}$$

Determine the case from **Table 7-9**:

Is the downstream end of the pipe submerged ($\text{EGL}_a \geq \text{Crown}$)?

93.63 ft $\geq$ 93.50 ft is true. Pipe is submerged (case A). Use full pipe flow.

With submerged outlet:

$$V = Q/A = 3.5 \text{ cfs} / (\pi (0.75 \text{ ft})^2) = 1.98 \text{ fps}$$

$$V^2/2g = (1.98 \text{ fps})^2 / (64.4 \text{ ft/s}^2) = 0.06 \text{ ft}$$

Inflow pipe exit loss:

$$H_o = (0.4) V^2/2g = (0.4) (0.06 \text{ ft}) = 0.02 \text{ ft}$$

$$\text{EGL}_o = \text{EGL}_a + H_o = 93.63 \text{ ft} + 0.02 \text{ ft} = 93.65 \text{ ft}$$

$$\text{HGL}_o = \text{EGL}_o - V^2/2g = 93.65 \text{ ft} - 0.06 \text{ ft} = 93.59 \text{ ft}$$

Repeat Step 3 – Estimate the HGL_i and EGL_i at the upstream end of MH 2 pipe

Required information to determine the HGL_i and EGL_i:

$$\text{Pipe Length, } L = 300 \text{ ft}$$

$$\text{Invert} = 93.50 \text{ ft}$$

$$\text{Crown} = 93.50 \text{ ft} + 1.5 \text{ ft} = 95.00 \text{ ft}$$

Estimate the pipe loss, H_f :

$$S_f = [(Q_n)/(K_Q D^{2.67})]^2 = [(3.5 \text{ cfs})(0.013)/(0.46)(1.5 \text{ ft})^{2.67}]^2 = 0.0011 \text{ ft/ft}$$

$$H_f = S_f L = (0.0011 \text{ ft/ft}) (300 \text{ ft}) = 0.33 \text{ ft}$$

$$\text{EGL}_i = \text{EGL}_o + \text{pipe loss} = 93.65 \text{ ft} + 0.33 \text{ ft} = 93.98 \text{ ft}$$

$$\text{HGL}_i = \text{EGL}_i - V^2/2g = 93.98 \text{ ft} - 0.06 \text{ ft} = 93.92 \text{ ft}$$

$$E_i = \text{EGL}_i - \text{Invert} = 93.98 \text{ ft} - 93.50 \text{ ft} = 0.48 \text{ ft}$$

Determine the flow condition at the upstream end of the pipe from **Table 7-10**:

HGL_i (93.92 ft) > Crown (95.00 ft) is not true, therefore, pipe is not flowing full, not condition A.

Estimate d_n and d_c :

$$d_n = 0.72 \text{ ft}$$

$$d_c = 0.71 \text{ ft}$$

Invert + d_c (93.50 ft + 0.71 ft = 94.21 ft) $\geq$ HGL_i (93.92 ft) is true; therefore, pipe is supercritical partial flow, condition D. Pipe losses not carried upstream. Recompute HGL_i and EGL_i .

$$V = 4.14 \text{ fps}$$

$$V^2/2g = (4.14 \text{ fps})^2 / (64.4 \text{ ft/s}^2) = 0.27 \text{ ft}$$

$$HGL_i = d_n + \text{Invert} = 0.72 \text{ ft} + 93.50 \text{ ft} = 94.22 \text{ ft}$$

$$EGL_i = HGL_i + V^2/2g = 94.22 \text{ ft} + 0.27 \text{ ft} = 94.49 \text{ ft}$$

$$E_i = EGL_i - \text{Invert} = 94.49 \text{ ft} - 93.50 \text{ ft} = 0.99 \text{ ft}$$

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

Repeat Step 4 – Calculate the EGL and HGL in the MH 2 pipe

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

Compute E_{aio} :

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

Compute discharge intensity (DI) for the outflow pipe:

$$DI = Q/[A(Dg)^{0.5}] = 3.5 \text{ cfs} / [(\pi/4)(1.5 \text{ ft})^2((1.5 \text{ ft})(32.2 \text{ ft/s}^2))^{0.5}] = 0.29$$

Compute E_{ais} :

$$E_{ais} = (DI)^2(D) = (0.29)^2(1.5 \text{ ft}) = 0.13 \text{ ft}$$

Compute E_{aiu} :

$$E_{aiu} = 1.6 (D)(DI)^{0.67} = 1.6 (1.5 \text{ ft}) (0.29)^{0.67} = 1.05 \text{ ft}$$

Select maximum: $E_{ai} = 1.05 \text{ ft}$

Compute E_a :

For benching:

$C_B = -0.85$; $E_{ai} / D = 1.05 \text{ ft} / 1.5 \text{ ft} = 0.70 \text{ ft} < 1$, therefore, assume half benched, unsubmerged values in **Table 7-8**.

For angled inflow:

$C_\theta = 0.0$; because flow is plunging and $\Theta_w = 180$ degrees.

For plunging flow:

$$z_k = 94.00 \text{ ft} - 93.50 \text{ ft} = 0.50 \text{ ft}$$

$$h_k = (z_k - E_{ai}) / D = (0.50 \text{ ft} - 1.05 \text{ ft}) / 1.5 \text{ ft} = -0.37$$

$$C_P = \Sigma Q_k h_k / Q = ((3.50 \text{ cfs})(-0.37)) / (3.50 \text{ cfs}) = 0, \text{ result not greater than } 0.$$

Check to ensure net energy. Is $E_{ai} > E_i$? Since $1.05 \text{ ft} > 0.99 \text{ ft}$ OK

$$H_a = (E_{ai} - E_i) (C_B + C_\theta + C_P) = (1.05 \text{ ft} - 0.99 \text{ ft})(-0.85 + 0.0 + 0.0) = 0, \text{ result not greater than } 0.$$

$$E_a = E_{ai} + H_a = 1.05 \text{ ft} + 0.00 \text{ ft} = 1.05 \text{ ft}$$

Compute EGL_a:

$$\text{EGL}_a = E_a + \text{Invert} = 1.05 \text{ ft} + 93.5 \text{ ft} = 94.55 \text{ ft}$$

Ground elevation = 100.00 ft > 94.55 ft by more than 1 ft OK

Repeat Step 2 – Estimate the HGL_o and EGL_o at the downstream end of the MH 1 pipe

General information to determine case:

$$D = 1.0 \text{ ft}$$

$$Q = 1.6 \text{ cfs}$$

$$\text{Invert} = 94.00 \text{ ft}$$

$$\text{Crown} = \text{Invert} + D = 94.00 \text{ ft} + 1.0 \text{ ft} = 95.00 \text{ ft}$$

$$\text{EGL}_a = 94.55 \text{ ft}$$

Determine case from **Table 7-9**:

Is the downstream end of the pipe submerged ($\text{EGL}_a \geq \text{Crown}$)?

$94.55 \text{ ft} \geq 95.00 \text{ ft}$ is not true, not case A.

Estimate d_n and d_c :

$$d_n = 0.58 \text{ ft}$$

$$d_c = 0.54 \text{ ft}$$

$\text{Invert} + d_n \geq \text{EGL}_a > \text{Invert} + d_c$ ($94.00 \text{ ft} + 0.58 \text{ ft} = 94.58 \text{ ft} \geq (94.55 \text{ ft}) > (94.00 \text{ ft} + 0.54 \text{ ft} = 94.54 \text{ ft})$) is true, case C, unsubmerged

With unsubmerged outlet:

$$V = 3.4 \text{ fps}$$

$$V^2/2g = (3.4 \text{ fps})^2 / (64.4 \text{ ft/s}^2) = 0.18 \text{ ft}$$

Inflow pipe exit loss:

$$H_o = (0.4) V^2/2g = (0.4) (0.18 \text{ ft}) = 0.08 \text{ ft}$$

$$\text{EGL}_o = \text{Invert} + \max (\text{EGL}_a + H_o \text{ or } d_n + V^2/2g)$$

$$\text{EGL}_a + H_o = 94.55 \text{ ft} + 0.08 \text{ ft} = 94.63 \text{ ft}$$

$$\text{Invert} + d_n + V^2/2g = 94.00 \text{ ft} + 0.58 \text{ ft} + 0.18 \text{ ft} = 94.76 \text{ ft}$$

$$\text{EGL}_o = 94.76 \text{ ft}$$

$$HGL_o = EGL_o - V^2/2g = 94.76 \text{ ft} - 0.18 \text{ ft} = 94.58 \text{ ft}$$

Repeat Step 3 – Estimate the HGL_i and EGL_i at the upstream end of the MH 1 pipe

Required information to determine the HGL_i and EGL_i :

Pipe length, $L = 300 \text{ ft}$

Invert = 95.50 ft

Crown = 95.50 ft + 1.0 ft = 96.50 ft

Estimate the pipe loss, H_f :

$S_f = 0.005 \text{ ft/ft}$

$H_f = S_f L = (0.005 \text{ ft/ft}) (300 \text{ ft}) = 1.50 \text{ ft}$

$EGL_i = EGL_o + \text{pipe loss} = 94.76 \text{ ft} + 1.50 \text{ ft} = 96.26 \text{ ft}$

$HGL_i = EGL_i - V^2/2g = 96.26 \text{ ft} - 0.18 \text{ ft} = 96.08 \text{ ft}$

$E_i = EGL_i - \text{Invert} = 96.26 \text{ ft} - 95.50 \text{ ft} = 0.76 \text{ ft}$

Determine the flow condition at the upstream end of the pipe from **Table 7-10**:

HGL_i (96.08 ft) > Crown (96.50 ft) is not true, therefore, pipe is not flowing full, not condition A.

Estimate d_n and d_c :

$d_n = 0.58 \text{ ft}$

$d_c = 0.54 \text{ ft}$

Invert + d_n (95.50 ft + 0.58 ft = 96.08 ft) $\geq HGL_i$ (96.08 ft) > Invert + d_c (95.50 ft + 0.54 ft = 96.04 ft) is true; therefore, pipe is subcritical partial flow, condition C. Therefore, losses are carried upstream as computed.

Repeat Step 4 – Calculate the EGL and HGL in the MH 1 pipe

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

Compute E_{aio} :

$H_i = (K_o)(V^2/2g) = (0.2)(0.18 \text{ ft}) = 0.04 \text{ ft}$

$E_{aio} = E_i + H_i = 0.76 \text{ ft} + 0.08 \text{ ft} = 0.80 \text{ ft}$

Compute discharge intensity (DI) for the outflow pipe:

$DI = Q/[A(Dg)^{0.5}] = 1.6 \text{ cfs} / [(\pi/4)(1.0 \text{ ft})^2((1.0 \text{ ft})(32.2 \text{ ft/s}^2))^{0.5}] = 0.35$

Compute E_{ais} :

$E_{ais} = (DI)^2(D) = (0.35)^2(1.0 \text{ ft}) = 0.12 \text{ ft}$

Compute E_{aiu} :

$E_{aiu} = 1.6 (D)(DI)^{0.67} = 1.6 (1.0 \text{ ft}) (0.35)^{0.67} = 0.79 \text{ ft}$

Select maximum: $E_{ai} = 0.80 \text{ ft}$

Compute E_a :

For benching:

$C_B = 0.0$, no inflow pipes, benching not a factor.

For angled inflow:

$C_\Theta = 0.0$; no inflow pipes, angled inflow not a factor.

For plunging flow:

$$z_k = 101.00 \text{ ft} - 95.50 \text{ ft} = 5.50 \text{ ft}$$

$$h_k = (z_k - E_{ai}) / D = (5.50 \text{ ft} - 0.80 \text{ ft}) / 1.0 \text{ ft} = 4.7$$

$$C_P = \Sigma Q_k h_k / Q = ((1.6 \text{ cfs})(4.7)) / (1.6 \text{ cfs}) = 4.7$$

Check to ensure net energy. Is $E_{ai} > E_i$? Since $0.80 \text{ ft} > 0.76 \text{ ft}$ OK

$$H_a = (E_{ai} - E_i) (C_B + C_\Theta + C_P) = (0.80 \text{ ft} - 0.76 \text{ ft})(0.0 + 0.0 + 4.7) = 0.18 \text{ ft}$$

$$E_a = E_{ai} + H_a = 0.80 \text{ ft} + 0.18 \text{ ft} = 0.98 \text{ ft}$$

Compute EGL_a :

$$EGL_a = E_a + \text{Invert} = 0.98 \text{ ft} + 95.5 \text{ ft} = 96.48 \text{ ft}$$

$$HGL_a = EGL_a - V^2/2g = 96.48 \text{ ft} - 0.18 \text{ ft} = 96.30 \text{ ft}$$

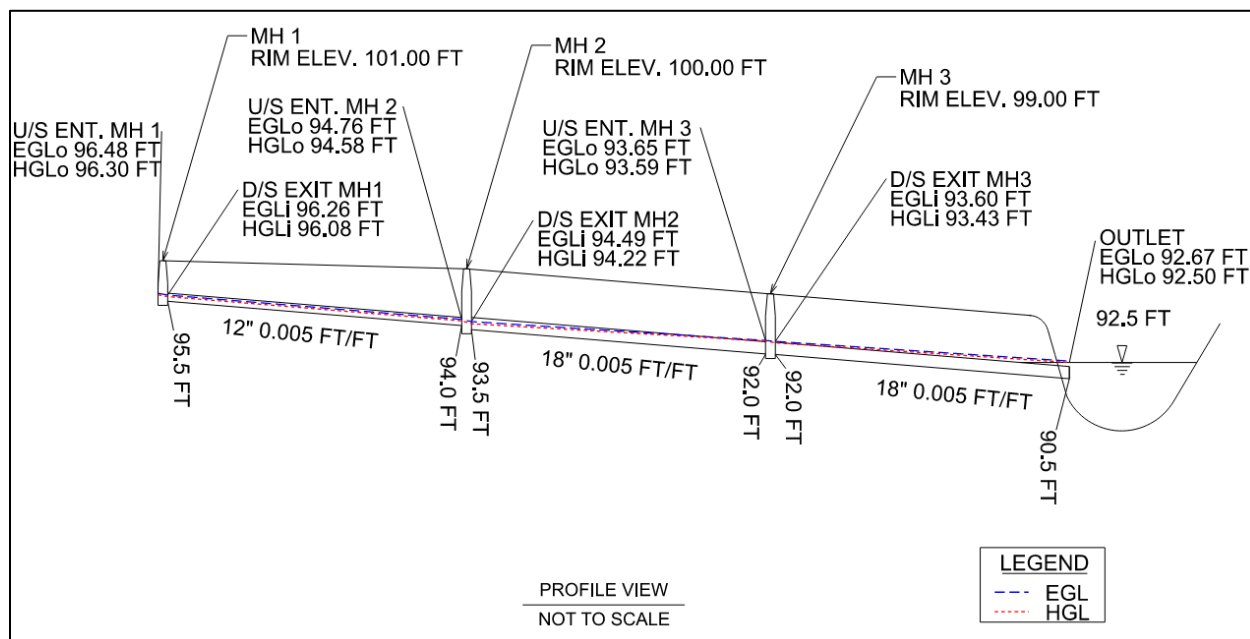
Ground elevation = $101.0 \text{ ft} > 96.48 \text{ ft}$, by more than 1 ft OK

Conclusions: The calculations show that the HGL elevation is 4.7 ft below the ground elevation at the MH 1 pipe. The HGL should not exceed the ground elevation and should be at least 1 ft below the gutter pan in depressed roadways. **Figure 7-15** shows the EGL and HGL through the system.

Storm Sewer Hydraulic and Energy Grade Line Calculations
Job Name: _____

Date: _____ Route: _____
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Computed by: _____ Checked by: _____
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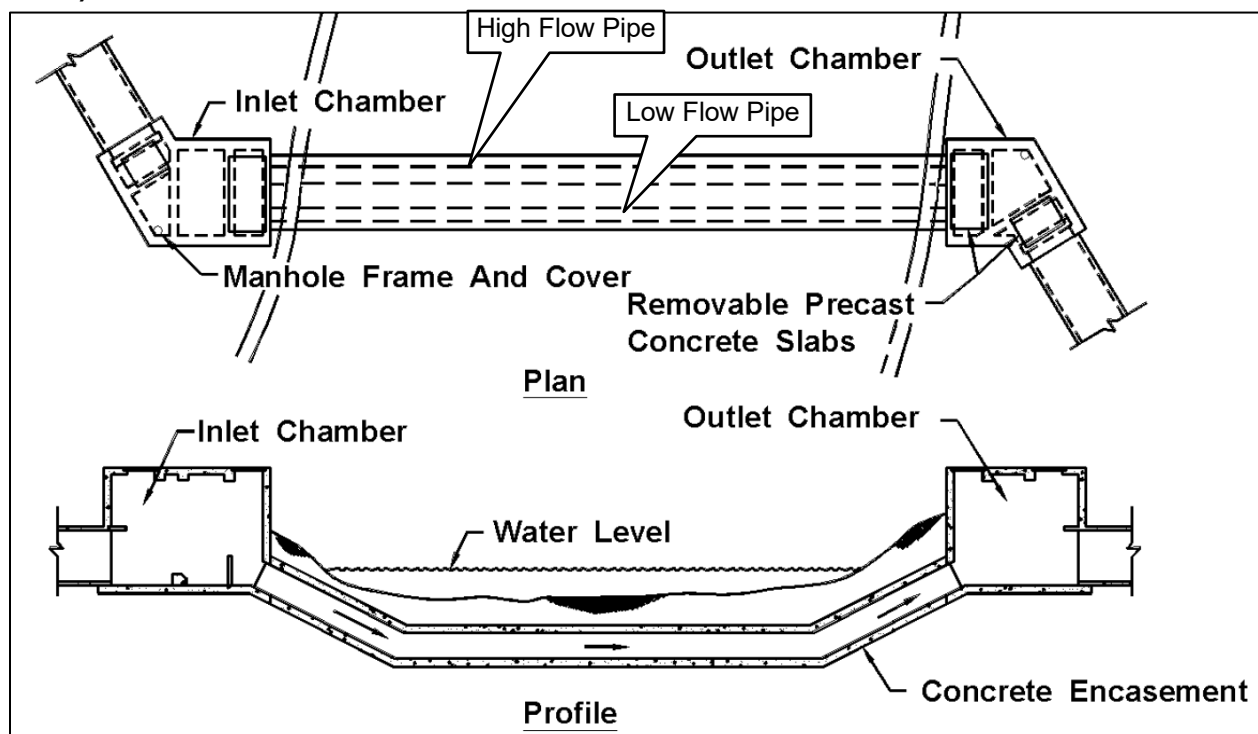
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Figure 7-15. Example Profile with Calculated EGL Values

7.6.5 Inverted Siphon (Sag Culvert, Submerged Culverts)

An inverted siphon, or sag culvert, carries flow under an obstruction such as a sanitary sewer, water main, or any other utility or structure in the path of the storm sewer line. The storm sewer invert is lowered at the obstacle and raised again after the crossing. A minimum of two barrels with minimum velocity of 4 to 5 fps is recommended. The minimum velocities are to be met for the 100% AEP (1-year) storm for the low flow barrel and the design storm AEP for the high flow barrel. One barrel is a low flow pipe that will convey the flow up to the 100% AEP (1-year) storm. The second high flow barrel will convey the remainder of the flow up to the design flow rate. The inlet structure is designed to divert flow between the two barrels. At the outlet structure, the high flow barrel invert is raised to prevent the backflow of sediment from the low flow barrel from accumulating in the high flow barrel. Locate siphon chambers outside of travel lanes for future maintenance, where practical. Include stop log slots in the inlet and outlet chamber to isolate the low flow barrel for maintenance. A plan and profile schematic of a double barrel siphon is shown in **Figure 7-16**.

Figure 7-16. Plan and Profile Schematic of Double Barrel Siphon (FHWA HEC-22, 2024)



The major components of a double barrel siphon are described as follows.

Inlet Chamber - The inlet chamber collects flow from the upstream system and splits the flow between the low flow and high flow pipes. The flow split is achieved using a diversion weir or by raising the inlet of the high flow pipe above the 100% AEP (1-year) water level in the chamber. Maintenance access is required at this chamber to inspect and clean the siphon.

Low Flow Pipe - The low flow pipe conveys 100% of the flow up to the 100% AEP (1-year) storm without diversion to the high flow barrel. The pipe conveys additional flow during the design AEP storm. The pipe is designed to maintain a sediment transport velocity of 4 to 5 fps. The slope of the downward side of the sag can vary, but a 45-degree slope is recommended for ease of maintenance. The pipe slope on the rising side of the sag is recommended to be no greater than 22.5 degrees to enhance sediment transport and to use common pipe fittings.

High Flow Pipe - The high flow pipe conveys the remainder of the flow up to the design flow rate that is not conveyed in the low flow pipe. The inlet is protected from sediment accumulation by a diversion weir or an offset in the invert relative to the chamber floor. The outlet invert is set equal to the 100% AEP (1-year) water level in the downstream chamber.

Outlet Chamber - The outlet chamber re-combines flow from the low and high flow pipes. Maintenance access is required at this chamber for siphon inspection and cleaning.

7.6.5.1 Siphon Design Procedure

Siphon design is required to balance scour velocity requirements with total hydraulic head loss through the system. Some portions of the analysis require iterative solutions to balance the total energy in the inlet chamber when the flow is split between the low and high flow pipes. Hydraulic losses due to friction, minor losses, and weirs are calculated as part of the analysis. The design is required to consider the upstream backwater impacts. General guidance for the development of a siphon design follows.

Step 1 – Determine Design Flow Rates

Develop design flow rates at the inlet chamber using the methodologies described in Chapter 3. The required flow rates are the 100% AEP (1-year) storm and the design storm AEP for the system.

Step 2 – Determine Downstream Boundary Condition

Calculate hydraulic losses in the siphon system from downstream to upstream. Use a starting water surface elevation at the outlet from the downstream chamber. Starting water surface elevations are required for both the 100% AEP (1-year) storm and the design storm AEP flow condition.

Step 3 – Design the Low Flow Pipe

Design the low flow pipe to convey the flow from the 100% AEP (1-year) storm while maintaining the minimum full flow pipe velocity. Establish the pipe size, length, vertical and horizontal geometry to connect the upstream and downstream storm water system while avoiding the obstacle. Ensure proper clearance under the obstacle. Slope the downward and rising pipes to promote sediment transport and aid in maintenance.

Using the 100% AEP (1-year) storm, calculate the hydraulic losses through the system to establish a water level in the inlet chamber. This level establishes the hydraulic control for the high flow pipe. Hydraulic losses include friction, exit, entrance, and bends. The design is revisited if the initial sizing of the outlet chamber changes after the high flow pipe is designed.

Step 4 – Design the High Flow Pipe

The design of the high flow pipe is the most challenging portion of the design process. Design the high flow pipe simultaneously with the inlet chamber and the hydraulic control (e.g. weir, elevated pipe, etc.) within the inlet diversion chamber. Develop an initial layout of the high flow pipe using a vertical and horizontal alignment similar to the low flow pipe. Use the AEP design storm flow rate for design of the high flow pipe. The flow is split within the inlet chamber by balancing the total energy through the low flow and high flow pipes. The flow split analysis is an iterative process that requires an initial estimate of the flow distribution between the low and high flow pipes. Maintain the minimum velocity in both pipes.

Step 5 – Evaluate Upstream Impacts

The siphon is part of the storm water system and does not require an independent analysis for harmful interference. However, total hydraulic head loss through the siphon must consider the potential backwater impacts on the upstream stormwater system. Provide open channel conditions for the AEP design flow in the inlet chamber.

Additional information on siphon design can be found in:

- US Bureau of Reclamation, Design of Small Canal Structures, 1978
- McGraw Hill, Water Supply and Sewerage, Sixth Edition, 1991

7.6.6 Underdrains

For information on underdrains, refer to Section 4.06 of the MDOT Road Design Manual. Design underdrains using the principles in this chapter.

7.7 MAINTENANCE

7.7.1 Drainage Structures

Drainage structure maintenance follows the work methods described in the MDOT Maintenance Activity Guide, Activity 1220, Drainage Structure Cleanout. Contact the MDOT Maintenance Services Section Engineer for more information on trunkline projects.

7.7.2 Storm Sewers

Storm sewer maintenance involves the removal of sand, silt, or debris and the maintenance of a tight seal at each pipe joint. Abrasive material may be present in the water (or some chemical that has a deleterious effect on the pipe) which causes the pipe material to degrade. Reline the pipe to preserve its integrity. For major reconstruction projects, inspect the entire storm system prior to design to understand existing conditions and if remedial action is needed.

7.8 ADVANCED HYDRAULICS

This section presents potential hydraulic situations and design approaches for unique conditions that are not typically found in highway and roadway systems. The majority of this discussion focuses on the hydraulic elements associated with deep or shallow stormwater tunnel systems. These hydraulic elements and the analysis approach may be found in other systems and the concepts described may be applied to those situations. For example, a stormwater system may be experiencing frequent and unexplained blow off of manhole lids. This phenomenon may be related to surge or venting problems that can be analyzed using the tools associated with stormwater tunnel analysis.

This section is intended to help understand design conditions that may require the use of advanced hydraulic analysis and provides a high-level description of the methods and tools that can be used as part of the analysis. Due to the complexity of the required analysis, specific example problems are not provided.

For trunkline projects, contact the Hydraulic Unit Supervisor before developing the approach or analysis for systems discussed in this section.

7.8.1 Stormwater Tunnel Systems

Stormwater tunnel systems are designed for storage, conveyance, or a combination of the two. Stormwater tunnels are generally constructed relatively deep to avoid existing utilities and infrastructure. Due to the depth of stormwater tunnels, a gravity outlet is not always possible and the stormwater tunnel is dewatered using a pump station. The major components of a stormwater tunnel system are described as follows and shown graphically in **Figure 7-17**.

Stormwater Tunnel - This main component of the stormwater storage system is usually constructed as a single diameter with a constant slope. In most situations, the stormwater tunnel is a single trunk line, but there are situations where a branching system is required.

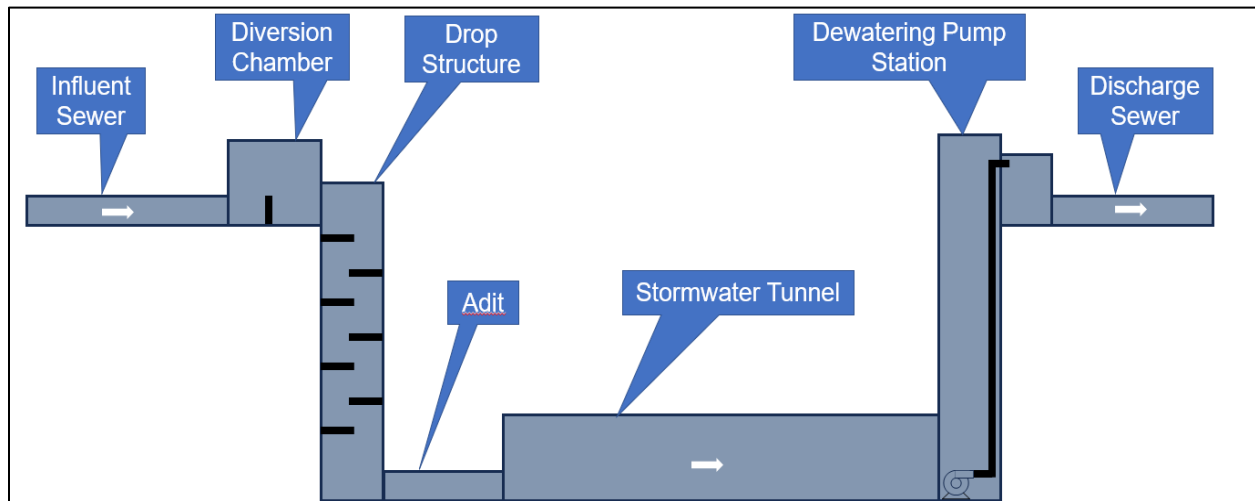
Drop Shafts - Due to the depth of stormwater tunnel systems, stormwater drops a significant distance. Drop shafts are designed to dissipate the hydraulic energy associated with these drops. The drop shaft can be designed to include provisions for stormwater tunnel maintenance access and stormwater tunnel venting. Locate drop shafts outside of travel lanes.

Adit - An interconnecting pipe called an adit conveys flow from the drop shaft to the stormwater tunnel. These are smaller than the stormwater tunnel diameter and may also be designed to reduce air entrainment in the stormwater tunnel.

Near Surface Diversion Chambers - If the stormwater tunnel is designed to store excess stormwater that an existing system cannot accommodate (peak shave), a separate chamber with a diversion weir may be required. A diversion chamber is a convenient location for stormwater tunnel isolation gates.

Dewatering Pump Station - In most situations, stormwater tunnels cannot dewater by gravity due to their depth relative to local existing sewer systems. To overcome this hydraulic head differential, a pump station is used to lift the flow out of the stormwater tunnel and discharge into the local system. If the stormwater tunnel is used for storage, the rated capacity of the pump station is significantly lower than the peak stormwater tunnel inflow rate.

Figure 7-17. Major Components of a Stormwater Tunnel System



7.8.2 Hydraulic Analysis of Stormwater Tunnel Systems

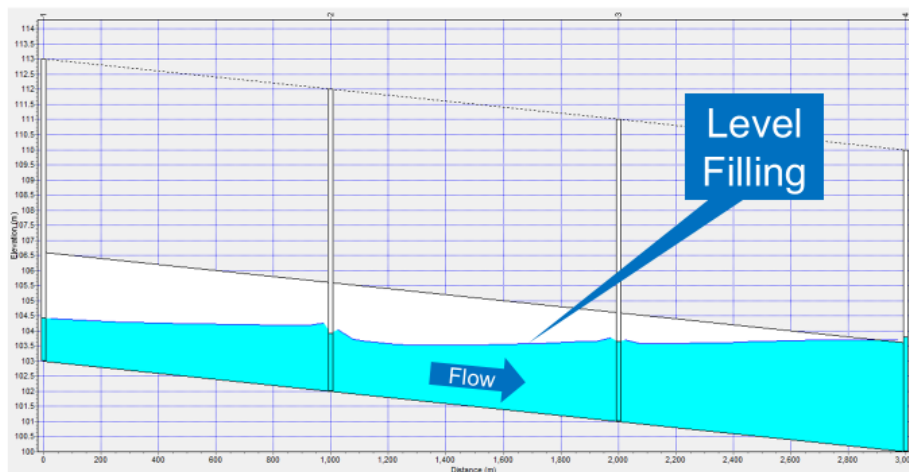
Stormwater tunnels are designed with discharge rates that are significantly lower than the inflow rate to the system. The limited outflow rates may be the result of the following conditions which are required to be analyzed to verify the safety of the stormwater tunnel design:

- There is a limited discharge rate due to a planned downstream restriction in the system such as a pump station or orifice outlet. This situation generally occurs in stormwater tunnel systems designed for storage.
- There is a limited discharge rate due to an unplanned reduction in outlet capacity. This situation occurs when a downstream boundary rapidly rises forcing the stormwater tunnel system hydraulic head to rise in-kind.
- A local capacity limitation occurs when flow at a point in the system exceeds the capacity of the stormwater tunnel. This situation can lead to localized trapped air pockets or the formation of hydraulic surge.
- Non-uniform filling of the stormwater tunnel can result in trapped air pockets between inlet/vent locations. These trapped air pockets can pressurize under stormwater tunnel surcharge conditions resulting in an air/water geyser when the trapped air pocket reaches a vent location.
- Coastal waves or wind induced seiche events can cause a rapid rise in the outlet boundary condition of a storm sewer. This condition can reduce outlet flow (or reverse flow) generating hydraulic surge that travels upstream through the system.

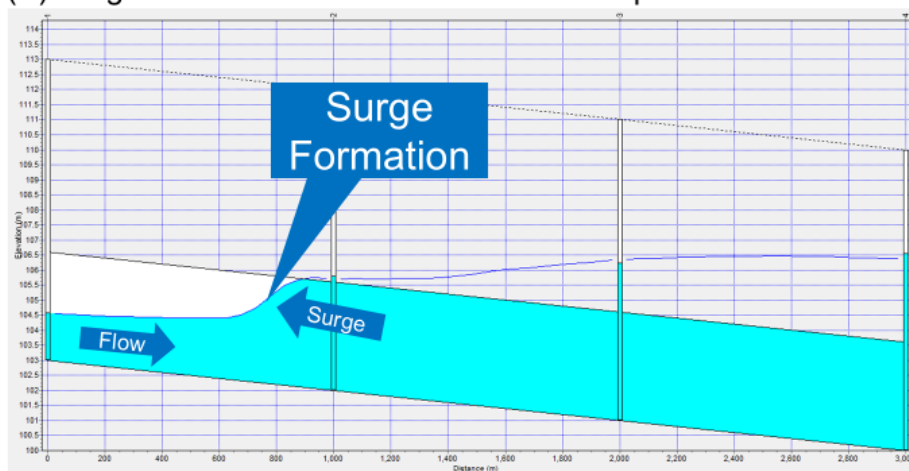
During initial stormwater tunnel filling, a mismatch of inflow to outflow is not a problem and the excess inflow is stored in the stormwater tunnel system. During the initial filling, the water level in the stormwater tunnel is relatively level. As the downstream end of the stormwater tunnel begins to pressurize, flow at the downstream end of the system travels upstream which can cause the formation of hydraulic surge. If the hydraulic surge forms and impacts the upstream end of the stormwater tunnel, the momentum of the flow may travel up the drop shaft resulting in a surface geyser. Trapped air pockets are common in a rapidly filling stormwater tunnel. The phases of a rapidly filling stormwater tunnel are shown in **Figure 7-18**.

Figure 7-18. Phases of Rapidly Filling Stormwater Tunnel

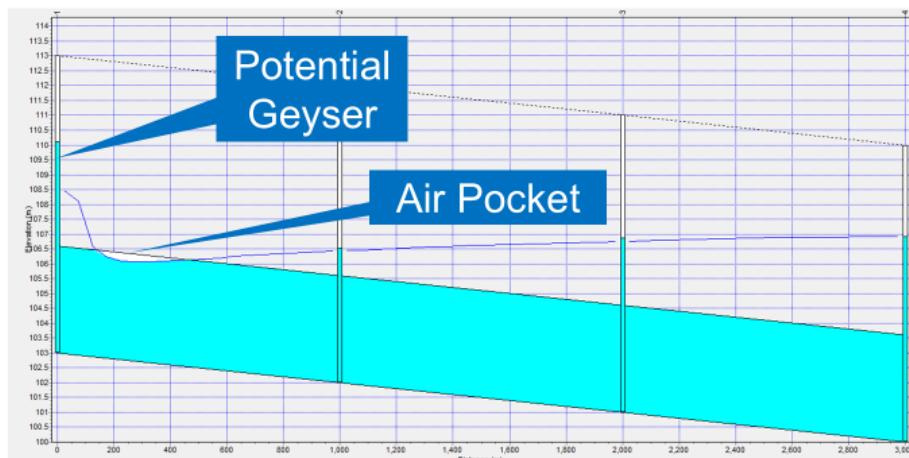
(A) Initial tunnel filling occurs with relatively level water surface



(B) Surge formation after downstream tunnel pressurizes



(C) Trapped air pocket and geyser formation



Due to the method of stormwater tunnel construction, inflow to a stormwater tunnel system is concentrated at discrete points in the system and are up to 1,200 feet apart. Longer spacing distances may be approved depending on inspection and maintenance requirements for a system. Longer spacing between inflow points results in high inflow rates at concentrated locations, limits the opportunities for access to the stormwater tunnel, and minimizes venting locations along the stormwater tunnel.

The hydraulic process within a stormwater tunnel system is complex and rapidly changing. Hydraulic surge and the formation of trapped air pockets are transient in nature and require the use of fully dynamic models that are specifically developed for surge analysis. The models are capable of tracking hydraulic surge fronts in the system between specified model nodes. Hydraulic surge analysis requires a model to track surge under open channel, pressurized, and transitional flow conditions. The mixed flow requirement prevents the use of available transient (water hammer) models for use in surge analysis. As of 2024, the Illinois Transient Model (ITM) is the only available non-proprietary model currently. ITM is available for download from the Florida International University website: [The Illinois Transient Model \(ITM\)](#). Other software can be used with approval from the Hydraulic Unit Supervisor for trunkline projects.

When analyzing a tunnel system, the following conditions are required to be verified using the model software:

- If the stormwater tunnel is used for storage, verify the stormwater tunnel storage volume is adequate for the design condition based on the results of the surge model. The storage volume must also be validated using the Modified Puls method (refer to Chapter 8). Typically, the stormwater tunnel should have approximately 5% excess storage volume to mitigate surge formation and to account for uncertainty in the hydrologic calculations.
- Verify the stormwater tunnel design provides adequate conveyance capacity to safely transport the inflows.
- Review model results to identify the formation of hydraulic surge in the system. Determine if any surge formation can result in trapped pressurized air pockets or the release of air water geysers to the ground surface.
- If hydraulic surge is identified for the design condition, develop surge mitigation for the stormwater tunnel. Surge mitigation may include restricted maximum inflow rates, active inflow control gates, relief overflows from the stormwater tunnel, or increased stormwater tunnel volume. Surge control measures can also be used to control the formation of trapped air pockets.
- Calculate stormwater tunnel air venting rate requirements based on volumetric displacement and stormwater tunnel surge requirements. Volumetric displacement is based on inflow rates during stormwater tunnel filling. Drop shafts become unavailable for venting as they become submerged. If hydraulic surge is present in the stormwater tunnel, rapid rise and fall of the water level in the drop shafts may be the dominant requirement for venting. Surge venting includes inflow or outflow of air from the drop shaft (up and down).
- Verify the stormwater tunnel slope meets the minimum scour velocities to convey sediment downstream and prevent sediment/debris accumulation. Stormwater

tunnels used for storage generate sediment deposits as the stormwater tunnel fills and velocity decreases to near zero. To offset these issues, the slope of the stormwater tunnel is set to provide adequate velocity to maintain suspension of sediment during initial filling and to resuspend sediment previously deposited. A circular cross-section is most effective in moving sediment efficiently.

Stormwater tunnels used for storage often have limited alternate surface storage or conveyance after they are completely full. Due to this condition, stormwater tunnels are designed to higher design storm conditions and analyzed for extreme storm conditions. Each application of a stormwater tunnel design is unique and may have special local considerations. Analyze the following list of additional storm event conditions:

7.8.2.1 Design Storms

These design conditions represent the minimum design for the system. Flow from these events must be fully contained within the stormwater tunnel system.

- Stormwater tunnel systems designed for conveyance must safely pass the design storm AEP for the roadway classification based on **Table 7-2**. The design must consider any downstream outlet limitations. For example, the receiving system downstream of the stormwater tunnel must be verified that it can accept discharge from the stormwater tunnel under free outlet conditions. Any backwater from the downstream system must be accounted for in the stormwater tunnel design.
- Stormwater tunnel systems designed for storage must be sized to store flow from the 1% AEP (100-year) event.

7.8.1.2 Extreme Event Analysis

This analysis applies to a stormwater tunnel system that is used for storage. The purpose of extreme event analysis is to understand the impacts to the system including the extent of flooding during an extreme condition and impacts to public safety. The extreme events are not intended to dictate design of the stormwater tunnel system, but rather provide guidance for managing and understanding the impact of extreme events on the system.

7.8.1.2.1 Resiliency Design Storm

NOAA Atlas 14 provides average design storm rainfall depths for each AEP and duration. The primary value represents a 90% confidence interval value that is recommended for design. The values shown in parentheses represent the upper and lower bounds of the 90% confidence interval. For resiliency simulations, use the upper bound of this range of values.

7.8.1.2.2 Large Historic Event

Large historic events that exceed the 1% AEP (100-year) event are required to be evaluated if rainfall data is available. Events from anywhere in the region can be used for this analysis.

7.8.1.2.3 Emergency Conditions

Emergency conditions can prevent dewatering of the stormwater tunnel during the design event. For example, a power failure of the dewatering pump station may occur. Other unique conditions for each stormwater tunnel system should be considered and evaluated. These conditions are required to be simulated using the design storm event.

7.8.3 Drop Shaft Design

Abrupt offset in the stormwater pipe invert results in significant energy loss that, if left unmanaged, can result in structural damage to the system due to impact forces or through erosion of the pipe and associated structures. Sudden drops entrain a significant volume of air that can bulk up the flow volume resulting in reduced conveyance capacity. The following discussion focuses on the design of drop shafts for stormwater tunnel systems that concentrate flow into discrete drop locations. This discussion can be applied to a collection system with hydraulic drops that exceed 15 feet. In these situations, special drop structures must be designed.

When designing drop shafts, consider the following features:

- Safely dissipate energy due to the vertical drop.
- Minimize the introduction of entrained air into the downstream collection system by minimizing turbulence or providing a stilling area to remove the entrained air.
- Provide required venting to remove locally entrained air and potentially manage air from the downstream collection system. Air from the downstream system is the result of volumetric displacement of air due to filling or hydraulic surge.
- Provide maintenance access to the downstream system. The size of the access needed depends on the maintenance activities expected at the individual drop shafts. These activities may include a simple maintenance staff access in a man-cage or insertion of maintenance equipment such as a bobcat or other sediment removal equipment.
- Provide security measures such as fencing, locks, or other measures at drop shafts and stormwater tunnel access points. Coordinate with the Maintenance Services Section and the Bureau of Safety and Security Administration.

Several drop shaft configurations have been studied and successfully used on projects around the country. These shaft configurations have been vetted through physical modeling, computational fluid dynamics modeling, and actual service in the field. Technical research publications provide guidance for standard sizing and configuration based on flow. Common drop shaft types that have been widely used successfully are

described as follows. Other less common drop shaft types may be better suited for specific applications.

- **Plunge** - This design is a simple flow drop. The drop shaft is sized large enough to allow for a free drop of the water while providing sufficient cross-sectional area for air venting.
- **Vortex** - This design accelerates the flow velocity tangentially to the drop shaft causing the flow to spiral down on the inside walls of the drop shaft. The spiral minimizes air entrainment and provides an air core for venting.
- **Baffle** - This design includes a series of alternating spaced shelves that allow the water to drop using multiple short drops. Each shelf includes an air vent to prevent choking of the flow on individual shelves due to air bulking of the flow.
- **Helicoidal** - This design is a fabricated spiral ramp down the entire length of the drop shaft.
- Other drop shaft designs are variations of these common types.

Selection of the drop shaft configuration for a specific system depends on a number of factors. The selection process balances the needs of hydraulic performance and constructability. If multiple drop shafts are constructed for a single project, a common design is beneficial to minimize the variation in construction methods. Consider the following factors when selecting a drop shaft configuration:

- Magnitude of peak design flow dictates the size of the drop shaft components. Balance the hydraulic design with constructability and maintenance requirements.
- Total depth of hydraulic drop will impact the selection. For example, the deeper the drop, the more baffle shelves or helicoidal rotations are required.
- Access requirements at each drop shaft vary. For example, some shafts do not require access while others require insertion of heavy equipment.
- Direction of flow approach precludes some drop shaft designs. An upstream junction chamber can realign flow approaching the drop shaft from multiple directions.
- Availability of ROW impacts the location of the drop shaft and its proximity to the deep stormwater tunnel. A drop shaft centered on the stormwater tunnel will have different design requirements than an off-line drop shaft connected with an adit.
- Geotechnical considerations have a big impact on the method of construction and the size of the excavation.
- Consider repurposing an excavation used for other construction activities for a drop shaft. For example, if a large excavation is required for pump station construction or for stormwater tunnel boring machine access, this oversized excavation can be available for the drop shaft.

The final analysis and design of the drop shaft is based on the following design guidance. If constructability requirements force a compromise of standard designs, additional analysis is required.

- Design can be based on proven drop shaft design. A literature search of specific standard drop shaft designs provides guidance for design based on peak flow rates. These standard designs are required to be based on proven designs that have been tested in the field or within a laboratory physical model environment. Any literature or design standards are required to be approved by the Hydraulic Unit Supervisor for trunkline projects.
- If conditions for a specific drop shaft location require deviation from standard drop shaft designs, a separate physical model study or computational fluid dynamics (CFD) model is required to be developed to verify the effectiveness of the proposed design. Approval from the Hydraulic Unit Supervisor is required for trunkline projects.

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