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Fourth Edition

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16. Abstract
This manual provides a comprehensive and practical guide for the design of storm drainage systems associated with transportation facilities. Information is provided for the design of storm drainage systems that collect, convey, and discharge stormwater flowing within and along the highway right-of-way.
Methods and procedures are given for the hydraulic design of storm drainage systems. Design methods are presented for evaluating rainfall and runoff magnitude, pavement drainage, gutter flow, inlet design, median and roadside ditch flow, structure design, and storm drain piping. Procedures for the design of detention facilities are also presented, along with an overview of stormwater pumping stations and urban water quality practices.

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Table of Contents

List of Figures

List of Tables

Acknowledgments

The cover image is a collage of photos starting from the top left and proceeding clockwise: detention pond, combination inlets, storm drain outlet, and urban water quality inlet. Source: Roger Kilgore.

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Notice

Non-Binding Contents

Quality Assurance Statement

Glossary

Access Hole:	A hole through which one can access an underground element for repairs or inspections.
Bench:	The elevated bottom of an access hole to help streamline flow through the structure.
Bypass Flow:	Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade.
Check Storm:	A lesser frequency event used to assess hazards at critical locations.
Check Valve:	Watertight valve used to prevent back flow.
Combination Inlet:	Use of both a curb opening inlet and a grate inlet.
Convolution:	The multiplication-translation-addition process used to route a rainfall-excess hyetograph using the unit hydrograph as the routing model.
Cover:	Distance from the outside top of a culvert or storm drain to the final grade of the ground surface.
Critical Depth:	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.
Critical Flow:	The open-channel flow condition where the specific energy of flow is at a minimum and the Froude number for the flow is one.
Cross Slope:	The rate of change of roadway elevation with respect to distance perpendicular to the direction of travel. Also known as transverse slope.
Crown:	The inside top elevation of a conduit. Also called the soffit or obvert.
Curb-opening Inlet:	A discontinuity in the curb structure which is covered by a top slab.
Detention Time:	The time required for a drop water to pass through a detention facility when the facility is filled to design capacity.
Direct Runoff:	The total runoff hydrograph minus base flow.
Drainage Inlet:	Receptor for surface water collected in ditches and gutters, which serves as the mechanism whereby surface water enters storm drains; refers to all types of inlets such as grate inlets, curb inlets, slotted inlets, etc.
Emergency Spillway:	Structure designed to allow controlled release of storm flows more than the design discharge from a detention facility.

Energy Grade Line (EGL):	The energy state at a channel or conduit section would be the sum of the pressure, velocity, and elevation heads. The energy grade line describes a conceptual link of the energy states between two (or more) channel or conduit locations. The differences in total energy between these two locations would be associated with energy losses. The slope of the energy grade line is often referred to as the friction slope.
Equivalent Cross Slope:	An imaginary straight cross slope having a conveyance capacity equal that of the given compound cross slope.
Extended Detention Dry Pond:	Depressed basin that temporarily stores a portion of the stormwater runoff following a storm event and releases stormwater over a longer period than detention ponds.
Flanking Inlet:	Inlet placed on either side of a low point inlet. Flanking inlets limit the spread of water onto the roadway if the low point inlet becomes clogged or is exceeded in its capacity.
Flap Gate:	A gate that restricts water from flowing back into the discharge pipe and discourages entry into the outfall line.
Flow Line:	The bottom elevation of an open channel or closed conduit.
Freeboard:	The vertical distance from the water surface at the design discharge to a pre-determined component of the roadway or channel.
Grate Inlet:	Parallel and/or transverse bars arranged to form an inlet structure.
Gutter:	Portion of the roadway structure used to intercept pavement runoff and carry it along the roadway shoulder.
Hydraulic Grade Line (HGL):	A line coinciding with the level of flowing water in an open channel. 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 energy grade line elevation minus the velocity head.
Hydraulic Jump:	Hydraulic phenomenon in open channel flow where supercritical flow changes to sub-critical flow. This results in an abrupt rise in the water surface elevation.
Hydraulic Radius:	The cross-sectional flow area of a channel or conduit divided by its wetted perimeter.
Hydrograph:	A plot of flow versus time.
Hydroplaning:	Separation of the vehicle tire from the roadway surface due to a film of water on the roadway surface.
Hyetograph:	A plot of the rainfall intensity (or depth) versus time.
Infiltration Basin:	An excavated area which impounds stormwater flow and gradually exfiltrates it through the basin floor.

Infiltration Trench:	Shallow excavation that has been backfilled with a coarse stone media. The trench forms an underground reservoir which collects runoff and exfiltrates it to the subsoil.
Inundation:	Presence of water in excess of water film (sheet flow) including ponding, overtopping, and concentrated flowing water.
Intensity-duration-frequency:	A graphical, tabular, or mathematical relation between the rainfall intensity, storm duration, and exceedance frequency.
Invert:	The inside bottom elevation of a culvert, storm drain, or other hydraulic structure.
Junction Boxes:	Formed control structures used to join sections of storm drains.
Longitudinal Slope (Grade):	The rate of change of elevation with respect to distance in the direction of travel or flow. The longitudinal grade may represent the slope of a roadway along the profile grade line, or along a designated offset, such as a gutter grade.
Major System:	This system provides overland relief for stormwater flows exceeding the capacity of the minor system and is composed of pathways that are provided, knowingly or unknowingly, for the runoff to flow to natural or constructed receiving channels such as streams, creeks, or rivers.
Mass Rainfall Curve:	The cumulative precipitation plotted over time.
Minor System:	Portion of the storm drainage system that is normally designed to carry runoff from the more frequent storm events. This includes curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc.
Open Channel Flow:	Flow exposed to atmospheric pressure.
Open Channel:	A natural or constructed structure that conveys water with the top surface in contact with the atmosphere.
Orifice Flow:	Flow of water through an opening driven by the water surface on the upstream end that is above the top of the opening.
Permissible Shear Stress:	The stress required to initiate movement of the channel bed or shear stress lining material.
Pressure Flow:	Flow not exposed to atmospheric pressure.
Profile Grade:	The trace of a vertical plane intersecting a particular surface of the road; usually along the longitudinal centerline of the roadway at the top of finished pavement. Profile grade means either elevation or gradient of such trace according to the context.
Routing:	The process of transposing an inflow hydrograph through a structure or channel and determining the outflow hydrograph from the structure or channel.

Sand Filters: Shallow excavations replaced with a sand bed to capture particles and water quality constituents.

Scupper: A small opening (usually vertical) in the bridge deck, curb, or barrier through which surface runoff water can flow.

Shallow Concentrated Flow: Flow that has concentrated in rills or small gullies.

Shear Stress: Stress or drag developed at the boundary of the waterbody by flowing water.

Sheet Flow: Shallow flow on the watershed surface that occurs prior to the flow concentrating into rills.

Slotted Inlet: A section of pipe cut along the longitudinal axis with transverse bars spaced to form slots.

Soffit: The inside top of a conduit. (See Crown.)

Specific Energy: The total energy head in open channel flow measured above the channel bed or conduit invert.

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

Steady Flow: Flow that remains constant with respect to time.

Storm Drain: A storm drainage system component that receives runoff from inlets and conveys the runoff to some point. Storm drains are closed conduits or open channels.

Storm Drainage System: System that collects, conveys, and discharges stormwater.

Subcritical Flow: Flow characterized by low velocities, large depths, mild slopes, and a Froude number less than 1.0.

Supercritical Flow: Flow characterized by high velocities, shallow depths, steep slopes, and a Froude number greater than 1.0.

Superelevation The transverse slope provided to reduce the tendency of a vehicle to overturn or to skid laterally outwards by raising the pavement outer edge with respect to inner edge.

Synthetic Rainfall Events: Artificially developed rainfall distribution events.

Time of Concentration: The time for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point.

Total Dynamic Head: The combination of static head, velocity head, and various head losses in the discharge system caused by friction, bends, obstructions, etc.

Tractive Force: Drag or shear on a streambed or bank caused by passing water which tends to move soil particles along with the streamflow.

Uniform Flow: Flow of constant cross section and velocity through a reach of channel or conduit at a given time. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.

Unit Hydrograph:	The direct runoff hydrograph produced by a storm of given duration such that the volume of excess rainfall and direct runoff is 1 inch (1 cm).
Unsteady Flow:	Flow of variable discharge and velocity through a cross-section with respect to time.
Varied Flow:	Flow where the flow rate and depth change along the length of the channel.
Water Quality Inlet:	Pre-cast storm drain inlet (oil and grit separator) that removes sediment, oil and grease, and large particulates from paved area runoff.
Weir Flow:	Flow of water exposed to the atmosphere over an obstruction.
Wet Ponds:	A pond designed to store a permanent pool during dry weather.
Wetted Perimeter:	The length of contact between the flowing water and the channel or conduit at a specific cross-section.

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Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACPA	American Concrete Pipe Association
AEP	Annual Exceedance Probability
AISI	American Iron and Steel Institute
AOP	Aquatic Organism Passage
BFE	Base Flood Elevation
BMP	Best Management Practice
CAD	Computer Aided Design
CFR	Code of Federal Regulations
CU	Customary Units (English)
CWA	Clean Water Act
DEM	Digital Elevation Model
DOT	Department of Transportation
EO	Executive Order from the Federal Register
FAHP	Federal-Aid Highway Program
FDC	Flow Duration Curve
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
GIS	Geographic Information System
H&H	Hydrology and Hydraulics
HDG	Highway Drainage Guidelines
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HWM	High Water Mark
LiDAR	Light Detection and Ranging
NASEM	National Academies of Sciences, Engineering, and Medicine
NCHRP	National Cooperative Highway Research Program
NED	National Elevation Database
NEH	National Engineering Handbook
NEPA	National Environmental Policy Act
NFIP	National Flood Insurance Program

NHI	National Highway Institute
NPDES	National Pollutant Discharge Elimination System
NRC	National Research Council
NRCS	Natural Resources Conservation Service
O&M	Operation and Maintenance
ROW	Right-of-Way
SCS	Soil Conservation Service (now NRCS)
SI	System International (Metric)
SWPPP	Stormwater Pollution Prevention Plan
TAM	Transportation Asset Management
TDH	Total dynamic head
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
USEPA	United States Environmental Protection Agency
USFS	United States Forest Service
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey

Symbols

a	Gutter depression
A	Drainage area
A	Cross-sectional area of flow
A	Minimum distance from back wall to trash rack
A_c	Contributing drainage area
A_q	Clear opening area of the grate
A_o, A_i	Outlet and inlet storm drain cross-sectional areas
A_o	Orifice area
A_w	Area of flow in depressed gutter section
A'_w	Area of flow in a specified width of the depressed gutter
b	Access hole or junction chamber diameter
b	Width of spillway
B	Maximum distance between a pump and the back wall
B	Bottom width of channel
B	Cross-sectional area of flow of basin
C_o	Orifice coefficient
C_W	Weir coefficient
CN	Curve number
d	Trench depth
d_i	Depth at lip of curb opening
d_o	Effective head on the center of the orifice throat
D	Pump, orifice, or storm drain diameter
D_{HW}	Design high water elevation
D_i	Inflowing pipe diameter
D_o	Outlet pipe diameter
E	Efficiency of an inlet
E_a	Access hole energy level
E_i	Energy head for access hole outlet pipe
E_{ai}	Initial access hole energy level
E_o	Ratio of flow in a depressed gutter section to total gutter flow
E'_o	Ratio of flow in a portion of a depressed gutter section to total gutter flow
E_t	Total energy
EGL_a	Access hole energy grade line elevation
EGL_i	Energy grade line elevation at upstream end of a pipe run
EGL_o	Energy grade line elevation at downstream end of a pipe run
ΔE	Total energy lost
Fr	Froude number
g	Gravitational acceleration
G_i	Grade of roadway

h	Height of curb-opening inlet
h	Vertical distance of plunging flow from the flow line of the higher elevation inlet pipe to the center of the outflow pipe
h_L	Head or energy loss
h_o, h_i	Outlet and inlet velocity heads
H	Wetted pipe length
H	Head above weir crest excluding velocity head
H_{ah}	Head loss at access holes or inlet structures (approximate method)
H_b	Bend loss
H_c	Height of weir crest above channel bottom
H_C	Contraction loss
H_e	Expansion loss
H_f	Friction loss
H_i	Junction loss
H_l	Losses through fittings, valves, etc.
H_o	Head measured from centroid of orifice to water surface elevation
H_s	Maximum static head
H_t	Storage depth
HGL	Hydraulic grade line elevation
I_a	Initial abstraction
k	Intercept coefficient
K	Vertical curve constant
K	Conveyance
K_b	Bend shear stress parameter
K_c	Storm drain contraction coefficient
K_e	Expansion coefficient
K_o	Initial head loss coefficient based on relative access hole size
L	Horizontal length of curve, flow length, length of basin at base length of pipe, weir length, or length of wet well
L_o	Length of increased shear stress due to the bend
L_T	Curb-opening length required to intercept 100 percent of the gutter flow
n	Manning's roughness coefficient
n_b	Manning's roughness coefficient in the channel bend
P	Depth of precipitation
P	Perimeter of the grate disregarding the side against the curb
P	Wetted perimeter
q_p	Peak flow
Q	Flow
Q_b	Bypass flow
Q_D	Depth of direct runoff
Q_i	Inflow, peak inflow rate, or inlet interception flow capacity
Q_{ic}	Interception capacity of curb

Q_{ig}	Interception capacity of grate
Q_l	Lateral flow
Q_o	Outflow
Q_s	Submerged flow
Q_s	Flow rate in the gutter section above the depressed section
Q'_s	Flow rate on one side of a composite V-ditch beyond the depressed section
Q_w	Flow rate in the depressed section of the gutter
Q'_w	One-half flow rate in depressed section of a composite V-ditch
r	Ratio of width to length of basin at the base
r	Pipe radius
R	Hydraulic radius
R_c	Radius to centerline of open channel
R_f	Ratio of frontal flow intercepted to total frontal flow
R_s	Ratio of side flow intercepted to total side flow
R_v	Runoff coefficient
S	Surface slope
S_c	Critical slope
S_e	Equivalent cross slope
S_f	Friction slope
S_L	Longitudinal slope
S_L	Main channel slope
S_o	Energy grade line slope
S'_w	Cross slope of gutter measured from cross slope of pavement
S_w	Cross slope of depressed gutter
S_x	Cross slope
t	Travel time
t_b	Time base of the unit hydrograph
t_c	Time of concentration
t_i	Duration of basin inflow
t_L	Lag time
t_p	Time to peak of the hydrograph
T	Width of flow (spread)
T'	Hypothetical spread that is correct if contained within S_{x1} and S_{x2}
T'	One-half of the total spread in a composite V-ditch
T_R	Duration of unit excess rainfall (Snyder unit hydrograph method)
T_s	Width of spread from junction of depressed gutter section and normal gutter section to limit of spread in both a standard gutter section and a composite V-ditch
T_s	Detention basin storage time
T_w	Width of circular gutter section
V	Velocity
V	Storage volume

V_c	Critical velocity
V_d	Channel velocity downstream of outlet
V_o	Gutter velocity where splash-over first occurs
V_o	Average storm drain outlet velocity
V_l	Lateral velocity
V_r	Inflow volume of runoff
V_s	Storage volume estimate
V_1	Velocity upstream of transition
V_2	Velocity downstream of transition
W	Width of gutter or width of basin at base
w	Trench width
y	Flow depth
y_c	Critical depth of flow in conduit
y_n	Normal depth of flow in conduit
Z	Elevation above a given datum
z	Horizontal distance for side slope of trapezoidal channel
α	Angle
Δ	Angle of curvature
Δd	Water surface elevation difference in a channel bend
ΔS	Change in storage
Δt	Time interval
γ	Unit weight of water
τ	Average shear stress
τ_b	Bend shear stress
τ_d	Maximum shear stress
τ_p	Permissible shear stress
θ	Angle between the inflow and outflow pipes
θ	Angle of v-notch

Chapter 1 - Introduction

This Fourth Edition of the Urban Drainage Design Manual provides technical information for understanding, assessing, and addressing drainage design for transportation infrastructure. This chapter describes the purpose and scope, organization, target audience, and units used in the manual.

1.1 Purpose and Scope

This manual provides a comprehensive and practical guide for the design of storm drainage systems associated with transportation facilities. The focus is to support the design of storm drainage systems that collect, convey, and discharge stormwater flowing within and along the highway right-of-way. The manual covers the design of most types of drainage systems associated with highways except for 1) cross drainage facilities such as culverts and bridges and 2) subsurface facilities. For culverts and bridges, the Federal Highway Administration (FHWA) has prepared two reference manuals in their Hydraulic Design Series (HDS): *Hydraulic Design of Highway Culverts* (HDS-5) (FHWA 2012a) and *Hydraulic Design of Safe Bridges* (HDS-7) (FHWA 2012b). The FHWA addresses subdrainage design in *Geotechnical Aspects of Pavements* (FHWA 2006b).

This manual provides methods and procedures for the hydrologic and hydraulic design of storm drainage systems. Design methods include evaluating rainfall and runoff magnitude, as well as the design of pavement drainage, gutter flow, inlet design, median and roadside ditch flow, drainage structure design, and storm drain piping. Methods also include procedures for the design of detention facilities, review of urban water quality practices, and review of stormwater pump stations.

A goal of this manual is to support planning, implementation, and stewardship of sustainable, resilient, and reliable transportation networks. The FHWA describes sustainability as considering three primary values or principles: social, environmental, and economic (FHWA 2022c). The goal of sustainability is the satisfaction of basic social and economic needs, both present and future, and the responsible use of natural resources, all while maintaining or improving the well-being of the environment on which life depends.

Commonly, society views sustainability through a lens of balancing the needs of the environment with the economic needs of roadway and bridge development. Balancing the environment with social values results in what is bearable by both society and the environment while balancing the social and economic results in what is equitable. Sustainability results when all three values (social, environmental, and economic) are satisfied and in balance. For FHWA, a sustainable highway project satisfies basic social and economic needs, makes responsible use of natural resources, and maintains or improves the well-being of the environment.

Considering transportation equity for underserved populations is important element of a sustainable approach to highways. Transportation equity is relevant under all three primary values of sustainability. Past Federal transportation investments have too often failed to consider transportation equity for all community members, including traditionally underserved and underrepresented populations (USDOT 2022). “Underserved populations” include minority and low-income populations but may also include many other demographic categories that face challenges engaging with the transportation process and receiving equitable benefits (See FHWA 2015). The U.S. Department of Transportation (USDOT or Department) has committed to pursuing a comprehensive approach to advancing equity for all (USDOT 2022.; and Executive Order 13985, 86 FR 7009 (2021)). Equity in transportation seeks the consistent and systematic fair, just, and impartial treatment of all individuals, including individuals who belong to traditionally underserved communities or populations (USDOT 2022).

Resilient and reliable design of storm drainage systems for transportation facilities is also essential in addressing the significant and growing risk presented by climate change. (USDOT 2021). In the transportation context, this risk is many-faceted, including risks to the safety, effectiveness, equity, and sustainability of the Nation's transportation infrastructure and the communities it serves. The USDOT recognizes that the United States has a "once-in-a-generation" opportunity to address this risk, which is increasing over time (USDOT 2021; see also Executive Order 14008 on Tackling the Climate Crisis at Home and Abroad, 86 FR 7619 (2021)). Addressing the risk of climate change is also closely interlinked with advancing transportation equity because of the disproportionate impacts of climate change on vulnerable populations, including older adults, children, low-income communities, and communities of color. The USDOT intends to lead the way in addressing the climate crisis.

The FHWA believes that this manual will be useful for aligning and integrating these concepts and principles of sustainability within the context of the design of storm drainage systems associated with transportation facilities.

1.2 Organization

This manual consists of 12 chapters, a glossary, list of acronyms, reference section, and appendices. This chapter, **Chapter 1**, provides discussion of the purpose, background, organization, and units.

Chapter 2 provides an overview of Federal policy as it relates to urban stormwater drainage analysis and design. This context guides work in stormwater management through a series of statutes and regulations.

Chapter 3 describes high-level concepts of system planning and outlines considerations for successful design. These include data requirements, agency coordination, concept development, and design.

Chapter 4 outlines hydrologic procedures for estimating rainfall and flow amounts that will drive the type, size, and configuration of the drainage system. The chapter includes selection of a design storm which establishes the overall capability of the drainage system to manage runoff.

Chapters 5, 6, and 7 describe detailed methods and information for designing the surface collection components of a comprehensive stormwater drainage system—roadway pavement drainage, roadside and median channels, and inlets, respectively. Each of these captures water from the land surface to preserve safe roadway facilities.

Chapters 8 and 9 address the subsurface system of storm drain structures and conduits, respectively. These components convey the collected stormwater to an offsite or discharge location.

Chapter 10 describes the design of detention and retention facilities when needed to manage stormwater quantity. These facilities can attenuate flood peaks and redistribute flood volumes.

Chapter 11 outlines the selection and design of stormwater quality best management practices (BMPs). These tools assist designers in improving the water quality of discharges offsite or to receiving waters to mitigate potential negative water quality impacts.

Chapter 12 addresses pump stations. Stormwater drainage systems primarily rely on gravity to convey stormwater away from roadways safely. When this is not an option, stormwater pump stations provide an alternative.

1.3 Target Audience

The target audience of this manual includes a wide cross-section of users with Federal, State, and local highway agencies, and consultants with roadway and drainage design responsibilities. While an understanding of basic hydrologic and hydraulic principles will be helpful, readers with varying backgrounds and expertise, including those with limited knowledge in urban drainage design, will find the manual useful.

Those with an interest in addressing the growing risk presented by climate change to transportation infrastructure may also find this manual helpful. It provides information they may find valuable as they explore ways to implement climate and resilience strategies in the design of storm drainage systems for transportation facilities.

The FHWA provides additional reference information on hydrologic topics in *Highway Hydrology* (HDS-2) (FHWA 2002). Other supporting resources include *Design and Construction of Urban Stormwater Management Systems* (ASCE 1992) and numerous basic hydrology and hydraulic textbooks.

This manual does not have the force and effect of law and it is not meant to bind the public in any way. The FHWA intends any descriptions of processes and approaches to provide illustrative insights into the underlying scientific and engineering concepts and practices rather than any proscribed guidance or requirements.

1.4 Units in this Manual

This manual uses English customary units (CU). However, in limited situations both CU and SI (metric) units are used or only SI units are used because these are the predominant measure used nationwide and globally for such topics. In these situations, the manual provides the rationale for the use of units. Appendix A summarizes information on units and unit conversions.

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Chapter 2 - Federal Policy for Urban Storm Drainage

Federal policy related to urban storm drainage sets the context for planning, design, construction, operations, and maintenance of roadways and their associated stormwater drainage infrastructure. This chapter provides background on applicable FHWA specific statutes and regulations and provides an overview of other Federal statutes and regulations that may affect roadway projects and urban storm drainage.

Context for Roadways and Urban Storm Drainage

Federal policy—in the form of statutes and regulations—establishes the "guard rails" and "signage" for the development of transportation infrastructure that serves to facilitate the movement of both people and goods. Taken together, these statutes and regulations, administered by multiple Federal agencies, reflect national values for economic well-being and environmental stewardship. This manual provides information on methods and tools to realize these values in the planning, development, construction, and operations and maintenance of urban storm drainage that supports the nation's transportation infrastructure.

2.1 Federal Highways and Urban Drainage: National Overview

The FHWA has the primary responsibility for Federal policy on highways. Legislation for the Federal road system dates back over a century. The Federal-Aid Road Act of 1916 created the Federal-Aid Highway Program that funded State highway agencies so they could make road improvements "to get the farmers out of the mud." This 1916 Act charged the Bureau of Public Roads with implementing the program. The growth of the Federal highway system, including the addition of the Interstate Highway System and concerns about how all these highways affected the environment, city development, and the ability to provide public mass transit, led to the 1966 establishment of the U.S. Department of Transportation (USDOT). The same enabling legislation renamed the Bureau of Public Roads to the FHWA. Currently, the FHWA continues to administer U.S. Federal policy on highways, but also coordinates extensively with other Federal agencies on environmental policies and permits, floodplains, and other compliance issues related to highway program and project delivery.

Other agencies influence urban storm drainage policy. At the Federal level, the Federal Emergency Management Agency (FEMA) oversees the National Floodplain Insurance Program (NFIP). The U.S. Fish and Wildlife Service (USFWS) and the National Oceanic and Atmospheric Administration (NOAA) National Marine Fisheries Service (NMFS) administer and enforce the Endangered Species Act (ESA). Almost every project involving work or activities in rivers is subject to the Clean Water Act (CWA) of 1972, which the U.S. Environmental Protection Agency (USEPA) administers in coordination with State governments.

2.2 FHWA Statutes and Regulations

The FHWA provides financial and technical assistance to State and local governments to ensure that the Nation's roads and highways continue to be among the safest and most technologically sound in the world. The FHWA authority for the subject matter of this manual includes the following statutes and regulations. The section below provides a synopsis of these various authorities as well as pertinent Congressional findings and statements, policy, and guidance.

2.2.1 FHWA Statutes

The FHWA operates under the statutory authority of Title 23 (Highways) of the United States Code (U.S.C.). For the purposes of this manual, relevant sections include:

- **Standards [23 U.S.C. § 109].** It is the intent of Congress that Federally funded projects to resurface, restore, and rehabilitate highways shall "be constructed in accordance with standards to preserve and extend the service life of highways and enhance highway safety." [23 U.S.C. § 109(n)]. Designs for new, reconstructed, resurfaced, restored, or rehabilitated highways on the National Highway System must consider, among other criteria, the "constructed and natural environment of the area." [Id. at (c)(1)(a)].
- **Maintenance [23 U.S.C. § 116].** Preventive maintenance is eligible for Federal assistance under Title 23 if a State Department of Transportation (DOT) can demonstrate that it is a "cost-effective means of extending the useful life of a Federal-aid highway." [23 U.S.C. § 116(e).]
- **National highway performance program [NHPP] [23 U.S.C. § 119].** The NHPP allows the FHWA to provide Federal-aid funds for "[c]onstruction, replacement …, rehabilitation, preservation, and protection (including … protection against extreme events) of bridges on the National Highway System." [23 U.S.C. § 119(d)(2)(B)]. The NHPP also allows Federal-aid funds for "[c]onstruction, replacement …, rehabilitation, preservation, and protection (including … protection against extreme events) of tunnels on the National Highway System." [Id. at (d)(2)(C)].
- **Surface transportation block grant [STBG] program [23 U.S.C. § 133].** The STBG program allows the FHWA to provide Federal-aid funds for protection of "bridges (including approaches to bridges and other elevated structures) and tunnels on public roads" including "painting, scour countermeasures, seismic retrofits, impact protection measures, security countermeasures, and protection against extreme events." [23 U.S.C. § 133(b)(10)]. The STBG program also allows Federal-aid funds for "inspection and evaluation of bridges and tunnels and other highway assets." [Id.]
- **Metropolitan transportation planning [23 U.S.C. § 134].** In the context of metropolitan transportation planning, Congress has found that it "is in the national interest … to encourage and promote the safe and efficient management, operation, and development of surface transportation systems … within and between States and urbanized areas" including taking "resiliency needs" into consideration. [23 U.S.C. § 134(a)(1)].
- **National bridge and tunnel inventory and inspection standards [23 U.S.C. § 144]**. Congress has found that "continued improvement to bridge conditions is essential to protect the safety of the traveling public." [23 U.S.C. § 144(a)(1)(A)]. Congress has further found that "the systematic preventative maintenance of bridges, and replacement and rehabilitation of deficient bridges, should be undertaken." [Id. at (a)(1)(B)]. In addition, Congress has also declared that "it is in the vital national interest" to use a "data-driven, risk-based approach" toward meeting these ends." [Id. At (a)(2)(B)]. Considering these findings and declarations, Section 144 requires the FHWA to maintain an inventory of bridges and tunnels on public roads both "on and off Federal-aid highways." [Id. at (b)]. The FHWA is also required to "establish and maintain inspection standards for the proper inspection and evaluation of all highway bridges and tunnels for safety and serviceability." [Id. at (h)(1)(A).] Section 144 also provides an exception to the requirement to obtain a bridge permit from the U.S. Coast Guard for certain bridges over a limited subset of navigable waters. [Id. at (c)(2)].

- **National goals and performance management measures [23 U.S.C. § 150].** Congress has declared that it is "in the interest" of the United States to focus the Federal-aid highway program on certain national transportation goals including Infrastructure Condition, or the objective to "maintain … highway infrastructure in a state of good repair;" and System Reliability, or the objective to "improve the efficiency of the surface transportation system." [23 U.S.C. § 150(b)].
- **PROTECT Program [23 U.S.C. § 176].** The Promoting Resilient Operations for Transformative, Efficient, and Cost-Saving Transportation (PROTECT) program allows the FHWA to provide grants for resilience improvements through: (i) formula funding distributed to States; (ii) competitive planning grants; and (iii) competitive resilience improvement grants. [23 U.S.C. § 176(b)]. Eligible activities under the PROTECT program include, among others, "resurfacing, restoration, rehabilitation, reconstruction, replacement, improvement, or realignment of" certain existing surface transportation facilities and "the incorporation of natural infrastructure." [23 U.S.C. §§ 176(c)(1) and 176(d)(4)(A)(ii)(II)].
- **Bridge Replacement, Rehabilitation, Preservation, Protection, and Construction Program (or Bridge Formula Program) (Division J, title VIII, Highway Infrastructure Program heading, paragraph (1)).** The Bridge Formula Program provides funding to help repair approximately 15,000 highway bridges. In addition to providing funds to states to replace, rehabilitate, preserve, protect, and construct highway bridges, the Bridge Formula Program has dedicated funding for Tribal transportation facility bridges as well as "off-system" bridges, which are generally locally-owned facilities not on the federal-aid highway system.
- **Bridge Investment Program (23 U.S.C. § 124).** The Bridge Investment Program provides financial assistance for eligible projects with program goals to improve the safety, efficiency, and reliability of the movement of people and freight over bridges; improve the condition of bridges; and provide financial assistance that leverages and encourages non-Federal contributions from sponsors and stakeholders involved in the planning, design, and construction of eligible projects.
- **National Culvert Removal, Replacement, and Restoration Grants Program (49 U.S.C. §§ 6703)].** The National Culvert Removal, Replacement, and Restoration Grant program established an annual competitive grant program to award grants to eligible entities for projects for the replacement, removal, and repair of culverts or weirs that would meaningfully improve or restore fish passage for anadromous fish.
- **Research and technology development and deployment [23 U.S.C. § 503].** In carrying out certain highway and bridge infrastructure and research and development activities, the FHWA must "study vulnerabilities of the transportation system to … extreme events and methods to reduce those vulnerabilities." [23 U.S.C. § 503(b)(3)(B)(viii)].
- **Statutory Definition of "Resilience." [23 U.S.C. § 101(a)(24)].** Section 11103 of the Bipartisan Infrastructure Law (BIL), enacted as the Infrastructure Investment and Jobs Act, Pub. L. 117-58 (Nov. 15, 2021), added a definition of "resilience," which applies throughout Title 23 of the U.S. Code. With respect to a project, "resilience" means a project with the ability to anticipate, prepare for, and or adapt to changing conditions and or withstand, respond to, and or recover rapidly from disruptions, including the ability: (A) to resist hazards or withstand impacts from weather events and natural disasters, or reduce the magnitude or duration of impacts of a disruptive weather event or natural disaster on a project; and (B) to have the absorptive capacity, adaptive capacity, and recoverability to decrease project vulnerability to weather events or other natural disasters. 23 U.S.C. § 101(a)(24). *See also* FHWA Order 5520 (FHWA 2014b).

2.2.2 FHWA Regulations

The FHWA's regulations are found within the Code of Federal Regulations (CFR), Title 23, Highways (23 CFR). The FHWA requires compliance with Federal law and the regulations in Chapter I, Subchapter A, Part 1 of 23 CFR for a project to be eligible for Federal-aid or other FHWA participation or assistance. [23 CFR 1.36]. The following FHWA regulations apply to highway projects and actions interacting with and within rivers and floodplains (paraphrased for brevity):

Scope of the statewide and nonmetropolitan transportation planning process [23 CFR 450.206]. State DOTs must "carry out a continuing, cooperative, and comprehensive statewide transportation planning process that provides for consideration and implementation of projects, strategies, and services that will … improve the resiliency and reliability of the transportation system..." [23 CFR 450.206(a)].

Asset Management Plans [23 CFR Part 515]. Part 515 establishes processes that a State DOT must use to develop an asset management plan. Two notable provisions include:

- **Section 515.7(b).** "A State DOT shall establish a process for conducting life-cycle planning for an asset class or asset sub-group at the network level (network to be defined by the State DOT). As a State DOT develops its life-cycle planning process, the State DOT should include future changes in demand; information on current and future environmental conditions including extreme weather events, climate change, and seismic activity; and other factors that could impact whole of life costs of assets."
- **Section 515.7(c).** A State DOT shall establish a process for developing a risk management plan. This process shall, at a minimum, produce information including: Identification of risks that can affect condition of NHS pavements and bridges and the performance of the NHS, including risks associated with current and future environmental conditions, such as extreme weather events, climate change, seismic activity, and risks related to recurring damage and costs as identified through the evaluation of facilities repeated damaged by emergency events carried out under Part 667 of title 23 of the CFR. Additional information that must be produced is specified in the regulation at 23 CFR 515.7(c).
- In addition, BIL Section 11105 amended 23 U.S.C. Section 119(e)(4) to require State DOTs to consider extreme weather and resilience as part of the life-cycle planning and risk management analyses within a TAMP (FHWA 2022d).

Design Standards [23 CFR Part 625]. Part 625 describes structural and geometric design standards.

- **Sections 625.3(a)(1), 625.3(b), and 625.4(b)(3).** The FHWA, in cooperation with SDOTs, has approved the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications. Based on the FHWA's approval, certain National Highway System (NHS) projects must follow those LRFD Specifications, including sections related to hydrology, hydraulics, and bridge scour. Among other standards, policies, and specifications listed in 23 CFR 625.4, FHWA has also approved the AASHTO Policy on Geometric Design of Highways and Streets (AASHTO 2018).
- **Section 625.3(a)(2).** Non-NHS projects must follow State DOT standard(s) and specifications on drainage, bridges, and other topics.

Location and Hydraulic Design of Encroachments on Flood Plains [23 CFR Part 650, Subpart A]. One of the FHWA's important river-related regulations, 23 CFR Part 650, Subpart A sets forth policies and procedures for location and hydraulic design of highway encroachments in base (1-percent chance) floodplains. Section 650.111 sets forth requirements for location hydraulic studies to identify the potential impact of the highway alternatives on the base floodplain; these studies are commonly used during the National Environmental Policy Act (NEPA) process. The regulations prohibit significant encroachments on base floodplains unless the FHWA determines that such encroachment is the only practicable alternative. [23 CFR 650.113(a)]. This finding must be included in the NEPA documents for a project and supported information including the reasons for the finding and considered alternatives. [Id.]. The procedures also provide minimum standards for Interstate Highways, set freeboard requirements to account for debris and scour, and require highway encroachments to be consistent with certain established design flood standards for hydraulic structures, including standards from FEMA and State and local governments related to administration of the National Flood Insurance Program (NFIP). [23 CFR 650.115(a)]. Notably, the policies and procedures in this Subpart apply to encroachments in all base floodplains, not just the floodplains regulated by the Federal Emergency Management Agency (FEMA) in the NFIP. [23 CFR 650.107]. Additionally, the Subpart incorporates a requirement for project-by-project risk assessments or analyses. [23 CFR 650.115(a)(1)]. Notable sections include:

- **Section 650.103 [Policy].** This section states that "it is the policy of the FHWA: (a) To encourage a broad and unified effort to prevent uneconomic, hazardous or incompatible use and development of the Nation's flood plains, (b) To avoid longitudinal encroachments, where practicable, (c) To avoid significant encroachments, where practicable, (d) To minimize impacts of highway agency actions which adversely affect base flood plains, (e) To restore and preserve the natural and beneficial flood-plain values that are adversely impacted by highway agency actions, (f) To avoid support of incompatible flood-plain development, (g) To be consistent with the intent of the Standards and Criteria of the National Flood Insurance Program, where appropriate, and (h) To incorporate "A Unified National Program for Floodplain Management" of the Water Resources Council into FHWA procedures." [23 CFR 650.103].
- **Section 650.115 [Hydraulic Design Standards].** This regulation applies to all Federal-aid projects, whether on the NHS or Non-NHS. Federal, State, local, and AASHTO standards may not change or override the design standards set forth under § 650.115 — although certain State and local standards must also be satisfied under the same section. The section also requires development of a "Design Study" for each highway project involving an encroachment on a floodplain. [23 CFR 650.115(a)].
- **Section 650.117 [Content of Design Studies].** This regulation requires studies to contain the "hydrologic and hydraulic data and design computations." [23 CFR 650.117(b)]. As both hydrologic and hydraulic factors and characteristics lead to scour formation, data and computations applicable to scour should be provided as well. Project plans must show the water surface elevations of the overtopping flood and base flood (i.e., 100-year flood) if larger than the overtopping flood. [23 CFR 650.117(c)].

Executive Order 14030, Climate-Related Financial Risk, and Executive Order 13690, Establishing a Federal Flood Risk Management Standard and a Process for Further Soliciting and Considering Stakeholder Input (80 FR 6425). Project applicants should be aware that DOT and FHWA, as of 2022, are in the process of developing guidance and considering updates to floodplain requirements, including redefining the appropriate flood hazard area to account for future climate conditions.

National Bridge Inspection Standards [23 CFR 650 Subpart C]. This regulation implements requirements of 23 U.S.C. § 144. In addition to the inspection and inventory requirements, the regulation specifically focuses on scour at bridges.

Mitigation of Impacts to Wetlands and Natural Habitat [23 CFR Part 777]. This regulation provides policy and procedures for the evaluation and mitigation of adverse environmental impacts to wetlands and natural habitat resulting from Federal-aid funded projects.

2.3 Other Federal Agency Statutes and Regulations

Civil engineering projects are subject to numerous Federal laws, policies, and regulations. This section describes some of the common Federal statutes, regulations, and other authoritative guidance that may apply highway projects.

2.3.1 Rivers and Harbors Act of 1899 [33 U.S.C. § 401 and § 403]

River and coastal highway engineering projects are subject to Section 9 [33 U.S.C. § 401] and Section 10 [33 U.S.C. § 403] of the Rivers and Harbors Act of 1899. Section 9 of this Act restricts the construction of any bridge, dam, dike, or causeway over or in U.S. navigable waterways. Except for bridges and causeways under Section 9 [33 U.S.C. § 401], the U.S. Army Corps of Engineers (USACE) is responsible for maintaining the standards set by and for issuing permits under the Rivers and Harbors Act. Authority to administer Section 9, applying to bridges and causeways, was redelegated to the U.S. Coast Guard under the provisions of the Department of Transportation Act of 1966 (as discussed below).

2.3.2 General Bridge Act of 1946 [33 U.S.C. §§ 525-533]

The General Bridge Act of 1946 requires the location and plans of bridges and causeways across the navigable waters of the United States be submitted to and approved by the U.S. Coast Guard prior to construction. [33 U.S.C. § 525]. The USACE may also impose conditions relating to maintenance and operation of the structure. [Id.]. The General Bridge Act of 1946 is cited as the legislative authority for bridge construction in most cases. Although the General Bridge Act of 1946 originally provided authority for issuing bridge permits to the USACE, subsequent legislation transferred these responsibilities from the USACE to the U.S. Coast Guard.

2.3.3 Transportation Act of 1966 [Public Law 89-670]

The Transportation Act of 1966 transferred the U.S. Coast Guard (USCG) to USDOT. One of USCG's newly assigned duties was to issue bridge permits. This, along with the Rivers and Harbors Act and General Bridge Act, made the USCG responsible for ensuring that bridges and other waterway obstructions do not interfere with the navigability of waters of the United States without express permission of the United States Government. Subsequent legislation amended 23 U.S.C. § 144 to provide certain exceptions to USCG's authority under 33 U.S.C. § 401 and 33 U.S.C. § 525 for bridges constructed, reconstructed, rehabilitated, or replaced using Federal-aid funds. [23 U.S.C. § 144(c)(2)].

2.3.4 National Environmental Policy Act [42 U.S.C. § 4321 et seq.]

The National Environmental Policy Act of 1969 (NEPA) establishes the continuing policy of the Federal government to use all practicable means and measures "to foster and promote the general welfare, … create and maintain conditions under which man and nature can exist in productive harmony, and fulfill the social, economic, and other requirements of present and future generations of Americans." [42 U.S.C. § 4331]. To achieve this goal, NEPA creates a requirement for Federal agencies to consider the environmental impacts of their actions before undertaking them. [42 U.S.C. § 4332(C)].

Section 102(2)(C) of NEPA requires that Federal agencies develop a detailed statement on proposals for major Federal actions significantly affecting the quality of the human environment. [42 U.S.C. § 4332(C)]. Environmental impact statements address items including "the environmental impact of" and "alternatives to" the proposed action." [Id.] FHWA implements NEPA according to the Council on Environmental Quality (CEQ) NEPA regulations at 40 CFR Part 1500 et seq. and the FHWA-FRA-FTA joint regulations at 23 CFR Part 771.

2.3.5 Clean Water Act [33 U.S.C. §§ 1251-1387]

Almost every project involving work or activities in rivers is subject to the Clean Water Act (CWA) of 1972, which is administered by the U.S. Environmental Protection Agency (USEPA) in coordination with State governments. The CWA is the primary Federal statute governing protection of the Nation's surface waters. Engineering of highways in the river environment is often subject to Section 404 of the CWA, which regulates the discharge of dredged or fill material in waters of the United States, including wetlands. [33 U.S.C. § 1344]. This includes the use of dredged or fill material for development, water resource projects, and infrastructure development (e.g., roads, bridges, etc.). The USACE handles the day-to-day administration and enforcement of the Section 404 program, including issuing permits. In circumstances where Section 404 is triggered, permit applicants also obtain a Section 401 certification from the State in which the discharge of dredged or fill material originates. [13 U.S.C. § 1341]. The Section 401 certification assures that materials discharged to waters of the United States will comply with relevant provisions of the CWA, including water quality standards. In addition, Section 402 of the CWA establishes the National Pollutant Discharge Elimination System (NPDES) Program. [33 U.S.C. § 1342]. The NPDES Program requires a permit for discharges of pollutants into waters of the United States, including storm water discharges.

2.3.6 Endangered Species Act [16 U.S.C. §§ 1531-1544]

Highway engineering projects have the potential to impact Federally-listed fish, wildlife, and plants. The purposes of the Endangered Species Act of 1973 (ESA) include conserving "the ecosystems upon which endangered species and threatened species depend" and providing "a program for the conservation of such endangered species and threatened species." [16 U.S.C. § 1531]. It is the policy of Congress that all Federal agencies shall seek to conserve endangered and threatened species and shall utilize their authorities in furtherance of the purposes of the ESA [Id.]. The U.S. Fish and Wildlife Service (USFWS) and the NOAA National Marine Fisheries Service (NMFS) administer the ESA. The USFWS and NMFS conduct consultations with the lead Federal agency when a proposed project may affect Federally endangered or threatened species. USFWS or NMFS involvement in a project depends on the affected species and the nature and extent of anticipated impacts (direct and indirect) to that species and its designated critical habitat. If anticipating a "take" of a Federally-listed species, USFWS or NMFS will issue a biological opinion, the terms and conditions of which are binding on the lead Federal agency. [16 U.S.C. § 1536].

2.3.7 National Historic Preservation Act [54 U.S.C. § 300101 et seq.]

River highway engineering projects are often subject to the National Historic Preservation Act of 1966 (NHPA). Section 106 of the National Historic Preservation Act (NHPA) (commonly called "Section 106") requires Federal agencies to consider the impacts on historic properties of projects that they carry out, approve, or fund. [54 U.S.C. § 306108]. The implementing regulations for the Section 106 process are found in 36 CFR Part 800. Those regulations provide that Federal agencies, in consultation with the Advisory Council on Historic Preservation, the State Historic Preservation Officers (SHPO), and certain other interested parties, identify and assess adverse effects to historic properties and seek ways to avoid, minimize, or mitigate those effects. [36 CFR 800.4-800.6]. Under Section 106, "historic property" is defined as any prehistoric or historic district, site, building, structure, or object included in, or eligible to be included in, the National

Register of Historic Places [36 CFR 800.16(l)(1); see also 54 U.S.C. 300311 and 302102]. The responsibilities of SHPOs are set forth at 54 U.S.C. § 302303.

In addition to Section 106, Section 4(f) of the U.S. Department of Transportation Act of 1966 [23 U.S.C. § 138 and 49 U.S.C. § 303] requires that the FHWA not approve the use of historic sites for a project unless there is no prudent and feasible alternative and the project incorporates all possible planning to minimize harm, or any impacts to historic sites are determined to be *de minimis*. The FHWA's regulations for implementation of Section 4(f) are found at 23 CFR Part 774.

2.3.8 National Flood Insurance Act of 1968 [42 U.S.C. § 4001 et seq.]

The National Flood Insurance Act of 1968 instituted the National Flood Insurance Program (NFIP) to help indemnify and reduce impacts associated with floods. The NFIP adopted the area subject to a 1 percent chance or greater of being flooded in any given year (also known as the 100-year flood) as the standard, or base flood, for mapping floodplains. [See, e.g., 44 CFR 9.4]. The area inundated by the 100-year flood determines the Special Flood Hazard Area (SFHA) on Flood Insurance Rate Maps (FIRMs) developed by FEMA and used to determine flood insurance rates for structures. [See, e.g., 44 CFR 59.1, which defines "area of special flood hazard"]. FEMA implements the NFIP using its regulations found in Title 44 of the CFR.

The FHWA's policies require projects to be consistent with the Standards and Criteria in the NFIP, where appropriate. [23 CFR 650.115(a)(5)]. To assist State DOTs in complying with this policy, the FHWA developed coordination procedures for Federal-aid highway projects with encroachments in NFIP-regulated floodplains. FEMA agreed to these procedures by signing a 1982 Memorandum of Understanding with the FHWA.

2.3.9 Wild and Scenic Rivers Act [16 U.S.C. § 1271 et seq.].

This Act establishes a policy to preserve designated rivers "in free-flowing condition" and to protect "their immediate environments … for the benefit and enjoyment of present and future generations." [16 U.S.C. § 1271]. Section 7(a) provides that "no department or agency of the United States shall assist by loan, grant, license, or otherwise in the construction of any water resources project that would have a direct and adverse effect on the values for which such river was established." [16 U.S.C. § 1278(a)]. A water resources project is "any dam, water conduit, reservoir, powerhouse, transmission line, or other project works under the Federal Power Act ... or other construction of developments which would affect the free-flowing characteristics of a Wild and Scenic River or Study River." [36 CFR 297.3]. "Federal assistance means any assistance by an authorizing agency including, but not limited to, ... [a] license, permit, or other authorization granted by the Corps of Engineers, Department of the Army, pursuant to the Rivers and Harbors Act of 1899 and section 404 of the Clean Water Act (33 U.S.C. 1344)." [Id.]

2.3.10 Fish and Wildlife Coordination Act [16 U.S.C. §§ 661-666c]

The Fish and Wildlife Coordination Act (FWCA) requires adequate consideration for the "conservation, maintenance, and management of wildlife resources" whenever the "waters of any stream or other body of water are impounded, diverted, the channel deepened, or the stream or other body of water otherwise controlled or modified for any purpose … including navigation and drainage, by any department or agency of the United States. [16 U.S.C. § 663(a)]. This generally includes consultation with the USFWS, the NMFS, and State wildlife agencies for activities that affect, control, or modify waters of any stream or bodies of water in order to minimize the adverse impacts of such actions on fish and wildlife resources and habitat. This consultation is generally incorporated into the process of complying with Section 404 of the Clean Water Act, NEPA, or other Federal permit, license, or review requirements.

2.3.11 Migratory Bird Treaty Act [16 U.S.C. § 703 et seq.].

The protection of all migratory birds is governed by the Migratory Bird Treaty Act (MBTA) [16 U.S.C. §§ 703-712], which generally prohibits the take of any migratory bird or any part, nest, or eggs of any such bird. [16 U.S.C. § 703(a)]. Under the MBTA, it is illegal to "take, kill, possess, transport, or import migratory birds or any part, nest, or egg of any such bird" unless authorized by a valid permit from the USFWS. [Id.]. The regulation at 50 CFR 10.13 includes a list of migratory birds protected by the MBTA.

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Chapter 3 - System Planning

Storm drainage is an integral component in highway and transportation networks. Drainage design for highway facilities strives to maintain compatibility and minimize interference with existing drainage patterns, control flooding of the roadway surface for design flood events, and minimize potential environmental impacts from highway-related stormwater runoff. To meet these goals, the planning and coordination of storm drainage systems begins in the early planning phases of transportation projects. Conducted before beginning design, successful system planning produces a final system design that evolves smoothly through the preliminary and final design stages of a transportation project.

3.1 Design Objectives

Highway storm drainage design aims to provide for safe and reliable passage of vehicles during the design storm event. The drainage system collects stormwater runoff from the roadway surface and right-of-way (ROW), conveys it along and through the ROW, and discharges it to an adequate receiving water body without causing adverse on- or off-site impacts.

Traffic safety relates directly to surface drainage. Rapid removal of stormwater from the pavement minimizes the conditions which can result in the hazards of hydroplaning. Surface drainage is a function of the hydrology, transverse and longitudinal pavement slope, pavement roughness and voids, inlet spacing, and inlet capacity.

Stormwater conveyance systems (storm drain piping, ditches, channels, pumps, etc.) provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without surcharging inlets or otherwise causing surface flooding. Designers also consider erosion potential in the design of open channels and ditches used for stormwater conveyance.

Water collected from and conveyed through the roadway corridor discharges offsite to receiving waters or to other storm drainage systems. Federal, State, and local regulations often guide the quality and quantity of the discharge. (Chapter 2 describes certain relevant Federal statutes and regulations.) To meet these regulatory requirements, storm drainage systems often rely on detention or retention basins and other best management practices (BMPs) for the control of discharge quantity and quality.

3.2 Minor versus Major Systems

A complete storm drainage system design includes consideration of both minor and major drainage systems. This manual focuses on the minor system, which consists of the components typically considered part of the "storm drainage system." Some designers refer to the minor system as the "convenience" or "primary" system. These components include curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc. The minor system is typically designed to carry runoff from an intermediate design storm event such as the 0.1 annual exceedance probability (AEP) (10-year return period) event.

The major system provides natural or constructed overland relief for stormwater flows exceeding the capacity of the minor system and may be intentionally designed or unintentional. Some designers refer to the major system as the "secondary" system. The major system usually carries stormwater during larger, more infrequent storm events, such as the 0.04, 0.02, and 0.01 AEP storms. The major system provides pathways for runoff to flow to natural or constructed receiving channels such as streets, streams, creeks, rivers, and ditches. The designer considers (at least in a general sense) the flow pathways and related depths and velocities of the major system under less frequent or check storm conditions.

Typically, design effort focuses on components of the minor system. However, when infrastructure owners and designers also consider the functioning of the major storm drainage system, they can provide more resilient transportation infrastructure and associated drainage features when storms exceeding the design magnitude for the minor system inevitably occur. This is especially relevant today as the FHWA and others seek to ensure the transportation network is resilient and reliable for all users despite the risk associated with a changing climate. (USDOT 2021).

3.3 Design Approach

Storm drainage system design evolves as a part of the highway design process. Figure 3.1 summarizes the primary activities of the process: data collection, agency coordination, concept development, concept refinement and design, and final design documentation. The first two activities continue throughout the design process. The next three activities represent progressively comprehensive stages of the design process: concept development, hydrologic and hydraulic (H&H) design, and final design.

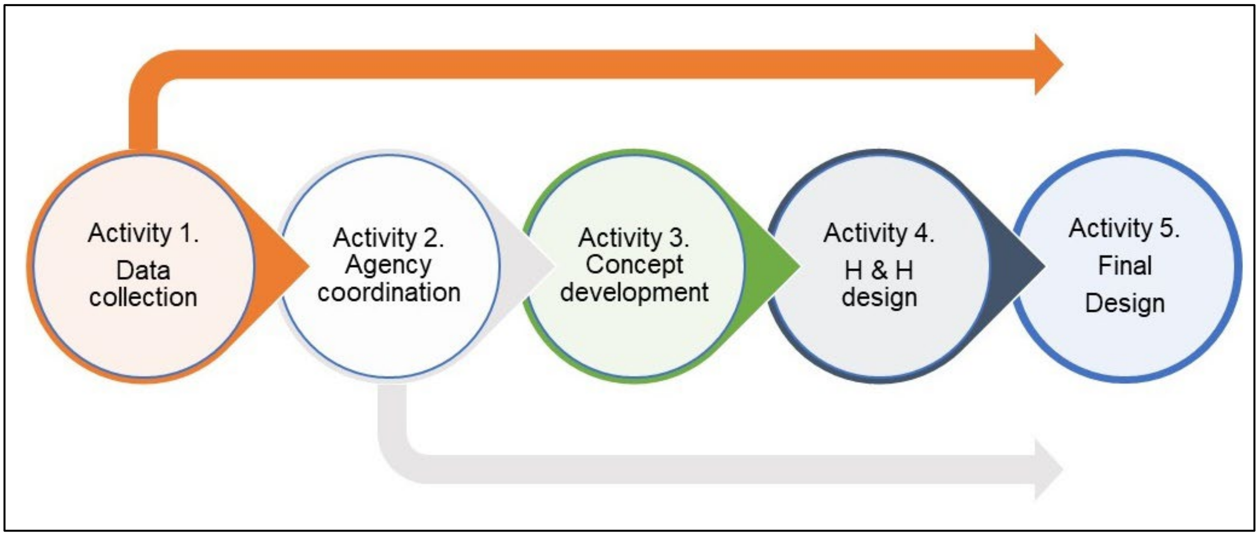

Figure 3.1. Drainage design process overview.

3.3.1 Data Collection

Initial data collection involves assembling and reviewing technical data and background information pertinent to the design. As the design progresses, the designer collects additional data needed for subsequent steps.

The design of storm drainage systems accumulates certain basic data discussed in the following paragraphs. The type of project, project drainage needs, and potential project risks inform the types and extent of supporting information needed.

- Watershed maps: These identify topographic features, watershed boundaries, existing drainage patterns, and ground cover. These maps are typically available from U.S. Geological Survey (USGS), local river authorities, drainage districts, or other planning agencies maps, field surveys, or aerial photography.
- Land use maps: These identify existing and potential future land uses. They are typically available from the internet and local zoning or planning agencies.

- Soils maps: These identify soil types and hydrologic soil groups. They are available in county soil surveys available online and from local U.S. Natural Resources Conservation Service (NRCS) offices.
- Flood histories and high-water marks: These may be available from local offices of the USGS, National Weather Service (NWS), Federal Emergency Management Agency (FEMA), U.S. Army Corps of Engineers (USACE), local planning agencies, river authorities, and drainage districts. Residents or Department of Transportation (DOT) regional or district maintenance offices may also be able to provide this information.
- Descriptions of existing drainage facilities: These includes size, shape, material, invert information, age, condition, etc. As-built information for existing drainage facilities may be available from the local facility owner. If unavailable, the DOT can perform field surveys to obtain this information.
- Design and performance data for existing drainage systems: These may be available from the facility owner. If unavailable for the existing system, the designer can develop the needed information to determine how the existing system will function under the new loading.
- Utility plans and descriptions: These may be available from the utility owner. If unavailable, the DOT can perform field surveys to determine critical design information.
- Existing right-of-way information: These may be available from the appropriate highway agency ROW office or local tax maps.
- Federal, State, and local regulatory requirements: Typical regulatory authorities include USACE, U.S. Environmental Protection Agency (USEPA), State environmental regulatory agencies, and local governments. (Chapter 2 describes Federal statutes and regulations.)

3.3.2 Agency Coordination

Agency coordination includes consultation with regulatory agencies and other stakeholders and continues through the design process. Before designing a storm drainage system, it is good practice to coordinate with regulatory agencies or others that have interests in drainage matters. Regulatory agency involvement may come from any level of government (Federal, State, or local). These agencies are generally concerned with potential impacts resulting from highway drainage, and center on stormwater quantity and quality issues. Chapter 2 discusses the Federal regulatory context.

Others with interests in storm drainage systems include local municipalities and developers. Local municipalities may wish to use portions of the system to provide for new or better drainage, or to augment old municipal drainage systems. Local municipalities may be interested in developing cooperative projects where a mutual economic benefit may exist. Local municipalities may also be aware of proposed private development in the vicinity of the road project which may affect drainage design. These groups may wish to improve or change drainage patterns, redirect stormwater to the ROW, or propose joint projects which could affect the highway storm drainage system. Early planning and coordination help identify and facilitate cooperative projects that are beneficial to both the transportation agency and other stakeholders.

The concerns of local community members, for example, about drainage facility impacts on their home or business, are also important. Community member concerns typically include impacts of roadway interruption and redirection of existing drainage patterns, the potential for flow concentration and increased flooding, and water quality impacts to both surface water and groundwater. Local government entities and the public hearing process usually facilitate communication and coordination with local community members.

3.3.3 Concept Plan Development

A concept plan typically provides a preliminary layout for the proposed storm drainage system identifying the basic components of the intended design superimposed on a project base map. The base map identifies the watershed areas and subareas, land use and cover types, soil types, existing drainage patterns, and other topographic features. Underground utility locations (and elevations if available), a preliminary roadway plan and profile, and locations of existing and proposed structures supplement this base information.

The concept plan illustrates how stormwater will be collected, conveyed, and discharged through both the minor and major systems. The designer may include preliminary sizes of significant drainage features. The basic components in the concept plan generally include:

- Elements of the major drainage system.
- Outfalls for the minor drainage system.
- Primary underground pipes and surface channels and ditches.
- Inlets.
- Stormwater quantity controls.
- Stormwater quality BMPs.

As a part of the development of the conceptual storm drainage plan, designers consider several additional issues. Whenever possible, designers avoid deep cuts and utilities. Designers address maintenance of traffic and construction related impacts and, in some cases, provide temporary drainage and traffic bypasses and other traffic control related activities. Designers also consider construction sequencing as it relates to the constructability of laterals and storm mains. Some instances may dictate a trunk line on both sides of the roadway with very few laterals, while other instances may call for a single trunk line.

3.3.4 Hydrologic and Hydraulic (H&H) Design

Once the concept is approved, the drainage designer refines the H&H aspects of the design. Design changes in the overall project development process, including input from regulatory and review agencies, result in the need for system refinement. This stage generally proceeds in the following sequence:

1. Compute runoff parameters and quantities based on the concept layout (see Chapter 4).
2. Estimate pavement drainage conditions (see Chapter 5) and roadside and median channel conditions (see Chapter 6).
3. Refine inlet location and spacing (see Chapter 7).
4. Refine the storm drain system layout including access holes, connecting mains, outfall control structures, and any other system components (see Chapters 8 and 9).
5. Size pipes, channels, pump stations, discharge control structures, and other storm drain system components.
6. Compute and review the hydraulic grade line (see Chapter 9).
7. Revise plan and recompute design parameters, as necessary.

3.3.5 Final Design

The final design includes preparation of final design documentation and construction plans. The sponsoring agency or the applicable local or State DOT Drainage Design Manual typically describes final design documentation requirements, which can vary with project type and scope. The AASHTO Drainage Manual (AASHTO 2014) provides a general description of design documentation.

3.4 Stormwater Drainage System Components

Minor storm drainage systems collect stormwater runoff from the roadway surface and ROW, convey it along and through the ROW, and discharge it to a receiving body without causing adverse on- or off-site environmental impacts. Major storm drainage systems provide flood conveyance and discharge for floods exceeding minor storm system capacity. As described in Section 3.2, this function is typically provided by streets, surface swales, ditches, streams, and other flow conduits to provide a relief mechanism and flow path for flood waters. The following sections describe the components of the minor storm drainage system.

3.4.1 Collection

Collection components, including gutters, ditches, and inlets, concentrate water on the roadway or bridge and move it away from the traveled way. Roadside and median ditches intercept runoff and carry it to a storm drain or channel. Ditch design considers not only hydraulic capacity but also safety for vehicles that may leave the roadway. If necessary, the design provides channel linings to control erosion in ditches.

Gutters intercept pavement runoff and carry it along the roadway edge to an adequate storm drain inlet. Designers install curbs in combination with gutters where runoff from the pavement surface would erode fill slopes or where ROW requirements or topographic conditions will not permit the development of roadside ditches. Urban settings typically use curbed pavement sections. Some other areas use gutters without curbs.

Drainage inlets receive surface water collected in ditches and gutters and serve as the mechanism for surface water to enter storm drains. Along the edge of the roadway, designers size and locate storm drain inlets to limit the spread of surface water onto travel lanes. Inlets may be grated, curbed, slotted, etc.

Roadway geometry and the ability to control the spread of water on the roadway surface determine the location of drainage inlets. Generally, placing inlets at low points in the gutter grade, intersections, crosswalks, cross-slope reversals, and on side streets prevents the water from flowing onto the main road. Additionally, placing inlets up-gradient of bridges prevents drainage onto bridge decks; placing inlets down-gradient of bridges prevents the flow of water from the bridge onto the roadway surface. Chapter 7 discusses inlet design.

3.4.2 Conveyance

Upon reaching the main storm drainage system, storm drains connected by access holes or other structures convey stormwater along and through the ROW to its discharge point. Storm drains receive runoff from inlets and convey it to some point where it is discharged into a channel, waterbody, or other piped system. Storm drains can be closed conduit or open channel; they consist of one or more pipes or conveyance channels connecting two or more inlets.

Access holes, junction boxes, and inlets serve as access structures and alignment control points in storm drainage systems. Access structure spacing and storm drain deflection are critical design parameters related to these structures. Maintenance tasks often dictate spacing limits. In addition, these structures are located at the intersections of two or more storm drains, when there is a change in the pipe size, and at changes in alignment (horizontal or vertical). Chapter 9 describes the design of storm drains.

Where gravity drainage is impossible or not economically justifiable, designers can use stormwater pump stations. For example, stormwater pump stations may be the only alternative for draining depressed roadway sections. Chapter 12 introduces the design of stormwater pumping stations.

3.4.3 Discharge Controls

Prior to discharging stormwater offsite or to receiving waters, designers may need to employ discharge controls to reduce the quantity (peak or volume) and improve the quality of discharge. For example, such controls may be required by Federal and state laws including Federal laws described in Chapter 2 such as the Clean Water Act. Detention and retention facilities control the quantity of runoff discharged to receiving waters. Runoff quantity can be reduced by storing runoff in detention/retention basins, storm drainage pipes, swales and channels, or other storage facilities. Outlet controls on these facilities reduce the rate of stormwater discharge. These controls can be useful where existing downstream receiving channels are inadequate to handle peak flow rates from the highway project, where highway development would contribute to increased peak flow rates and aggravate downstream flooding problems, or as a technique to reduce the size and associated cost of outfalls from highway storm drainage facilities. Chapter 10 discusses the analysis and design of detention and retention facilities.

Water quality controls improve the quality of stormwater discharges from highway storm drainage systems and mitigate potential impacts on receiving waters. They include extended detention ponds, wet ponds, infiltration trenches, infiltration basins, porous pavements, sand filters, water quality inlets, vegetative practices, erosion control practices, and wetlands. Water quality constituents typically associated with highway runoff include suspended solids, heavy metals, nutrients, and organics. Chapter 11 describes tools and methods for analyzing urban stormwater quality.

Chapter 4 - Urban Hydrologic Procedures

This chapter provides an overview of hydrologic methods and procedures commonly used in urban highway drainage design. The chapter condenses information from *Highway Hydrology*, Hydraulic Design Series 2 (HDS-2) (FHWA 2002). The chapter introduces the methods and procedures, their data demands, and their limitations. Most of these procedures can be applied using commonly available computer programs. HDS-2 contains additional information and detail on the methods described.

4.1 Rainfall (Precipitation)

Rainfall, along with watershed characteristics, determines the flows for storm drainage design. The following sections describe three representations of rainfall the designer can use to derive flood flows: uniform rainfall intensity, variable rainfall intensity, and synthetic design storm events.

4.1.1 Uniform Rainfall Intensity

Although rainfall intensity varies during precipitation events, many of the procedures that designers use to derive peak flow (see Section 4.2) are based on an assumed uniform rainfall intensity. Intensity is the rate of rainfall and is typically given in units of inches per hour (millimeters per hour).

Federal, State, and local agencies have developed Intensity-Duration-Frequency (IDF) curves throughout the United States through frequency analysis of rainfall events for thousands of rainfall gages. The IDF curve, as illustrated in Figure 4.1, provides a summary of a site's rainfall characteristics by relating storm duration and exceedance probability (frequency) to rainfall intensity (assumed constant over the duration). To interpret an IDF curve, find the rainfall duration along the horizontal axis, go vertically up the graph until reaching the proper return period, then go horizontally to the left and read the intensity off the vertical axis. Most highway agency drainage manuals contain regional IDF curves and NOAA Atlas 14 provides a national source of IDF curves.

4.1.2 Variable Rainfall Intensity (Hyetograph)

Rainfall intensity in any given storm varies over time. Figure 4.2 depicts a mass rainfall curve showing the accumulation of rainfall during an actual or design storm. The instantaneous intensity is the slope of the mass rainfall curve at a particular time. For hydrologic analysis, designers typically divide the storm into convenient time increments and determine the average intensity over each of the selected periods. The designer then plots these results as a rainfall hyetograph, like that presented in Figure 4.2.

Hyetographs provide greater flexibility in creating design storms than a uniform rainfall intensity by specifying the precipitation variability over time. Designers use them in conjunction with hydrographic (rather than peak flow) methods. In storm drain design, hyetographs are relevant for volumetric applications such as storage routing of hydrographs. Hyetographs allow for simulation of actual rainfall events and calibration of hydrologic models, which can provide valuable information on the relative flood risks of different events. The National Climatic Data Center (NCDC) at NOAA often makes available hyetographs of actual storms.

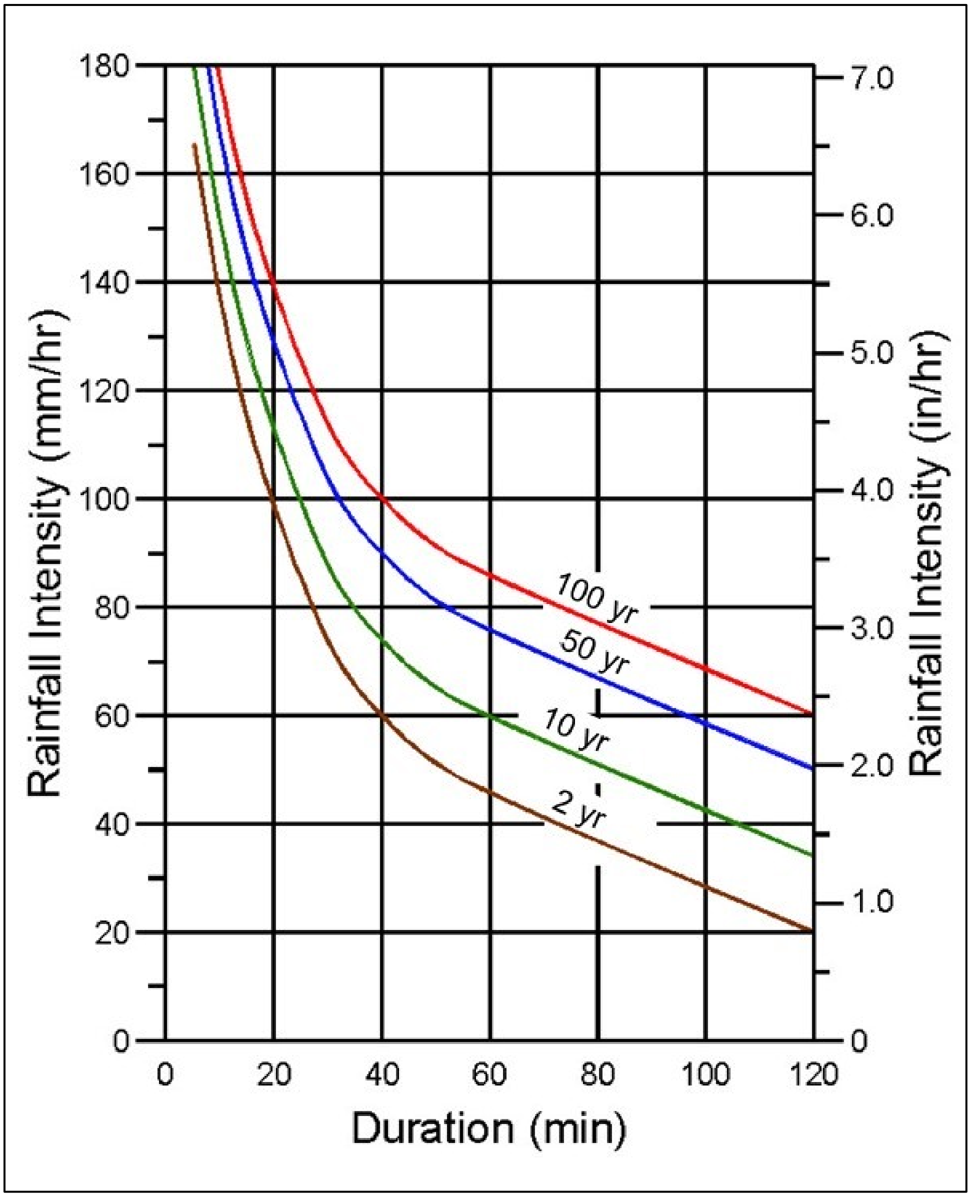

Figure 4.1. Example IDF curves.

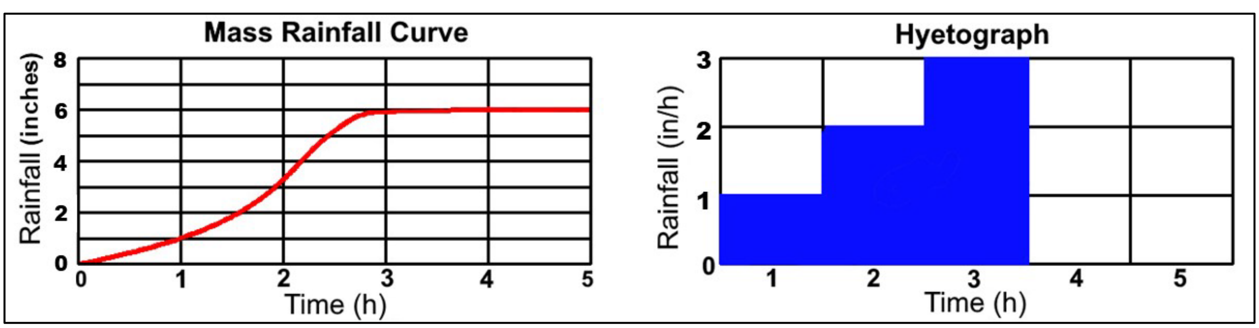

Figure 4.2. Example mass rainfall curve and corresponding hyetograph.

4.1.3 Synthetic Design Storm Events

Designers typically base drainage design on synthetic, rather than actual, rainfall events. The U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), developed and documented 24-hour rainfall distributions, which is described in HDS-2. The SCS 24-hour rainfall distributions are widely used synthetic hyetographs and incorporate the intensity-duration relationship for the design AEP. This approach assumes that the maximum rainfall for any duration within the 24-hour duration has the same AEP. For example, a 0.1 AEP, 24-hour design storm contains the 0.1 AEP rainfall depths for all durations up to 24 hours as derived from IDF curves. Other sources of rainfall distributions exist including NOAA Atlas 14.

4.2 Peak Flow

Peak flows are generally adequate for design and analysis of conveyance systems such as storm drains or open channels. This section discusses methods used to derive peak flows for both gaged and ungaged sites. The NRCS (SCS) peak flow method is another approach that calculates peak flow as a function of drainage basin area, potential watershed storage, and the time of concentration. This rainfall-runoff methodology separates total rainfall into direct runoff, retention, and initial abstraction. HDS-2 provides more detailed discussion on this method.

4.2.1 Statistical Analysis

Designers use statistical analysis to evaluate peak flows where adequate gaged streamflow data exist. Frequency distributions, used in the analysis of hydrologic data, include the normal distribution, the log-normal distribution, the Gumbel extreme value distribution, and the log-Pearson type III distribution. The log-Pearson type III distribution is a three-parameter gamma distribution with a logarithmic transform of the independent variable. Designers use it widely for flood analyses because the data frequently fit the assumed population. This flexibility led the United States Geological Survey (USGS) to recommend its use as the standard distribution for flood frequency studies by all U.S. Government agencies, as documented in Bulletin 17C (England et al. 2019). Figure 4.3 presents an example of a log-Pearson type III distribution frequency curve (FHWA 2002). Designers do not commonly use statistical analysis methods in urban drainage design due to the lack of adequate streamflow data. Consult HDS-2 (FHWA 2002) for additional information on these methods.

4.2.2 Rational Method

One of the most used approaches for the calculation of peak flow from small areas is the Rational Method, given as:

$$Q = \frac{CIA}{K_u} \tag{4.1}$$

where:

Q	=	Flow, ft^3/s (m^3/s)
C	=	Dimensionless runoff coefficient
I	=	Rainfall intensity, in/h (mm/h)
A	=	Drainage area, ac (ha)
K_u	=	Unit conversion constant, 1.0 in CU (360 in SI)

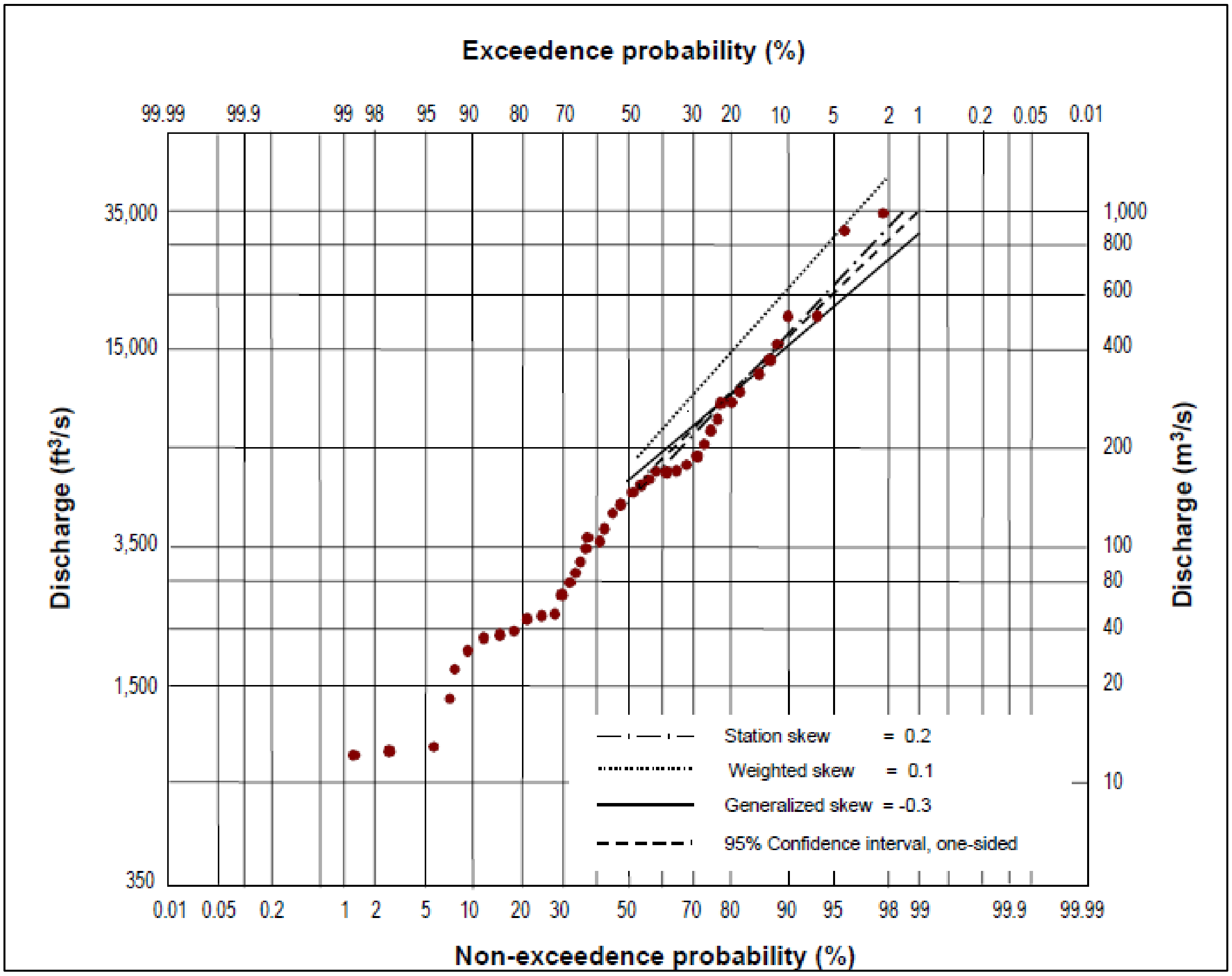

Figure 4.3. Log-Pearson type III distribution analysis, Medina River, Texas.

As described in HDS-2 (FHWA 2002), the Rational Method, assumes:

- Peak flow occurs when the entire watershed contributes to the flow.
- Rainfall intensity is the same over the entire drainage area.
- Rainfall intensity is uniform over a time duration equal to the time of concentration, t_c. (The time of concentration is the time water takes to travel from the hydraulically most remote point of the basin to the point of interest.)
- Frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 0.1 AEP rainfall intensity produces the 0.1 AEP peak flow.
- The coefficient of runoff is the same for all storms of all recurrence probabilities.

Rational Method

Emil Kuichling, City Engineer of Rochester, New York, developed the Rational Method in 1889. The methodology is based rainfall-runoff observations of small urban watersheds ranging from 25 to 357 acres. The assumptions emerge from his findings.

Related to the last assumption, designers sometimes use a frequency-of-event correction factor as a modifier to the Rational Method runoff coefficient. Some agencies recommend use of this coefficient, but FHWA does not endorse its use. The intent of the correction factor is to compensate for the reduced effect of infiltration and other hydrologic abstractions during less frequent, higher intensity storms. The frequency-of-event correction factor is multiplied times the runoff coefficient, C, to produce an adjusted runoff coefficient.

Because of the inherent assumptions, HDS-2 recommends application of the Rational Method to drainage areas smaller than 200 ac (80 ha) and provides additional information on the Rational Method (FHWA 2002).

4.2.2.1 Runoff Coefficient

The runoff coefficient, C, in equation 4.1 is a function of the ground cover and other characteristics that influence hydrologic abstraction. Table 4.1 summarizes typical values for C. If the basin contains varying amounts of different land cover or other abstractions, a weighted coefficient can be calculated through areal weighting as follows (FHWA 2002):

$$C = \sum \frac{C_x A_x}{A_{total}} \qquad (4.2)$$

where:

x = Subscript designating values for incremental areas with consistent land cover

Example 4.1: Calculation of the runoff coefficient.

Objective: Estimate weighted runoff coefficient, C, for existing and proposed conditions.

Given: The following existing and proposed land uses:

Existing conditions (unimproved):

Land Use	Area (ac)	Runoff Coefficient
Unimproved Area	22.1	0.25
Grass	21.2	0.22
Total	43.3	-

Proposed conditions (improved):

Land Use	Area (ac)	Runoff Coefficient
Paved	5.4	0.90
Lawn	1.6	0.15
Unimproved Area	18.6	0.25
Grass	17.7	0.22
Total	43.3	-

Step 1. Determine weighted C for existing (unimproved) conditions.

Using equation 4.1:

Weighted $C = \Sigma (C_x A_x) / A = [(22.1) (0.25) + (21.2) (0.22)] / 43.3 = 0.235$

Step 2. Determine weighted C for proposed (improved) conditions.

Using equation 4.1:

Weighted C = [(5.4) (0.90) + (1.6) (0.15) + (18.6) (0.25) + (17.7) (0.22)] / 43.3 = 0.315

Solution: The weighted runoff coefficients, C, for existing and proposed conditions are 0.235 and 0.315, respectively.

Table 4.1. Runoff coefficients for the Rational Method (ASCE 1960).

Land Use Category	Type of Drainage Area	Runoff Coefficient, C*
Business	Downtown areas	0.70 - 0.95
	Neighborhood areas	0.50 - 0.70
Residential	Single-family areas	0.30 - 0.50
	Multi-units, detached	0.40 - 0.60
	Multi-units, attached	0.60 - 0.75
	Suburban	0.25 - 0.40
	Apartment dwelling areas	0.50 - 0.70
Industrial	Light areas	0.50 - 0.80
	Heavy areas	0.60 - 0.90
Open	Parks, cemeteries	0.10 - 0.25
	Playgrounds	0.20 - 0.40
	Railroad yard areas	0.20 - 0.40
	Unimproved areas	0.10 - 0.30
Lawns	Sandy soil, flat, 2%	0.05 - 0.10
	Sandy soil, average, 2 - 7%	0.10 - 0.15
	Sandy soil, steep, 7%	0.15 - 0.20
	Heavy soil, flat, 2%	0.13 - 0.17
	Heavy soil, average, 2 - 7%	0.18 - 0.22
	Heavy soil, steep, 7%	0.25 - 0.35
Streets	Asphaltic	0.70 - 0.95
	Concrete	0.80 - 0.95
	Brick	0.70 - 0.85
Other impervious	Drives and walks	0.75 - 0.85
	Roofs	0.75 - 0.95

*Higher values are generally appropriate for steeply sloped areas and less frequent AEPs because infiltration and other losses have a proportionally smaller effect on runoff in these cases.

4.2.2.2 Rainfall Intensity

The Rational Method uses rainfall intensity, duration, and frequency curves. Federal, State, and local agencies have developed IDF curves for locations across the country and State highway agency drainage manuals typically document those applicable within their jurisdiction. NOAA and NRCS have created regional rainfall intensity curves that can also be used to create IDF relationships for design.

4.2.2.3 Time of Concentration

Designers use many methods to estimate time of concentration including the velocity or segment method. The velocity method calculates the flow velocity within individual segments of the flow path, e.g., sheet flow, shallow concentrated flow, and open channel flow. The time of concentration can be calculated as the sum of the travel times within the various consecutive flow segments. For additional discussion on establishing the time of concentration for inlets and drainage systems, see Section 9.2.2 of this manual.

Sheet flow is the shallow runoff on a planar surface with a uniform depth across the sloping surface. This usually occurs at the headwater of streams over relatively short distances, rarely more than about 300 ft, but most often less than 100 ft (NRCS 2010). Ragan (1971) suggests sheet flow occurs for distances 72 feet or less. Designers commonly estimate sheet flow with a version of the kinematic wave equation (FHWA 2002):

$$t_t = \frac{K_u}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8} \quad (4.3)$$

where:

t_t	=	Sheet flow travel time, min
n	=	Roughness coefficient
L	=	Flow length, ft (m)
P_2	=	2-year, 24-hour rainfall depth, inches (mm)
S	=	Surface slope, ft/ft (m/m)
K_u	=	Unit conversion constant, 0.42 in CU (5.5 in SI)

Table 4.2 summarizes Manning's roughness coefficients. Equation 4.3 is the modified version of the sheet flow equation. An iterative version of the equation replaces rainfall depth with rainfall intensity (FHWA 2002).

Shallow concentrated flow develops as sheet flow concentrates in rills and then gullies of increasing proportions. Designers can estimate the velocity of such flow using a relationship between velocity and slope, as described in HDS-2 (FHWA 2002) as follows:

$$V = K_u k S_p^{0.5} \quad (4.4)$$

where:

V	=	Velocity, ft/s (m/s)
k	=	Intercept coefficient
S_p	=	Slope, percent
K_u	=	Unit conversion constant, 3.28 in CU (1.0 in SI)

Table 4.3 summarizes intercept coefficients for shallow concentrated flow.

Table 4.2. Manning's roughness coefficient (n) for overland sheet flow (FHWA 2002).

Surface Description	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipe	0.024
Cement rubble surface	0.024
Fallow (no residue)	0.05
Cultivated soils: Residue cover # 20%	0.06
Cultivated soils: Residue cover > 20%	0.17
Cultivated soils: Range (natural)	0.13
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Woods: Light underbrush *	0.40
Woods: Dense underbrush *	0.80

*When selecting n, consider cover to a height of about 1 inch (30 mm). This is only part of the plant cover that will obstruct sheet flow.

Table 4.3. Intercept coefficients for velocity vs. slope relationship (FHWA 2002).

Land Cover/Flow Regime	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

Open channel and pipe flow occurs when water further collects in gullies, channels, or pipes. Such channels can be identified when they are shown on maps or are visible on aerial photographs. Velocity calculations typically use cross-section geometry and roughness for all channel reaches in the watershed. Designers commonly use Manning's equation to estimate average flow velocities in pipes and open channels as follows:

$$V = \frac{K_u}{n} R^{2/3} S^{1/2} \tag{4.5}$$

where:

n	=	Roughness coefficient
V	=	Velocity, ft/s (m/s)
R	=	Hydraulic radius (measured as the flow area divided by the wetted perimeter), ft (m)
S	=	Slope, ft/ft (m/m)
K_u	=	Unit conversion constant, 1.49 in CU (1 in SI)

Table 4.4 provides representative values of Manning's roughness coefficient for channels and pipes. For a circular pipe flowing full, the hydraulic radius equals one-fourth of the diameter.

Table 4.4. Typical range of Manning's coefficient (n) for channels and pipes.

Conduit Category	Conduit Material	Manning's n *
Closed conduits	Concrete pipe	0.010 - 0.015
	CMP	0.011 - 0.037
	Plastic pipe (smooth)	0.009 - 0.015
	Plastic pipe (corrugated)	0.018 - 0.025
Pavement/gutter sections	Concrete, asphalt	0.012 - 0.016
Small open channels	Concrete	0.011 - 0.015
	Rubble or riprap	0.020 - 0.035
	Vegetation	0.020 - 0.150
	Bare soil	0.016 - 0.025
	Rock cut	0.025 - 0.045
Natural channels/streams (top width at flood stage less than 100 ft (30 m)	Fairly regular section	0.025 - 0.050
	Irregular section with pools	0.040 - 0.150

*Lower values usually apply to well-constructed and maintained (smoother) pipes and channels.

For a wide rectangular channel (W > 10 d), the hydraulic radius approximately equals the depth. To calculate the travel time:

$$t = \frac{L}{60V} \tag{4.6}$$

where:

t	=	Travel time for a given segment, min
L	=	Flow length for the given segment, ft (m)
V	=	Velocity for the given segment, ft/s (m/s)

Example 4.2: Calculation of time of concentration.

Objective: Estimate time of concentration, t_c, for the given area.

Given: The following flow path characteristics:

Flow Segment	Length (ft)	Slope (ft/ft)	Segment Description
1 (sheet flow)	223	0.010	Bermuda grass
2 (shallow conc.)	259	0.006	Grassed waterway
3 (flow in conduit)	479	0.008	15-inch (380 mm) concrete pipe

The 2-yr 24-h rainfall depth is 4.35 inches.

Step 1. Calculate travel times for each segment, starting at the downstream end, using the 0.1 AEP (10-year) IDF curve.

Step 1a. Calculate travel time for segment 1.

Obtain Manning's n roughness coefficient from Table 4.2:

n = 0.41

Determine the sheet flow travel time using equation 4.3:

$$t_t = \frac{K_u}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8} = \frac{0.42}{(4.35)^{0.5}}\left(\frac{0.41(223)}{\sqrt{0.01}}\right)^{0.8} = 47.1\,\text{min (use 47 minutes)}$$

Step 1b. Calculate travel time for segment 2.

Obtain intercept coefficient, k, from Table 4.3: k = 0.457 and K_u = 3.281

Determine the concentrated flow velocity from equation 4.4:

$V = K_u\, k\, S_p^{0.5} = (3.281)\,(0.457)\,(0.6)^{0.5} = 1.16$ ft/s

Determine the travel time from equation 4.6:

$t_{t2} = L / (60\ V) = 259/[(60)(1.16)] = 3.7$ min

Step 1c. Calculate travel time for segment 3.

Obtain Manning's n roughness coefficient from Table 4.3: n = 0.011

Determine the pipe flow velocity from equation 4.5 (assuming full flow):

$V = (1.49 / 0.011)\,(1.25 / 4)^{0.67}\,(0.008)^{0.5} = 5.58$ ft/s

Determine the travel time:

$t_{t3} = L/(60\ V) = 479/[(60)(5.58)] = 1.4$ min

Step 2. Determine the total travel time by summing the individual travel times.

$t_c = t_{t1} + t_{t2} + t_{t3} = 47.1 + 3.7 + 1.4 = 52.2$ min; use 52 min

Solution: The estimated time of concentration is 52 minutes.

Example 4.3: Rational Method peak flow.

Objective: Estimate 0.1 AEP (10-year) peak flow using the Rational Method and the IDF curve shown in Figure 4.1.

Given: Land use conditions from example 4.1 and the following times of concentration:

Condition	Time of concentration, t_c (min)	Weighted C (from example 4.1)
Existing (unimproved)	88	0.235
Proposed (improved)	66	0.315

Area = 43.36 ac (17.55 ha)

Step 1. Determine rainfall intensity, I, from the 0.1 AEP IDF curve for each time of concentration.

Rainfall intensity:

Existing condition (unimproved) 1.9 in/h

Proposed condition (improved) 2.3 in/h

Step 2. Determine peak flow rate, Q.

Existing condition (unimproved):

$Q = CIA / K_u = (0.235)(1.9)(43.3) / 1 = 19.3\ ft^3/s$

Proposed condition (improved):

$Q = CIA / K_u = (0.315)(2.3)(43.3) / 1 = 31.4\ ft^3/s$

Solution: The existing and proposed condition peak flows are 19.3 ft^3/s (0.55 m^3/s) and 31.4 ft^3/s (89 m^3/s), respectively.

4.2.3 USGS Regression Equations

Designers commonly use regression equations to estimate peak flows at ungaged sites or sites with limited data. The USGS has developed and compiled regional regression equations which are included in a computer program called the National Streamflow Statistics program (NSS) and in StreamStats. NSS allows quick and easy estimation of peak flows throughout the United States (Reis 2007). Local equations may also be available. HDS-2 provides additional information on regression equations (FHWA 2002).

4.2.3.1 Rural Equations

The rural equations are based on watershed and climatic characteristics within specific regions of each State that can be obtained from topographic maps, rainfall reports, and atlases. These regression equations generally take the form:

$$RQ_T = a\,A^b B^c\ C^d \tag{4.7}$$

where:

RQ_T	=	T-year rural peak flow, ft^3/s (m^3/s)
a	=	Regression constant
b, c, d	=	Regression coefficients
A, B, C	=	Basin or meteorological characteristics

The USGS, State highway, and other agencies conducted a series of studies to develop rural equations for all States. These equations do not apply where dams and other hydrologic modifications have a significant effect on peak flows. HDS-2 presents other limitations (FHWA 2002).

4.2.3.2 Urban Equations

Designers can adapt a rural peak flow for urban conditions with the three-parameter or seven-parameter nationwide urban regression equations developed by USGS. NSS can calculate peak flows with both versions. The USGS urban equations are based on urban runoff data from 269 basins in 56 cities and 31 States and validated at 78 additional sites in the southeastern United States. The equations provide reasonable estimates of peak flows with recurrence intervals between 2 and 500 years (Sauer et al. 1983). The USGS has quantified the accuracy of the urban equations, like the rural equations, with standard errors that are in the range of 35 to 50 percent when compared to site-specific estimates from gage records. HDS-2 (FHWA 2002) provides more detail on urban regression equations.

4.3 Design Hydrographs

This section discusses methods used to develop a design hydrograph. Application of hydrograph methods often necessitates computer programs such as the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), the Stormwater Management Model (SWMM), WinTR-20, WinTR-55, among others, to generate runoff hydrographs. Practitioners perform hydrographic analyses when flow routing is important such as in the design of stormwater detention, other water quality facilities, and pump stations. Large storm drainage systems might also use hydrographic analyses to evaluate flow routing and more precisely reflect flow peaking conditions in each segment of complex systems. HDS-2 contains additional information on hydrographic methods (FHWA 2002).

4.3.1 Unit Hydrograph Methods

A unit hydrograph is the direct runoff hydrograph resulting from a unit volume of excess rainfall. The unit volume is 1 inch in CU and 1 millimeter in SI system. Although the unit volume is given in units of depth, users of the method call it a volume because it is that unit depth applied over the area of the watershed. Depth times area is volume.

The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is also equal to that unit volume, as described in HDS-2 (FHWA 2002). As do many methods, application of a unit hydrograph assumes uniform rainfall intensity and duration over the entire watershed. Therefore, the modeler chooses a design storm (rainfall intensity and duration) suited for the size of watershed under consideration. Additionally, storm movement can affect the runoff characteristics of the watershed. Storms moving down long and narrow watersheds produce a higher peak runoff rate and a longer time to peak. Acknowledging these assumptions, the drainage area limitation of unit hydrograph applications is a maximum of

1,000 mi^2 (FHWA 2002). This section discusses the NRCS dimensionless unit hydrograph and Snyder unit hydrograph methods.

4.3.1.1 NRCS Dimensionless Unit Hydrograph

The NRCS developed a synthetic unit hydrograph procedure widely used in their conservation and flood control work. This unit hydrograph is based upon an analysis of many natural unit hydrographs from a broad range of geographic locations and hydrologic regions. To construct the standard NRCS unit hydrograph, the designer estimates the peak flow and the time to peak.

For the development of the NRCS Unit Hydrograph, the curvilinear unit hydrograph is approximated by a triangular unit hydrograph with similar characteristics. Figure 4.4 compares the two dimensionless unit hydrographs. Even though the time base of the triangular unit hydrograph is 8/3 of the time to peak and the time base of the curvilinear unit hydrograph is five times the time to peak, the area under the two unit hydrograph types is the same.

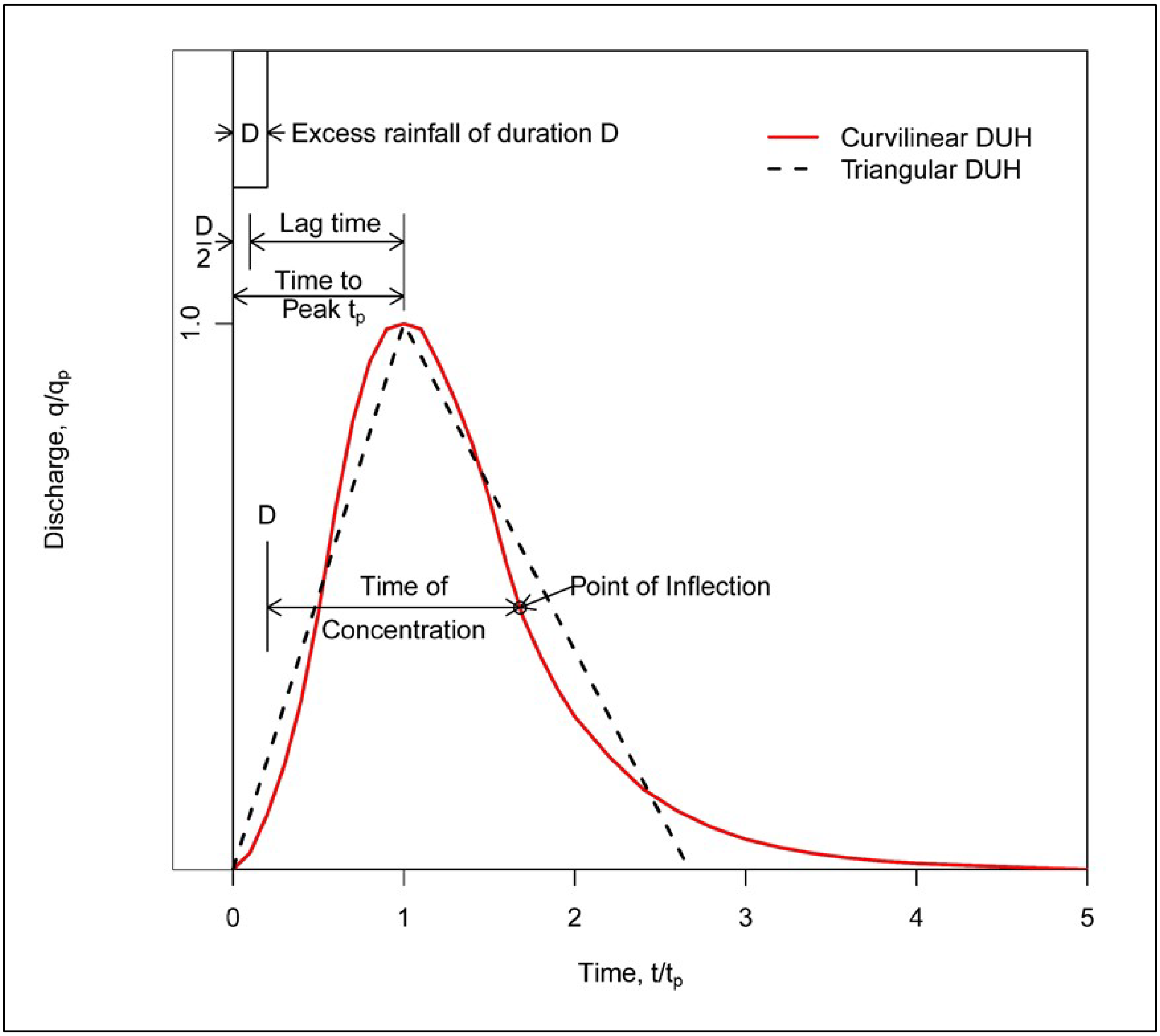

Figure 4.4. Dimensionless curvilinear NRCS unit hydrograph and equivalent triangular hydrograph.

The area under a hydrograph equals the volume of direct runoff, Q_D, which is 1 inch (millimeter) for a unit hydrograph. To calculate the peak flow:

$$q_p = \frac{K_u K_p A Q_D}{t_p} \tag{4.8}$$

where:

q_p	=	Peak flow, ft^3/s (m^3/s)
A	=	Drainage area, mi^2 (km^2)
Q_D	=	Volume of direct runoff (= 1 for unit hydrograph), inch (mm)
t_p	=	Time to peak, h
K_p	=	Peaking constant equal to 484, dimensionless
K_u	=	Unit conversion constant, 1 in CU (0.0043 in SI)

The peaking constant reflects a unit hydrograph with 3/8 of its area under the rising limb. For mountainous watersheds, the fraction could be expected to be greater than 3/8, and therefore the constant may be near 600. For flat, swampy areas, the constant may be on the order of 300.

Time to peak, t_p, can be expressed in terms of time of concentration, t_c, as follows:

$$t_p = \frac{2}{3} t_c \tag{4.9}$$

Expressing q_p in terms of t_c rather than t_p yields:

$$q_p = \frac{K_u K_p A Q_D}{t_c} \tag{4.10}$$

where:

K_u	=	Unit conversion constant, 1.5 in CU (0.00645 in SI)

Example 4.4:	NRCS dimensionless unit hydrograph.
Objective:	Estimate the peak and time to peak for the NRCS dimensionless unit hydrograph.

Given: The following watershed conditions:

Watershed is commercially developed.

Watershed area = 0.463 mi^2 (1.2 km^2)

Time of concentration = 1.34 h

Q_D = 1.0 inch (for unit hydrograph, 1 mm for SI)

Step 1. Calculate peak flow using equation 4.10.

$q_p = (K_u\ K_p\ A\ Q_D) / t_c = [1.5\ (484)\ (0.463)\ (1.0)] / 1.34 = 251\ ft^3/s$

Step 2. Calculate time to peak using equation 4.9.

$t_p = (2/3)\ t_c = (2/3)\ (1.34) = 0.89$ h

Solution:	The key parameters for this application are a peak flow of 251 ft^3/s (7.1 m^3/s) and a time to peak of 0.89 hours.

4.3.1.2 Snyder Unit Hydrograph

The Snyder method uses empirical terms and the physiographic characteristics of the drainage basin to determine a unit hydrograph. Figure 4.5 illustrates the key parameters that determine the hydrograph shape (lag time, unit hydrograph duration, peak flow, and hydrograph time widths of 50 percent and 75 percent of the peak flow). The designer adjusts the selected parameters so that the volume of the hydrograph equals one inch (millimeter) of direct runoff. Symbols used in the figure are:

T_R = Duration of unit excess rainfall, h
t_L = Lag time from the centroid of the unit rainfall excess to the peak of the unit hydrograph, h
t_p = Time to peak flow of the unit hydrograph, h
t_b = Time base of the unit hydrograph, h
W_{50}, W_{75} = Time width of unit hydrograph at discharge equal to 50 and 75 percent, h

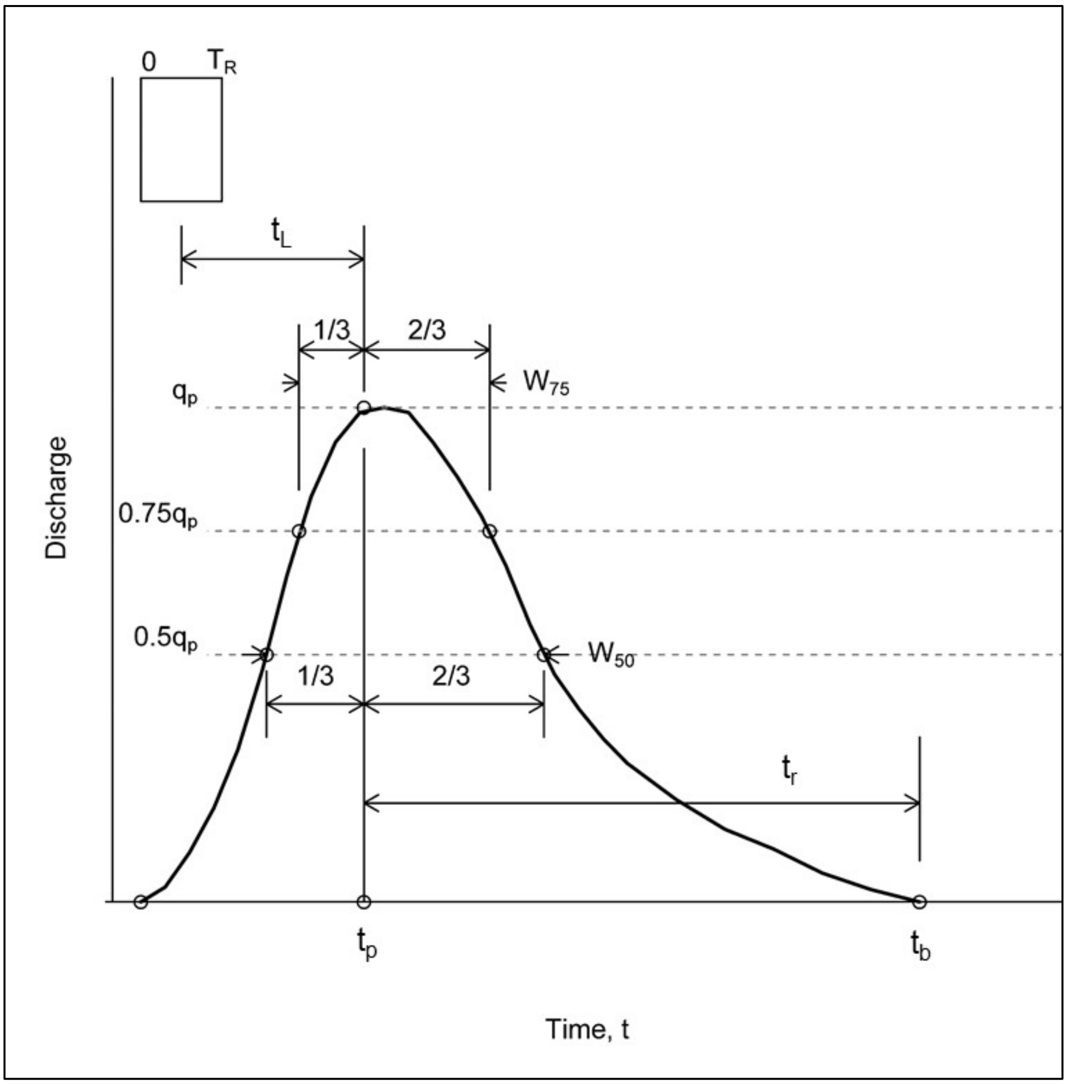

Figure 4.5. Snyder synthetic hydrograph definition.

Snyder initially applied this unit hydrograph for watersheds in the Appalachian highlands; however, the general method has been successfully applied throughout the country. HDS-2 provides additional information and an example problem that describes the procedures for computing the Snyder Synthetic Unit Hydrograph (FHWA 2002).

4.3.2 USGS Nationwide Urban Hydrograph

The USGS nationwide urban hydrograph method uses information developed by the USGS that approximates the shape and characteristics of hydrographs. Information needed for using this method include: 1) dimensionless hydrograph ordinates, 2) lag time, and 3) peak flow. HDS-2 (FHWA 2002) provides more detail on the USGS urban hydrograph method.

4.3.3 Continuous Simulation

In drainage design, peak flow and unit hydrographs typically meet the needs of designers but in some complex applications of stormwater management water quality analysis, designers use continuous simulation. Continuous simulation evaluates the entire hydrologic cycle based on the historical record and is distinctly different from single-event models as described in the preceding sections. Continuous simulation involves less simplification of processes and fewer assumptions and may produce more robust hydrologic estimates.

Depending on applicable design standards or requirements, or both, hydrologic design applications may rely on low flow statistics, flood-volume statistics, or flow duration curves. These applications use daily time series data. Modelers can use a calibrated continuous simulation model and a long-term precipitation record to produce a simulated streamflow series as described in HDS-2 (FHWA 2002). There are many implementations of continuous simulation.

Chapter 5 - Roadway Pavement Drainage

The paved surface of the driving lanes is the critical area of interaction between vehicles and the roadway; effective drainage of the pavement facilitates safe roadway use. Water on the pavement adversely affects safe use of the road, interrupts traffic, reduces skid resistance, increases potential for hydroplaning, and limits visibility by splash and spray from traffic (AASHTO 2014).

Pavement drainage design involves knowledge and consideration of surface drainage, gutter flow, and inlet capacity. The design of these elements depends on the design rainfall frequency and the allowable spread of stormwater on the pavement surface. This chapter presents information for the design of roadway features to meet the desired safety and serviceability levels. The chapter draws on HEC-12, Drainage of Highway Pavements (FHWA 1984), and AASHTO's Drainage Manual (2014) for most of the information presented.

5.1 Spread

The width of pavement (outward from the face of curb) covered by water during rainfall is called "spread." Spread at any point on the roadway relates to the intensity and duration of rainfall. Allowable spread represents the maximum width of pavement covered during the design storm. Designers also evaluate spread associated with a larger (less frequent) storm.

5.1.1 Design Frequency and Spread

Highway storm drainage provides safe passage for vehicles under a set of conditions called a design storm. The drainage system, including the curb and gutter and additional width such as parking lanes, conveys stormwater to pavement inlets to provide reasonable safety for traffic and pedestrians at a reasonable cost. As spread from the curb increases, the risk of traffic accidents increases because of hydroplaning and spray, along with nuisance to pedestrian, bicycle, and scooter traffic.

When specifying the AEP or return period for the design storm and the allowable spread during such a storm, designers make decisions regarding risk of accidents and traffic delays, and acceptable costs for the drainage system. Risk associated with water on traffic lanes increases with increasing traffic volume and traffic speed not only for safety reasons but also because of increased potential for delay. The safety of the public represents the primary consideration for designers in specifying the design frequency and design spread.

State and local Departments of Transportation (DOTs) establish design storm and spread criteria based on consideration of:

- Functional classification of the roadway, and possibly traffic volume within the functional classification.
- Traffic speed, which is a primary factor in hydroplaning when water is on the pavement.
- Existing and projected traffic volume, which may be an indicator of the economic importance of keeping the highway open to traffic. The costs of traffic delays and accidents increase with increasing traffic volumes.
- Cost. Balance between desirable and practicable criteria is sometimes necessary because of cost, and because of external factors such as constrained right-of-way (ROW) or utilities. The costs and feasibility of providing for a given design frequency and spread may vary significantly between projects. In some cases, it may be practicable to significantly

upgrade the drainage design and reduce risks at moderate costs. In other instances, costs may be very sensitive to the criteria selected for design.

Designers consider inconvenience, hazards, and nuisances to pedestrian, bicycle, and other personal transport traffic. In some places, such as in commercial areas, this consideration may assume major importance. Local design practice may also be a major consideration since it can affect the feasibility of designing to higher standards, and it influences public perception of acceptable practice.

Designers also consider the relative elevation of the highway and surrounding terrain where water can be drained only through a storm drainage system, as in underpasses and depressed roadway sections. They consider the potential for ponding to undesirable depths when selecting the frequency and spread criteria and in checking the design against storm runoff events of lesser frequency than the design event.

Selection of design criteria for intermediate types of facilities may be the most difficult. For example, some arterials with relatively high traffic volumes and speeds may not have shoulders which will convey the design runoff without encroaching on the traffic lanes. In these instances, practitioners typically assess the relative risks and costs of various design spreads to select appropriate design criteria. Table 5.1 provides example minimum design frequencies and spread based on the type of highway and traffic speed.

Along with the situations covered in Table 5.1, for depressed sections and underpasses where ponded water can be removed only through the storm drainage system, designers frequently consider additional criteria including a 0.02 AEP event. The use of a more severe event, such as a 0.01 AEP event to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a “check storm.”

Table 5.1. Example minimum design frequency and spread.

Road Classification	Context	Design Frequency (AEP)	Design Spread
Interstate	Varies	0.02	Shoulder
Principal arterial (divided or bi-directional)	Design speed < 45 mph	0.1	Shoulder + 3 ft
	Design speed > 45 mph	0.1	Shoulder
	Sag vertical curve	0.02	Shoulder + 3 ft
Major or minor collector	Design speed < 45 mph	0.1	1/2 Driving Lane
	Design speed > 45 mph	0.1	Shoulder
	Sag vertical curve	0.1	1/2 Driving Lane
Local streets	Low ADT	0.2	1/2 Driving Lane
	High ADT	0.1	1/2 Driving Lane
	Sag Point	0.1	1/2 Driving Lane

5.1.2 Check Storm Frequency and Spread

Practitioners typically use a check storm any time runoff could cause severe flooding during less frequent (larger) events. Where ponding to undesirable depths could occur, as when a series of inlets terminates at a sag vertical curve, the designer checks the performance of gutters and inlets with an event more severe than the design event.

Designers base selection of the frequency for the check storm on the same considerations used to select the design storm, i.e., the consequences of spread exceeding that chosen for design and the potential for ponding. Where no significant ponding can occur, check storms are typically unnecessary. During the service life of a roadway, there is always a risk that a storm more severe than the design and check storm will occur once or multiple times. When the consequences of such events are potentially unacceptable, a designer may employ risk-based design concepts (FHWA 2016). With risk-based design, the designer evaluates the consequences of design flow exceedance and balances the number of occurrences of the consequences against cost and other goals of society.

Each State and locality determine standards for the criteria for spread during the check event. Examples of criteria for a check storm event include: 1) one lane open to traffic, and 2) one lane free of water. In the first, water may partially of fully cover the lane but at a depth that drivers can drive through at a reduced speed.

5.2 Surface Drainage

To facilitate surface drainage, designers avoid flat roadway surfaces. Slope may be parallel to the roadway, transverse to it, or both. The slope of the roadway in a longitudinal direction is the longitudinal grade. Designers refer to the overall vertical alignment of the roadway, including tangent grades and vertical curves, as the profile grade line. The longitudinal grade may represent the slope of the profile grade line, or along a designated offset, such as a gutter grade. Transverse slope is referred to as the “cross slope.”

When rain falls on a sloped pavement surface, it forms a thin film of water that increases in depth as it flows downgrade. Factors influencing the depth of water on the pavement include the length of the flow path, the texture of the pavement surface, the surface slope, and rainfall intensity. As the depth of water on the pavement increases, the potential for hydroplaning increases. Anderson et al. (1995) provides additional technical information on the mechanics of surface drainage. This section describes the following surface drainage topics:

- Hydroplaning.
- Longitudinal pavement slope.
- Cross or transverse pavement slope.
- Surface conveyance.
- Superelevation transition.
- Other surface features affecting drainage.

5.2.1 Hydroplaning

When a rolling vehicle tire meets a film of water on the roadway, it has the potential to lose contact with the roadway, a condition known as hydroplaning. Ideally, the water is channeled through the tire tread pattern and through the surface roughness of the pavement. Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is insufficient to conduct the water away, and the water begins to build up in front of the tire. As this wedge of water builds up in front of the tire, the wedge produces a force that can lift the tire off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers no shear resistance, the tire loses its ability to exert stopping or steering force on the vehicle. The driver has then lost control of the vehicle. Hydroplaning can occur at speeds of 55 mph with a water depth of 0.08 inches (Anderson et al. 1995). However, depending on a variety of criteria, hydroplaning may occur at lower speeds and depths.

Hydroplaning is a function of the water depth, roadway geometry, vehicle speed, tread depth, tire inflation pressure, and pavement texture. The AASHTO Model Drainage Manual (AASHTO 2000) provides methods of calculating when it can occur. In problem areas, hydroplaning hazard may be reduced by:

- Designing the highway geometry to reduce the drainage path lengths of flow over the pavement to prevent flow buildup.
- Increasing the pavement surface texture depth. Grooving of concrete pavement and the use of asphalt-aggregate surface treatment (chip seal) are examples of surfaces that inhibit hydroplaning. An increase of pavement surface texture will increase the drainage capacity at the tire pavement interface.
- Using open graded asphaltic pavements. The open texture prevents the buildup of hydrostatic pressure and the corresponding lifting force by allowing the water to be forced out from under the tire.
- Adding drainage structures to capture water from the pavement to reduce the film of water and reduce the hydroplaning potential.

NASEM (2021) provides an in-depth discussion of hydroplaning and the related variables.

5.2.2 Longitudinal Grade

The AASHTO Policy on Geometric Design (AASHTO 2018) provides suggested minimum roadway longitudinal grades for safe pavement drainage on tangent grade sections. Based on its citation in 23 CFR 625.4, this Policy contains specific criteria and controls for the design of NHS projects. [See 23 CFR 625.3(b) and 625.4(a)]. In addition, to create safe roadways designers consider the following general statements:

- A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to a spread problem if vegetation builds up along the pavement edge.
- Desirable gutter grades are greater than or equal to 0.5 percent for curbed pavements with an absolute minimum of 0.3 percent. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles (TxDOT 2019).
- To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent is maintained within 50 ft of the low point of the curve. At the low point, the grade passes through zero. This is accomplished when the vertical curve constant, K, is equal to or less than 167 (50 in SI).

The vertical curve constant, K, is:

$$K = L / (G_2 - G_1) \tag{5.1}$$

where:

K	=	Vertical curve constant ft/percent (m/percent)
L	=	Horizontal length of curve, ft (m)
G_i	=	Grade of roadway, percent

Lessons from Experience: Avoiding Slippery Slopes

Designers may benefit from considering these lessons for specific situations:

- On highways where three or more lanes are sloped in the same direction, increase the cross slope of the lowest lanes. For example, slope the two lanes adjacent to the crown line at typical slope and increase the successive lanes, or portions of them, by 0.5 to 1 percent per lane, but not more than a total cross slope of 4 percent.
- When depressed medians are present, slope inside lanes toward the median if conditions warrant.
- Avoid introducing water from median areas to the travel lanes.
- Increase cross slopes in vertical curves where grade passes through zero and in flat sections where it is small.
- Slope shoulders to drain away from the pavement, except with raised, narrow medians and in superelevated curves. Shoulders with a greater cross slope than the roadway assist in reducing the depth of water in the travel lanes.

5.2.3 Cross (Transverse) Slope

The AASHTO Policy on Geometric Design of Highways and Streets specifies an acceptable range of cross slopes (AASHTO 2018). [See 23 CFR 625.3(b) and 625.4(a)]. Summarized in Table 5.2, these cross slopes balance the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort and safety. Cross slopes of 2 percent or less have little effect on driver effort in steering or on friction demand for vehicle stability (Gallaway et al. 1979). In areas of intense rainfall, a somewhat steeper cross slope (2.5 percent) may be used to facilitate drainage. When a designer considers deviating from values in Table 5.2, the AASHTO policy provides other information.

Table 5.2. Typical pavement cross slopes.

Surface Type *	Range in Rate of Surface Slope, ft/ft (m/m)
High-type surface (2-lanes)	0.015 - 0.020
High-type surface (3 or more lanes, each direction)	0.015 – 0.04 (increase 0.005 to 0.010 per lane)
Intermediate-type surface	0.015 - 0.030
Low-type surface	0.020 - 0.060
Bituminous or concrete shoulders	0.020 - 0.060
Shoulders with curbs	≥ 0.040

*High-, intermediate-, and low-type surfaces describe the relative heights of skid-resisting surface textural elements.

5.2.4 Surface Conveyance

Surface drainage includes conveyance features adjacent to the paved roadway that collects water from the pavement and carries it parallel to the roadway, although some surface drainage features may flow skewed from or perpendicular to the roadway. Parallel features include curb and gutter sections and median and roadside channels.

5.2.4.1 Curb and Gutter

Designers typically use curbs at the outside edge of pavements for low-speed highway facilities, and in some instances, adjacent to shoulders on moderate to high-speed facilities. Curbs serve several purposes:

- Contain surface runoff within the roadway and away from adjacent properties.
- Prevent erosion on fill slopes.
- Provide pavement delineation.
- Enable orderly development of adjacent property.
- Redirect errant vehicles back into the roadway.

Gutters formed in combination with curbs come in various widths. Gutters often have the same cross slopes as that of the pavement or they may have a steeper cross slope than the shoulder or adjacent lane (if present).

For stormwater runoff from cut slopes and other areas that drain toward the roadway, designers provide alternative drainage paths, where practical, to prevent such flow from entering the roadway. Where curbs are not needed for traffic purposes, shallow ditches at the edge of pavement offer advantages over curbed sections and provide channel capacity that is not dependent on spread on the pavement. Section 5.3 provides a detailed discussion of curb and gutter sections.

5.2.4.2 Roadside and Median Channels

Designers commonly use roadside channels with uncurbed roadway sections to convey runoff from the highway pavement and adjacent areas. Right-of-way limitations and access requirements often preclude the use of roadside channels on urban arterials. Designers typically use roadside channels in depressed roadway sections, and other locations with sufficient ROW and where driveways or intersections are infrequent.

To prevent runoff from median areas from running across the travel lanes, designers slope median areas and inside shoulders to a center swale. This design option is particularly useful for high-speed facilities and for facilities with more than two lanes of traffic in each direction. Chapter 6 provides a detailed discussion of roadside and median channels.

5.2.5 Other Surface Features Affecting Drainage

Other roadway surface features affect roadway drainage. These include bridge decks, median barriers, superelevated roadways, roundabouts, and porous pavements.

5.2.5.1 Bridge Decks

Design of bridge deck drainage addresses several challenges unique to the bridge environment in providing for and maintaining adequate drainage systems:

- Deck structural and reinforcing steel tends to corrode from deicing chemicals, so that designs focus on draining treated water to slow the corrosion.

- Water on bridge decks freezes before surface roadways leading to ice-covered bridges.
- Bridge deck drainage gutters may have lower capacity than roadway gutters because cross slopes may be flatter.
- Parapet-type bridge rails can accumulate roadway debris.
- Scuppers and other deck drains generally have smaller open areas than many roadway inlets and can be easily clogged by debris.
- Bridges lack auxiliary lanes or additional width beyond the travel lanes.
- Bridges over water may be subject to water quality restrictions limiting direct discharge to the water body.
- Bridges crossing over land, including other roadways or bridges, may have to convey water to a safe discharge location.

To address these difficulties in providing for deck drainage systems, designers focus on intercepting and redirecting gutter flow from the roadway leading to a bridge before it reaches a bridge whenever possible. For similar reasons, designers avoid zero longitudinal grades and sag vertical curves on bridges. HEC-21 includes detailed coverage of bridge deck drainage systems (FHWA 1993).

5.2.5.2 Median Barriers

When designing shoulder areas adjacent to median barriers, designers typically slope them toward the center to prevent drainage from running across the traveled pavement. Where designers use median barriers, they provide inlets or slotted drains to collect water accumulated against barriers at low points or where spread exceeds allowable on grade. For superelevated curves, designers provide inlets to collect water before it is redirected back to the roadway by the superelevated roadway geometry.

5.2.5.3 Superelevated Horizontal Curves

Designers can accomplish changes to the roadway direction in the horizontal plane by using circular curves. In many cases, horizontal curves involve super elevating the roadway to facilitate the safe passage of vehicles through a change in direction as shown in Figure 5.1. Superelevation is the transverse slope provided to reduce the tendency of a vehicle to overturn or to skid laterally outwards by raising the pavement outer edge with respect to inner edge.

As shown in Figure 5.1, the designer provides superelevation by steadily reducing the cross slope on the outer side of the curve until it is zero, and then steadily sloping the outer side of the roadway toward the inner side, until the inner side cross slope is continuous across the entire roadway. The designer steadily rotates the entire roadway further toward the inner side, until reaching the desired cross slope. The full superelevation cross slope depends on design speed and radius of curvature. State DOT design manuals provide maximum recommended values, for example, 8 percent in temperate areas where snow and ice are uncommon and 6 percent in areas where snow and ice are more common. Roadway designers commonly begin the transition from normal crown to superelevated section before the beginning of the horizontal curve, achieve full superelevation within the curve, and transition from superelevated back to normal crown.

Superelevation transitions present several challenges for drainage design:

- The increase in contributing pavement runoff when all lanes drain to the same side.
- The flat cross slope during the transition.
- Potential low or flat slope slopes in the longitudinal grade of the roadway gutter.

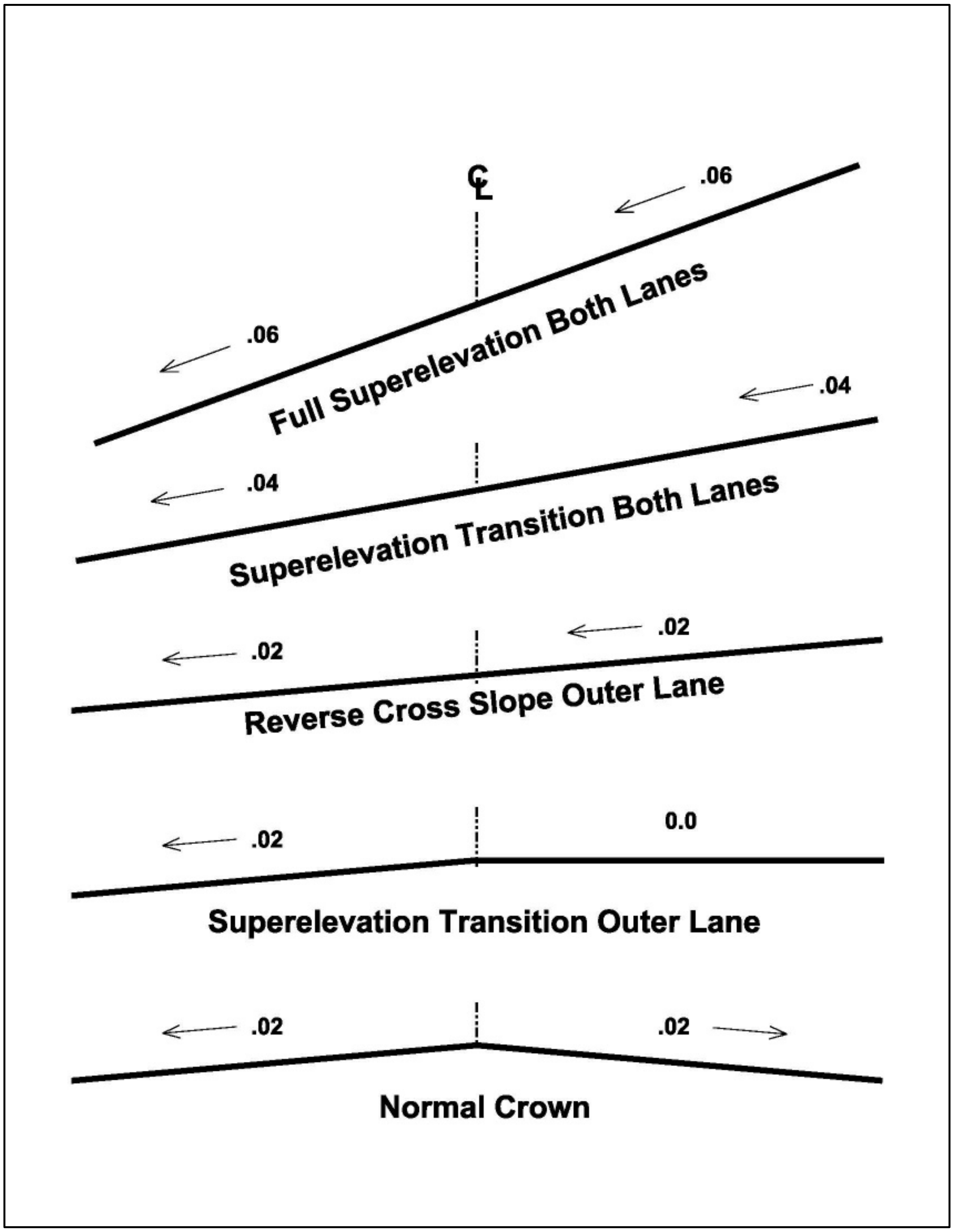

Figure 5.1 Transition from normal crown to superelevated curve to the left.

Fully superelevated sections present pavement drainage challenges by increasing the contributing drainage area to one side. The entire roadway section drains a single direction, rather than each half of the roadway draining toward its edge. The increased drainage length may be partially offset by increased cross slope in some cases. In addition to increasing the width of roadway draining to the edge, 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 depth of surface flow.

The section of roadway where the cross slope in the right (outer) lane passes through zero is important for drainage design because ponding can occur and available head for inlets to remove water would also be zero. Flow in the roadway is then governed by longitudinal grade only. If there is no longitudinal grade in the same location as a zero cross slope water can build up on the pavement in that location. Designers seek to avoid conditions where water is directed across the roadway from the outer side of the curve to the inner side.

The drainage designer also considers the relative changes in the roadway profile grade and the gutter profile grade as these do not change at the same rates 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).

In superelevation transitions, the roadway design may rotate the roadway cross-section about the centerline, inner edge, or outer edge. If rotated about the centerline or outer edge, the gutter profile on the inner side of the curve may be depressed by the increase of cross slope past the normal crown slope at full superelevation. Combining the increased drainage distance of a superelevated section and a depressed gutter profile grade line can result in significant increase in spread, and in depth of runoff on the inner side of curves. If the roadway is rotated about either edge, the centerline profile grade line will show a “hump” or “dip” to reflect the transition and curve that may affect drainage.

Because of the many possible situations for superelevation transitions and the importance of good drainage for safe roadways, drainage and roadway geometry designers coordinate to minimize problem areas created by superelevated curves and transitions. They consider the profile grade line, both 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.

5.2.5.4 Roundabouts

Designers increasingly use circular “roundabout” intersections as a traffic management technique. Roundabouts can present unique drainage situations. The profile grade lines of the roadways converging or crossing at a roundabout are adjusted to intersect at a common elevation at the center of the roundabout. Traffic turns to the right to enter the roundabout, curves to the left while traveling in the roundabout, turns to the right to exit the roundabout. Traffic proceeds through roundabouts at speeds lower than the approaching roadway sections. The lower speed facilitates the “reversing” motion of entering, transiting, and leaving the roundabout.

Typically, the central island of a roundabout is elevated, and roadways approaching a roundabout have a center crown with cross slopes of 0.02. The cross slope on the roundabout circle is an adverse superelevation (sloping from the outside of the curve rather than the inside) throughout (NASEM 2007, NASEM 2010). Researchers recommend that the central island be elevated in all classifications of roundabouts (mini, single-lane, and multi-lane). If the central island is not elevated, cross slope toward the central island will create ponding near the island, and drainage structures will be needed to alleviate the ponding (NASEM 2010). Multi-lane roundabouts, like other multi-lane features, exhibit wider pavement sections, and longer accumulation distances for runoff, than do single-lane features.

The adverse cross slope within the roundabout directs water toward the outside of the roundabout. Because the outer edge of the roadway within the roundabout has a larger circumference than the inner edge, as water flows toward the outer edge of the roundabout the increase of depth resulting from increasing pavement area is mitigated by the larger circumference.

Among the intended purposes of roundabouts is to limit traffic speed; because of this, they are inherently low-speed features. The tendency to limit speed also serves to mitigate hydroplaning, even in cases where water ponds on the pavement in depths greater than would be desirable on other roadway features.

Gutter design at the outer edge of the roundabout determines the spread conditions. The gutters either direct water to inlets within the roundabout or to gutters on roadways connected to the roundabout. Because roundabouts direct water to the periphery and slow traffic speeds, roundabouts can reduce the potential for hydroplaning compared to other intersection configurations.

5.2.5.5 Porous Pavement

Primarily to improve stormwater runoff quality, roadway designers sometimes use porous or permeable pavements that allow stormwater into the pavement so that it slowly runs through the pavement rather than quickly off over the pavement surface. Porous pavements do not generally affect stormwater runoff quantities for moderate to large design events because the quantity of water redirected through the pavement is small. However, there may be some quantity effect for smaller storms (Harvey and Smith 2018).

5.3 Flow in Gutters

A pavement gutter is a structure at the edge of the roadway that conveys water during a storm runoff event. It may include a portion or all of a travel lane. Figure 5.2 illustrates typical curb and gutter and shallow swale sections. Conventional curb and gutter sections commonly have a triangular shape with the curb forming the near-vertical leg of the triangle. Conventional gutters typically have a uniform cross slope (Figure 5.2, a.1), a composite cross slope (Figure 5.2, a.2), or a parabolic section (Figure 5.2, a.3). Shallow swale gutters typically have V-shaped or circular sections as illustrated in Figure 5.2 and are often used in paved median areas.

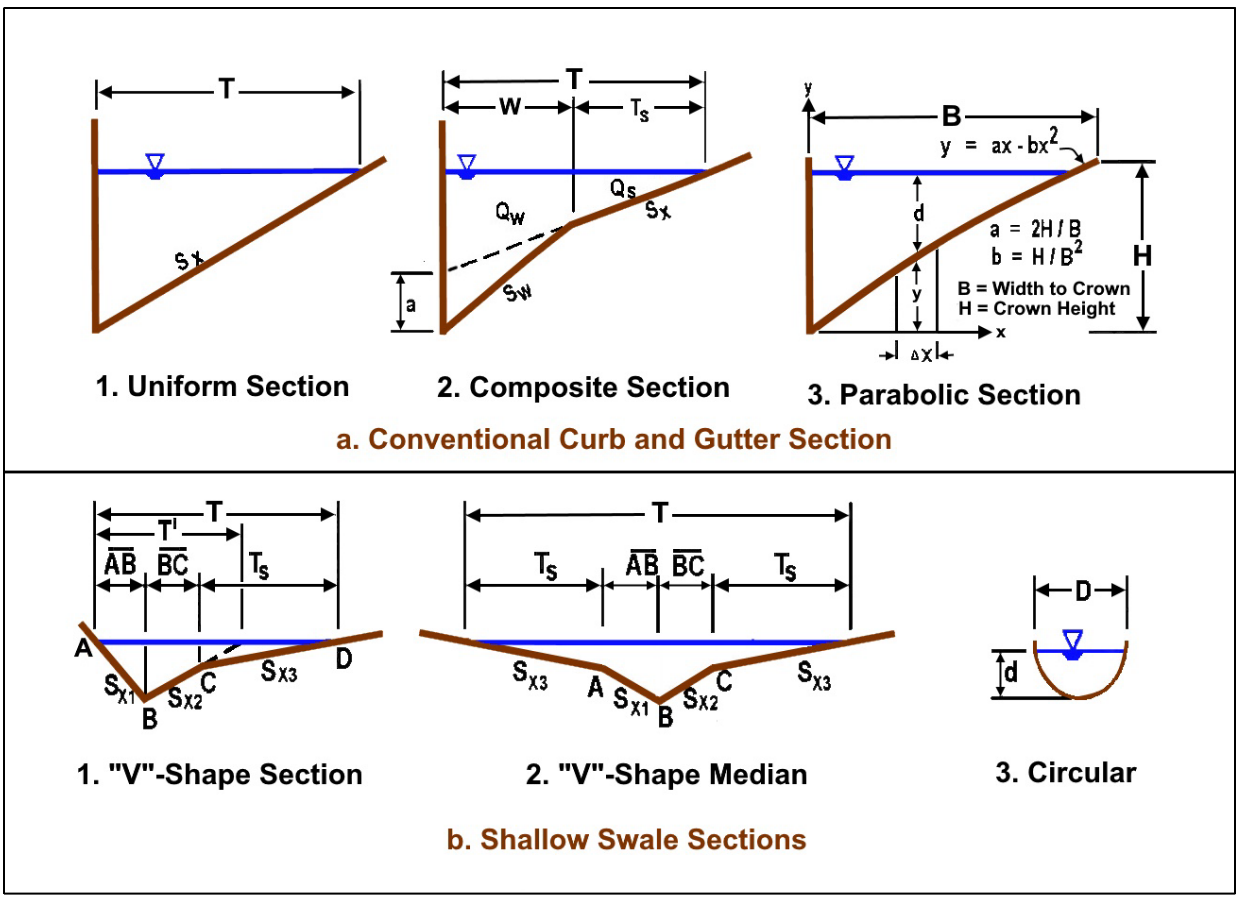

Figure 5.2. Typical stormwater conveyance sections.

5.3.1 Capacity Relationship

Gutter flow calculations estimate the spread of water on the shoulder, parking lane, or pavement section. Izzard stated that the hydraulic radius used in Manning's equation does not adequately describe the gutter cross-section, particularly where the top width of the water surface may be more than 40 times the depth at the curb, and modified Manning's equation for gutter flow (Izzard 1946):

$$Q = \frac{K_u}{n} S_x^{1.67} S_L^{0.5} T^{2.67} \tag{5.2}$$

where:

K_u	=	Unit conversion constant, 0.56 in CU (0.376 in SI)
n	=	Manning's coefficient
Q	=	Flow rate, ft^3/s (m^3/s)
T	=	Width of flow (spread), ft (m)
S_x	=	Cross slope, ft/ft (m/m)
S_L	=	Longitudinal grade, ft/ft (m/m)

Equation 5.2 neglects the resistance of the curb face since this resistance is negligible. Table 5.3 summarizes typical Manning's coefficients for paved gutters. In terms of velocity, the relationship is:

$$V = \frac{2K_u}{n} S_x^{0.67} S_L^{0.5} T^{0.67} \tag{5.3}$$

Designers often use spread on the pavement and flow depth at the curb as criteria for spacing pavement drainage inlets. Equation 5.2 can be solved in terms of T to estimate a spread width given a flow rate:

$$T = \left[\frac{Q n}{K_u S_x^{1.67} S_L^{0.5}} \right]^{0.375} \tag{5.4}$$

Table 5.3. Manning's n for street and pavement gutters (FHWA 1977).

Type of Gutter or Pavement	Manning's n *
Concrete gutter, troweled finish	0.012
Asphalt pavement: smooth texture	0.013
Asphalt pavement: rough texture	0.016
Asphalt pavement: smooth texture with concrete gutter	0.013
Asphalt pavement: rough texture with concrete gutter	0.015
Concrete pavement: float finished	0.014
Concrete pavement: broom finished	0.016

*For gutters with small slope, where sediment may accumulate, increase "n" by 0.002.

5.3.2 Conventional Curb and Gutter Sections

Conventional curb and gutter sections include a very short wall-like curb with a near-vertical face, and a toe section sloped upward to form the invert of the gutter. Gutters begin at the inside base of the curb and usually extend from the curb face toward the roadway centerline 1.0 to 3.0 ft. As illustrated in Figure 5.2, gutters can have uniform, composite, or curved sections; most commonly, they are uniform.

5.3.2.1 Uniform Cross Slope

Uniform gutter sections have a cross slope equal to the cross slope of the shoulder or travel lane adjacent to the gutter. Equations 5.2 and 5.3 describe the gutter flow characteristics in a uniform triangular gutter as illustrated in the following example.

Example 5.1:	Computation of triangular gutter flow.
Objective:	Estimate the spread given a flow rate of 1.8 ft^3/s (0.051 m^3/s) and find the gutter flow given a spread of 8.2 ft (2.5 m).

Given: Gutter section illustrated in Figure 5.2 (a.1).

S_L = 0.010 ft/ft (m/m)
S_x = 0.020 ft/ft (m/m)
n = 0.016

Step 1. Compute spread, T, using equation 5.4.

$T = [(Q\ n)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375} = [(1.8)(0.016)/\{(0.56)(0.020)^{1.67}(0.010)^{0.5}\}]^{0.375} = 9.0$ ft

Step 2. Compute Q using equation 5.2.

$Q = (K_u/n)\ S_x^{1.67}\ S_L^{0.5}\ T^{2.67} = (0.56/0.016))(0.020)^{1.67}(0.010)^{0.5}\ (8.2)^{2.67} = 1.4\ ft^3\ /s$

Solution:	The spread given a flow rate of 1.8 ft^3/s (0.051 m^3/s) is 9.0 ft (2.7 m). The gutter flow given a spread of 8.2 ft (2.5 m) is 1.4 ft^3/s (0.040 m^3/s).

5.3.2.2 Composite Gutters

Gutters with composite sections are depressed in relation to the adjacent pavement slope. Most commonly, designers use gutters with composite sections as the flow approaches a storm drain inlet. Depressed gutters can present a hazard to bicycle traffic.

Because the cross-section is no longer a simple triangle, the total flow is conceptually divided between the flow in the depressed section, Q_w, and the flow in the side section, Q_s. Based on the composite channel geometry, the fraction of the flow in the depressed and side sections are represented as:

$$Q_w = Q\ E_o \tag{5.5}$$

$$Q_s = Q\ (1-E_o) \tag{5.6}$$

where:

Q	=	Total gutter flow rate, ft^3/s (m^3/s)
Q_w	=	Flow in the depressed section of the gutter, ft^3/s (m^3/s)
Q_s	=	Flow in the gutter section above the depressed section, ft^3/s (m^3/s)
E_o	=	Ratio of flow in the depressed section (usually the width of a grate) to total gutter flow

The ratio of flow in the depressed section to the total flow is:

$$E_o = \frac{1}{1 + \dfrac{S_w / S_x}{\left(1 + \dfrac{S_w / S_x}{(T/W) - 1}\right)^{2.67} - 1}} \quad (5.7)$$

where:

S_w	=	Cross slope in the depressed section, ft/ft (m/m)

As shown in Figure 5.2 (a.2) the depressed section cross slope is:

$$S_w = S_x + \frac{a}{W} \quad (5.8)$$

The following example demonstrates computing flow and spread in a composite gutter.

Example 5.2: Composite gutter flow.

Objective: Estimate: A) the flow in a composite gutter at a spread of 8.2 ft (2.5 m) and B) the spread in the gutter at a flow of 4.2 ft^3/s (0.12 m^3/s).

Given: Gutter section illustrated in Figure 5.2 (a.2) with:

W	=	2 ft (0.61 m)
S_L	=	0.01 ft/ft (m/m)
S_x	=	0.02 ft/ft (m/m)
n	=	0.016
a	=	2 inches (51 mm) (gutter depression)

Step A1. Compute the cross slope of the depressed gutter, S_w, and the width of spread from the junction of the gutter and the road to the limit of the spread, T_s.

S_w = a / W + S_x = [(2)/(12)]/(2) + (0.020) = 0.103 ft/ft

T_s = T - W = 8.2 - 2.0 = 6.2 ft

Step A2. Compute Q_s from equation 5.2 using T_s.

$Q_s = (K_u/n)\, S_x^{1.67}\, S_L^{0.5}\, T_s^{2.67} = (0.56/0.016)\,(0.02)^{1.67}\,(0.01)^{0.5}\,(6.2)^{2.67} = 0.66\ ft^3/s$

Step A3. Determine the gutter flow, Q.

T / W = 8.2 / 2 = 4.10

S_w / S_x = 0.103 / 0.020 = 5.15

Using equation 5.7:

$$E_o = \frac{1}{1 + \dfrac{S_w / S_x}{\left(1 + \dfrac{S_w / S_x}{(T/W) - 1}\right)^{2.67} - 1}} = \frac{1}{1 + \dfrac{5.15}{\left(1 + \dfrac{5.15}{(4.10) - 1}\right)^{2.67} - 1}} = 0.7$$

Using equation 5.6:

$Q = Q_s / (1 - E_o) = 0.66 / (1 - 0.70) = 2.3$ ft^3/s

Step B1. Select an initial estimate of Q_s.

There is not a direct computational solution for estimating spread from flow in a composite gutter, therefore, an iterative approach is used. Select an initial estimate of $Q_s = 1.4$ ft^3/s.

Step B2. Compute Q_w.

$Q_w = Q - Q_s = 4.2 - 1.4 = 2.8$ ft^3/s

Step B3. Determine W/T ratio.

$E_o = Q_w / Q = 2.8 / 4.2 = 0.67$

$S_w / S_x = 0.103 / 0.020 = 5.15$

Solve for T/W using equation 5.7:

$$0.67 = \frac{1}{1 + \dfrac{5.15}{\left(1 + \dfrac{5.15}{(T/W) - 1}\right)^{2.67} - 1}}$$

T/W = 4.48

Step B4. Compute spread based on the assumed Q_s.

$T = W (T/W) = 2.0 (4.48) = 9.0$ ft

Step B5. Compute T_s.

$T_s = T - W = 9.0 - 2.0 = 7.0$ ft

Step B6. Use equation 5.2 to determine Q_s for computed T_s.

$Q_s = (K_u/n) S_x^{1.67} S_L^{0.5} T^{2.67} = (0.56/0.016) (0.02)^{1.67} (0.01)^{0.5} (7.0)^{2.67} = 0.92$ ft^3/s

Step B7. Compare computed Q_s with assumed Q_s.

Q_s assumed = 1.4 > 0.92, Computed Q_s not close to assumed. Try again.

Step B8. Try a new assumed Q_s and repeat steps B1 through B7.

Assume $Q_s = 1.85$ ft^3/s

$Q_w = 4.2 - 1.85 = 2.35$ ft^3/s

$E_o = Q_w / Q = 2.35 / 4.2 = 0.56$

$S_w / S_x = 5.15$

$T / W = 5.55$

$T = 2.0 (5.55) = 11.1$ ft

$T_s = 11.1 - 2.0 = 9.1$ ft

$Q_s = 1.85$ ft^3/s

Q_s assumed = Q_s computed. Computations completed.

Solution:	A) The estimated discharge of the gutter for the given spread is 2.3 ft^3/s (0.065 m^3/s). B) The estimated spread of the composite gutter for the given discharge is 11.1 ft (3.38 m).

5.3.2.3 Gutters with Curved Sections

Older city streets or highways with curved pavement sections sometimes have curved gutter sections. Where the pavement cross-section is curved, gutter capacity varies with the configuration of the pavement. For this reason, discharge-spread or discharge-depth-at-the-curb relationships developed for one pavement configuration do not apply to another section with a different crown height or half width. Appendix B includes procedures for developing conveyance curves for parabolic pavement sections.

5.3.3 Shallow Swale Sections

Where traffic control does not demand curbs, designers typically use a small swale section of circular or V-shape to convey runoff from the pavement. For example, designers use swales to convey runoff from pavement on fills to protect the embankment from erosion. Small swale sections may have sufficient capacity to convey the flow to a location suitable for interception.

5.3.3.1 V-Sections

Equation 5.2 can be used to compute the flow in a shallow V-shaped section by estimating the cross slope, S_x, using the following equation:

$$S_x = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \tag{5.9}$$

The following example demonstrates the analysis of a V-shaped shoulder gutter.

Example 5.3:	V-shaped roadside shoulder gutter.
Objective:	Find spread for a design flow of 1.77 ft^3/s (0.050 m^3/s).

Given: V-shaped roadside gutter (Figure 5.2 (b.1)) with:

S_L	=	0.01 ft/ft (m/m)
S_{x1}	=	0.25 ft/ft (m/m)
S_{x3}	=	0.02 ft/ft (m/m)
n	=	0.016
S_{x2}	=	0.04 ft/ft (m/m)
T_{BC}	=	2.0 ft (0.61 m)

Step 1. Calculate S_x using equation 5.8 assuming all flow is contained entirely in the V-shaped gutter section determined by S_{x1} and S_{x2}.

$S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2}) = (0.25) (0.04) / (0.25 + 0.04) = 0.0345$ ft/ft

Step 2. Using equation 5.4, find the hypothetical spread, T', assuming all flow contained entirely in the V-shaped gutter.

$T' = [(Q\ n)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375} = [(1.77)(0.016)/\{(0.56)(0.0345)^{1.67}(0.01)^{0.5}\}]^{0.375} = 6.4$ ft

Step 3. Determine if T' is within S_{x1} and S_{x2}.

$d_B = T_{BC}\ S_{x2} = (2)\ (0.04) = 0.08$ ft

$T_{AB} = d_B\ /\ S_{x1} = (0.08)\ /\ (0.25) = 0.32$ ft

$T_{AC} = T_{AB} + T_{BC} = 0.32 + 2.0 = 2.32$ ft

2.32 ft < T' therefore, spread falls outside V-shaped gutter section. An iterative solution technique is used to solve for the section spread, T, as illustrated in the following steps.

Step 4. Solve for the depth at point C, y_c, and compute an initial estimate of the spread along T_{BD}.

$y_C = d_B - T_{BC}\ (S_{x2})$

From the geometry of the triangle formed by the gutter, an initial estimate for d_B is determined as:

$(d_B\ /0.25) + (d_B\ /0.04) = 6.4$ ft

$d_B = 0.22$ ft

$y_c = 0.22 - (2.0)\ (0.04) = 0.14$ ft

$T_s = y_c\ /\ S_{x3} = 0.14/\ 0.02 = 7$ ft

$T_{BD} = T_s + T_{BC} = 7 + 2 = 9$ ft

Step 5. With T_{BD}, develop a weighted slope for S_{x2} and S_{x3}.

2.0 ft at S_{x2} (0.04) and 7.0 ft at S_{x3} (0.02)

[(2.0) (0.04) + (7.0) (0.02)] / 0.90 = 0.024 ft/ft

Use this slope along with S_{X1}, find S_x using equation 5.9:

$S_x = (S_{x1}\ S_{x2})\ /\ (S_{x1} + S_{x2}) = [(0.25)\ (0.024)]\ /\ (0.25 + 0.024) = 0.022$ ft/ft

Step 6. Using equation 5.2, compute the gutter spread using the composite cross slope, S_x.

$T = [(Q\ n)/(K_u\ S_x^{\ 1.67}\ S_L^{\ 0.5})]^{\ 0.375} = [(1.77)(0.016)/\{(0.56)(0.022)^{1.67}(0.01)^{0.5}\}]^{0.375} = 8.5$ ft

This 8.5 ft is lower than the assumed value of 9.0 ft.

Therefore, assume T_{BD} = 8.3 ft and repeat step 5 and step 6.

Step 5 (repeated). 2.0 ft at S_{x2} (0.04) and 6.3 ft at S_{x3} (0.02).

[(2.0) (0.04) + (6.3) (0.02)] / 8.30 = 0.0248

Use this slope along with S_{X1}, find S_x using equation 5.6:

$S_x = [(0.25)\ (0.0248)]\ /\ (0.25 + 0.0248) = 0.0226$ ft/ft

Step 6 (repeated). Using equation 5.2 compute the spread.

$T = [(Q\ n)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375} = [(1.77)(0.016)/\{(0.56)(0.0226)^{1.67}(0.01)^{0.5}\}]^{0.375} = 8.31$ ft

Solution: This value of T is close to the assumed value, therefore, OK.

The following example illustrates analysis of a V-shaped median gutter resulting from a roadway with an inverted crown section.

Example 5.4: V-shaped median shallow swale.

Objective: Find A) spread for the design flow of 24.7 ft^3/s (0.70 m^3/s) and B) compute the flow for a spread of 23.0 ft (7.0 m).

Given: V-shaped gutter as illustrated in Figure 5.2 (b.2) with:

T_{AB} = 3.28 ft (1 m)
T_{BC} = 3.28 ft (1 m)
S_L = 0.01 ft/ft (m/m)
n = 0.016
S_{x1} = 0.25 ft/ft (m/m)
S_{x2} = 0.25 ft/ft (m/m)
S_{x3} = 0.04 ft/ft (m/m)

Step A1. Assume spread remains within middle "V" (A to C) and compute S_x.

$S_x = (S_{x1}\ S_{x2}) / (S_{x1} + S_{x2}) = (0.25)\ (0.25) / (0.25 + 0.25) = 0.125$ ft/ft

Step A2. Compute the spread from equation 5.4.

$T = [(Q\ n)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375} = [(24.7)(0.016)/\{(0.56)\ (0.125)^{1.67}\ (0.01)^{0.5}\}]^{0.375} = 7.65$ ft

Since "T" is outside S_{x1} and S_{x2} an iterative approach is used to compute the spread.

Step A3. Treat one-half of the median gutter as a composite section and solve for T' equal to one-half of the total spread.

Q' for T' = ½ Q = 0.5 (24.7) = 12.4 ft^3/s

Step A4. Try $Q'_s = 1.8\ ft^3/s$.

$Q'_w = Q' - Q'_s = 12.4 - 1.8 = 10.6\ ft^3/s$

Step A5. Using equation 5.5, determine the W/T' ratio.

$E'_o = Q'_w/Q' = 10.6/12.4 = 0.85$

$S_w / S_x = S_{x2} / S_{x3} = 0.25 / 0.04 = 6.25$

W/T' = 0.33 from trial-and-error

Step A6. Compute spread based on assumed Q'_s.

T' = W / (W/T') = 3.28 / 0.33 = 9.94 ft

Step A7. Compute T_s based on assumed Q'_s.

T_s = T' - W = 9.94 - 3.28 = 6.66 ft

Step A8. Use equation 5.2 to determine Q'_s for T_s.

$Q'_s = (K_u/n)\ S_{x3}^{1.67}\ S_L^{0.5}\ T_s^{2.67} = (0.56/0.016)\ (0.04)^{1.67}\ (0.01)^{0.5}\ (6.66)^{2.67} = = 2.56\ ft^3/s$

Step A9. Check computed Q'_s with assumed Q'_s.

Q'_s assumed = 1.8 < 2.56 = Q'_s computed

Therefore, try a new assumed Q'_s and repeat steps 4 through 9.

Assume $Q'_s = 0.04$

$Q'_w = 12.0\ ft^3/s$

$E'_o = 0.97$

$S_w/S_x = 6.25$

$W/T' = 0.50$ by iteration, as in example 5.2 (step 6)

$T' = 6.56$ ft

$T_s = 1.0$ ft

$Q_s = 0.39\ ft^3/s$

Q_s computed = 0.39 close to 0.40 = Q_s assumed, therefore OK.

$T = 2\ T' = 2\ (6.56) = 13.12$ ft

Step B1. Compute half-section top width.

Analyze in half-section using composite section techniques. Double the computed half width flow rate to get the total discharge:

$T' = T/2 = 23 / 2 = 11.5$ ft

$T_s = T' - 3.28 = 8.22$ ft

Step B2. Determine Q_s.

Using equation 5.2:

$Q_s = (K_u/n)\ S_x^{1.67}\ S_L^{0.5}\ T_s^{\ 2.67} = (0.56/0.016)\ (0.04)^{1.67}\ (0.01)^{0.5}\ (8.22)^{2.67} = 4.56\ ft^3/s$

Step B3. Determine flow in half-section.

$T'/W = 11.5/ 3.28 = 3.5\ 1$

$S_w / S_x = 0.25 / 0.04 = 6.25$

$E_o = 1 / \{1 + (S_w/S_x) / [(1 + (S_w/S_x) / (T'/W - 1))^{2.67} - 1]\}$

$= 1 / \{1 + (6.25) / [(1 + (6.25) / (3.5 - 3.28))^{2.67} - 1]\}$

$= 0.814 = Q'_w / Q = 1 - Q'_s / Q'$

$Q' = Q'_s / (1 - 0.814) = 4.56 / (1 - 0.814) = 24.5\ ft^3/s$

$Q = 2\ Q' = 2\ (24.5) = 49\ ft^3/s$

Solution: A: Estimated spread is 13.12 ft (4.0 m). B: Estimated flow is 49 ft^3/s (1.4 m^3/s).

5.3.3.2 Circular Sections

Flow in shallow circular gutter sections can be represented by the relationship:

$$\frac{d}{D} = K_u \left[(Qn) / \left(D^{2.67} S_L^{0.5} \right) \right]^{0.488} \qquad (5.10)$$

where:

d	=	Depth of flow in circular gutter, ft (m)
D	=	Diameter of circular gutter, ft (m)
K_u	=	Unit conversion constant, 0.972 in CU (1.179 in SI)

The width of circular gutter section T_w is represented by the chord of the arc which can be computed using:

$$T_W = 2\left(r^2 - (r-d)^2\right)^{0.5} \quad (5.11)$$

where:

T_w	=	Width of circular gutter section, ft (m)
r	=	Radius of circular gutter, ft (m)

Example 5.5:	Circular channels.
Objective:	Find flow depth and top width of a circular gutter.

Given: A circular gutter swale as illustrated in Figure 5.2, b.3 with:

D	=	4.92 ft (1.5 m)
S_L	=	0.01 ft/ft (m/m)
n	=	0.016
Q	=	17.6 ft^3/s (0.50 m^3/s)

Step 1. Determine flow depth.

Use equation 5.10:

$d/D = K_u [(Q\ n)/ (D^{2.67} S_L^{0.5})]^{0.488} = (0.972) [((17.6) (0.016))/ ((4.92)^{2.67} (0.01^{0.5})]^{0.488} = 0.20$

$d = D\ (d/D) = 4.92(0.20) = 0.98$ ft

Step 2. Using equation 5.11, determine T_w.

$T_w = 2 [r^2 - (r - d)^2]^{1/2} = 2 [(2.46)^2 - (2.46 - 0.98)^2]^{1/2} = 3.93$ ft

Solution:	Estimated depth is equal to 0.98 ft (0.3 m) and estimated top width is 3.93 ft (1.2 m).

5.3.4 Flow in Sag Vertical Curves

As gutter flow approaches the low point in a sag vertical curve, the flow can exceed the allowable design spread values because of the continually decreasing gutter slope. Check the spread in these areas to ensure it remains within allowable limits. If the computed spread exceeds design values, additional inlets can be provided to reduce the flow as it approaches the low point. Chapter 7 discusses sag vertical curves and measures for reducing spread in more detail.

In vertical curve cases where a negative grade goes to a lesser (flatter) grade, spread may increase because conveyance in the gutter decreases.

5.3.5 Relative Flow Capacities

Examples 5.1 and 5.2 illustrate the advantage of a composite gutter section. The capacity of the section with a depressed gutter in the examples is 70 percent greater than that of the section with a straight cross slope with all other parameters held constant.

Equation 5.2 can be used to examine the relative effects of changing the values of spread, cross slope, and longitudinal slope on the capacity of a section with a straight cross slope.

To examine the effects of cross slope on gutter capacity, equation 5.2 can be transformed as follows into a relationship between S_x and Q. Let:

$K_1 = n / (K_u S_L{}^{0.5} T^{2.67})$

Then,

$S_x{}^{1.67} = K_1 Q$

and

$$\left(\frac{S_{x1}}{S_{x2}}\right)^{1.67} = \frac{K_1 Q_1}{K_2 Q_2} = \frac{Q_1}{Q_2} \tag{5.12}$$

Similar transformations can be performed to evaluate the effects of changing longitudinal slope and width of spread on gutter capacity resulting in:

$$\left(\frac{S_{L1}}{S_{L2}}\right)^{0.5} = \frac{Q_1}{Q_2} \tag{5.13}$$

$$\left(\frac{T_1}{T_2}\right)^{2.67} = \frac{Q_1}{Q_2} \tag{5.14}$$

Figure 5.3 illustrates that the effects of cross slope are also relatively significant. At a cross slope of 4 percent, a gutter has 10 times the capacity of a gutter of 1 percent cross slope. A gutter at 4 percent cross slope has 3.2 times the capacity of a gutter at 2 percent cross slope. Designers generally have little latitude to vary longitudinal slope to increase gutter capacity but slope changes, which change gutter capacity, are frequent.

5.3.6 Gutter Flow Time

The flow time in gutters is an important component of the time of concentration for the contributing drainage area to an inlet. To find the gutter flow component of the time of concentration, a method for estimating the average velocity in a reach of gutter is needed. The velocity in a gutter varies with the flow rate and the flow rate varies with the distance along the gutter, i.e., both the velocity and flow rate in a gutter are spatially varied. Designers can estimate the time of flow using an average velocity obtained by integration of the Manning's equation for the gutter section with respect to time (see Appendix B for the derivation):

$$V_a = K_u\ K_G \left[\left(T_2^{2.67} - T_1^{2.67}\right) / \left(T_2^2 - T_1^2\right)\right] \tag{5.15}$$

where:

V_a	=	Average velocity in the gutter section between T_1 and T_2 locations, ft/s (m/s)
T_1	=	Upstream spread, ft (m)
T_2	=	Downstream spread, ft (m)
K_G	=	Gutter geometry parameter
K_u	=	Unit conversion constant, 0.84 in CU (0.564 in SI)

The gutter geometry parameter is:

$$K_G = (S_L^{0.5}\ S_X^{0.67}) / n \tag{5.16}$$

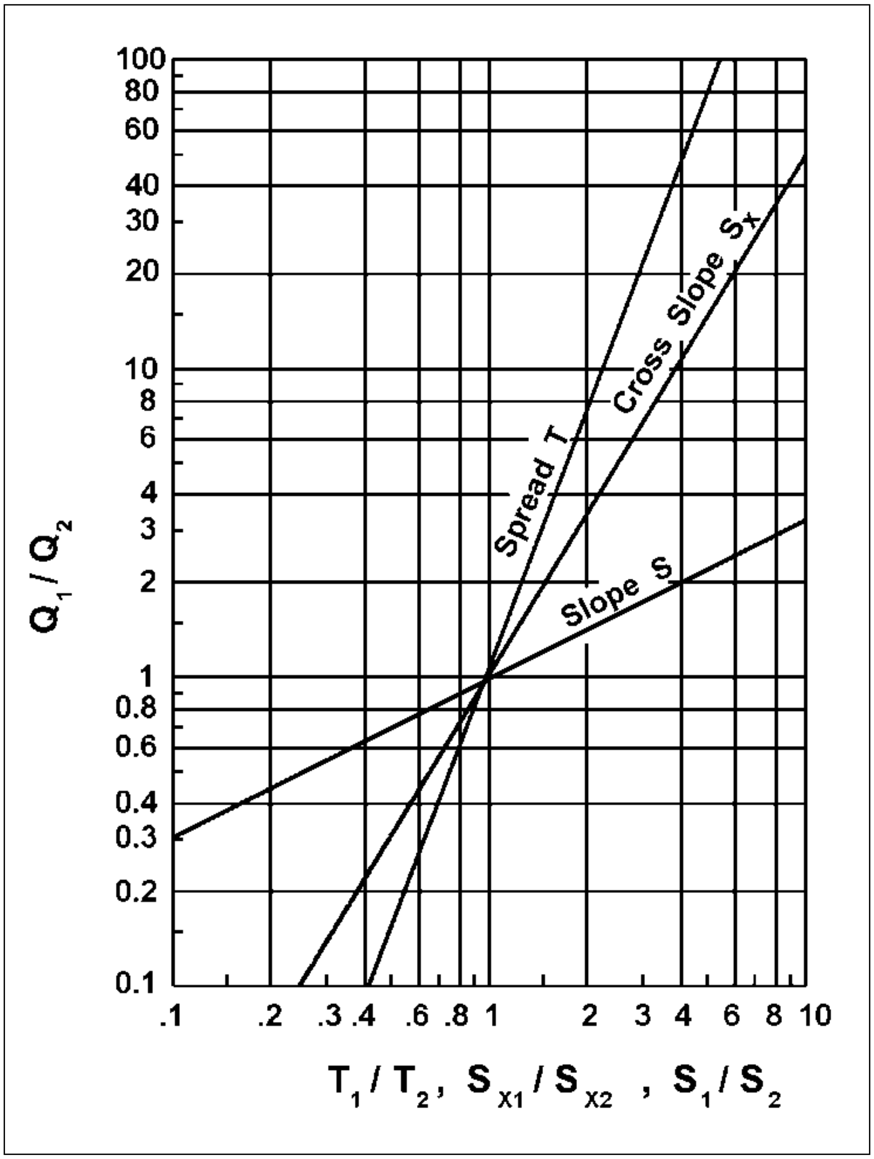

Figure 5.3. Relative effects of spread, cross slope, and longitudinal slope on gutter capacity.

Example 5.6:	Gutter flow time.
Objective:	Find the travel time in the gutter for an inlet spacing of 330 ft (100 m).

Given: A triangular gutter section:

T_1	=	3.28 ft (1.0 m)
T_2	=	9.84 ft (3.0 m)
S_L	=	0.03 ft/ft (m/m)
S_x	=	0.02 ft/ft (m/m)
n	=	0.016

Step 1. Compute the gutter geometry parameter.

Use equation 5.16:

$K_G = (S_L^{0.5} S_x^{0.67}) / n = (0.03^{0.5}\ 0.02^{0.67}) / 0.016 = 0.79$

Step 2. Estimate the average velocity in the gutter.

Use equation 5.15:

$V_a = K_u K_G [(T_2^{2.67} - T_1^{2.67}) / (T_2^2 - T_1^{2.})] = (0.83)(0.79)[((9.84)^{2.67} - (3.28)^{2.67}) / ((9.84)^2 - (3.28)^{2.})]$

$= 3.22$ ft/s

Step 3. Estimate the travel time in the gutter.

$t = L/V = 330 / 3.22 / 60 = 1.7$ min

Solution: Travel time in the gutter between the two spread locations is 1.7 minutes.

Chapter 6 - Roadside and Median Channels

Roadside and median channels are free water surface conveyance channels designed and constructed to collect stormwater from paved and other surfaces within the right-of-way (ROW) and, in some situations, adjacent areas. These channels provide a flow path to where stormwater can enter the offsite surface drainage system. Before entering the offsite drainage system, these channels may route water to an outfall structure or to a detention or retention basin or other storage component.

Designers commonly specify roadside and median channels with triangular or trapezoidal cross-sections. Construction techniques or the need for ongoing maintenance activities over the life of a transportation project may influence the exact cross-section shape, resulting in more rounded cross-sections. The surface of the channel may be vegetated or covered with other protective linings.

Limited ROW width, existing roadside development, and established adjacent land uses often constrain urban roadways and the associated roadside and median channels. Private and public utilities potentially share the ROW with stormwater conveyance features. Such elements include sanitary sewer access holes, water distribution valves, utility poles and conduits, illumination poles, traffic signal poles, signs, pedestrian sidewalks, bicycle paths, on-street parking lanes, flexible traffic barriers, and accessibility features.

Some of these features present potential safety hazards to the traveling public. While safety concerns guide the placement of those potential fixed objects, the location of some of these ancillary features within the boundaries of roadside channels is often unavoidable. Median channels for the conveyance of stormwater frequently share the median of divided roadways with illumination standards, signal standards, and flexible traffic barriers. Accommodating all the features needed for a safe and functional roadway encourages cooperation among the engineering team during the design phase. Designers of drainage features will likely find other references such as the AASHTO Roadside Design Guide (AASHTO 2011) helpful.

Although designed to carry stormwater, roadside and median channels also carry sediment, pollutants, and debris. During certain seasons, channels adjacent to residential areas may also contain plant material including bloom from trees in the spring and nuts and leaves in the fall that accumulate in streets and are washed into the drainage facilities. Over time, the accumulation of sediment, debris, and plant materials can obstruct flow in channels and medians. By understanding the adjacent land use, and anticipating debris type and occurrence, the designer can facilitate movement of these unintended materials through the channel or provide access for removal, or both.

This chapter presents concepts and relationships for the design of roadside and median channels beginning with open channel flow and channel design parameters. Next, the chapter presents the concepts and design steps for stable channel design. Finally, the chapter provides an overall process for designing roadside and median channels.

6.1 Open Channel Flow Concepts

The analysis and design of roadside and median channels rely on the principles of open channel flow. The following sections present summaries of several open channel flow concepts. Chow (1959), FHWA (2008), and many other hydraulic references provide more complete and specific discussion of open channel flow concepts.

6.1.1 Energy

Conservation of energy is a basic principle in open channel flow. As shown in Figure 6.1, the total energy (or head) at a given location in an open channel is expressed as the sum of the potential energy (channel bottom elevation plus depth) and kinetic energy (velocity head). (Pressure head is zero in open channel flow and, therefore, excluded from consideration.) The total energy at any cross-section along the channel can be approximated as:

$$E_t = Z + y + \frac{V^2}{2g} \tag{6.1}$$

where:

E_t	=	Total energy, ft (m)
Z	=	Elevation of the channel bottom above a given datum, ft (m)
y	=	Flow depth, ft (m)
V	=	Mean velocity, ft/s (m/s)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

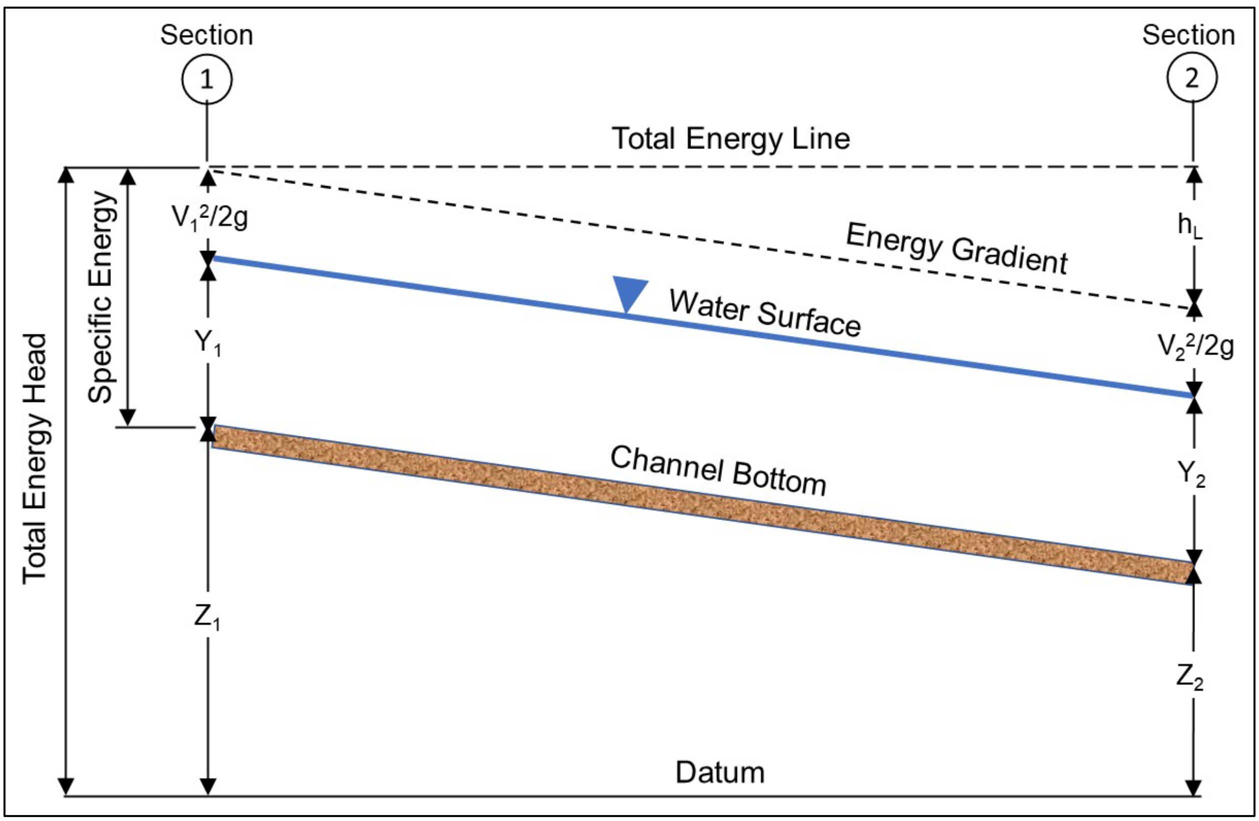

Figure 6.1. Total energy in open channels.

Figure 6.1 depicts a channel schematic highlighting two cross-section locations. Balancing energy between cross-section 1 and a downstream cross-section 2, the energy equation becomes:

$$Z_1 + y_1 + \frac{V_1^2}{2g} = Z_2 + y_2 + \frac{V_2^2}{2g} + h_L \tag{6.2}$$

where:

h_L = Energy (head) lost between section 1 and 2 to friction and turbulence, ft (m)

The energy equation states that the total energy head at an upstream cross-section is equal to the total energy head at a downstream cross-section plus the energy head loss between the two sections.

6.1.2 Critical, Subcritical, and Supercritical Flow

Hydraulic engineers classify open channel flow situations into critical, subcritical, and supercritical flow regimes using the **specific energy** of the flow and a dimensionless Froude number. The specific energy, E, is the energy head relative to the channel bottom. It is the sum of the flow depth and velocity head:

$$E = y + \frac{V^2}{2g} \tag{6.3}$$

For a given discharge and channel roughness, the specific energy changes with channel slope. At mild slopes, flow moves through a channel relatively slowly and with greater depths. As slope increases, velocity increases and depth decreases. Figure 6.2 illustrates the change in specific energy relative to the depth for three different flow rates, q_1, q_2, and q_3. The figure reveals that for each of the curves there is a depth at which the specific energy is a minimum.

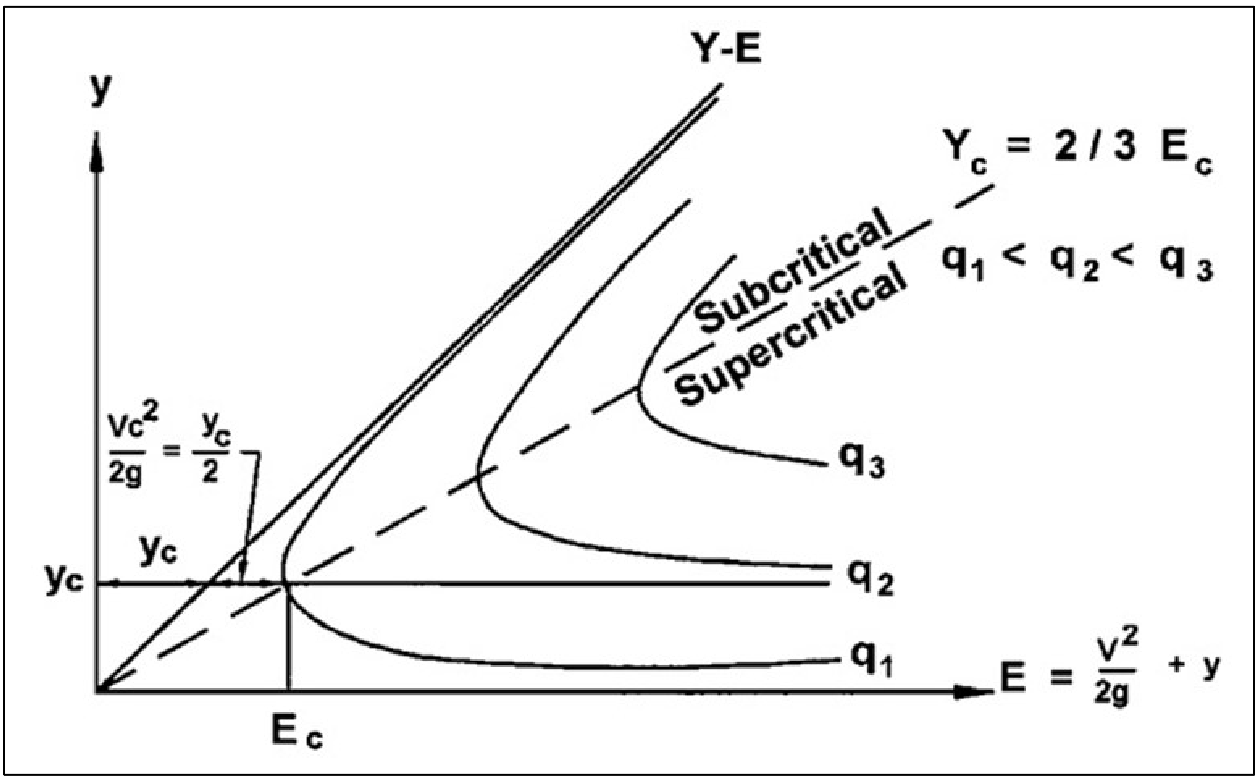

Figure 6.2. Specific energy diagram.

Critical flow occurs when the specific energy is at its minimum for a given flow and the corresponding depth is the **critical depth**. Figure 6.2 shows that at critical flow, the critical depth is two-thirds of the specific energy. The velocity head is the remaining one-third.

Subcritical flow occurs when the depth is greater than critical depth and supercritical flow occurs when the depth is less than critical depth. Hydraulic engineers use the Froude number to identify critical, subcritical, or supercritical flow conditions in a channel. The Froude number is calculated for rectangular channels using the following equation:

$$Fr = \frac{V}{\sqrt{gy}} \tag{6.4}$$

where:

V	=	Mean velocity of flow, ft/s (m/s)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)
y	=	Flow depth, ft (m)

When the Froude number is less than one, the flow is classified as **subcritical**. Depth is greater than critical depth, the velocity is slower than critical velocity, and small water surface disturbances travel both upstream and downstream. Downstream conditions control the flow rate. The control may be a structure or obstruction in the channel, or the control may be the channel roughness. Subcritical flow generally occurs on mild slopes.

When the Froude number is greater than one, the flow is classified as **supercritical**. Depth is less than critical depth, the velocity is faster than critical velocity, and small water surface disturbances cannot move upstream and are swept downstream. Upstream conditions control the flow rate. Supercritical flow occurs only very rarely in natural channels but is common in constructed channels on relatively steep slopes.

When the Froude number is equal to one, the flow is critical. When the Froude number is close to one, small changes in the flow rate, channel geometry, or channel slope can initiate a change in flow regime. A change from subcritical flow to supercritical flow results in higher velocities than anticipated and a change from supercritical to subcritical flow results in higher depths than anticipated. Considering these changes and any resulting impacts on flow depth or channel stability will help engineers design roadside and median channels that serve their intended function.

When the Froude number is greater than one (flow is supercritical) and a change in channel condition, such as an obstacle or a reduction in channel slope, occurs, flow may transition abruptly from supercritical to subcritical in a **hydraulic jump**. A hydraulic jump results in a rapid increase in depth and reduction in velocity. In making this transition, the jump can dissipate a significant portion of the flow energy.

The turbulence of a hydraulic jump may threaten the stability of a roadside or median channel. Exposing an unprotected channel boundary material, e.g., soil, to the turbulence of a hydraulic jump may result in undesirable scour and erosion of the boundary, altering the channel shape and resulting in long-term or recurrent channel maintenance problems. In addition, as the flow varies over time (e.g., from the beginning of a runoff event to its peak and recession), the location of a jump along the channel can vary significantly as the Froude number varies with flow.

For these reasons, designers typically avoid hydraulic jump conditions in roadside and median channels. The designer may consider the potential for a hydraulic jump in cases where the Froude number is greater than one or where the slope of the channel bottom changes abruptly from steep to mild. If hydraulic jumps cannot be avoided, accounting for their likely presence allows the designer to apply measures, such as protective linings, that create a more sustainable design.

Lessons from Experience: Hydraulic Jumps

Jumps are common in the field, especially where there is concrete lining. The jumps are generally small and poorly developed, but they are often a source of channel problems. Because the slope in a roadside or median channel is driven by the ROW width and the grade of the edge of pavement, jump conditions occur but designers can recognize and mitigate these conditions.

6.1.3 Steady Uniform Flow

Typical roadside design practice assumes that flow conditions are both steady and uniform though there are situations where more complex situations occur. **Steady flow** does not change over time and **uniform flow** does not vary in depth over the length of a channel. Therefore, steady uniform flow neither varies with time nor depth within a given channel segment. Such conditions are unlikely to occur in natural channels but can be assumed in prismatic roadside and median channels. Prismatic channels are those with slope and cross-section geometry that do not change significantly over their length.

Unsteady flow varies with time. Channels experience unsteady flow in every runoff event as runoff increases over time, reaches a peak, and decreases. As a channel receives the runoff, flow and depth in the channel will increase, reach a peak, and decrease. State Department of Transportation (DOT) staff may use hydrographs in the design of a roadside channel to evaluate how it performs over a range of flows.

Nonuniform (varied) flow occurs when either flow rate or depth, or both, vary along the channel. **Gradually varied** flow is nonuniform flow in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected in estimating the water surface profile in the channel. A typical example of gradually varied flow is the stream channel condition upstream of a culvert with ponded flow. **Rapidly varied** flow is nonuniform flow in which the depth and velocity change so that vertical accelerations cannot be neglected. An example of a rapidly varied flow is the flow profile through a constricted bridge opening.

6.1.4 Manning's Equation

Water flows in an open channel because of the force of gravity. Friction between the water and the channel boundary, and energy loss from turbulence near the boundary, resist the water flow and cause the loss of energy over distance. In the idealized model of steady uniform flow there are no significant accelerations, streamlines are straight and parallel, and the pressure distribution is hydrostatic. This is the simplest flow condition to analyze, but one that does not occur in the real world. However, for many applications, the flow is essentially steady and any changes in width, depth, or direction (resulting in nonuniform flow) are sufficiently small that the flow can be considered uniform.

The depth of flow in steady uniform flow is the **normal depth**. Designers commonly use Manning's equation to characterize steady uniform flow conditions and to determine normal depth in a channel. Manning's equation for discharge is:

$$Q = \frac{K_u}{n} A\, R^{2/3}\, S_o^{1/2} \tag{6.5}$$

where:

K_u	=	Unit conversion constant, 1.486 in CU (1.0 in SI)
Q	=	Discharge rate, ft/s³ (m/s³)
A	=	Cross-sectional flow area, ft² (m²)
R	=	Hydraulic radius, ft (m)
S_o	=	Energy grade line slope, ft/ft (m/m)
n	=	Manning's roughness coefficient

Applied appropriately, Manning's equation is a useful and reliable representation of steady uniform flow in open channels. Whenever the steady uniform flow model is appropriate, or a reasonable approximation of that condition exists, calculating the discharge capacity of a given channel section using Manning's equation is straightforward. However, when working with a discharge from a hydrologic analysis, designers sizing a channel to convey that discharge will typically use an iterative trial and comparison process.

Manning's n

Manning's n is a way of representing energy loss in fluid flow. When using Manning's equation, the value of n accounts for energy losses attributable to all causes (boundary shear, form drag, turbulence, etc.).

Manning's roughness coefficient, n, is a critical input for evaluating Manning's equation. Designers usually select an appropriate Manning's n value based on tables or procedures in reference materials such as textbooks and hydraulic design manuals. For channels with rigid, manufactured boundaries such as concrete, designers can reasonably assume that the Manning's n value is constant with different applications and with time. For channels lined with vegetation or flexible materials, on the other hand, the Manning's n value can vary quite dramatically. Factors influencing this variance can include the type of vegetation, its height relative to flow depth, and its state of growth. Seasonal variations in vegetative vigor, mowing and vegetation management policies and procedures, as well as roadside maintenance procedures such as "blading" slopes and ditches, may involve a wide range of changes to the hydraulic texture of a channel boundary.

Over many decades, researchers have compiled typical Manning's n values for a wide range of channel conditions. Table 6.1 provides a tabulation of typical Manning's n values for various channel linings that can be used in roadside channel design including rigid linings, no linings, and rolled erosion control productions (RECPs). Designers will find more information on selecting Manning's n values for roadside and median channels in HEC-15 (FHWA 2005).

6.1.5 Superelevation in Bends

Flow around a bend in an open channel induces centrifugal forces because of the change in flow direction (Chow 1959). This results in **superelevation** (transverse slope) of the water surface at the outside of bends. Without adequate freeboard, superelevation can cause the flow to splash up or over the side of the channel. Superelevation may also expose channel linings to higher shear stresses. Designers can estimate superelevation using:

$$\Delta d = \frac{V^2\, T}{g\, R_c} \tag{6.6}$$

where:

Δd	=	Difference in water surface elevation between the inner and outer banks of the channel in the bend, ft (m)
V	=	Average velocity, ft/s (m/s)
T	=	Surface width of the channel, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)
R_c	=	Radius to the centerline of the channel, ft (m)

Table 6.1. Typical channel lining Manning's roughness coefficients (FHWA 2005).

Lining Category	Lining Type	Manning's n Minimum	Manning's n Typical	Manning's n Maximum
Rigid	Concrete	0.015	0.013	0.011
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Element	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.025	0.020	0.016
	Rock Cut	0.045	0.035	0.025
RECP	Open-weave textile	0.028	0.025	0.022
	Erosion control blanket	0.045	0.035	0.028
	Turf reinforcement mat	0.036	0.030	0.024

Equation 6.6 is valid for subcritical flow conditions. The elevation of the water surface at the outer channel bank will be $\Delta d/2$ higher than the centerline water surface elevation and the elevation of the water surface at the inner channel bank will be $\Delta d/2$ lower than the centerline water surface elevation.

Under supercritical flow conditions, the water surface is not influenced by conditions or geometry downstream, including bends in the channel. Bends intended to change the direction of supercritical flow may result in potentially undesirable hydraulic conditions such as standing waves oblique to the direction of flow, oblique hydraulic jumps, and directional jets. If supercritical flow conditions approaching a bend are unavoidable, the designer can consider introducing a controlled hydraulic jump to induce subcritical conditions prior to changing direction (Chow 1959).

6.2 Channel Design Parameters

For roadside and median channels, the designer uses information including discharge frequency, available space, elevation change, vegetation type, freeboard, and shear stress. This section provides information for selecting or computing these design elements.

6.2.1 Discharge Frequency

State DOTs typically design roadside and median drainage channels to convey the estimated 0.2 to 0.1 AEP design discharges (FHWA 2005). However, designers can consider and mitigate potential consequences of discharges exceeding the design discharge. For temporary channel linings, designers can use a lower design discharge, e.g., the 0.5 AEP discharge.

6.2.2 Channel Geometry

Engineers often design highway drainage channels to be trapezoidal or triangular. Figure 6.3 summarizes these channel geometries, but during construction and through years of maintenance activities, these shapes can vary. Channel geometry (e.g., depth, bottom width, top width, and side slopes) are frequently subject to the influence of the roadway profile grade at the edge of pavement, the available ROW width, and the adjacent land use and profile along the ROW line.

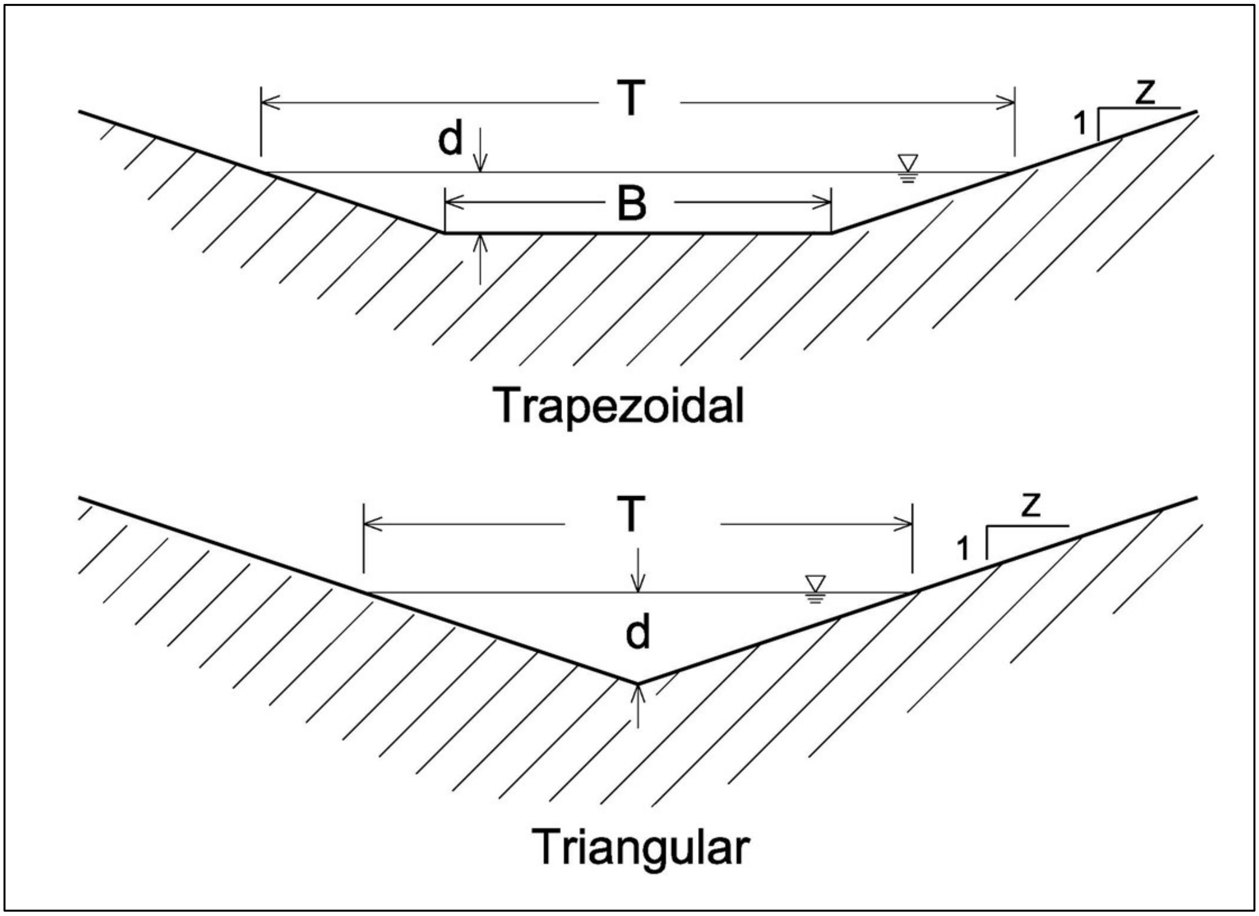

Figure 6.3. Channel geometries.

To avoid unstable side slopes, designers specify channel side slopes for roadside and median channels that do not exceed the angle of repose of either the soil or lining material, or both. Designers compute cross-section area, wetted perimeter, and flow top width for triangular (B = 0) and trapezoidal channels using:

$$A = Bd + zd^2 \tag{6.7}$$

$$P = B + 2d\,(z^2+1)^{0.5} \tag{6.8}$$

$$T = B + 2dz \tag{6.9}$$

where:

A	=	Cross-sectional area of the channel, ft^2 (m^2)
P	=	Wetted perimeter of the channel, ft (m)
T	=	Surface width of the channel, ft (m)

B	=	Bottom width, ft (m)
d	=	Maximum flow depth, ft (m)
z	=	Horizontal side slope dimension 1:z (V:H), dimensionless

Channels immediately adjacent to the roadway may be influenced by roadside safety needs or features such as traffic barriers. For example, geometric design criteria frequently specify channel side slopes that are **traversable** by errant vehicles (1V:3H or flatter with additional considerations) (FHWA 2005). In areas where roadside safety may be of concern, the Roadside Design Guide also allows for flatter channel side slopes, considering the functional classification of the roadway and traffic volume (AASHTO 2011). Design of roadside and median channels, the highway geometric and pavement design, and the design of appurtenances (e.g., signing, signals, illumination, and the accommodation of utilities) work together to ensure proper balancing of function, safety, utility, and drainage needs.

Lessons from Experience: Channel Shape and Construction

Engineers frequently design roadside and median channels in urban areas as triangles or trapezoids. However, at relatively small scale, they are often difficult to construct consistently. After completion of roadway construction, the actual shape of a channel cross-section is likely to be more rounded, possibly approximating a circle or parabola. This may be largely a factor of the equipment and methods used for construction.

If construction does not significantly alter the width and depth dimensions of the section, a consequential difference in performance among those shapes is unlikely.

Example 6.1:	Application of Manning's equation
Objective:	Estimate channel capacity and velocity for a channel lined with a turf reinforcement mat with an n value of 0.03.

Given: A trapezoidal channel (as shown in Figure 6.3) with the following characteristics:

S_o	=	0.01 ft/ft (m/m)
B	=	2.6 ft (0.8 m)
z	=	3
d	=	1.6 ft (0.5 m)

Step 1. Estimate the channel parameters.

Using equations 6.7, 6.8, and 6.9:

$A = Bd + zd^2 = (2.6)(1.6) + (3)(1.6)^2 = 11.8\ ft^2$

$P = B + 2d(z^2+1)^{0.5} = (2.6) + (2)(1.6)(3^2+1)^{0.5} = 12.7\ ft$

$R = A/P = 11.8/12.7 = 0.93\ ft$

Step 2. Compute the flow capacity using equation 6.5.

$Q = (K_u/n)\ A\ R^{(2/3)}\ S_o^{(1/2)} = (1.486/0.030)(11.8)(0.93)^{(2/3)}(0.01)^{(1/2)} = 55.7\ ft^3/s$

Step 3. Compute the flow velocity.

$V = Q/A = 55.7/11.8 = 4.7\ ft/s$

Solution:	The maximum flow for the channel is 55.7 ft^3/s (1.58 m^3/s) with an average velocity of 4.7 ft/s (1.4 m/s).

6.2.3 Channel Slope

The road profile and adjacent land use constraints often heavily influence channel slopes. However, if channel stability conditions warrant and geometry is feasible, the designer may choose to adjust the channel gradient slightly to achieve a more stable condition.

For steeper channel gradients, designers can use flexible or rigid linings to maintain stability. Knowledge of the material likely to constitute the channel bottom (usually the soil) informs the application of flexible linings. Soil erodibility is an important consideration. Linings, such as stone riprap, wire-enclosed riprap, and gabion mattress can be suitable for protecting very steep channels. See HEC-15 (FHWA 2005) for more information on slope limitations for different lining types. Section 6.3.2 discusses channel lining materials.

6.2.4 Freeboard

Freeboard is the vertical distance from the water surface at the design discharge to a pre-determined component of the roadway or channel. In a permanent roadside or median channel, a designer may use the bottom of the pavement structure base course as the relevant roadway component for defining freeboard. The need for freeboard depends on the consequences of overflows escaping the channel. At a minimum, appropriate freeboard prevents debris, waves, superelevation changes, or fluctuations in water surface from overflowing the sides. However, to accommodate the large variations in flow caused by shocks, standing waves, splashing, and surging in a steep channel, the designer can consider a freeboard height equal to the total energy depth. For temporary channels, freeboard is optional.

6.3 Stable Channel Design

Designers can use stable channel design concepts to specify channel geometry and lining types that result in long-term stability and low maintenance. A channel is stable when the material or the channel lining forming the channel boundary effectively resists the erosive forces of the flow. HEC-15 describes the principles of rigid boundary hydraulics and provides a detailed presentation of stable channel design concepts related to the design of roadside and median channels (FHWA 2005). This section provides a summary of significant concepts.

6.3.1 Shear Stress

The force of friction and the turbulence generated by flowing water (the same forces that cause head loss) result in **shear stress** or **tractive force** on the channel boundaries. The bed material resists this shear stress either by cohesion of the material itself, or by inertia and interlocking of cohesionless particles. To maintain stability, tractive force theory says that the flow-induced shear stress should not produce a force greater than the resisting force of the bed material. The force resisting the movement of the bed material is the **permissible** or **critical shear stress** of the bed material. In a uniform flow, the applied shear stress is equal to the effective component of the gravitational force acting on the body of water parallel to the channel bottom. The estimated average shear stress is:

$$\tau = \gamma R S \tag{6.10}$$

where:

τ	=	Average shear stress, lb/ft^2 (N/m^2)
γ	=	Unit weight of water, 62.4 lb/ft^3 at 60° F (9.81 kN/m^3 at 15° C)

R	=	Hydraulic radius, ft (m)
S	=	Average bed slope or energy slope, ft/ft (m/m)

The maximum shear stress for a straight channel occurs where flow is deepest (on the channel bed) (Chow 1959). It is computed using maximum depth instead of hydraulic radius:

$$\tau_d = \gamma\, d\, S \tag{6.11}$$

where:

τ_d	=	Maximum shear stress, lb/ft^2 (N/m^2)
d	=	Maximum depth of flow, ft (m)

Because shear stress is related to depth of flow and flow is shallower at the channel edges than in the middle of the channel, shear stress is not uniformly distributed along the wetted perimeter of a channel. The maximum shear on the side of a channel is estimated by the following (FHWA 2005):

$$\tau_s = K_1 \tau_d \tag{6.12}$$

where:

τ_s	=	Side shear stress on the channel, lb/ft^2 (N/m^2)
K_1	=	Ratio of channel side to bottom shear stress
τ_d	=	Shear stress in the channel at maximum depth, lb/ft^2 (N/m^2)

The value K_1 depends on the size and shape of the channel. For triangular channels with rounded bottoms, there is no sharp discontinuity along the wetted perimeter. Therefore, computation of shear stress at any point on the side slope is related to the depth at that point using equation 6.11.

The work of Anderson et al. (1970) led to the development of estimates for K_1 in trapezoidal and triangular channels (FHWA 2005):

$$\begin{array}{ll} K_1 = 0.77 & z \leq 1.5 \\ K_1 = 0.066\, z + 0.67 & 1.5 < z < 5 \\ K_1 = 1.0 & 5 \leq z \end{array} \tag{6.13}$$

The z value is the horizontal dimension 1:z (V:H) of side slope. Side slopes steeper than 1:3 (V:H) are at greater risk for failure because of the potential for erosion of the side slopes.

For noncohesive linings such as gravel or riprap the resisting ability of the lining is reduced because the material has the potential to roll or slide out of place. While the reduced shear stress on the channel sides might suggest increased stability in that region of the channel, this may be diminished by the steepness of side slope. For example, when designing a trapezoidal channel lined with gravel or riprap having side slopes steeper than 1:3 the appropriate rock size for the side slopes is estimated as:

$$D_{50,\,sides} = \left(\frac{K_1}{K_2}\right) D_{50,\,bottom} \tag{6.14}$$

where:

D_{50} = Riprap or bed material median size, ft (m)
K_1 = Ratio of shear stresses on the sides and bottom of a trapezoidal channel
K_2 = Ratio of tractive force on the sides and bottom of a trapezoidal channel

K_2 is a function of the side slope angle and the stone angle of repose and is determined from equation 6.15. HEC-15 (FHWA 2005) provides the angle of repose for gravels and riprap of different types.

$$K_2 = \sqrt{1 - \left(\sin^2\Theta\right) / \left(\sin^2\Phi\right)} \qquad (6.15)$$

where:

Θ = Angle of side slope
Φ = Angle of repose for the channel lining material

Flow around bends also creates secondary currents which impose higher shear stresses on the channel sides and bottom compared to straight reaches. Areas of high shear stress in bends are illustrated in Figure 6.4.

The maximum shear stress in a bend is a function of the ratio of channel curvature to the top (water surface) width. This ratio increases as the bend becomes sharper and the maximum shear stress in the bend increases. The bend shear stress can be computed using the following relationship:

$$\tau_b = K_b \, \tau_d \qquad (6.16)$$

where:

τ_b = Bend shear stress, lb/ft^2 (N/m^2)
K_b = Ratio of channel bend to bottom shear stress
τ_d = Maximum channel shear stress, lb/ft^2 (N/m^2)

K_b can be determined from the following equation from Young et al. (1996) adapted from Lane (1955):

$$\begin{aligned} K_b &= 2.00 && R_c/T \leq 2 \\ K_b &= 2.38 - 0.206\left(R_c / T\right) + 0.0073\left(R_c / T\right)^2 && 2 < R_c/T < 10 \\ K_b &= 1.05 && 10 \leq R_c/T \end{aligned} \qquad (6.17)$$

where:

K_b = Ratio of channel bend to bottom shear stress
R_c = Radius to the centerline of the channel, ft (m)
T = Top (water surface) width of channel, ft (m)

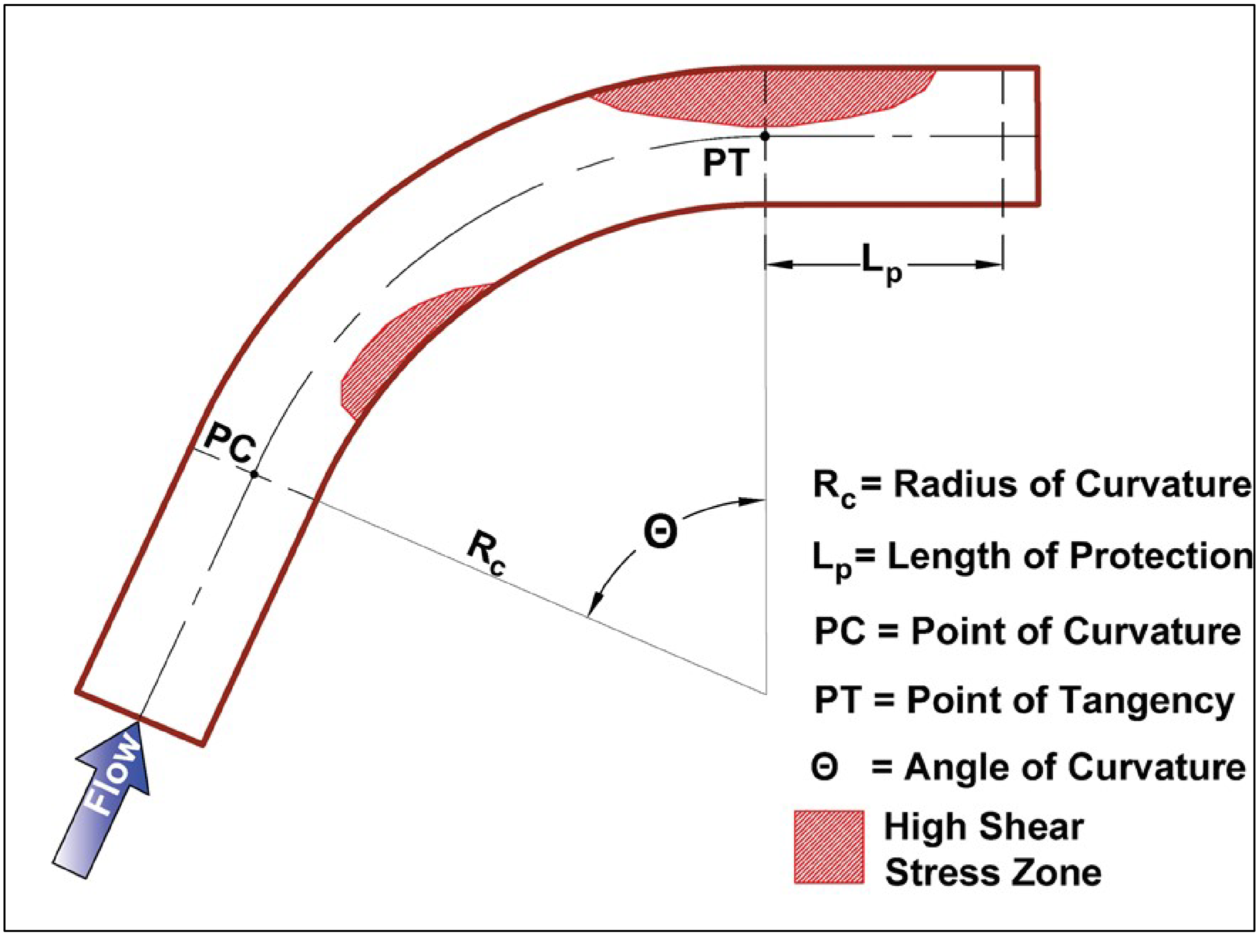

Figure 6.4. Shear stress distribution in channel bends. Source: HEC-15.

The increased shear stress produced by the bend persists downstream of the bend a distance L_p, as shown in Figure 6.4. This distance can be computed using the following relationship:

$$L_p = (K_u\ R^{7/6}) / n_b \tag{6.18}$$

where:

L_p	=	Length of protection (length of increased shear stress due to the bend) downstream of the curve point of tangency, ft (m)
n_b	=	Manning's n in the channel bend
R	=	Hydraulic radius, ft (m)
K_u	=	Unit conversion constant, 0.604 in CU (0.736 in SI)

6.3.2 Lining Materials

Lining materials may be classified as flexible or rigid. Flexible linings can conform to and sustain changes in channel shape while maintaining the overall integrity of the channel. Common flexible lining materials include vegetation and riprap. In contrast, rigid linings cannot change shape and tend to fail when a portion of the channel lining is damaged. Channel shape may change due to frost-heave, slumping, piping, and other causes. Concrete is a common rigid lining material. Flexible linings are generally less expensive, may have a more natural appearance, and are typically more environmentally acceptable. However, flexible linings are limited in the erosive forces they can sustain without damage to the channel and lining. A rigid lining can typically provide higher capacity and greater erosion resistance. In some cases, rigid linings may be the only feasible alternative.

Flexible linings can be either long-term, transitional, or temporary. Designers use long-term flexible linings where the channel needs protection against erosion for the life of the channel. Long-term lining materials include cobbles, rock riprap, wire-enclosed riprap, gabion mattresses, vegetation, and turf reinforcement. State DOTs often choose established vegetation as the primary long-term channel management strategy; vegetation may be planned or incidental but will frequently occur eventually.

Designers use transitional flexible linings to provide erosion protection until long-term protection, usually vegetation, can be established. They use temporary channel linings without vegetation to line channels that might be part of a construction site erosion and sediment management strategy, or some other short-term channel situation. State DOT staff can select turf reinforcement either as a transitional approach or as part of a long-term strategy of providing additional structure to the soil/vegetation matrix.

Vegetation and Channel Linings

Vegetation, such as native or locally popular grasses, will, by nature, establish themselves in drainage channels; seeds from ground cover are ever-present and naturally distributed by flowing water. Unlined channels or those with flexible liners are often ideal growing environments for grasses, weeds, woody shrubs, and even trees. While vegetative cover almost always enhances the stability of channels against erosive forces and will help lock flexible liner materials in place, they may also reduce channel capacity by increasing flow resistance.

Even rigid channel linings such as concrete slope pavement will invariably accumulate vegetative growth in cracks and construction joints. In this case, vegetation can be destructive. Growing vegetation serves to open cracks in concrete. In cases of woody brush or trees, root heave can destroy the lining.

Designers can reduce future maintenance costs by considering intended effects of vegetation on all types of lining materials.

Typical turf reinforcement materials include gravel/soil mixes and turf reinforcement mats (TRMs). A TRM is usually a non-degradable rolled erosion control product (RECP) processed into a three-dimensional matrix. A TRM is stiffer, thicker, and denser than an erosion control blanket, which is typically a degradable product composed of an engineered distribution of natural or polymer fibers bound together to form a continuous mat. Open-weave textiles (OWT) are a degradable RECP composed of natural or polymer yarns more loosely woven into a matrix. RECPs are laid in the channel and secured with staples or stakes.

Construction of rigid concrete linings involves considerable effort, costly materials, and specialized construction equipment. As a result, the cost of rigid linings is typically higher than flexible linings. Rigid lining such as concrete paving or grouted riprap is susceptible to failure from undermining, particularly when placed on constructed or disturbed material such as embankments. It can also be subject to structural instability from overtopping, freeze-thaw cycles, swelling, and excessive soil pore water pressure. Thermal stress causes ubiquitous cracking of concrete. Over time, water invasion and erosion of the supporting soil through cracks often results in concealed void spaces under the concrete. These concealed void spaces present the danger of sudden and unexpected collapse during runoff events or under the load of errant vehicles.

Prefabricated modular linings, such as interlocking concrete paving blocks, can be an alternative if shipping distances are not excessive, however these often involve labor-intensive placement. Modular linings are classified as flexible as they can withstand some movement and erosion underneath the lining before failing.

In general, when selecting a lining, the designer considers the cost of the lining that affords satisfactory protection as the baseline for comparison, but will also evaluate constructability, aesthetics, and long-term service. In some regions, State DOTs often use vegetation alone or in combination with other types of linings. Thus, a channel might be lined with vegetation on flatter slopes and with more erosion resistant material on steeper slopes. In cross-section, the channel might be lined with a resistant material within the depth necessary to carry frequent flows and lined with vegetation above that depth for protection from less frequent flows.

6.3.3 Stable Channel Design Procedure

The FHWA presented the permissible tractive force (shear stress) approach as the recommended design procedure for channels with flexible linings in HEC-15 (FHWA 2005). The tractive force approach necessitates that the shear stresses on the channel bed and banks do not exceed the allowable amounts for the given channel boundary. Tractive force procedures based on shear stress concepts originated largely through research by the Bureau of Reclamation in the 1950s. Based on the actual physical processes involved in maintaining a stable channel, specifically the stresses developed at the interface between flowing water and materials forming the channel boundary, the tractive force procedure is a realistic model of the processes that affect channel linings.

Designers also employ a permissible velocity approach where they assume the channel is stable if the mean velocity in the channel is lower than the maximum permissible velocity for the given channel boundary condition. This approach approximates the physical processes affecting channel linings. Researchers first introduced permissible velocity procedures around the 1920s and the Soil Conservation Service (now the Natural Resource Conservation Service) further developed and widely uses the approach.

For stable channel design, this manual uses the permissible tractive force approach building on the shear stress descriptions in Section 6.3.1. When the permissible shear stress is greater than or equal to the computed shear stress, the lining is considered acceptable. The safety factor in the following equation provides for a measure of uncertainty and failure tolerance, and typically ranges from 1.0 to 1.5.

$$\tau_p \geq SF\ \tau_d \tag{6.19}$$

where:

τ_p	=	Permissible shear stress for the channel lining, lb/ft^2 (N/m^2)
SF	=	Safety factor

Flexible linings reduce the shear stress acting directly on the underlying soil surface. Therefore, the erodibility of the underlying soil is a key factor in the performance of flexible linings. Erodibility of noncohesive (granular) soils (plasticity index less than 10) primarily relates to particle size and specific gravity, while in cohesive (clay or clay-bound) soils it largely corresponds with the cohesion and density of the soil. Vegetative and RECP lining performance relates to how well they protect the underlying soil from shear stress, therefore, the protection offered by these lining types depends on soil type. Figure 6.5 summarizes the basic procedure for designing a flexible lining.

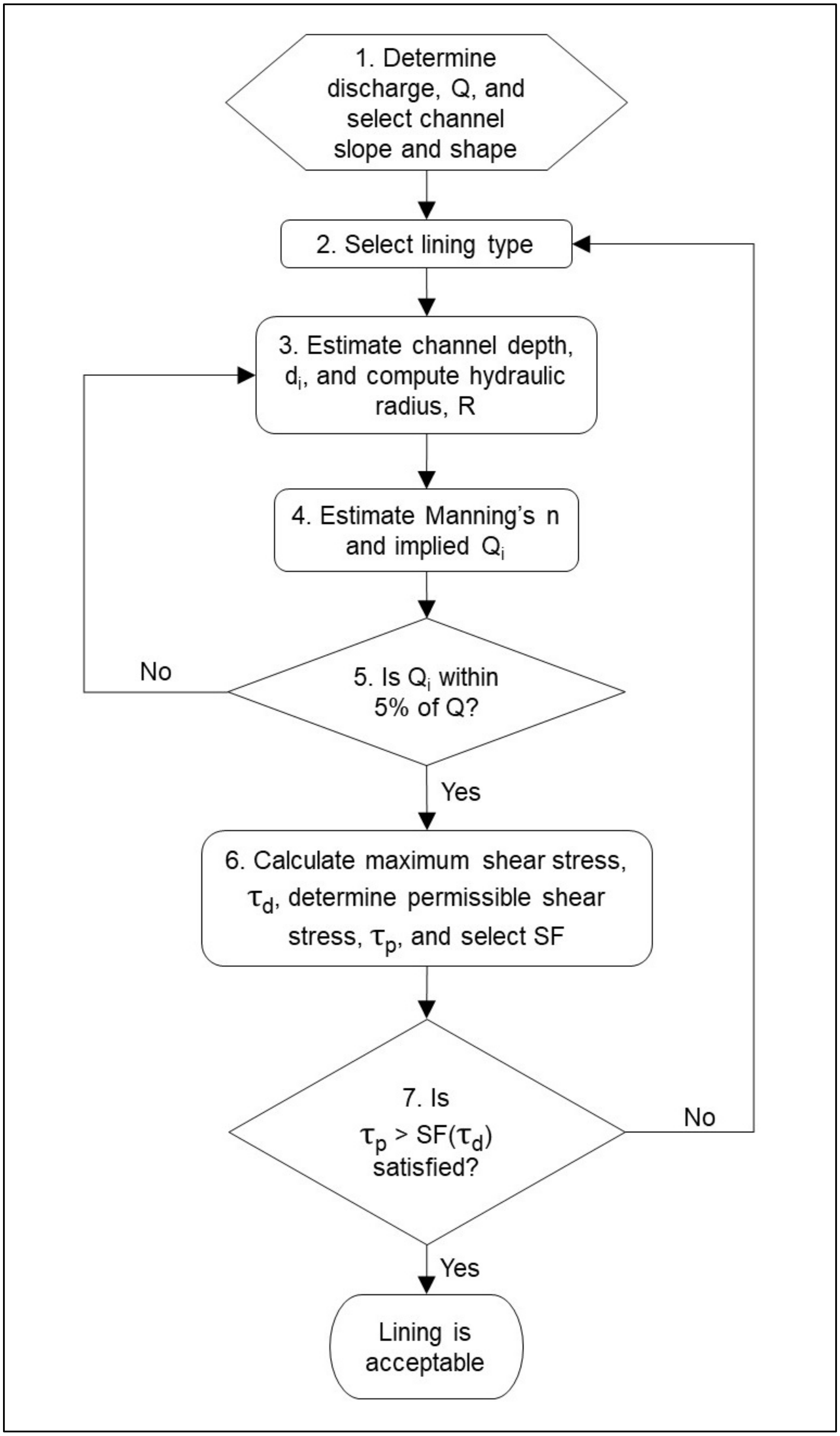

Figure 6.5. Flexible channel lining design process. Source: HEC-15.

Step 1. Select design hydrology and channel geometry.

Calculate a design discharge for the analysis. Select an initial channel cross-section shape and channel slope.

Step 2. Select a trial lining type.

Decide if a long-term lining is appropriate or whether a temporary or transitional lining is adequate. If the long-term lining is a constructed lining that provides full protection immediately, a transitional lining is not indicated. However, if the long-term lining is vegetation that will take time to provide full protection, a transitional lining is also indicated. For example, the designer could initially consider an established vegetated channel for the long-term and evaluate whether a bare soil (unlined) is adequate until vegetation is established. The designer evaluates both the long-term and temporary or transitional linings in separate analyses to determine their suitability for meeting design objectives.

Vegetated Channel Stability

Designers can improve the long-term performance of drainage features by planning for the establishment of temporary and permanent vegetation coverage. The first year or more following construction of a project, depending on the environment, presents the greatest challenge because the vegetation is not well established. Channels exhibiting erosion or sedimentation during that initial period may never stabilize and may present a recurring maintenance problem. In addition to maintenance issues, an eroding channel may present a safety hazard to vehicles that leave the roadway.

Step 3. Estimate the depth of flow.

The estimated depth may be based on physical limits of the channel and an initial estimate based on an assumed Manning's n value for the lining type and the design discharge. Depending on how close the initial estimate is to the final estimate, iterations of steps 3 through 5 may be necessary.

Step 4. Estimate implied discharge.

Estimate the implied discharge, Q_i, using the assumed Manning's n and estimated flow depth value from step 3.

Step 5. Compare implied and design discharges.

If Q_i is within 5 percent of the design discharge, Q, then proceed to step 6. The initial estimate of depth was appropriate. If not, return to step 3 and select a new estimated flow depth, d_{i+1}. This can be estimated from the following equation or any other appropriate method.

$$d_{i+1}=d_i\left(\frac{Q}{Q_i}\right)^{0.4} \qquad (6.20)$$

where:

d_{i+1}	=	Next estimate of depth for computing the implied discharge, ft (m)
d_i	=	Previous estimate of depth for computing the implied discharge, ft (m)
Q	=	Design discharge, ft³/s (m³/s)
Q_i	=	Implied discharge based on the previous estimate of depth, d_i, ft³/s (m³/s)

Step 6. Calculate shear stresses and select safety factor.

Estimate the shear stress at maximum depth, τ_d, using equation 6.11. Determine the permissible shear stress, τ_p, according to the methods described in HEC-15 (FHWA 2005). Select an appropriate safety factor.

Step 7. Compare the permissible shear stress to the calculated shear stress.

Using equation 6.19, compare the permissible and calculated shear stresses from step 6. If the permissible shear stress is adequate, then the lining is acceptable. If the permissible shear is inadequate, then return to step 2 and select an alternative lining type with greater permissible shear stress. As an alternative, a different channel shape may be selected that results in a lower depth of flow.

When the selected lining is stable the design process is complete. If desired, other linings may be tested before specifying the preferred lining.

HEC-15 details the tractive force stable channel design procedure for vegetative linings, RECPs, riprap/cobble, and gabion linings. HEC-15 also includes information on special considerations for steep-slope riprap design and design of composite linings.

Example 6.2: Channel shear stress and lining stability

Objective: Estimate the maximum shear stress in a channel with straight and bend sections and determine if the channel lining is stable.

Given: A trapezoidal channel with the following characteristics:

S_o = 0.01 ft/ft (m/m)
B = 3.0 ft (0.90 m)
z = 3

The channel reach consists of a straight section and a 90-degree bend with a centerline radius of 20.0 ft (6.1 m). The design discharge is 28 ft^3/s (0.79 m^3/s). Assume a vegetated lining with a Manning's n value of 0.030. Maximum depth in the channel is 1.3 ft based on the desired freeboard and channel dimensions.

Step 1. Select design hydrology and channel geometry.

The design hydrology (28 ft^3/s) and trapezoidal channel geometry were given above.

Step 2. Select trial lining type.

Assume a vegetative lining with a Manning's n value of 0.030 as given.

Step 3. Estimate the depth of flow.

Channel geometry and freeboard limit the maximum depth of flow to 1.3 ft. Assume an initial depth of 1 ft to estimate the implied discharge.

Step 4. Estimate implied discharge.

$A = Bd + zd^2 = (3.0)(1.0) + (3)(1.0)^2 = 6.0\ ft^2$

$P = B + 2d(z^2+1)^{0.5} = (3.0) + (2)(1.0)(3^2+1)^{0.5} = 9.3\ ft$

$R = A/P = 6.0/9.3 = 0.64\ ft$

$Q = (K_u/n)\ A\ R^{(2/3)}\ S_o^{(1/2)} = (1.486/0.030)(6.0)(0.64)^{(2/3)}(0.01)^{(1/2)} = 22.1\ ft^3/s$

Step 5. Compare implied and design discharges.

Implied discharge (22 ft^3/s) is less than the design discharge (28 ft^3/s) by more than 5 percent. Therefore, recompute with a higher estimate of design discharge and repeat step 4.

When using a 1.1 ft depth, the resulting implied discharge is 27 ft^3/s which is within the 5 percent tolerance of 28 ft^3/s. Continue with step 6 using 1.1 ft for the depth.

Step 6. Calculate shear stresses and select safety factor.

Permissible shear stress for vegetation depends on the type of vegetation and soils. Using the procedure in HEC-15 permissible shear stress is estimated for this channel as 2.7 lb/ft^2.

Safety factor selected for this channel is 1.2.

Maximum shear stress in the straight channel is computed as:

$\tau_d = \gamma dS = (62.4)\ (1.1)\ (0.01) = 0.69\ lb/ft^2$

Maximum shear stress in the bend is computed starting with computing the flow top width. Next, compute the ratio of bend to bottom shear stress and finally the bend shear stress:

$T = B + 2dz = 3.0 + 2(1.1)(3) = 9.6\ ft$

$K_b = 2.38 – 0.206(R_c/T) + 0.0073(R_c/T)^2 = 2.38 – 0.206\ (20/9.6) + 0.0073(20/9.6)^2 = 2.0\ lb/ft^2$

$\tau_b = K_b\ \tau_d = 2.0\ (0.69) = 1.4\ lb/ft^2$

Step 7. Compare the permissible shear stress to the calculated shear stress.

Use equation 6.19 to compare permissible to maximum shear stress in the bend of the channel:

$\tau_p \geq SF\ \tau_d = (1.2)\ (1.4) = 1.7\ lb/ft^2$

Since the maximum shear stress multiplied by the safety factor is less than the permissible shear stress, the channel is stable in the straight and bend portions of the channel.

Solution:	The maximum shear stress in the straight and bend sections are 0.69 lb/ft^2 (33.0 Pa) and 1.4 lb/ft^2 (67.0 Pa), respectively. Both are less than the permissible shear stress. Therefore, the vegetated channel lining is stable for the design flow.

6.4 General Design Procedure

This section presents a general procedure for designing roadside and median channels. Although each project is unique, the design steps outlined below will typically be applicable. State and local procedures may also be available and inform the design process.

Step 1. Establish a preliminary project drainage plan.

Chapter 3 discussed the development of a preliminary drainage concept plan. For proposed median or roadside channels, designers may take the following preliminary actions:

- Review municipal master drainage plan(s) and available outfall locations.
- Review available ROW, roadway schematic, and roadway profile; locate public utilities and traffic control/signage.
- Identify the locations of natural drainage divides and channel outlet points.

- Prepare existing and proposed plan and profile of the proposed channels. Include any constraints on design such as highway and road locations, culverts, and utilities.
- Collect any available site data such as soil types and topographic information.

Step 2. Obtain typical cross-section information.

Establish preliminary cross-section geometric parameters and controlling physical features considering the following:

- Roadway width, auxiliary lanes, shoulders, pedestrian facilities (e.g., sidewalks), and other roadway features when present.
- Adequate channel depth to drain the subbase and minimize freeze-thaw.
- Channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access.

Step 3. Select initial channel slope.

Plot initial slopes on the plan and profile. Note that highway grades often control slopes on roadside channels. Use the following guides when establishing initial channel slopes:

- Provide a channel slope with sufficient elevation drop to minimize ponding and sediment accumulation.
- Where possible, avoid features which may influence or restrict slope, such as utility (e.g., electricity and gas) structures.

Step 4. Check flow capacities and adjust sections.

Evaluate the hydraulic capacity of the channel and confirm that it meets the hydraulic design criteria using the techniques in Section 6.2. The following activities may be appropriate:

- Compute the design discharge at the downstream end of each channel segment (see Chapter 4).
- Set preliminary values for channel size, roughness, and slope as discussed in Section 6.2 based on long-term conditions and maintenance considerations.
- Determine the maximum allowable depth of channel including freeboard.
- Estimate the flow depth using the design discharge and channel characteristics.
- If the capacity is not adequate, consider possibilities for increasing capacity including:
    - Increasing bottom width.
    - Flattening channel side slopes.
    - Steepening channel slope.
    - Providing smoother channel lining.
    - Intercepting some flow before it reaches the channel.
    - Installing drop inlets and a parallel storm drainpipe beneath the channel to supplement channel capacity.

Step 5. Select channel protection.

Follow the procedure outlined in Section 6.3 to complete final design of channels that will involve a lining for stability.

Step 6. Check channel transitions and end of channel conditions.

At channel transitions, the designer may need to employ more detailed hydraulic evaluations because the assumption of uniform or gradually varied flow is not valid.

- Identify transition locations, e.g., significant changes in channel geometry (slope, roughness, or cross-section), channel bends, and channel inlets/outlets.
- Review hydraulic conditions upstream and downstream of the transition (flow area, depth, and velocity). If significant changes are observed, perform additional hydraulic evaluations to estimate flow conditions in the vicinity of the transition. Use the energy equation presented in equation 6.2 or other information in Chow (1959), FHWA (2005), FHWA (2006a), or other hydraulic references to evaluate transition flow conditions.
- Provide for gradual channel transitions to minimize the potential for sudden changes in hydraulic conditions at channel transitions.

Step 7. Analyze outlet points and downstream effects.

In this final step, the designer identifies possible adverse consequences for discharge of water downstream and considers appropriate mitigation. Table 6.2 summarizes possible adverse impacts to downstream and adjacent properties and potential mitigation approaches for each. To achieve a roadside channel system design that meets drainage requirements, protects roadside safety, and avoids adverse downstream consequences, the designer may make several trials of this design procedure before selecting a final design.

Table 6.2. Possible adverse channel outlet impacts and potential mitigation.

Adverse Impact	Potential Mitigation
Increase in discharge.	Enlarge outlet channel and/or install control structures to provide detention (see FHWA 2002).
Increase in flow velocity.	Install velocity control or energy dissipation structure (see FHWA 2006a).
Capture of sheet flow previously draining to a different outlet.	Increase channel capacity and/or improve lining of downstream channel.
Capture of concentrated or channel flow previously draining to a different outlet.	Avoid diversions of existing drainage patterns where possible. If not possible, see mitigations associated with increase in discharge and increase in velocity.
Decrease in outlet water quality.	Select lining types that may provide water quality improvements, e.g., vegetation, or provide other water quality mitigation approaches (see Chapter 11).

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Chapter 7 - Inlet Design

Stormwater inlets in an urban or roadway environment capture runoff from roadway surfaces to maintain a safe roadway and road corridor for vehicles, pedestrians, bicycles, and other forms of personal transport. This chapter describes the types, uses, and selection of inlets for a variety of applications. It includes information on the hydraulic performance of inlets on grade and in sag locations, as well as tools for selecting and sizing inlets. The chapter provides a section on locating inlets and a section on inlets in medians and at embankments. The chapter does not address bridge deck drainage inlets. The FHWA's HEC-21 document provides information on the analysis and design of bridge deck drainage (FHWA 1993).

7.1 Inlet Types, Uses, and Selection

Storm drain inlets collect runoff from gutter sections, paved medians, roadside ditches, and median ditches and discharge it to an underground storm drainage system. Inlet selection depends on the intended use, hydraulic efficiency, clogging potential, pedestrian and bicycle safety, loading conditions, cost, and other factors. Inlets that will experience vehicle loading, particularly grates, must be able to withstand traffic loads. Conversely, grates draining non traffic areas do not generally need to be as strong. However, engineers may select grates that withstand traffic loads in non-traffic areas to handle maintenance and construction equipment or errant vehicles. The following subsections discuss inlet types and uses, hydraulic efficiency, clogging potential, and pedestrian and bicycle safety.

7.1.1 Inlet Types and Uses

Inlet design and configuration are determined by their intended function. Figure 7.1 summarizes the four general inlet types:

- Grate inlets.
- Curb-opening inlets.
- Combination inlets.
- Slotted inlets.

Drainage designers use these inlet types for permanent installations. They also use barrier walls with drainage openings for temporary construction applications.

Grate inlets consist of an opening in the gutter or ditch covered by a grate. Grate inlets generally perform satisfactorily over a wide range of gutter grades. They lose capacity with increase in grade, but less than curb-opening inlets. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing. Their principal disadvantage is that they may be clogged by floating trash or debris. Grate inlets are a good choice where out-of-control vehicles might wander.

Lessons from Experience: Inlets in the Roadway

Grate inlets may extend into the pavement section which can make it more difficult for mechanized placement and compaction of subgrade and base for asphaltic pavement and for maintenance operations.

Curb-opening inlets are vertical openings in the curb covered by a top slab. They are most effective on flatter slopes, in sags, and with flows that carry significant amounts of floating debris. The interception capacity of curb-opening inlets decreases as the gutter grade steepens. Consequently, curb-opening inlets are most effective in sags and on grades less than 3 percent.

Combination inlets consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may extend upstream of the grate. Combination inlets provide the advantages of both curb-opening and grate inlets and perform as a high-capacity inlet. When the curb opening extends upstream of the grate in a "sweeper" configuration, the curb-opening inlet acts as a trash interceptor during the initial phases of a storm. Used in a sag configuration, the sweeper inlet can have a curb opening on both ends of the grate.

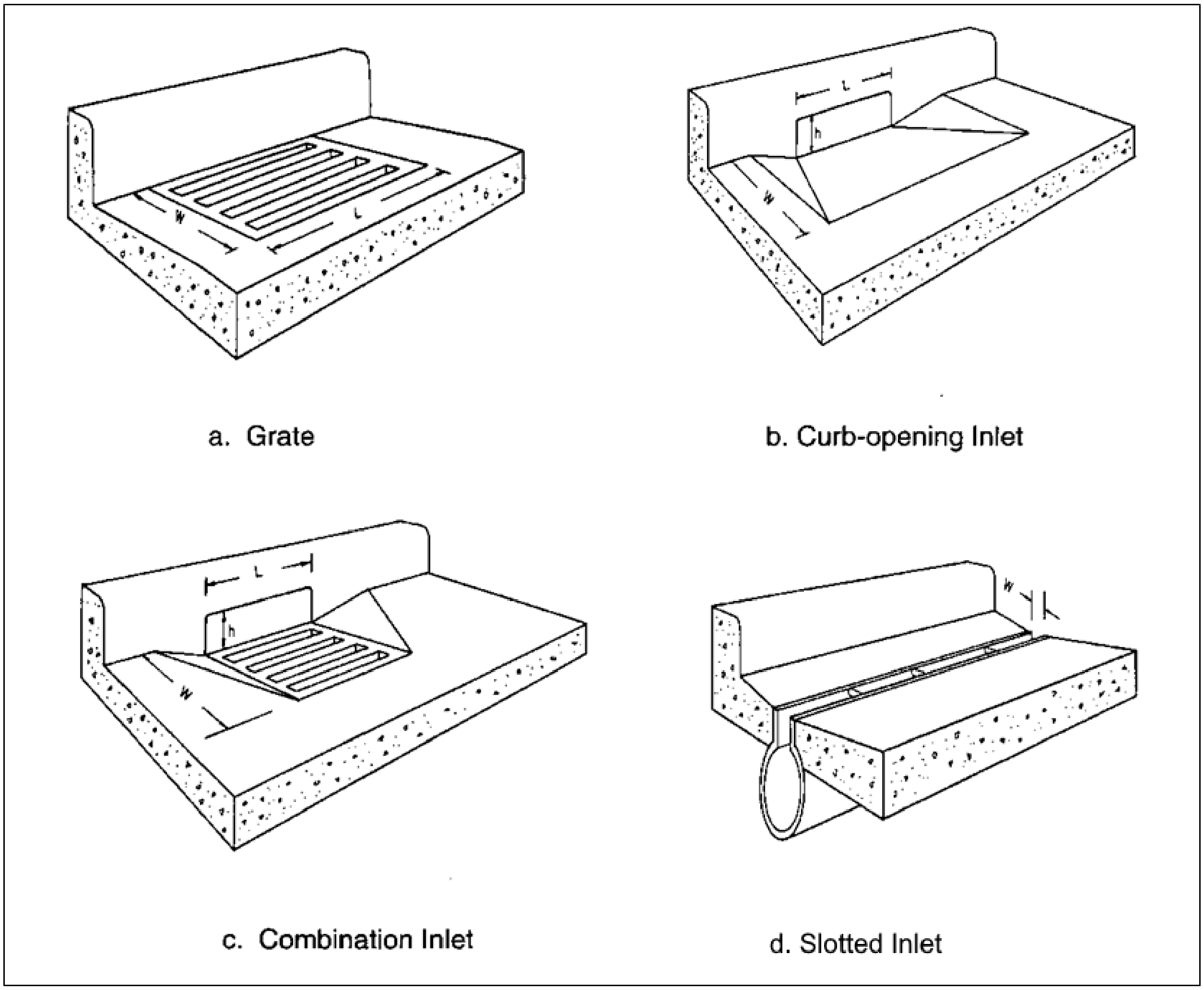

Figure 7.1. Storm drain inlet types.

Slotted inlets consist of a pipe cut along the longitudinal axis with bars perpendicular to the opening to maintain the slotted opening. Slotted inlets can be used in areas where it is desirable to intercept sheet flow before it crosses onto a section of roadway. Their principal advantage is that they can intercept flow over a wide section. However, the susceptibility of slotted inlets to clogging from sediments and debris makes them ill-suited for use in environments where significant sediment or debris loads may be present. Slotted inlets on a longitudinal slope exhibit the same hydraulic capacity as curb openings when debris is not a factor. Slotted drains may also be combined with grates.

Barriers and barrier walls used during construction temporarily separate vehicles from construction activities. Where pavement runoff flows toward the barriers, drainage designers provide for capture and diversion of this runoff from the traveled way. Typically, these barriers include pre-located rectangular openings to allow water to pass beneath the barriers. Although, these openings create a hydraulic configuration analogous to a curb and gutter, they are different in important ways:

- The openings in each barrier are generally prefabricated and not customized based on the drainage needs.
- The openings are located at regular, closely spaced intervals in a series of barriers that are not selected by the drainage designer.
- The barriers are often located close to the traveled way potentially reducing or eliminating flow in a gutter.

For drainage under temporary barriers, the drainage design estimates the contributing flow to, and interception capacity of, the barrier openings to assess water depth and spread to maintain safe vehicle use of the roadway. In addition to adapting the tools in this manual for curb inlets, drainage designers can use tools developed by researchers focused specifically on barrier wall drainage, e.g., Kranc et al. (2005).

7.1.2 Hydraulic Efficiency

Inlets primarily function to capture flow on the roadway or in the roadway corridor. Inlet characteristics, inlet location (e.g., on grade versus in a sag), and the flow characteristics (e.g., velocity, depth, and spread) determine the ability to intercept flow.

Several agencies and manufacturers of grates have investigated grate inlet interception capacity. On behalf of the FHWA, the U.S. Bureau of Reclamation (USBR) conducted tests on grate inlets and slotted inlets included in this document (Burgi and Gober 1977, Burgi 1978a, Burgi 1978b, Pugh 1980). Four of the grates selected for testing were rated highest in bicycle safety tests, three have designs and bar spacing similar to those proven bicycle-safe, and a parallel bar grate was used as a standard with which to compare the performance of others. Table 7.1 summarizes the grate types investigated.

Table 7.1. Inlet grate types and specifications.

Inlet Grate	Grate Type	Longitudinal Bar Spacing *	Transverse Bar Spacing *	Figure
P-1-7/8	Parallel bar	1-7/8 inch (48 mm)	–	Figure 7.2
P-1-7/8-4	Parallel bar	1-7/8 inch (48 mm)	4 inch (102 mm)	Figure 7.2
P-1-1/8	Parallel bar	1-1/8 inch (29 mm)	–	Figure 7.3
Curved Vane	Curved vane	3-1/4 inch (83 mm)	4-1/4 inch (108 mm)	Figure 7.4
45°- 60 Tilt-Bar	Tilt-bar	2-1/4 inch (57 mm)	4 inch (102 mm)	Figure 7.5
45°- 85 Tilt-Bar	Tilt-bar	3-1/4 inch (83 mm)	4 inch (102 mm)	Figure 7.5
30°- 85 Tilt-Bar	Tilt-bar	3-1/4 inch (83 mm)	4 inch (102 mm)	Figure 7.6
Reticuline	Honeycomb	Not applicable	Not applicable	Figure 7.7

*Spacing is on center.

The parallel bar grate (P-1-7/8) performs better hydraulically than all others but is not considered bicycle-safe. The curved vane and the P-1-1/8 grates have good hydraulic characteristics with high velocity flows. The other grates tested are hydraulically effective at lower velocities. Section 7.2.1 and Section 7.3.1 discuss the interception capacity of grate inlets on grade and in sags, respectively.

Lessons from Experience: Avoid Backwards Installation

The curved-vane and tilt bar grates work only when the vanes/tilt bars are angled toward the oncoming gutter flow. If these grates are removed for maintenance and replaced backwards, they will be ineffective. Consider designs that prevent the grate from being incorrectly installed, e.g., with offset anchor bolt holes.

Several agencies also contributed to research to determine the interception capacity of curb-opening inlets. Colorado State University derived design procedures documented in this manual from experimental work (Izzard 1946, Bauer and Woo 1964). Section 7.2.2 and Section 7.3.2 discuss the interception capacity of curb inlets on grade and in sags, respectively.

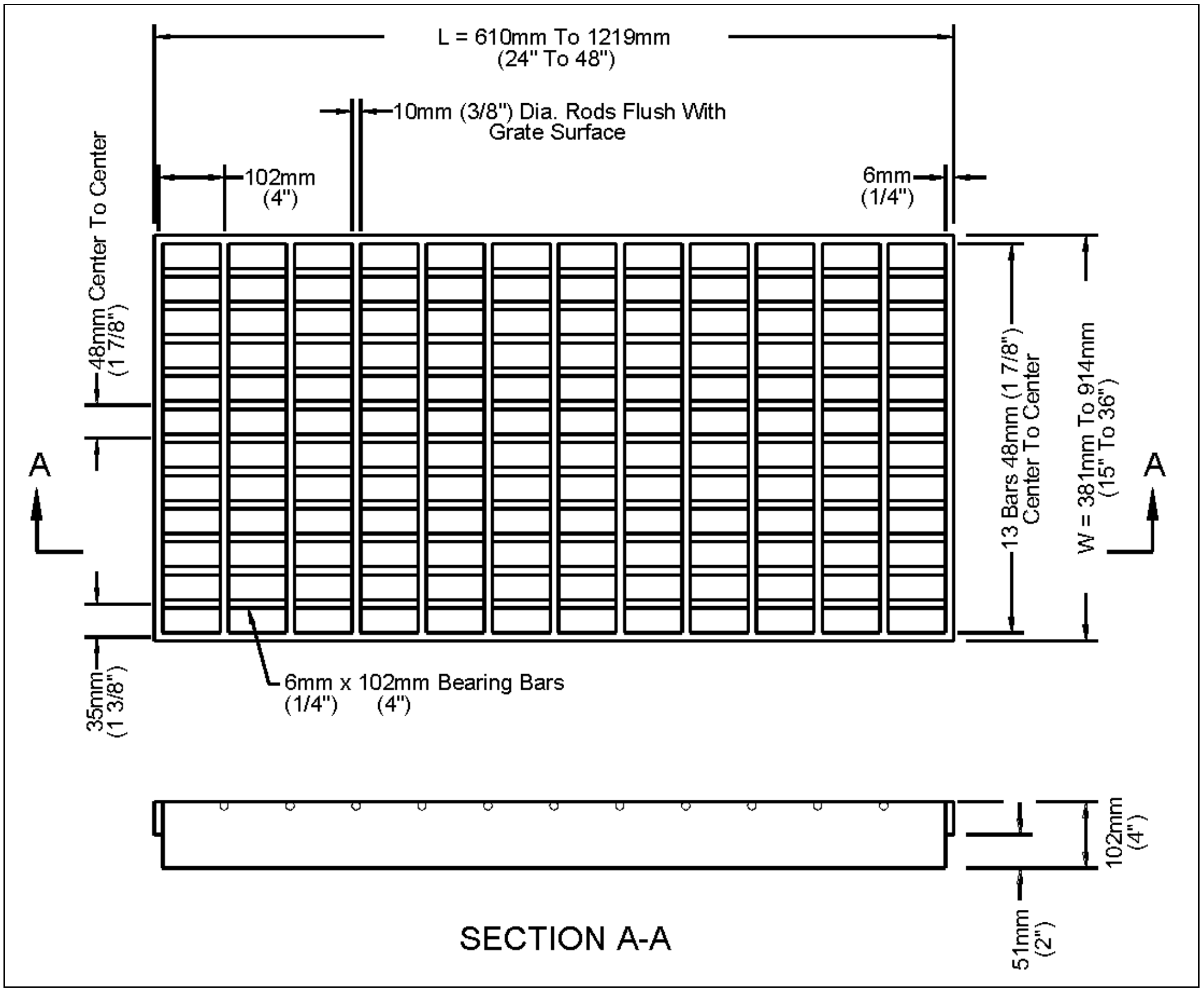

Figure 7.2. P-1-7/8-4 grate (P-1-7/8 is this grate without transverse rods).

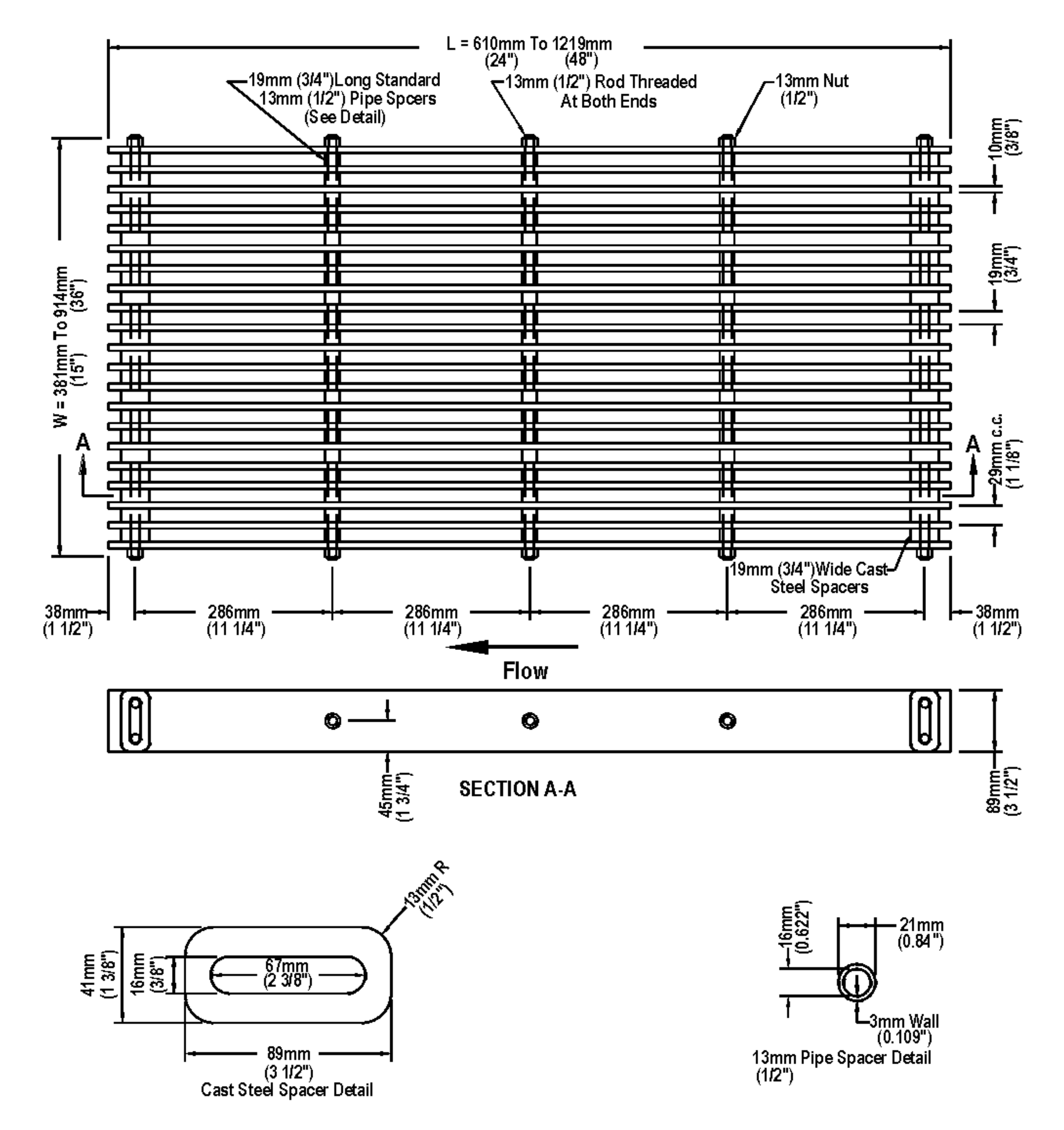

Figure 7.3. P-1-1/8 grate.

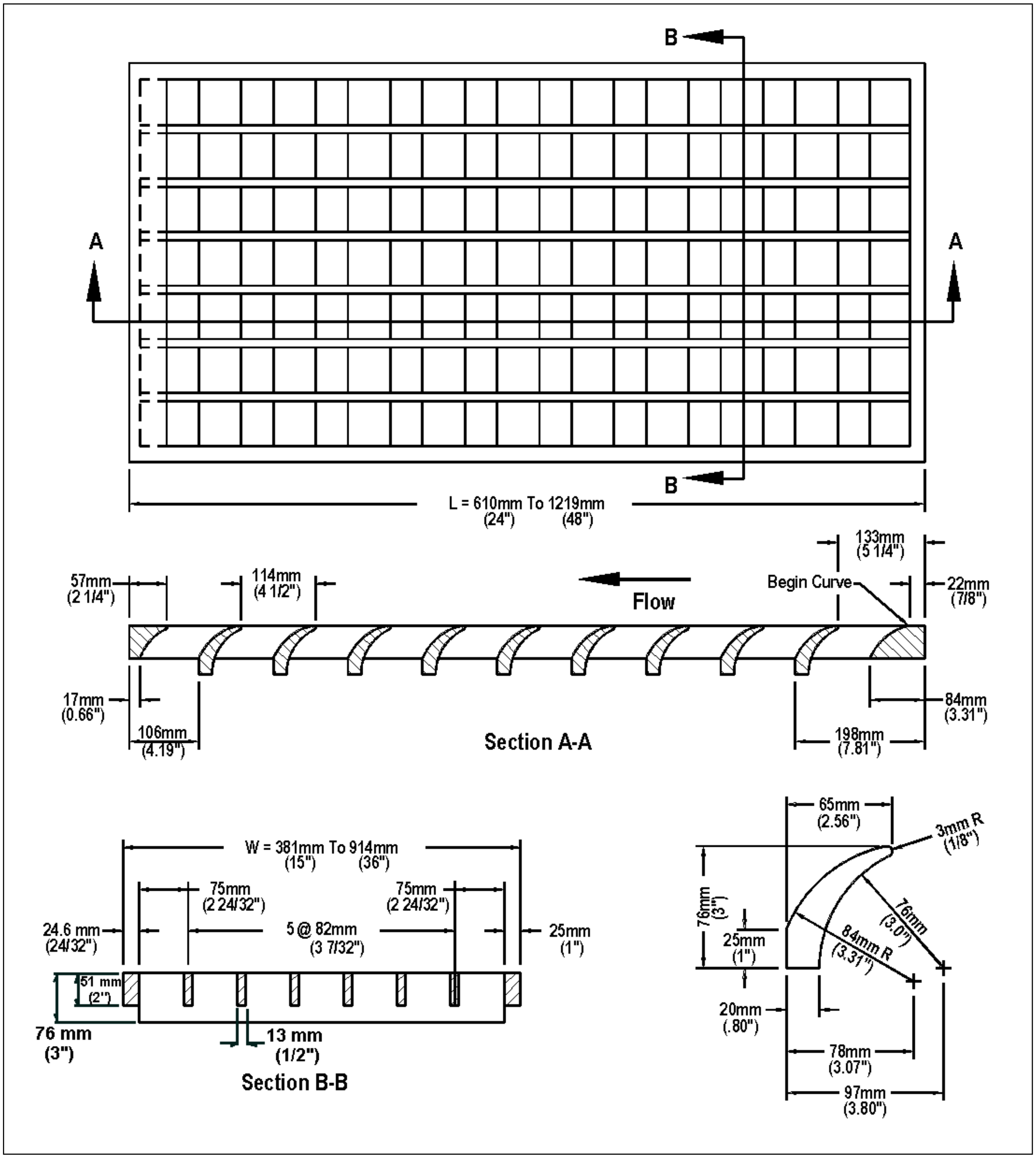

Figure 7.4. Curved vane grate.

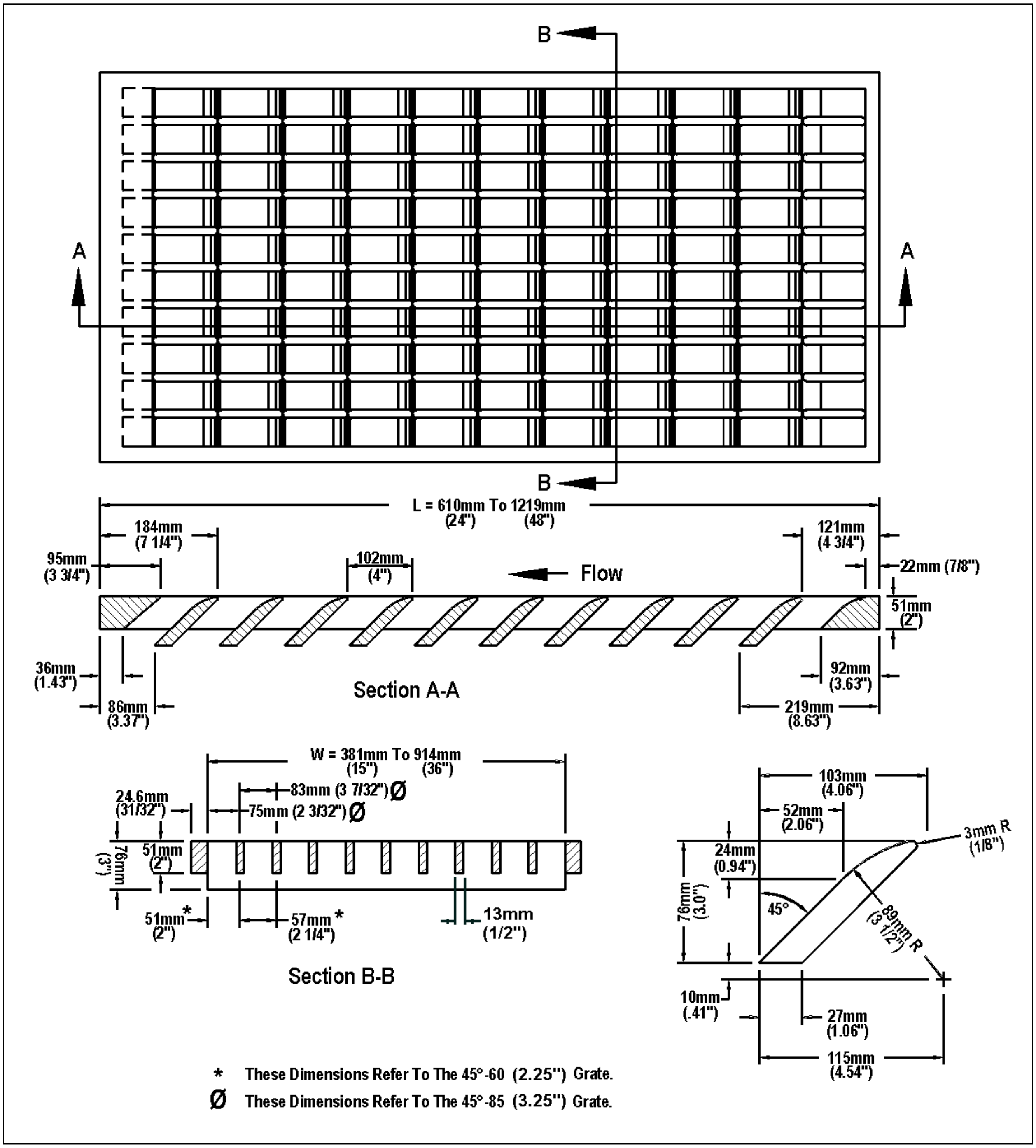

Figure 7.5. Tilt-bar grate: 45°- 60 (2.25 inch) and 45°- 85 (3.25 inch).

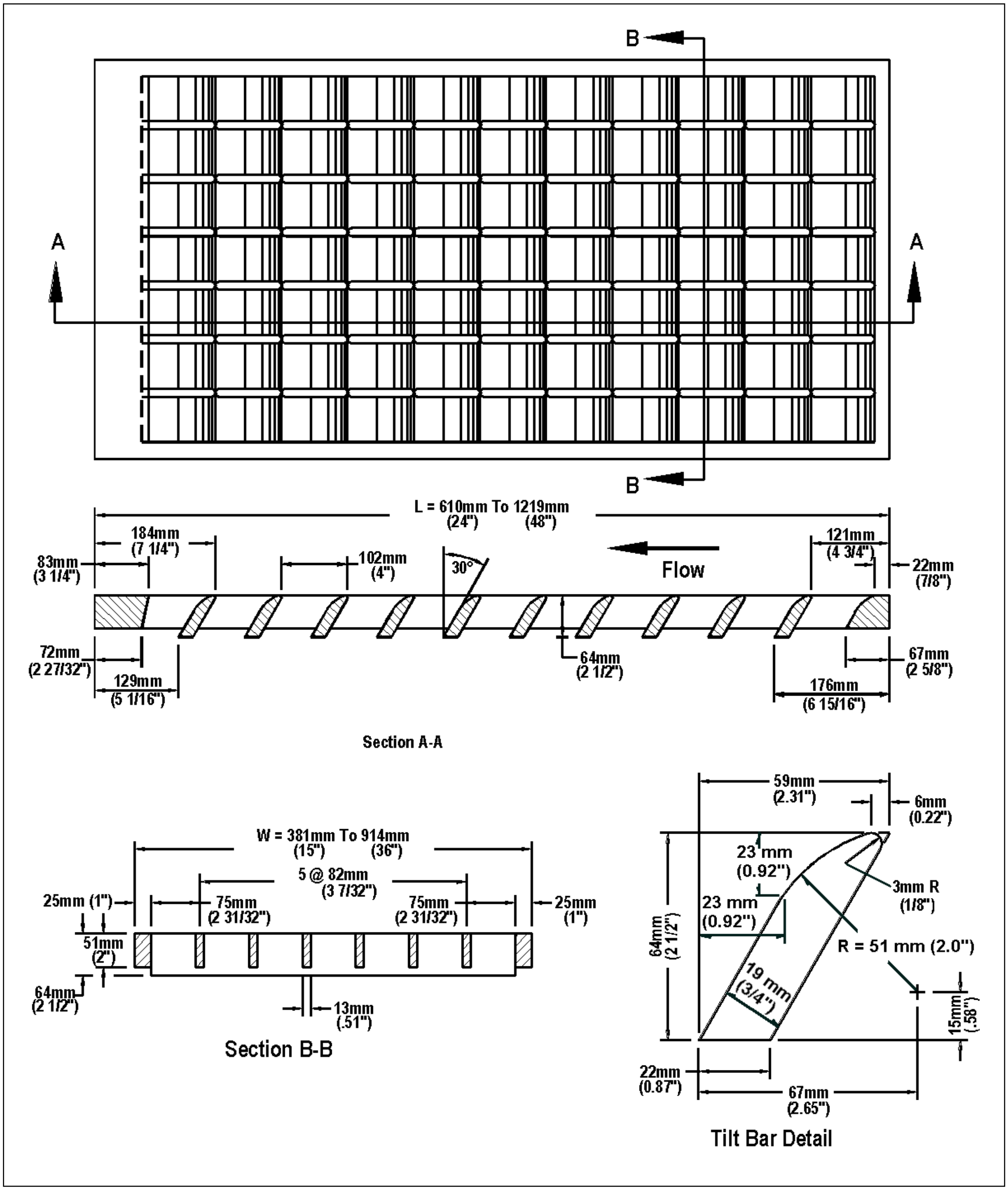

Figure 7.6. Tilt-bar grate: 30°- 85 (3.25 inch).

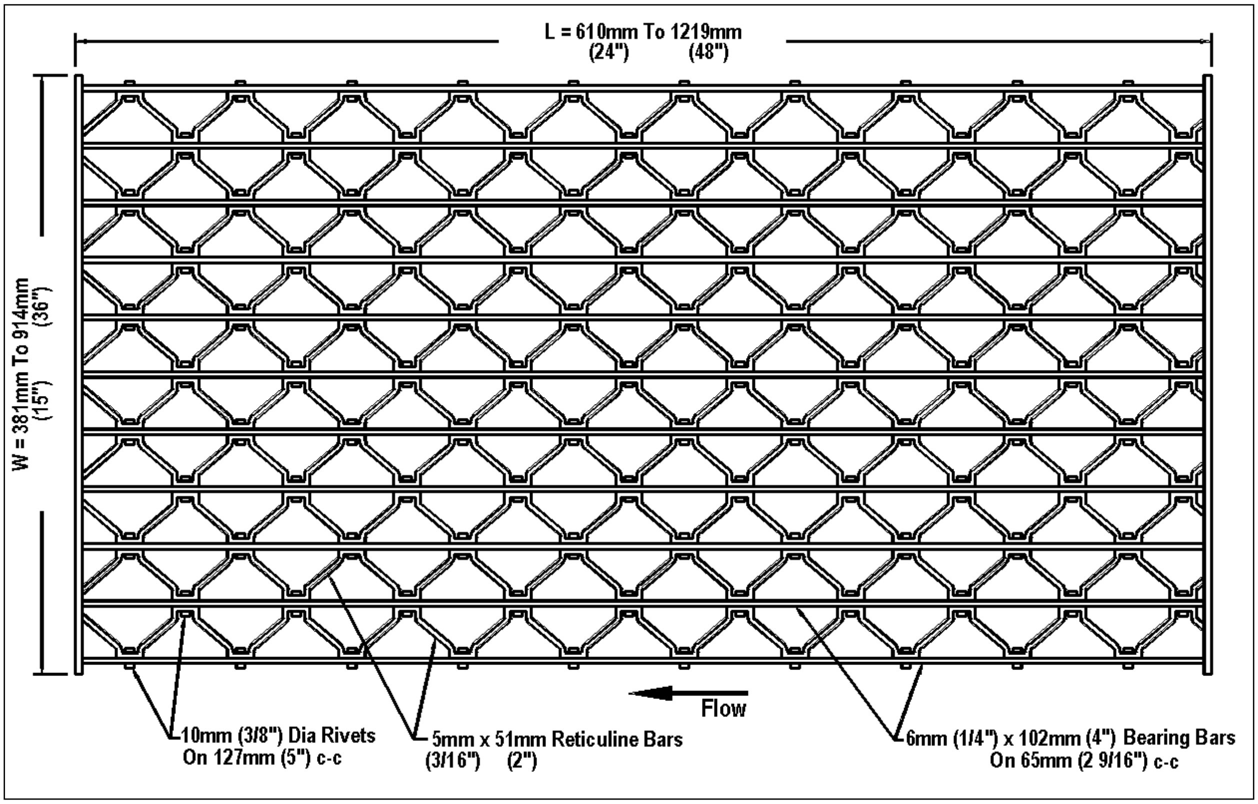

Figure 7.7. Reticuline grate.

7.1.3 Clogging Potential

All types of inlets, including curb-opening inlets, experience clogging, with those in low points (sag conditions) the most vulnerable. Attempts to simulate clogging tendencies in the laboratory demonstrate the importance of parallel bar spacing in debris handling efficiency. Grates with wider spacings of longitudinal bars pass debris more efficiently. Except for reticuline grates, testers did not conduct trials of grates with lateral bar spacing of less than 4 inches, so they did not supply information concerning debris handling capabilities of many grates. Problems with clogging are largely local since the quantity of debris varies significantly from one locality to another. Some localities contend with only a small amount of debris while others experience extensive clogging of drainage inlets. Since partial clogging of inlets on grade rarely causes major problems, local experience will indicate where an allowance for reduction in inlet interception capacity is advisable.

Table 7.2 provides a ranking of debris handling capabilities of various grates based on laboratory tests using simulated “leaves” (Burgi and Gober 1977). The table shows a clear difference in efficiency between the grates with the 3-1/4 inch (83 mm) longitudinal bar spacing and those with smaller spacings. The efficiencies shown in the table are suitable for comparisons between the grate designs tested are not an indication of field performance since the testing procedure did not simulate actual field conditions. Some local transportation agencies have developed factors for use of debris handling characteristics with specific inlet configurations. Curb-opening inlets have good debris handling capabilities and are less likely to clog compared with grate inlets.

Table 7.2. Average debris handling efficiencies of grates.

Rank	Grate	Longitudinal Slope (ft/ft)	
		0.005	0.04
1	Curved Vane	46	61
2	30°- 85 Tilt-Bar	44	55
3	45°- 85 Tilt-Bar	43	48
4	P-1-7/8	32	32
5	P-1-7/8-4	18	28
6	45°- 60 Tilt-Bar	16	23
7	Reticuline	12	16
8	P-1-1/8	9	20

7.1.4 Pedestrian, Bicycle, and ADA Safety

Table 7.3 ranks grates according to relative bicycle and pedestrian safety. Burgi and Gober (1977) and Burgi (1978a) established the bicycle safety ratings based on a subjective test program. However, all the grates are considered bicycle and pedestrian safe except the P-1-7/8 for most bicycles and the P-1-1/8 grate for very narrow racing bicycle tires. Designers will also consider the potential for scooters and other types of shared micromobility and personal transport systems and equipment that may have access to areas around drainage inlets.

Table 7.3. Inlet ranking for bicycle and pedestrian safety.

Rank	Grate Type
1	P-1-7/8-4
2	Reticuline
3	P-1-1/8
4	45° - 85 Tilt-Bar
5	45° - 60 Tilt-Bar
6	Curved Vane
7	30° - 85 Tilt-Bar

In addition, areas where pedestrians using various mobility assistance devices may be found, the Americans with Disabilities Act (ADA) has resulted in grates designated as ADA-compliant. Generally, ADA-compliant grates include smaller openings than non-compliant grates reducing their flow interception efficiency under some conditions. Like other grates, interception efficiency is grate specific. Lottes and Bojanowski (2015) compared the efficiency of a curved vane grate and an ADA-compliant grate with the same dimensions and confirmed a reduction in interception efficiency. However, the magnitude of reductions varies with the design.

7.2 Interception Capacity of Inlets on Grade

Inlet interception capacity, Q_i, is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet, E, is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency, E, is expressed by the following equation:

$$E = \frac{Q_i}{Q} \tag{7.1}$$

where:

E	=	Inlet efficiency
Q	=	Total gutter flow, ft^3/s (m^3/s)
Q_i	=	Intercepted flow, ft^3/s (m^3/s)

Bypass (carryover) flow is not intercepted by an inlet and is determined as follows:

$$Q_b = Q - Q_i \tag{7.2}$$

where:

Q_b	=	Bypass flow, ft^3/s (m^3/s)

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. Gutter flow characteristics also affect inlet interception capacity. The depth of water next to the curb is the major factor in the interception capacity of both grate inlets and curb-opening inlets. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate, and the velocity of flow in the gutter.

Interception capacity of a curb-opening inlet largely depends on flow depth at the curb and curb-opening length. Local gutter depression at a curb-opening or a continuously depressed gutter increase depth at the curb, interception capacity, and efficiency. Top slab supports placed flush with the curb line can substantially reduce the interception capacity of curb openings. Tests have shown that such supports reduce the effectiveness of openings downstream of the support by as much as 50 percent and, if debris is caught at the support, interception by the downstream portion of the opening may be reduced to near zero. However, Schalla (2016) and Muhammad (2018) concluded that, in the absence of debris, flush slab supports did not reduce interception. Where feasible, recessing intermediate top slab supports several inches from the curb line and rounding their shape reduces loss of interception capacity.

Slotted inlets function in essentially the same manner as curb-opening inlets, i.e., as weirs with flow entering from the side. Interception capacity depends on flow depth and inlet length. Efficiency depends on flow depth, inlet length, and total gutter flow.

The interception capacity of an equal length combination inlet consisting of a grate placed alongside a curb opening on a grade does not differ materially from that of a grate only. Interception capacity and efficiency depend on the same factors which affect grate capacity and efficiency. A combination inlet consisting of a curb-opening inlet placed upstream of a grate inlet (a sweeper configuration) has a capacity equal to that of the curb-opening length upstream of the grate plus that of the grate. However, capacity of the grate lowers because of the reduced spread and depth of flow over the grate resulting from the interception by the curb opening. This inlet configuration has the added advantage of intercepting debris that might otherwise clog the grate and deflect water away from the inlet.

The following sections present methods for estimating interception capacity of inlets on grade. For locally depressed inlets, the quantity of flow reaching the inlet depends on the upstream gutter section geometry and not the locally depressed section geometry.

7.2.1 Grate Inlets

Grates can be effective highway pavement drainage inlets where clogging with debris is not a problem. (See Section 7.1.3 for a discussion of clogging potential.) The FHWA sponsored research to develop interception efficiencies for grate inlets (Burgi and Gober 1977, Burgi 1978a, Burgi 1978b, Pugh 1980). Conceptually, flow approaching a grate inlet can be divided into frontal flow (flow in the gutter approaching the front edge of the grate) and side flow (flow in the gutter beyond the front edge). The research demonstrated that grates intercept all frontal flow until a velocity is reached at which water begins to splash over the grate. At velocities greater than "splash-over" velocity, grate efficiency in intercepting frontal flow is diminished. The research also demonstrated that grates also intercept a portion of the flow along the length of the grate, or the side flow, depending on the cross slope of the pavement, the grate length, and the flow velocity.

The ratio of frontal flow to total gutter flow for a uniform cross slope is expressed as:

$$E_o = \frac{Q_w}{Q} = 1-\left(1-W/T\right)^{2.67} \tag{7.3}$$

where:

E_o	=	Frontal flow ratio
Q	=	Total gutter flow, ft^3/s (m^3/s)
Q_w	=	Frontal flow in width W, ft^3/s (m^3/s)
W	=	Width of depressed gutter or grate, ft (m)
T	=	Total spread of water, ft (m)

The ratio of side flow, Q_s, to total gutter flow is:

$$Q_s / Q = 1-\left(Q_w / Q\right) = 1-E_o \tag{7.4}$$

When the velocity approaching the grate is less than the splash-over velocity, the grate will intercept essentially all frontal flow. Conversely, when the gutter flow velocity exceeds the splash-over velocity for the grate, only part of the flow will be intercepted. The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed as:

$$R_f = 1-K_u\left(V-V_o\right) \tag{7.5}$$

where:

K_u	=	Unit conversion constant, 0.09 in CU (0.295 in SI)
V	=	Velocity of flow in the gutter, ft/s (m/s)
V_o	=	Gutter velocity where splash-over first occurs, ft/s (m/s)

Figure 7.8 provides the splash-over velocity for several grate types for equation 7.5. The approaching gutter flow velocity is computed from the gutter equation (equation 5.3). If the splash-

over velocity is greater than the gutter flow velocity, R_f is 1.0 because capture efficiency cannot exceed 100 percent.

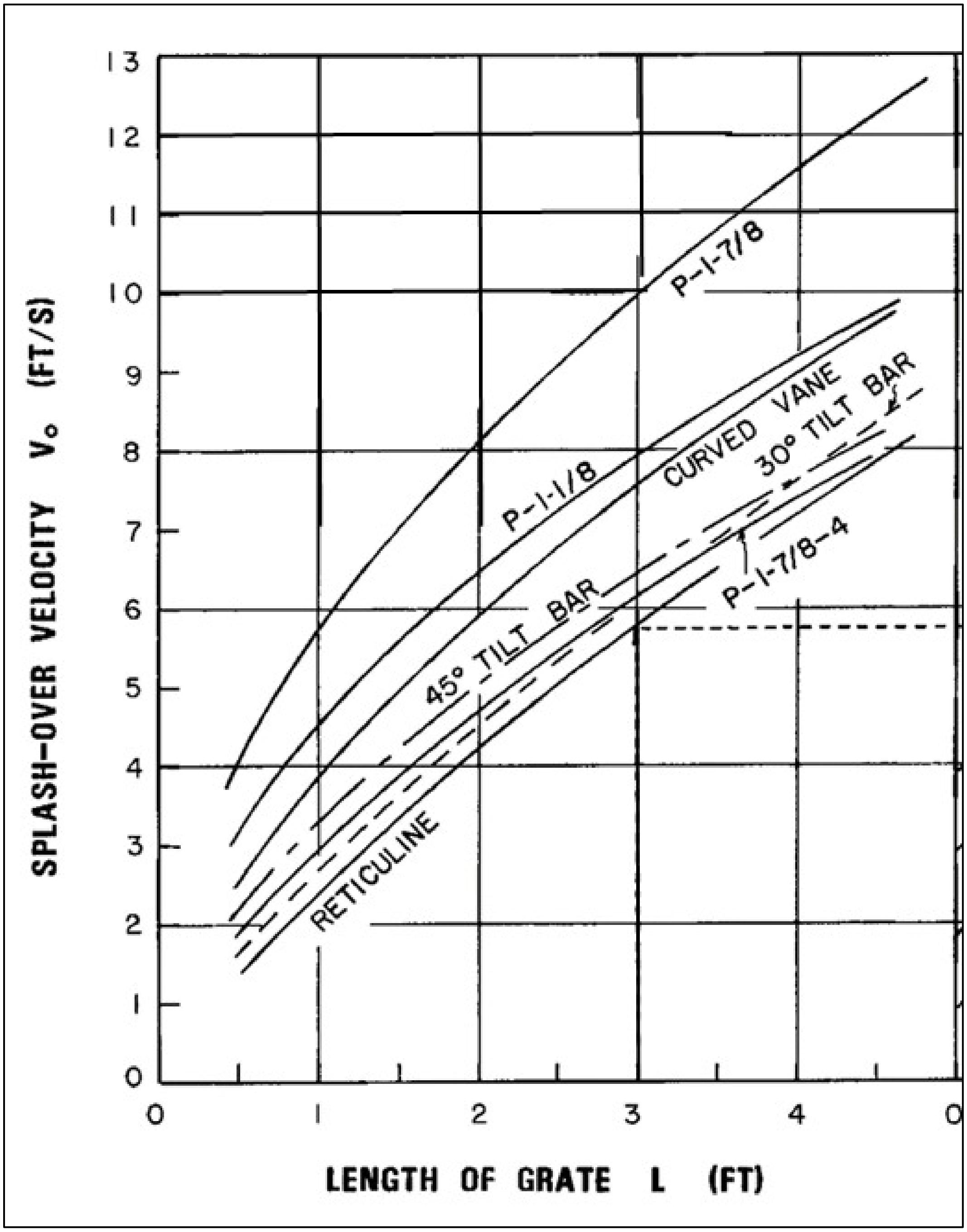

Figure 7.8. Splash-over velocity for grate inlets.

The inlet also intercepts part of the flow along the side of the grate. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, depends on the pavement cross slope, the grate length, and the flow velocity:

$$R_s = 1 / \left(1 + (K_u\ V^{1.8}) / (S_x L^{2.3})\right) \quad (7.6)$$

where:

K_u = Unit conversion constant, 0.15 in CU (0.0828 in SI)

Equation 7.6 underestimates side flow interception where the velocity is low, and the spread only slightly exceeds the grate width. Error due to this deficiency is small.

The total efficiency of a grate inlet, E, is expressed as:

$$E = R_f E_o + R_s \left(1 - E_o\right) \tag{7.7}$$

The first term on the right side of the equation is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

Frontal flow to total gutter flow ratio, E_o, for grate inlets in composite gutter sections assumes frontal flow width equal equals the depressed gutter section width which also equals the grate width. If the grate width is less than W, the frontal flow ratio, E_o, is modified to evaluate grate efficiency. As an approximation, the adjusted frontal flow ratio assumes that average velocity in the gutter is applicable to the entire gutter width. Then, the adjusted frontal flow ratio, E'_o, is calculated by multiplying E_o by a flow area ratio. The area ratio is the gutter flow area in a width equal to the grate width divided by the total flow area in the depressed gutter section and is applied as follows:

$$E'_o = E_o \left(A'_w / A_w\right) \tag{7.8}$$

where:

E'_o = Adjusted frontal flow area ratio for grates in composite cross-sections
A'_w = Gutter flow area in a width equal to the grate width, ft^2 (m^2)
A_w = Flow area in the depressed gutter width, ft^2 (m^2)

Equation 7.9 describes the interception capacity of a grate inlet on grade as the efficiency of the grate multiplied by the total gutter flow. The adjusted frontal ratio, E'_o is substituted for E_o when the grate width is less than the gutter width.

$$Q_i = E\,Q = Q\left[R_f E_o + R_s \left(1 - E_o\right)\right] \tag{7.9}$$

Example 7.1:	Interception capacity of a grate inlet on grade.
Objective:	Find the interception capacity of different sizes and types of grates.

Given: Given a uniform gutter section where bicycles are not permitted:

T = 9.84 ft (3.0 m)
S_L = 0.04 ft/ft (0.04 m/m)
S_x = 0.025 ft/ft (0.025 m/m)
W = 2 ft (0.61 m)
n = 0.016

Grates for evaluation:

a. P-1-7/8, 2.0 ft x 2.0 ft (0.61 m x 0.61 m)

b. Reticuline, 2.0 ft x 2.0 ft (0.61 m x 0.61 m)

c. P-1-7/8, 2.0 ft x 4.0 ft (0.61 m x 1.22 m)

d. Reticuline, 2.0 ft x 4.0 ft (0.61 m x 1.22 m)

Step 1. Estimate flow in the gutter.

Using the gutter equation (equation 5.2):

$Q = (K_u/n)\ S_x^{1.67}\ S_L^{0.5}\ T^{2.67} = (0.56)/(0.016)\ (0.025)^{1.67}\ (0.04)^{0.5}\ (9.84)^{2.67} = 6.62\ ft^3/s$

Step 2. Determine frontal flow ratio, E_0.

$W/T = 2.0/9.84 = 0.2$

From equation 7.3:

$E_o = 1 - (1 - W/T)^{2.67} = 1 - (1 - 0.2)^{2.67} = 0.45$

Step 3. Compute the gutter flow velocity.

Using the gutter equation (equation 5.3):

$V = (K_u/n)\ S_L^{0.5}\ S_x^{0.67}\ T^{0.67} = \{(1.11)/(0.016)\}\ (0.04)^{0.5}\ (0.025)^{0.67}\ (9.84)^{0.67} = 5.4\ ft/s$

Step 4. Determine the frontal flow efficiency.

Using equation 7.5 for the P-1-7/8 grate (2 ft x 2 ft):

From Figure 7.8, the splash-over velocity is approximately 8.2 ft/s.

$R_f = 1 - K_u\ (V - V_o) = 1 - 0.09\ (5.4 - 8.2) = 1.25$ which is taken as $R_f = 1.0$ because $R_f \leq 1.0$.

Repeat the process for the other three grates.

Step 5. Determine the side flow efficiency.

Using equation 7.6, for the P-1-7/8 grate (2 ft x 2 ft):

$R_s = 1/[1 + (K_u\ V^{1.8}) / (S_x\ L^{2.3})] = 1/[1 + (0.15)\ (5.4)^{1.8} / [(0.025)\ (2.0)^{2.3}] = 0.038$

Repeat the process for the other three grates.

Step 6. Compute the interception capacity.

Using equation 7.9, for the P-1-7/8 grate (2 ft x 2 ft):

$Q_i = Q\ [R_f\ E_o + R_s\ (1 - E_o)] = (6.62)\ [(1.0)\ (0.45) + (0.0.038)\ (1- 0.45)] = 3.15\ ft^3/s$

Repeat the process for the other three grates. Table 7.4 summarizes the results.

Table 7.4. Interception and efficiency results for example.

Grate Type and Size (width by length)	Frontal Flow Efficiency, R_f	Side Flow Efficiency, R_s	Interception, Q_i, (ft^3/s)
P-1-7/8 (2.0 ft by 2.0 ft)	1.0	0.038	3.15
Reticuline (2.0 ft by 2.0 ft)	0.88	0.038	2.80
P-1-7/8 (2.0 ft by 4.0 ft)	1.0	0.163	3.60
Reticuline (2.0 ft by 4.0 ft)	1.0	0.163	3.60

Solution:	Table 7.4 summarizes the interception capacities. The P-1-7/8 parallel bar grate intercepts 48 percent of the total flow compared with 42 percent for the reticuline grate. Increasing the length of the grates would not be cost-effective, because the increase in side flow interception is small.

7.2.2 Curb-Opening Inlets

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are less susceptible to clogging compared with grate inlets and offer little interference to traffic operation. They perform well compared with grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

Curb-opening heights vary in dimension; however, a typical maximum height is approximately 4 to 6 inches. The length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed as:

$$L_T = K_u Q^{0.42} S_L^{0.3} \left[1/\left(n S_x\right)\right]^{0.6} \tag{7.10}$$

where:

L_T	=	Curb-opening length required to intercept 100 percent of the gutter flow, ft (m)
K_u	=	Unit conversion constant, 0.6 in CU (0.817 in SI)
S_L	=	Longitudinal slope, ft/ft (m/m)
Q	=	Gutter flow, ft^3/s (m^3/s)

Alternative On-Grade Curb Opening Design Equation

Laboratory and computational fluid dynamic modeling research suggests that equation 7.10 underestimates the length to intercept 100 percent of the gutter flow, especially for longer (greater than or equal to 10 ft) inlets (Schalla 2016, Muhammad 2018, FHWA 2022a). Therefore, inlets designed with the equation may not capture the intended flow and may allow for additional bypass. The FHWA's researchers prepared an alternative approach that appears to remedy this problem (FHWA 2022a). The FHWA encourages designers to review the technical note describing the research, results, recommendations, and sample computations (FHWA 2022a).

For depressed curb-opening inlets, as shown in Figure 7.9, or curb openings in depressed gutter sections an equivalent cross slope, S_e, replaces S_x in equation 7.10. S_e is:

$$S_e = S_x + S_w^{'} E_0 \tag{7.11}$$

where:

S'_w	=	Cross slope of the gutter measured from the pavement cross slope, S_x, ft/ft (m/m)
E_o	=	Frontal flow ratio for the depressed section determined by the gutter configuration upstream of the inlet

S_w' is computed as a/W or equivalently, $S_w - S_x$, where S_w is shown in Figure 7.9. Using the equivalent cross slope, S_e, equation 7.10 becomes:

$$L_T = K_u Q^{0.42} S_L^{0.3} \left[1 / \left(n S_e\right)\right]^{0.6} \tag{7.12}$$

Increasing the cross slope or the equivalent cross slope reduces the required curb-opening length for 100 percent interception. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

The efficiency of curb-opening inlets shorter than the length required for total interception is applicable for either uniform cross slopes or composite cross slopes and is expressed as:

$$E = 1 - \left[1 - \frac{L}{L_T}\right]^{1.8} \tag{7.13}$$

where:

L = Curb-opening length, ft (m)

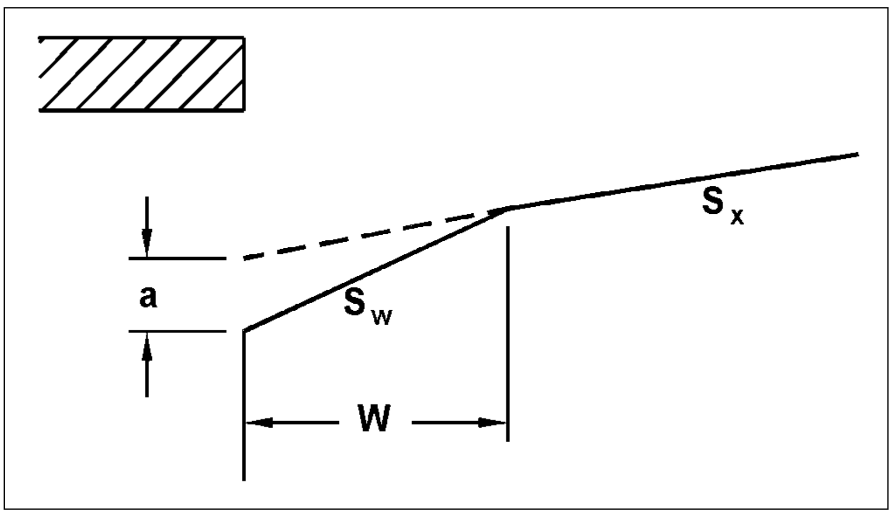

Figure 7.9. Depressed curb-opening inlet.

Example 7.2:	Interception capacity of a curb inlets on grade.
Objective:	Find the interception capacity of a curb inlet: A) without a depressed gutter section and B) with a depressed gutter section.

Given: A curb-opening inlet in the following situation:

S_L = 0.01 ft/ft (m/m)
S_x = 0.02 ft/ft (m/m)
n = 0.016
Q = 1.77 ft^3/s (0.050 m^3/s)
L = 9.84 ft (3.0 m)

For the gutter depression:

a = 1 inch (25.4 mm)
W = 2 ft (0.61 m)

Step A1. Determine the length of curb opening required for total interception of gutter flow without gutter depression.

Using equation 7.10:

$L_T = K_u\, Q^{0.42}\, S_L^{0.3}\, (1/(n\, S_x))^{0.6} = 0.6\, (1.77)^{0.42}(0.01)^{0.3}(1/[(0.016)(0.02)])^{0.6} = 23.9$ ft

Step A2. Compute the curb-opening efficiency without gutter depression.

Using equation 7.13:

$L / L_T = 9.84 / 23.9 = 0.41$

$E = 1 – (1 – L / L_T)^{1.8} = 1 – (1 – 0.41)^{1.8} = 0.61$

Step A3. Compute the interception capacity without gutter depression.

$Q_i = E\, Q = (0.61)\, (1.77) = 1.08\ ft^3/s$

Step B1. Determine the W/T ratio for the depressed gutter.

Determine spread, T (see example 5.2(B))

Assume $Q_s = 0.64\ ft^3/s$

$Q_w = Q – Q_s = 1.77 – 0.64 = 1.13\ ft^3/s$

$E_o = Q_w / Q = 1.13 / 1.77 = 0.64$

$S_w = S_x + a/W = 0.02 + (0.083/2.0) = 0.062$

$S_w/S_x = 0.062 / 0.02 = 3.1$

Use equation 5.7 to determine that W/T = 0.24

Then, T = W / (W/T) = 2.0 / 0.24 = 8.3 ft

$T_s = T – W = 8.3 – 2.0 = 6.3$ ft

Use the gutter flow equation (equation 5.2) to obtain Q_s.

$Q_s = (K_u/n)\, S_x^{1.67}\, S_L^{0.5}\, T_s^{2.67} = \{(0.56) / (0.016)\}\, (0.02)^{1.67}(0.01)^{0.5}\, (6.3)^{2.67} = 0.69\ ft^3/s$

Since this is close to the assumed Q_s no further iterations are necessary.

Step B2. Determine efficiency of curb opening with the depressed gutter section.

Using equation 7.11:

$S_e = S_x + S'_w\, E_o = S_x + (a/W)E_o = 0.02 + [(0.083)/(2.0)](0.64) = 0.047$

Using equation 7.12:

$L_T = K_U\, Q^{0.42}\, S_L^{0.3}\, [1/(n\, S_e)]^{0.6} = (0.6)\, (1.77)^{0.42}\, (0.01)^{0.3}\, [1/((0.016)(0.047))]^{0.6} = 14.3$ ft

Using equation 7.13:

$L/L_T = 9.84/14.3 = 0.69$

$E = 1 – (1 – L/L_T)^{1.8} = 1 – (1 – 0.69)^{1.8} = 0.88$

Step B3. Compute curb-opening interception.

Using equation 7.1:

$Q_i = Q\, E = (1.77)\, (0.88) = 1.56\ ft^3/s$

Solution:	A) the interception of the curb inlet without the depressed gutter is 1.08 ft^3/s (0.031 m^3/s). B) the interception of the same inlet with a depressed gutter is 1.55 ft^3/s (0.044 m^3/s), an increase of approximately 40 percent under these conditions.

The following example illustrates computation of the required curb-opening length to intercept 100 percent of the flow given the gutter flow.

Example 7.3:	Curb-opening length on grade.
Objective:	Find the minimum length of a locally depressed curb-opening inlet required to intercept 100 percent of the gutter flow.

Given: A curb-opening inlet in the following situation:

S_L = 0.01 ft/ft (m/m)
S_x = 0.02 ft/ft (m/m)
n = 0.016
Q = 2.26 ft^3/s (0.064 m^3/s)
T = 8.2 ft (2.5 m)

For the local depression:

a = 2.0 inches (51 mm)
W = 2.0 ft (0.61 m)
E_o = 0.70

Step 1. Compute the composite cross slope for the locally depressed section.

Using equation 7.11:

$S'_w = a/W = (2/12) / 2 = 0.0833$

$S_e = S_x + S'_w E_o = 0.02 + (0.0833)(0.7) = 0.078$

Step 2. Compute the length of curb-opening inlet required for 100 percent interception.

Using equation 7.12:

$L_T = K_U Q^{0.42} S_L^{0.3} (1 / n S_e)^{0.6} = (0.60)(2.26)^{0.42}(0.01)^{0.3} [1/ \{(0.016)(0.078)\}]^{0.6} = 11.7$ ft

Solution:	For the given conditions, the curb-opening length for 100 percent interception is 11.7 ft (3.56 m).

7.2.3 Slotted Inlets

Slotted inlets are effective pavement drainage inlets in some applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. Figure 7.10 illustrates an installation. Deposition in the pipe is the problem most encountered. The configuration of slotted inlets makes them accessible for cleaning with a high-pressure water jet. If use of a slotted inlet includes the need for customized curb or gutter, or both, the cost of customization may lead to other inlet options.

Flow interception by slotted inlets is like that of curb-opening inlets because both operate as a side weir with the flow subjected to lateral acceleration because of the cross slope of the pavement. The FHWA tested slotted inlets with slot widths greater than or equal to 1.75 inches and concluded that the length of slotted inlet required for total interception can be computed by equation 7.10. Similarly, FHWA concluded that equation 7.13 also applies to slotted inlets and can be used to obtain the inlet efficiency for a given inlet length.

For overland flow, FHWA research indicates that 1-, 1.75-, and 2.5-inch-wide slotted drain inlets can capture 100 percent of the approaching flow up to 0.025 $ft^3/s/ft$ for water depths ranging from 0.38 to 0.56 inches on slopes ranging from 0.005 to 0.09 ft/ft. During tests at a system capacity of 0.040 $ft^3/s/ft$, a small amount of splash-over occurred.

7.2.4 Combination Inlets

Figure 7.11 depicts a combination inlet that combines a grate inlet with a curb opening. When the curb opening and grate length are equal, as shown, the interception capacity of is the same as the grate alone. The benefit of the curb opening is to capture debris and reduce the clogging potential of the grate.

Figure 7.12 illustrates a sweeper combination inlet with the curb opening extended upstream of the grate. In this installation the curb opening also intercepts debris that might clog the grate and augments interception capacity. In a sweeper combination inlet, interception capacity equals the sum of the curb opening upstream of the grate plus the grate capacity. However, the grate interception is reduced because the flow captured by the curb opening reduces the frontal flow approaching the grate by the same amount. The following example illustrates computation of the interception capacity of a sweeper combination inlet.

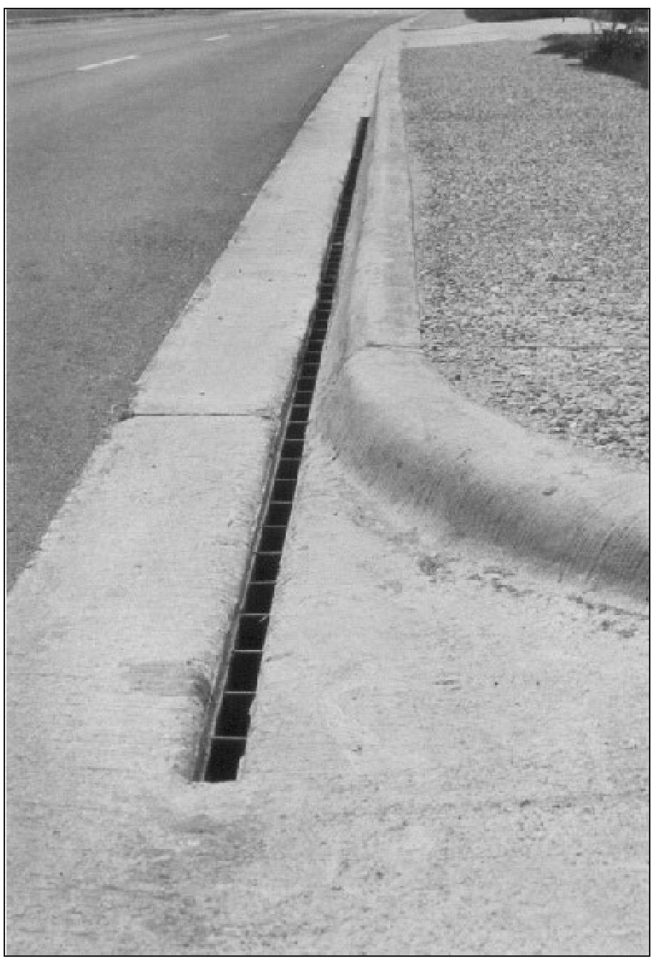

Figure 7.10. Slotted drain inlet at an intersection.

Figure 7.11. Combination curb-opening with 45-degree tilt-bar grate inlet.

Figure 7.12. Sweeper combination inlet.

Example 7.4: Sweeper combination inlet on grade.

Objective: Find the interception capacity of a combination inlet with the grate inlet located at the downstream end of the curb opening.

Given: The following composite gutter and flow information:

S_L	=	0.01 ft/ft (m/m)
S_x	=	0.02 ft/ft (m/m)
n	=	0.016
a	=	1 inch (25.4 mm)
W	=	2 ft (0.61 m)
Q	=	1.77 ft^3/s (0.050 m^3/s)

Curb-opening length = 9.84 ft (3.0 m)
Curved vane grate is 2 ft x 2 ft (0.61 m x 0.61 m)

Step 1. Compute the portion of the curb opening contributing to the interception capacity of the combination inlet.

L = 9.84 – 2.0 = 7.84 ft

Step 2. Determine the frontal flow ratio for the depressed portion of the gutter.

Determine spread, T (see example 5.2(B))

Assume $Q_s = 0.64$ ft^3/s

$Q_w = Q – Q_s = 1.77 – 0.64 = 1.13$ ft^3/s

$E_o = Q_w / Q = 1.13 / 1.77 = 0.64$

Step 3. Determine the efficiency of the curb opening with the depressed gutter section.

Using equation 7.11:

$S_e = S_x + S'_w E_o = S_x + (a/W)E_o = 0.02 + [(0.083)/(2.0)](0.64) = 0.047$

Using equation 7.12:

$L_T = K_U Q^{0.42} S_L^{0.3} [1/(n S_e)]^{0.6} = (0.6) (1.77)^{0.42} (0.01)^{0.3} [1/((0.016)(0.047))]^{0.6} = 14.3$ ft

$L / L_T = 7.84 / 14.3 = 0.55$

Using equation7.13:

$E = 1 – (1 – L / LT)^{1.8} = 1 – (1 – 0.55)^{1.8} = 0.76$

$Q_{ic} = E Q = (0.76)(1.77) = 1.35$ ft^3/s

Step 4. Compute the flow approaching the grate.

$Q_g = Q – Q_{ic} = 1.77 – 1.35 = 0.42$ ft^3/s

Step 5. Estimate the side flow approaching the grate.

Determine spread, T (see example 5.2(B))

Assume $Q_s = 0.01$ ft^3/s

$Q_w = Q – Q_s = 0.42 – 0.01 = 0.41$ ft^3/s

$E_o = Q_w / Q = 0.41/0.42 = 0.97$

$S_w = S_x + a/W = 0.02 + (0.083/2.0) = 0.062$

$S_w / S_x = 0.062 / 0.02 = 3.1$

From equation 5.7:

$W/T = 1/\{(1/[(1/(1/E_o – 1))(S_w/S_x)+1]^{0.375} – 1) (S_w / S_x) +1\}$

$= 1/\{(1/[(1/(1/0.97 – 1))(3.1)+1]^{0.375} – 1) (3.1)+1\} = 0.62$

$T = W / (W/T) = 2.0 / 0.62 = 3.2$ ft

$T_s = T – W = 3.2 -2.0 = 1.2$ ft

From equation 5.2:

$Q_s = (K_u/n) S_x^{1.67} S_L^{0.5} T_s^{2.67} = \{(0.56) / (0.016)\} (0.02)^{1.67}(0.01)^{0.5} (1.2)^{2.67} = 0.01$ ft^3/s

Since this matches the assumed Q_s no further iterations are necessary.

Step 6. Compute the interception capacity of the grate.

Estimate the flow area in the composite gutter approaching the grate:

$A = 0.5 T^2 S_x + 0.5 a W = (0.5)(3.2)^2(0.02) + (0.5)(0.083)(2.0) = 0.185$

Determine velocity, V.

$V = Q / A = 0.42 / 0.185 = 2.27$ ft/s

From Figure 7.8, splash-over velocity equals 6 ft/s.

From equation 7.5:

$R_f = 1 – K_u (V – V_o) = 1 – 0.09 (2.27 – 6.0) > 1.0$, therefore $R_f = 1.0$

From equation 7.6:

$R_s = 1 / (1 + (K_u V^{1.8})/(S_x L^{2.3})) = 1 / (1 + [(0.15) (2.27)^{1.8}]/[(0.02) (2.0)^{2.3}] = 0.13$

From equation 7.9:

$Q_{ig} = Q_g [R_f E_o + R_s (1-E_o)] = 0.42 [(1.0)(0.97) + (0.13)(1 – 0.97)] = 0.41$ ft^3/s

Step 7. Compute the total interception capacity. (Interception capacity of curb opening adjacent to grate is neglected.)

$Q_i = Q_{ic} + Q_{ig} = 1.35 + 0.41 = 1.76$ ft^3/s

Solution: The sweeper combination inlet intercepts 1.76 ft^3/s (0.536 m^3/s). This represents nearly 100 percent interception of the 1.77 ft^3/s (0.539 m^3/s) in the approaching gutter.

7.2.5 Comparison of Interception Capacity of Inlets on Grade

Many variables contribute to the interception capacity of inlets on grade including the inlet type and size, the approach gutter/pavement conditions, local or gutter depression, and clogging. Over a range of longitudinal slopes, Figure 7.13 compares curb-opening inlets, grates, and slotted drain inlets holding gutter flow at 3.2 ft^3/s, cross slope at 3 percent, and Manning's n at 0.016. The relationship between the curves shown may vary under different conditions but is useful for general comparisons.

For example, Figure 7.13 illustrates that the slotted inlets and curb-opening inlets shown lose interception capacity and efficiency as longitudinal slope increases. This occurs because depth at the curb becomes smaller as velocity increases. Factors that encourage flow toward the curb, such as increasing cross slope, gutter depression, or local inlet depression, increase interception. The effect of depression on curb-opening inlets is illustrated by comparing undepressed curb openings (curves 9 and 10) with depressed curb openings of the same length (curves 11 and 12). The depressed curb openings capture significantly higher flows for these conditions.

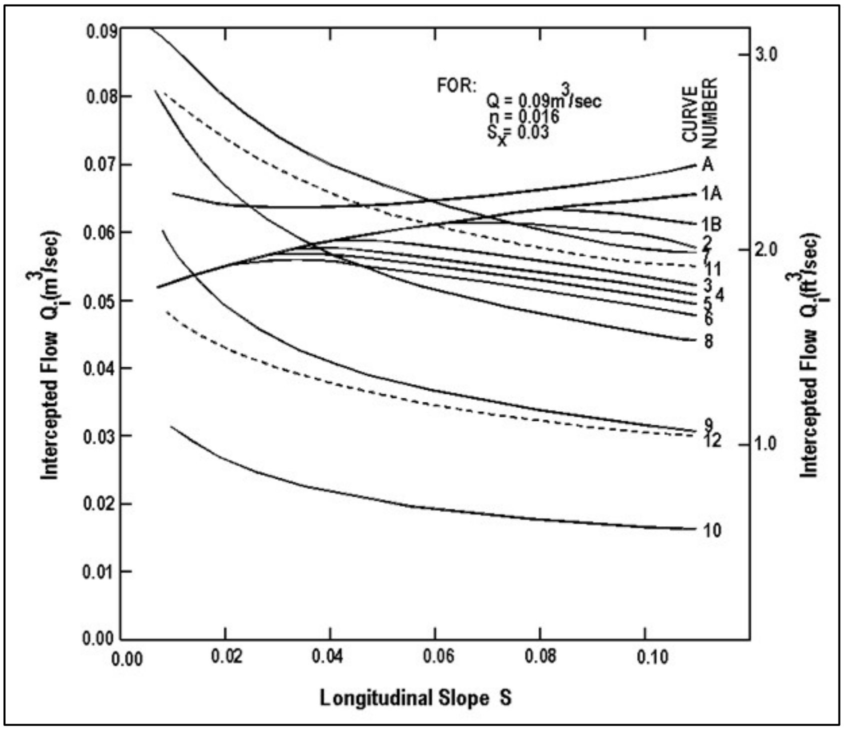

Figure 7.13. Comparison of inlet interception capacity on grade.

Curve Number Key

Grates (L = 4 ft, W = 2 ft)
A. P-1-7/8
Grates (L = 2 ft, W = 2 ft)
1. A. P-1-7/8, B. P-1-1/8
2. Curved vane
3. 45° inclined bar
4. P-1-7/8-4
5. 30° inclined bar
6. Reticuline
Slotted inlets (W ≥ 1.75 in)
Curb openings (h ≥ 4.25 in)
7. L = 20 ft
8. L = 15 ft
9. L = 10 ft
10. L = 5 ft
Depressed curb openings (W=2 ft, a=2 in)
11. L = 10 ft
12. L = 5 ft

Figure 7.13 also illustrates that interception for grate inlets can increase with increasing longitudinal slope until the splash-over velocity is reached. The grates represented by curves other than 1 and 1A show an increase in interception for an increase in slope because the flow spread is narrowing. However, at greater slopes, they experience a decrease in interception as the splash-over velocity for the grate is reached and flow begins to jump over the grate. The P-1-7/8 grates (curves 1 and 1A) show an increase in interception at the longitudinal slopes shown because the velocity in the gutter has not reached their splash-over velocity. Based on these performance characteristics curves, parallel bar grates and the curved vane grate are relatively efficient at higher velocities and the reticuline grate is least efficient. At low velocities, the grates perform equally well. However, some of the grates such as the reticuline grate are more susceptible to clogging by debris than the parallel bar grate.

Figure 7.13 demonstrates that inlet length increases interception. The 4 ft grate (curve A) intercepts more flow than the 2 ft grate (curve 1A). However, under these conditions, the doubling of length increases interception modestly. Wider grates could increase interception because side flow, not frontal flow, is escaping these grates. The figure reveals greater benefits of lengthening curb opening and slotted inlets under these conditions. Interception increases significantly for the 10 ft curb-opening inlets (curves 9 and 11) compared with the 5 ft counterparts (curves 10 and 12).

Over a range of gutter flow rates, Figure 7.14 compares the same inlet types holding longitudinal slope at 6 percent, cross slope at 3 percent, and Manning's n at 0.016. It shows, for example, that at a 6 percent slope, splash-over begins at about 0.7 ft^3/s on a reticuline grate. It also illustrates that with increased discharge, the interception capacity of all inlets increases, and inlet efficiency decreases.

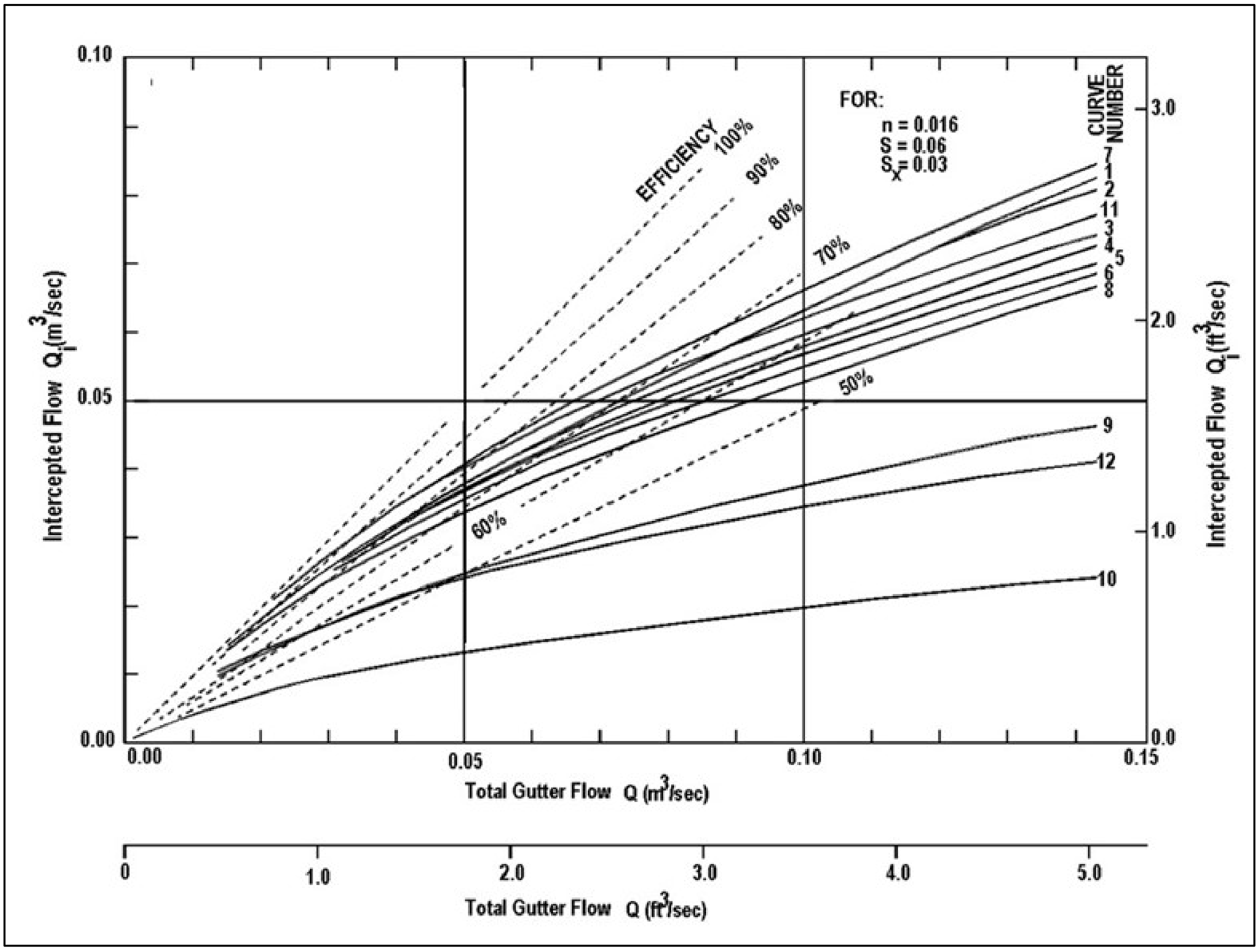

Figure 7.14. Comparison of inlet interception capacity, flow rate variable.

The performance characteristics shown in Figure 7.13 and Figure 7.14 neglect the effects of debris and clogging on inlets. Section 7.1.3 discusses clogging.

7.3 Interception Capacity of Inlets in Sag Locations

Inlets in sag locations operate as weirs under low depth conditions and as orifices at greater depths. Orifice flow begins at depths dependent on the grate size, the curb-opening height, or the slot width of the inlet. At depths between those at which weir flow prevails and those at which orifice flow prevails, flow is in a transition stage. At these depths, control may fluctuate between weir and orifice control. In the transition stage, designers estimate conditions assuming both weir and orifice control and design using the more conservative result.

The efficiency of inlets in passing debris is critical in sag locations because all runoff which enters the sag must be passed through the inlet. Total or partial clogging of inlets can result in hazardous ponded conditions. Because of the potential for clogging, designers prefer combination inlets or curb-opening inlets over grate inlets alone in sag locations.

7.3.1 Grate Inlets in Sags

The perimeter and clear opening area of the grate and the depth of water at the curb affect inlet capacity. 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. Larger grates will operate as weirs to greater depths than smaller grates.

When operating under weir flow conditions, designers estimate interception capacity as:

$$Q_i = C_w \sqrt{2g}\, P\, d^{1.5} \tag{7.14}$$

where:

Q_i	=	Intercepted flow, ft^3/s (m^3/s)
C_w	=	Weir coefficient (typically equal to 0.37)
P	=	Perimeter of the grate disregarding the side against the curb, ft (m)
d	=	Average depth across the grate, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

Figure 7.15 describes the computation of average depth across a grate. The figure also describes the grate width, W, and the grate length, L. The perimeter for estimating weir flow is L + 2W.

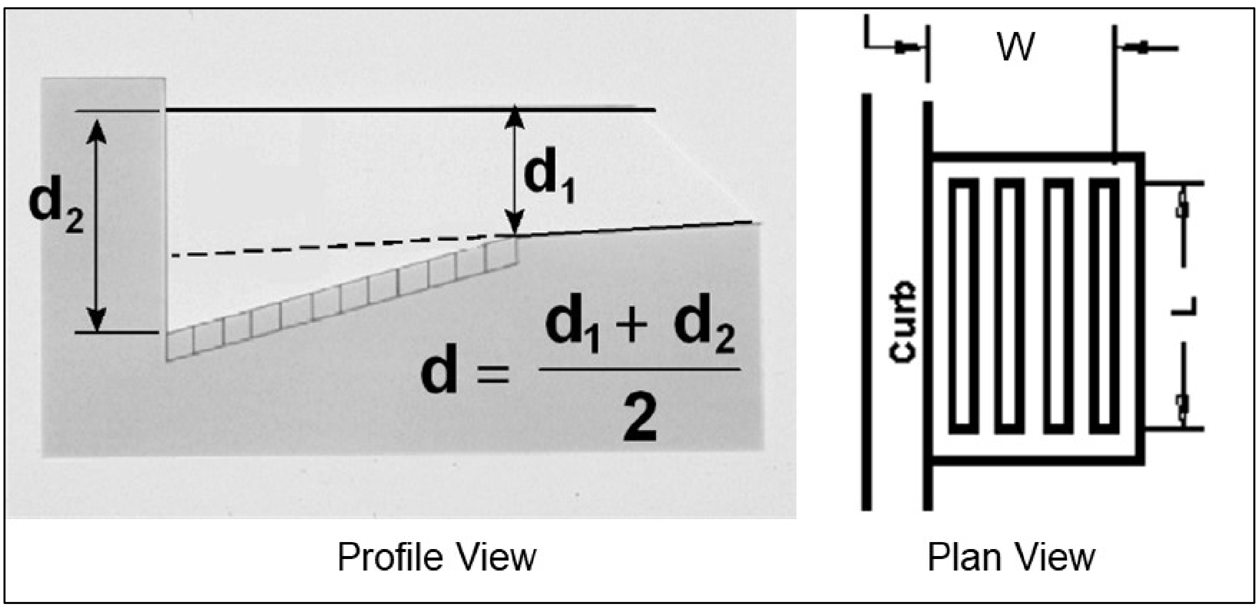

Figure 7.15. Profile and plan view grate definition sketch.

When operating under orifice flow conditions, designers estimate interception capacity as:

$$Q_i = C_o A_g (2gd)^{0.5} \tag{7.15}$$

where:

Q_i	=	Intercepted flow ft^3/s (m^3/s)
C_o	=	Orifice coefficient usually taken as 0.67
A_g	=	Clear opening area of the grate, ft^2 (m^2)
d	=	Average depth across the grate, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

The clear area of opening of a grate equals the total area (length times width) times an opening ratio. Burgi (1978b) tested three grate styles for the FHWA and showed that for flat bar grates, such as the P-1-7/8-4 and P-1-1/8 grates, the clear opening equals the total area of the grate less the area occupied by longitudinal and lateral bars. The curved vane grate performed about 10 percent better than a grate with a net opening equal to the total area less the area of the bars projected on a horizontal plane. That is, the projected area of the bars in a curved vane grate is 68 percent of the total area of the grate leaving a net opening of 32 percent, however the grate performed as a grate with a net opening of 35 percent. Tilt-bar grates were not tested, but extrapolation of the testing results indicates a net opening area of 34 percent for the 30-degree tilt-bar and zero for the 45-degree tilt-bar grate. However, the 45-degree tilt-bar grate has greater than zero capacity. The tilt-bar angle and curved vanes enhance interception on grade. However, because of the low opening ratio, tilt-bar and curved vane grates perform poorly in sump locations under orifice flow conditions. Table 7.5 summarizes grate opening ratios.

Table 7.5. Grate opening ratios.

Grate	Opening Ratio
P-1-7/8-4 *	0.8
P-1-7/8	0.9
P-1-1/8 *	0.6
Reticuline	0.8
Curved vane *	0.35
30° tilt-bar	0.34

*Laboratory tested.

Depending on the ponding depth and grate characteristics, designers can estimate flow interception based on either weir flow (equation 7.14) or orifice flow (equation 7.15). Under many circumstances, the designer may be unsure which to use. To avoid overestimating flow interception, or underestimating ponding depth, the designer calculates depth by both equations for a given flow and uses the highest value for the depth.

Clogging reduces the inlet interception. Clogging can be represented as a reduction of the perimeter length when using the weir flow equation and as a reduction of the opening area when using the orifice flow equation. However, a particular clogging scenario may change the effective opening area to a greater or lesser degree than it changes the effective perimeter. For example, an accumulation of leaves at a curb may block 25 percent of the grate opening area but a smaller percent of the grate perimeter because it only affects the perimeter nearest the curb.

Example 7.5: Grate inlet in a sag.

Objective: Find the grate size required to maintain allowable spread.

Given: The following uniform gutter and flow conditions:

S_x = 0.05 ft/ft (m/m)
S_W = 0.05 ft/ft (m/m)
n = 0.016
Q = 6.71 ft^3/s (0.19 m^3/s)
T = 9.84 ft (3.0 m^3/s) (allowable)

For the grate (P-1-7/8-4):

W = 2.0 ft (0.61 m)
Clogging = 50 percent of clogging from the curb (assumed)

Step 1. Determine the required grate perimeter without clogging for weir flow.

Depth at curb:

$d_2 = T\, S_x = (9.84)\,(0.05) = 0.49$ ft

Average depth over grate:

$d = d_2 – (W/2)\, S_W = 0.49 – (2.0/2)\,(0.05) = 0.44$ ft

Using equation 7.14:

$P = Q_i / (C_w\,(2g)^{0.5}\, d^{1.5}) = (6.71)/[(0.37)\,(64.4)^{0.5}\,(0.44)^{1.5}] = 7.66$ ft

Step 2. Determine the required grate perimeter with clogging for weir flow.

Assuming 50 percent clogging from the curb:

$P_{effective} = 7.66 = L + (0.5)\, 2W = L + 0.5\,(2)\,(2.0)$

Solving for L = 5.66 ft

Select a double 2 ft by 3 ft grate.

$P_{effective} = (0.5)\,(2)\,(2.0) + (6) = 8$ ft

Step 3. Check depth of flow at curb for weir flow.

Using equation 7.14:

$d = [Q/(C_w\,(2g)^{0.5}\, P)]^{0.67}\ [6.71/((0.37\,(64.4)^{0.5}\,(8.0)]^{0.67} = 0.43$ ft

Therefore, grate size meets spread requirements with clogging.

Step 4. Determine the required grate opening area without clogging for orifice flow.

Using equation 7.15:

$A_g = Q_i / [C_o\,(2gd)^{0.5})] = (6.71)/[(0.67)(2(32.2)(0.44)^{0.5}] = 2.15$ ft^2

Step 5. Determine the required grate opening area with clogging for orifice flow.

Assuming 50 percent clogging from the curb:

$A_{g\ effective} = 2.15 = (L)\,(W)\,(\text{opening ratio})\,(\text{clogging}) = L\,(2.0)\,(0.8)\,(0.5)$

Solving for L = 2.67 ft

Select a single grate 2 ft by 3 ft.

$A_{g\ effective} = (0.5)\,(2.0)\,(3.0) = 3$ ft^2

Step 6. Check depth of flow at curb for orifice flow.

Using equation 7.15:

$d = [Q_i/(C_oA_g)]^2/(2g) = [6.71 / ((0.67)\,(3.0))]^2/(2(32.16)) = 0.17$ ft

Because this depth for orifice flow is less than the depth computed using the weir equation (step 3), the weir equation is used for design and the orifice equation is disregarded.

Solution:	A double 2 ft by 3 ft (0.61 m by 0.91 m) grate 50 percent clogged from the curb is adequate to intercept the design storm flow and meet the design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet or curb-opening inlet in a sag where ponding can occur, and flanking inlets in long flat vertical curves.

7.3.2 Curb-Opening Inlets

Curb-opening inlets operate as weirs in sag vertical curve locations up to a ponding depth equal to the opening height. At depths above 1.4 times the opening height, the inlet operates as an orifice and between these depths, transition between weir and orifice flow occurs. The curb-opening height and length, and water depth at the curb affect inlet capacity.

The weir equation for curb-opening inlets without depression is:

$$Q_i = C_w \sqrt{2g}\, L\, d^{1.5} \tag{7.16}$$

where:

C_w	=	Weir coefficient (typically equal to 0.37)
L	=	Length of curb opening, ft (m)
d	=	Depth at curb, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

For non-depressed inlets with a uniform gutter, the curb inlet operates as a weir for depths up to the curb-opening height, h. The weir location for a curb-opening inlet that is not depressed is at the lip of the curb opening and its length is equal to that of the inlet, as shown in Figure 7.16.

At a given flow rate, the effective water depth at the curb can be increased by using a continuously depressed gutter, a locally depressed curb opening, or by increasing cross slope, thus decreasing the width of spread at the inlet.

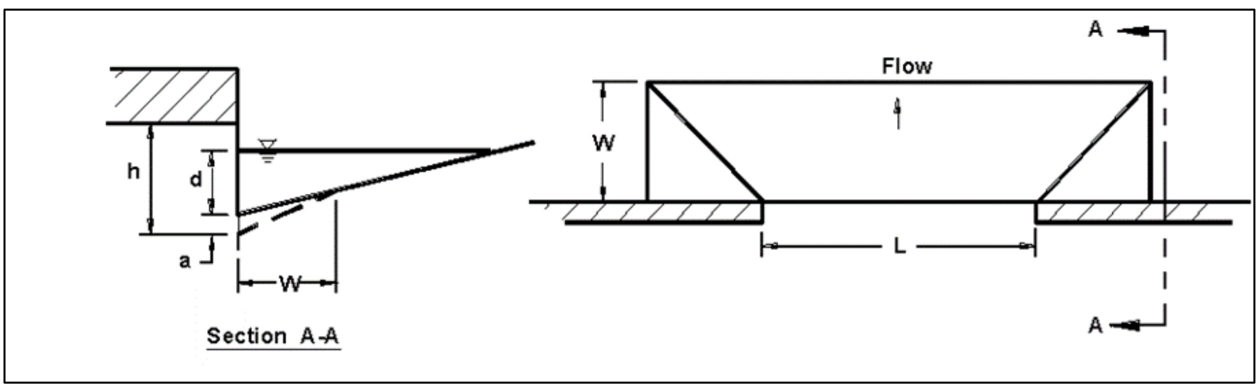

Figure 7.16. Curb-opening inlet definition sketch.

The weir location for a depressed curb-opening inlet is at the edge of the gutter, and the effective weir length depends on the width of the depressed gutter and the length of the curb opening. The interception capacity of a locally depressed curb-opening inlet operating as a weir is:

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

where:

C_w	=	Weir coefficient (typically equal to 0.29)
L	=	Length of curb opening, ft (m)
W	=	Lateral width of depression, ft (m)
d	=	Depth at the curb measured from the pavement cross-slope (S_x), ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

The weir coefficient for a curb-opening inlet is less than in equation 7.16 primarily because experimental tests did not include depth measurements taken at the weir and drawdown occurs between the point where measurements were made and the weir. The weir equation applies to depths at the curb. For weir flow at a depressed curb inlet, the depth at the curb for equation 7.17 must be less than the curb-opening height:

$$d_i \leq h = d + a \tag{7.18}$$

where:

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

Although laboratory experiments for FHWA did not include curb-opening inlets with a continuously depressed gutter, the FHWA expects that the effective weir length described in equation 7.17 would be equaled or exceeded with 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 non-depressed inlet produces intercepted flows which exceed the values for depressed inlets computed using equation 7.17. Since depressed inlets will perform at least as well as non-depressed inlets of the same length, equation 7.16 is most 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 for depressed and undepressed curb-opening inlets can be computed as:

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

where:

C_o	=	Orifice coefficient, usually taken as 0.67
d_o	=	Effective head on the center of the orifice throat, ft (m)
A_o	=	Clear area of opening, ft^2 (m^2)

The effective head on the center of the orifice throat, d_0, and the clear opening area, A_0, are computed based on the curb-opening configuration. Figure 7.17 illustrates a horizontal throat, an inclined throat, and a vertical throat. The clear opening area is the length of the curb-opening inlet times the height of the curb-opening inlet as represented in the figure. A limited throat width could reduce the capacity of the curb-opening inlet by causing the inlet to go into orifice flow at depths less than the height of the opening. Equation 7.19 applies to other configurations when the designer appropriately selects the effective head and clear opening area.

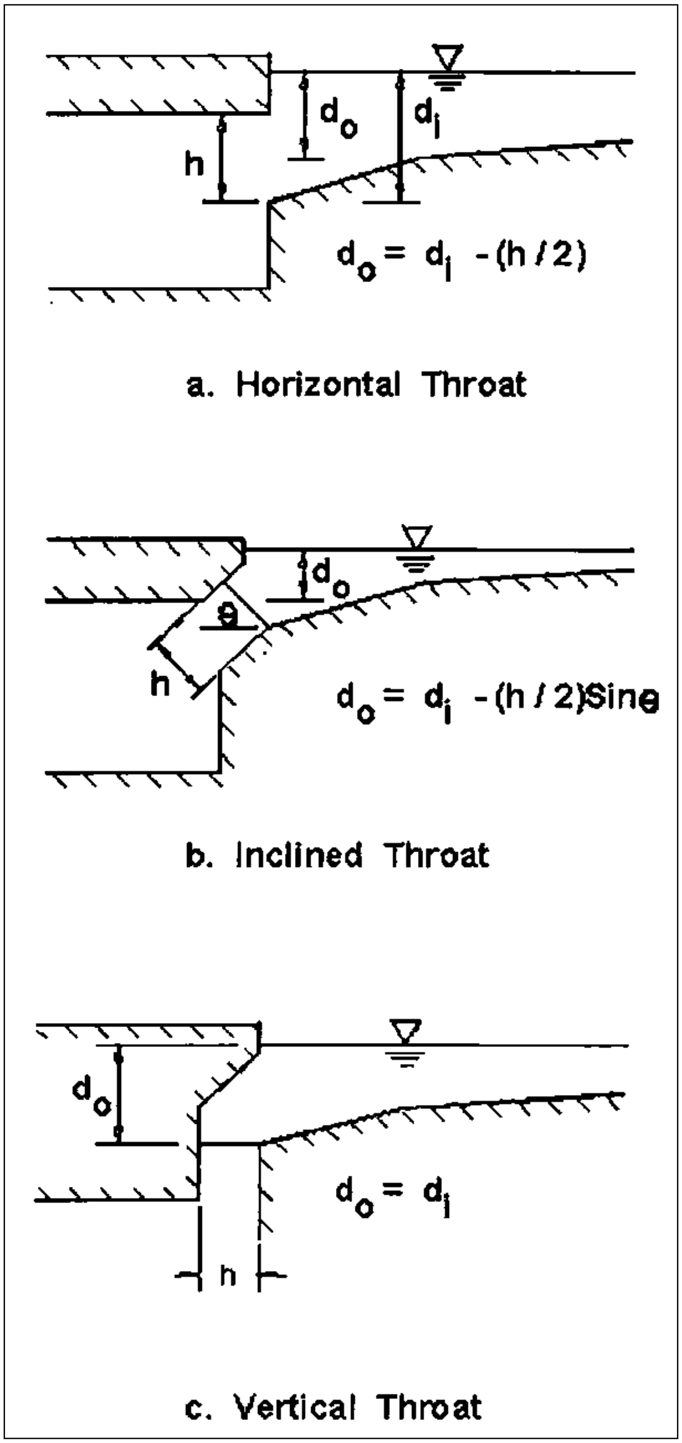

Figure 7.17. Curb-opening inlet throat configurations.

Example 7.6:	Curb inlet in a sag.
Objective:	Find curb inlet interception without and with local depression.

Given: Curb-opening inlet in a sump location with:

S_x = 0.02 ft/ft (m/m)
L = 8.2 ft (2.5 m)

h	=	0.43 ft (0.13 m)
T	=	8.2 ft (2.5 m)

For the local depression:

a	=	1 inch (25.4 mm)
W	=	2 ft (0.61 ft)

Step 1. Determine depth at curb for the undepressed inlet.

$D = T S_x = (8.2)(0.02) = 0.16$ ft

Since 0.16 ft < h = 0.43 ft weir flow controls

Step 2. Estimate Q_i for the undepressed inlet.

Use equation 7.16:

$Q_i = C_w (2g)^{0.5} L d^{1.5} = (0.37)(64.4)^{0.5} (8.2)(0.16)^{1.5} = 1.6$ ft^3/s

Step 3. Determine depth at curb for the depressed inlet.

$d_i = d + a = S_x T + a = (0.02)(8.2) + 1/12 = 0.25$ ft

Since 0.25 ft < h = 0.43 ft weir flow controls

Step 4. Estimate Q_i for the depressed inlet.

$P = L + 1.8 W = 8.2 + (1.8)(2.0) = 11.8$ ft

Use equation 7.17:

$Q_i = C_w (2g)^{0.5} (L + 1.8 W) d^{1.5} = (0.29)(64.4)^{0.5} (11.8)(0.16)^{1.5} = 1.7$ ft^3/s

Solution: The interception capacity in the sag of the undepressed inlet is 1.6 ft^3/s (0.045 m^3/s) and of the depressed inlet is 1.7 ft^3/s (0.048 m^3/s). In this case, local depression increased interception by approximately 7 percent.

7.3.3 Slotted Inlets

Slotted inlets operate as weirs for depths below approximately 2 inches and as orifices where the depth at the upstream edge of the slot is greater than about 5 inches. Between these depths, the flow conditions are in transition between weir and orifice flow. Interception capacity varies with flow depth, cross slope, slot width, and slot length. Slotted drains in sag locations are susceptible to clogging from debris. Designers generally avoid their use in sag locations because of the clogging potential.

The interception capacity of a slotted inlet operating as a weir can be computed as:

$$Q_i = C_w \sqrt{2g}\, L\, d^{1.5} \tag{7.20}$$

where:

C_w	=	Weir coefficient
L	=	Length of slot, ft (m)
d	=	Depth at curb measured from the normal cross slope, m (ft)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

The weir coefficient for a slotted inlet varies with flow depth and slot length, but a typical value is approximately 0.31.

The interception capacity of a slotted inlet operating as an orifice can be computed as:

$$Q_i = C_o L W (2gd)^{0.5} \tag{7.21}$$

where:

W	=	Width of slot, ft (m)
C_o	=	Orifice coefficient (typically equal to 0.8)
L	=	Length of slot, ft (m)
d	=	Depth of water at slot, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

For depths in the transition between weir flow and orifice flow, designers compute interception capacity using both the weir and orifice equations and use the lowest interception capacity (most conservative) for design. Similarly, when estimating ponding depth from a given flow rate, designers use both equations and use the highest depth for design.

Example 7.7:	Slotted inlet in a sag.
Objective:	Find the slotted inlet length to limit maximum depth at the inlet to 3.6 inches assuming no clogging.

Given: A slotted inlet located along a curb:

W	=	1.75 in (44.5 mm)
Q	=	4.9 ft^3/s (0.14 m^3/s)

Step 1. Estimate the inlet length assuming weir flow.

Use equation 7.20:

$L = Q_i / (C_w (2g)^{0.5} d^{1.5}) = 4.9 / ((0.31)(64.4)^{0.5} (0.3^{1.5})) = 12.0$ ft

Step 2. Estimate the inlet length assuming orifice flow.

Use equation 7.21:

$L = Q_i / (0.8 W (2 g d)^{0.5}) = 4.9 / ((0.8) (0.146) (2 (32.2) (0.3)^{0.5}) = 9.6$ ft

Solution:	The length is computed for both weir and orifice flow at the maximum allowable depth. Weir flow controls because it results in the longest length. A 12 ft (3.7 m) inlet length is needed to meet the depth requirements.

7.3.4 Combination Inlets

Combination inlets (described in Section 7.2.4) perform well in sags where hazardous ponding can occur. The curb-opening length in a combination inlet in sag can equal the length of the grate or can be longer than the grate on one or both ends of the grate in a variation of a sweeper configuration.

When ponded depths are sufficiently low that the grate inlet captures the flow operating as a weir, the designer may ignore the curb opening, and compute the interception as described in Section 7.3.1. If the grate becomes clogged and ineffective, the effectiveness of the combination inlet depends only on the curb opening; designers can estimate it as described in Section 7.3.2.

When the ponded depths are sufficiently high so that both the grate and the curb opening are operating in orifice flow, the capacity of the combination inlet equals the sum of the two operating independently using the orifice equations described in Section 7.3.1 and Section 7.3.2, respectively. Under these conditions, a trial-and-error solution best finds the depth that satisfies both orifice equations and totals to the design flow. In the transition between weir and orifice flow, trial-and-error solutions are also used to find the depth associated with a given design flow.

Example 7.8:	Combination inlet in a sag.
Objective:	Find the depth at the curb and spread for a clog-free combination inlet and a combination inlet with the grate 100 percent clogged.

Given: A combination inlet in a sag location with the following characteristics:

Q	=	5.3 ft^3/s (0.15 m^3/s)
S_x	=	0.03 ft/ft (m/m)

Grate inlet (P-1-7/8):

W	=	2 ft (0.61 m)
L	=	4 ft (1.22 m)

Curb-opening inlet:

L	=	4 ft (1.22 m)
h	=	3.9 inches (100 mm)

Step 1. Compute average depth over the grate with no clogging.

Assuming grate controls interception with weir flow:

$P = 2W + L = 2(2) + 4 = 8.0$ ft

From equation 7.14:

$d_{avg} = [Q_i / (C_w\, P)]^{0.67} = [(5.3) / \{(3.0)(8.0\}]^{0.67} = 0.36$ ft

Step 2. Compute associated curb depth and spread.

$d = d_{avg} + S_x\, (W/2) = 0.36 + 0.03\, (2/2) = 0.39$

$T = d / S_x = 0.39 / 0.03 = 13$ ft

Step 3. Compute depth at curb assuming 100 percent clogging of grate.

Assuming orifice flow and using equation 7.19:

$A_o = L\, h = 4\, (0.325) = 1.3\ ft^2$

$d_o = [Q_i / (C_o\, A_o\, (2g)^{0.5}]^2 = [5.3 / (0.67\, (1.3)\, (2(32.2))^{0.5}]^2 = 0.576$ ft

$d_i = d_o + h/2 = 0.576 + 0.325/2 = 0.74$ ft

Depth at curb is in transition. Orifice flow equation provides conservative estimate of depth.

Step 4. Compute associated spread.

$T = d / Sx = (0.74) / (0.03) = 24.7$ ft

Solution:	Depth at curb with no clogging and grate operating is 0.36 ft (0.11 m). With the grate clogged, the depth at the curb is 0.74 ft (0.23 m), approximately twice the amount without clogging. The spread is also twice as large.

7.4 Inlet Location

Designers determine inlet location based on geometric controls that result in inlets at specific locations (including flanking inlets in sag vertical curves) and that are based on roadway spread criteria. To adequately design the location of the inlets for a given project, designers need:

- Layout or plan sheets suitable for outlining drainage areas.
- Road profiles.
- Typical cross-sections.
- Grading cross-sections.
- Superelevation diagrams.
- Contour maps.

7.4.1 Geometric Controls

Roadway geometry determines the location of some drainage inlets. These locations are marked on plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass. Examples of such locations include:

- At all low points in the gutter grade.
- Immediately up-gradient of median breaks, entrance/exit ramp gores, cross walks, and street intersections, i.e., at any location where water could flow onto the roadway.
- Immediately up-gradient of bridges (to prevent pavement drainage from flowing onto bridge decks).
- Immediately down-gradient of bridges (to intercept bridge deck drainage).
- Immediately up-gradient of cross-slope reversals.
- Immediately up-gradient from pedestrian cross walks.
- At the end of channels in cut sections.
- On side streets immediately up-gradient from intersections.
- Behind curbs, shoulders, or sidewalks (to drain low areas).

In addition, designers place roadside channels or inlets to intercept runoff from areas draining toward the roadway. This applies to drainage from cut slopes, side streets, and other areas alongside the pavement. Curbed pavement sections and pavement drainage inlets are inefficient means for handling extraneous drainage.

7.4.2 Inlet Spacing on Continuous Grades

Design (allowable) spread drives spacing of storm drain inlets between inlets required by geometric or other controls. The interception capacity of the upstream inlet will establish the initial spread for the next inlet downstream on a continuous road grade. As flow is contributed to the gutter section in the downstream direction, spread increases. The next downstream inlet is located at or before the point where the spread in the gutter reaches the design spread. Therefore, the spacing of inlets on a continuous grade is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter/roadway cross slope.

For a continuous grade, the designer may establish a uniform design spacing between inlets if the drainage area consists of pavement only or has reasonably uniform runoff characteristics and is rectangular in shape. In this case, the designer assumes the time of concentration is the same for all inlets.

The following procedure and example illustrate the inlet spacing design process. Designers can implement calculations in a spreadsheet or use automated drainage design software to achieve the same goals. Regardless of the tools used, documentation of the process and results facilitates independent review and future revisions if conditions or criteria change.

Step 1. Plan the design sequencing.

In this first step, the designer begins the documentation process and plans the design sequencing. This step generally includes the following activities:

- Confirm the design criteria including the design frequency, allowable spread, and maximum inlet spacing.
- Mark plans with the location of inlets which are necessary to satisfy geometric controls such as the locations described in Section 7.4.1.
- Identify a high point to begin, at one end of the job, if possible, and progress toward the low point. Then begin at the next high point and work backward toward the same low point.
- Select a trial drainage area and inlet location below the high point and outline the area on the plan. Depending on the drainage area width, an initial trial might be approximately 300 to 500 ft long. Include any area that may drain over the curb and onto the roadway. However, where practical, drainage from large areas behind the curb should be intercepted before reaching the roadway or gutter.

Step 2. Calculate the gutter discharge.

In step 2, the designer estimates the drainage area to the trial inlet location and the design flow in the gutter approaching the inlet. This step generally includes the following activities:

- Describe the locations of the proposed inlets by number and station.
- Compute the drainage area to the inlet.
- Determine the runoff coefficient, C, the time of concentration, t_c, and rainfall intensity for the inlet using the procedures from Chapter 4. Note the minimum time of concentration applicable to the project.
- Calculate the design flow in the gutter using the Rational Method or other appropriate method from Chapter 4.

Step 3. Estimate spread.

In step 3, the designer uses the gutter characteristics and the design flow to estimate the gutter spread as the flow approaches the inlet. This step generally includes the following activities:

- From the roadway profile, determine the gutter longitudinal slope, S_L, at the inlet, considering any superelevation.
- From the cross-section, determine the cross slope, S_x, and the grate gutter width, W.
- Determine the total gutter flow approaching the inlet. The total flow is the flow computed in step 2 plus any bypass flow from the inlet up-gradient. For the most up-gradient inlet in a series, there is no bypass flow.
- Determine the spread, T, and depth at the curb using methods described in Chapter 5.
- Compare the spread with the allowable spread. Compare the depth at the curb with the actual curb height. If the calculated spread is near the allowable spread and the depth at

the curb is less than the actual curb height, continue to step 4. Otherwise, move the inlet to expand or decrease the drainage area to increase or decrease the spread, respectively. The drainage area can be expanded by increasing the length to the inlet and it can be decreased by decreasing the distance to the inlet. Then, repeat step 2 and step 3 until appropriate values are obtained.

Step 4. Select inlet and compute interception.

The designer selects an inlet type and size and then computes the interception and bypass. This step generally includes the following activities:

- Select the inlet type (grate, curb-opening, or combination) and dimensions.
- If using a grate or combination inlet, calculate W/T to initiate the computation of frontal versus side flow.
- Calculate the flow intercepted by the inlet, Q_i, using the tools in Section 7.2.
- Determine the bypass flow, Q_b.

Step 5. Repeat process for the next inlet.

The designer repeats steps 2 through 4 for each subsequent inlet down-gradient until the low point is reached. For long stretches at a constant gradient, a uniform spacing between inlets of a single type and size is desirable.

For inlet spacing in areas with changing grades, the spacing will vary as the grade changes. If the grade becomes flatter, inlets may be spaced at closer intervals because the spread will exceed the allowable. Conversely, for an increase in slope, the inlet spacing will become longer because of increased capacity in the gutter sections. Additionally, individual transportation agencies may have limitations for spacing due to maintenance constraints.

Example 7.9:	Inlet spacing on continuous grades.
Objective:	Find the maximum design inlet spacing for a 2 ft by 3 ft (0.61 m by 0.91 m) P-1-7/8-4 grate.

Given: The storm drainage system illustrated in Figure 7.18 with:

n	=	0.016
S_x	=	0.04 ft/ft (m/m)
S_L	=	0.03 ft/ft (m/m)
W	=	2.0 ft (0.61 m)
h	=	0.5 ft (0.15 m)) (curb height)

Design criteria:

Allowable spread = 6.6 ft (2.0 m)
Design storm = 0.1 AEP (10-year) event
Minimum time of concentration = 5 minutes
Maximum inlet spacing (for maintenance reasons) = 360 ft (110 m)

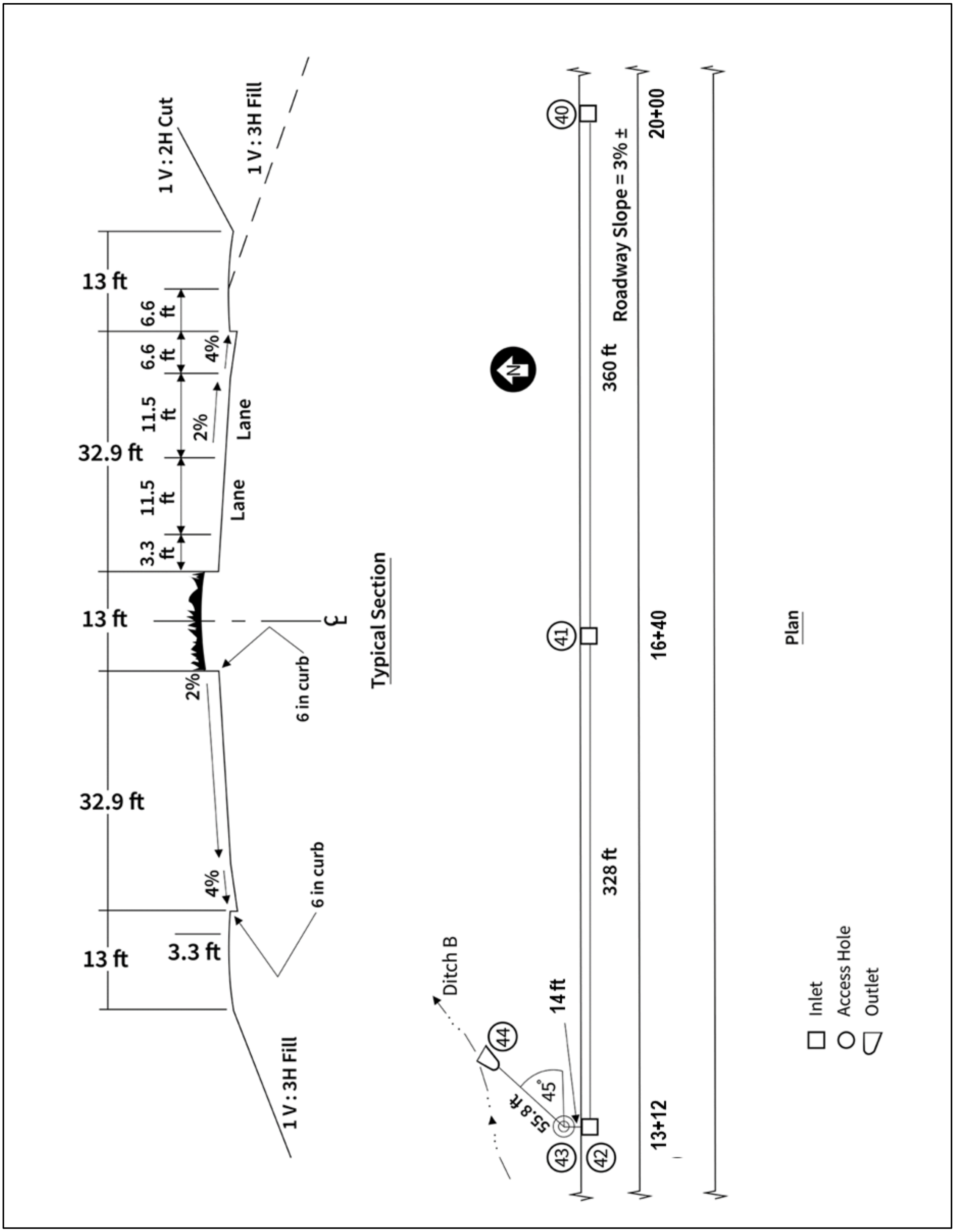

Figure 7.18. Storm drainage system for example.

Step 1. Plan the design sequencing.

Begin at the inlet (labeled # 40) located at station 20+00. The initial drainage area consists of a 42.7 ft wide roadway section with a length of 656 ft. The top of the drainage basin is located at station 26+56.

Step 2. Calculate the gutter discharge.

Drainage area = (656) (42.7) / 43,560 = 0.64 ac

Estimate runoff coefficient, time of concentration, and rainfall intensity using the Rational Method (Chapter 4).

Runoff coefficient, C = 0.73

Time of concentration, t_c, = 3.1 min (use 5 min minimum)

Rainfall intensity from Intensity-Duration-Frequency (IDF) curve: I = 7.1 in/h

Flow from the Rational Method:

$Q = CIA/K_u = (0.73)(7.1)(0.64)/(1) = 3.32$ ft^3/s

Step 3. Estimate spread.

Determine spread, T, using equation 5.4:

$T = [\{Qn\} / \{K S_x^{1.67} S_L^{0.5}\}]^{0.375} = [\{(3.32)(0.016)\} / \{(0.56)(0.04)^{1.67} (0.03)^{0.5}\}]^{0.375} = 5.99$ ft

T is less than the allowable spread.

$d = T S_x = (5.99)(0.04) = 0.24$ ft

Note: T is less than allowable spread and d is less than the curb height. Proceed to next step.

Step 4. Select inlet and compute interception.

Select a P-1-7/8-4 grate measuring 2 ft wide by 3 ft long.

W/T = 2/5.99 = 0.33

Using equation 7.3 for a uniform gutter:

$E_o = 1 - (1 - W/T)^{2.67} = 1 - (1 - 0.33)^{2.67} = 0.66$

Using equation 5.3:

$V = (K_u/n) S_L^{0.5} S_x^{0.67} T^{0.67} = \{1.11/(0.016)\} (0.03)^{0.5} (0.04)^{0.67} (1.83)^{0.67} = 4.61$ ft/s

Using equation 7.5:

$R_f = 1.0$

Using equation 7.6:

$R_s = 1 / [1 + (K_u V^{1.8})/(S_x L^{2.3})] = 1 / [1 + \{(0.15)(4.6)^{1.8}\}/\{(0.04)(3)^{2.3}\}] = 0.18$

Using equation 7.9:

$Q_i = Q [R_f E_o + R_s (1 - E_o)] = (3.32) [(1.0)(0.66) + (0.18)(1 - 0.66)] = 2.39$ ft^3/s

$Q_b = Q - Q_i = 3.32 - 2.39 = 0.93$ ft^3/s

Step 5. Repeat process for next inlets.

Next inlet: Inlet # 41 at station 16+40

Drainage area = (360)(42.7)/43560 = 0.35 ac

Estimate runoff coefficient, time of concentration, and rainfall intensity using the Rational Method (Chapter 4).

Runoff coefficient, C = 0.73

Time of concentration, t_c, = 1.7 min (use 5 min minimum)

Rainfall intensity from IDF curve: i = 7.1 in/h

Flow from the Rational Method:

$Q = CIA/K_u = (0.73)(7.1)(0.35)/(1) = 1.81$ ft^3/s

Total flow to inlet # 41: Q = 0.93 + 1.81 = 2.74 ft^3/s

T = 5.6 ft (T < T allowable)

d = (5.6) (0.4) = 0.22 ft (d < curb height)

Select P-1-7/8-4 grate 2 ft wide by 3 ft long.

$Q_i = 2.05$ ft^3/s

$Q_b = Q – Q_i = 2.74 – 2.05 = 0.69$ ft^3/s

Solution:	For the conditions, the inlet spacing on this continuous grade is limited by the maintenance limitation of 360 ft (110 m). The spread and depth are less than the allowable spread and curb height.

7.4.3 Flanking Inlets

Low (sag) points in the gutter profile need inlets to provide for drainage. In addition, good engineering practice includes flanking inlets (inlets located on one or both sides of a low point inlet) in a sag area that has no outlet except through the drainage system. Figure 7.19 illustrates the location of flanking inlets to act in relief of a low point inlet if it becomes clogged or to reduce spread at the sag.

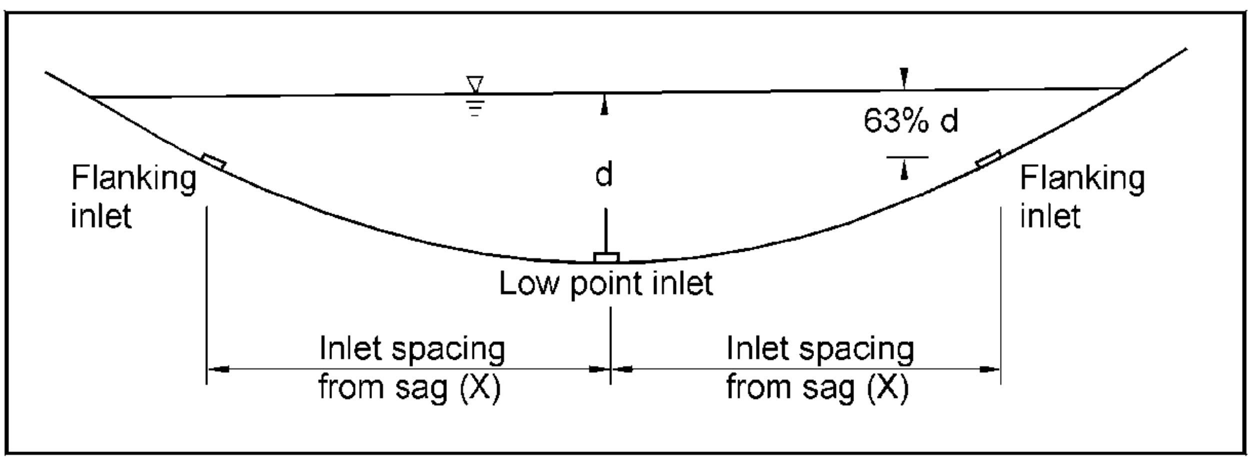

Figure 7.19. Location of flanking inlets.

Designers locate and size flanking inlets to receive the design flow when the primary inlet at the bottom of the sag is clogged without exceeding the allowable spread. If the flanking inlets are the same dimension as the primary inlet, they will each intercept one-half the design flow when they are located so that the depth of ponding at the flanking inlets is 63 percent of the depth of ponding at the low point. When flanking inlets differ from the primary inlet, the designer estimates ponding depths based on inlet performance to determine the capacity of the flanking inlet at the desired depths (AASHTO 2014).

The spacing required for the allowable depth at the curb in a vertical sag curve is:

$$X = (74\,dK)^{0.5} \tag{7.22}$$

where:

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

The vertical curve constant is computed from:

$$K = L / (G_2 - G_1) \tag{7.23}$$

where:

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

The AASHTO policy on geometrics specifies maximum K values for various design speeds and a maximum 167 ft/percent (50 m/percent) considering drainage (AASHTO 2018). [See 23 CFR 625.3(b) and 625.4(a)].

Example 7.10:	Flanking inlet spacing.
Objective:	Determine the location of flanking inlets to function in relief of the inlet at the low point.

Given: A sag vertical curve at an underpass on a 4-lane divided highway. The allowable spread criterion is to stay within the shoulder width of 9.84 ft (3.0 m).

G_1	=	-2.5 percent
G_2	=	+2.5 percent
L	=	500 ft (150 m)
S_x	=	0.02 ft/ft (m/m)

Step 1. Find the vertical curve constant, K.

$K = L / (G_2 - G_1) = 500 / (2.5 - (-2.5)) = 100$ ft/percent

Step 2. Determine depth at curb for design spread.

$d = S_x T = (0.02)(9.84) = 0.2$ ft

Step 3. Determine the flanking inlet locations.

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

Solution:	Flanker inlets located within 38.5 ft (11.7 m) from the sag meet the design spread criterion.

7.4.4 Spread in a Sag Curve

The information in Section 7.3 and Section 7.4.3 describes the interception capacity of inlets in sag locations. As the low point approaches, the longitudinal slope decreases increasing spread for any given flow. Except where inlets become clogged, spread on these low gradient approaches to the low point tends to become the critical drainage design criteria rather than the interception capacity of the sag inlet. AASHTO (2018) recommends that a gradient of 0.3 percent be maintained within 50 ft of the level point to provide adequate drainage. Inlet locations may be adjusted to avoid excessive spread in the sag curve.

In addition, inlets may be appropriate between the flanking inlets and the ends of the vertical curves. For major sag points, flanking inlets are added as a safety factor, and are not the primary means of intercepting flow. Designers include them to assist the sag point inlet in the event of clogging.

7.5 Median and Embankment Inlets

Removal of stormwater at locations other than the main road surface, but within the roadway corridor, drives the need for inlets in roadway medians and to protect roadway embankments. Design for these environments uses principles discussed earlier in this chapter as well as additional considerations.

7.5.1 Median and Roadside Ditch Inlets

Designers place inlets in medians and roadside ditches to remove water that could cause erosion and in roadside ditches at the intersection of cut and fill slopes to prevent erosion downstream of cut sections. In addition, designers provide median inlets to capture water so that it can be removed from the road corridor without crossing the roadway surface. Pipe culverts under one roadway and cross drainage culverts which are not continuous across the median are also used to redirect flow out of a median ditch.

Designers consider roadside safety when locating median and roadside inlets, pipes, and discontinuous cross drainage culverts. Drop inlets flush with the ditch bottom and traffic-safe bar grates placed on the ends of pipes reduce hazards to errant vehicles. Figure 7.20 illustrates a traffic-safe drop inlet in a median ditch like those used for pavement drainage.

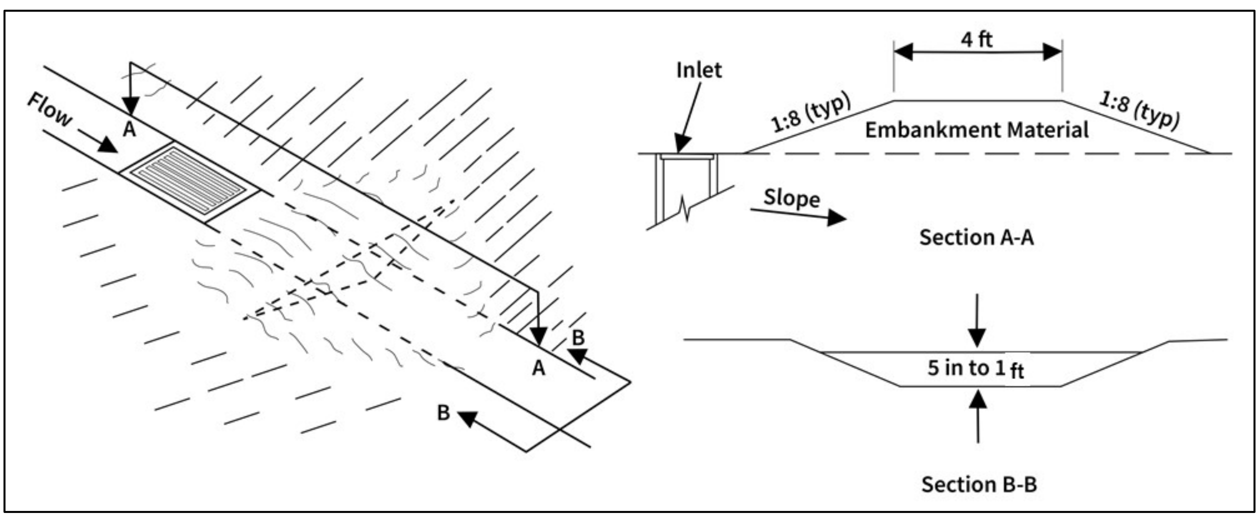

Figure 7.20. Median drop inlet with a downstream berm.

Designers balance drainage requirements, safety, clogging potential, and erosion potential. Where practical, continuous cross drainage structures across the median reduce clogging and erosion potential compared with non-continuous cross drainage. Because ditches tend to erode at drop inlets, paving around the inlets helps to prevent erosion and may increase the interception capacity of the inlet marginally by acceleration of the flow.

At times, designers use culverts to collect stormwater from medians. These generally need more water depth to intercept median flow than drop inlets. No test results are available on which to base design procedures for estimating the effects of placing grates on culvert inlets, but little effect is expected.

The interception capacity of drop inlets in median ditches on continuous grades depends on the approach depth and velocity, as well as the inlet dimensions and type. Chapter 6 discusses flow in median and roadside ditches and describes Manning's equation for open channels:

$$Q = \left(K_u / n\right) A R^{0.67} S_L^{0.5} \tag{7.24}$$

where:

Q	=	Discharge rate, ft^3/s (m^3/s)
K_u	=	Unit conversion constant, 1.49 in CU (1.0 in SI)
n	=	Hydraulic resistance variable
A	=	Cross-sectional area of flow, ft^2 (m^2)
R	=	Hydraulic radius (flow area/wetted perimeter), ft, (m)
S_L	=	Bed slope, ft/ft (m/m)

Interception capacity is based on the ratio of frontal flow to total flow in the median ditch. For a trapezoidal channel, the frontal flow ratio is:

$$E_o = W / \left(B + d z\right) \tag{7.25}$$

where:

E_o	=	Ratio of frontal to total flow
W	=	Inlet width, ft (m)
B	=	Bottom width of the trapezoidal channel, ft (m)
d	=	Flow depth in the channel, ft (m)
z	=	Channel side slope (1:z)

Placement of small berms down-gradient of drop inlets, as shown in Figure 7.20, increases interception by impeding bypass flow. If not overtopped, the berm provides for complete interception of the approach flow. In many cases, the berms are small (less than 6 inches high) and have traffic-safe slopes. Berm height for complete interception on continuous grades or the depth of ponding in sag vertical curves depends on the maximum ponding depth estimated for weir flow in a sag (equation 7.14) or orifice flow in a sag (equation 7.15). In contrast to weir flow at a grate against a curb, the effective perimeter of a grate in an open channel with a berm is 2(L + W) since one side of the grate is not adjacent to a curb and flow is captured from both sides. See Section 7.3.1 for computing ponding depth in a sag.

Inlet interception for median and ditch inlets on grade depends on the capture of frontal and side flow as described in Section 7.2.1 for grate inlets at a curb. For a ditch bottom that is approximately equal to the inlet width, the ditch side slope is used to estimate side flow capture. For a ditch bottom wider than the inlet width, the side slope is effectively zero. In this case, a low side slope, such as 1 percent, is used to estimate side flow capture. Experience indicates that both assumptions are conservative.

To avoid overtopping the berm because of the momentum of the water, designers add freeboard to the berm height. The height of freeboard varies with the situation; 0.5 ft is often a good starting point.

Example 7.11:	Median ditch inlet.
Objective:	Find the intercepted and bypassed flows at a 2 ft by 2 ft (0.61 m by 0.91 m) median ditch inlet with a P-1-7/8 parallel bar grate. Determine the berm height needed to provide 100 percent interception.

Given: A median ditch with the following characteristics:

B	=	2.0 ft (0.61 m)
n	=	0.03
z	=	6
S_L	=	0.03 ft/ft (0.03 m/m)
Q	=	9.9 ft^3/s (0.28 m^3/s)

Step 1. Estimate the channel parameters.

Using equations 6.7, 6.8, and 6.9 for a trapezoidal channel and an initial trial depth of 0.5 ft:

$A = Bd + zd^2 = (2.0)\ (0.5) + (6)\ (0.5)^2 = 2.50\ ft^2$

$P = B + 2d\ (z^2+1)^{0.5} = (2.0) + (2)\ (0.5)\ (6^2+1)^{0.5} = 8.08\ ft$

$R = A/P = 2.50/8.08 = 0.309\ ft$

Using equation 7.24:

$Q = (K_u/n)\ A\ R^{(2/3)}\ S_0^{(1/2)} = (1.49/0.03)\ (2.50)\ (0.309)^{(2/3)}\ (0.03)^{(1/2)} = 9.8\ ft^3/s$

This flow is close to 9.9 ft^3/s. No further iterations needed. A depth of 0.5 ft is satisfactory for further computations.

Step 2. Compute the ratio of frontal to total flow in trapezoidal channel.

Using equation 7.25:

$E_o = W / (B + dz) = (2.0 / [2.0 + (0.5)(6)] = 0.40$

Step 3. Compute frontal flow efficiency.

$V = Q/A = 9.9 / 2.50 = 3.96\ ft/s$

From Figure 7.8 the splash-over velocity is 8 ft/s, which is greater than the approaching velocity. Therefore, from equation 7.5:

$R_f = 1.0$

Step 4. Compute side flow efficiency.

Apply equation 7.6 to solve for R_s. This equation assumes side flow from only one side with the other side being at the curb. Therefore, applying this equation for two sides as in this median ditch is conservative.

$S_x = 1/z = 1/6 = 0.167$

$R_s = 1/[1 + (K_u V^{1.8})/(S_x L^{2.3})] = 1/[1 + (0.15)(3.96)^{1.8}/\{(0.167)\ (2.0)^{2.3}\}] = 0.315$

Note: When the ditch is wider than the inlet, S_x is zero at the edge of the inlet in the bottom of the median ditch. A flat slope, such as S_x = 0.01, can be used in the equation.

Step 5. Compute total efficiency.

$E = E_o R_f + R_s (1 – E_o) = (0.40)(1.0) + (0.315)(1 - 0.40) = 0.59$

Step 6. Compute interception and bypass flow.

$Q_i = E Q = (0.59)(9.9) = 5.8$ ft^3/s

$Q_b = Q - Q_i = (9.9) - (5.8) = 4.1$ ft^3/s

Step 7. Estimate the height of a downstream berm for 100 percent interception.

Assuming the depth results in weir flow, calculate the perimeter, and use equation 7.14:

$P = 2 (L+W) = 2 (2.0+ 2.0) = 8.0$ ft

$d = [Q_i / (C_w P)]^{0.67} = [(9.9) / \{(3.0)(8.0)\}]^{0.67} = 0.55$ ft

Solution: A P-1-7/8 inlet intercepts about 59 percent of the design flow for the given conditions. To capture 100 percent of the flow, the anticipated ponding depth is 0.55 ft (0.17 m). The berm will be that height plus freeboard to prevent the momentum of flow overtopping the berm.

7.5.2 Embankment Inlets

Where adequate vegetative cover cannot be established on embankment slopes to prevent erosion or where flow running down an embankment is concentrated, designers can use inlets with downdrains, swales, or chutes to protect the embankment. Inlets used at embankments and adjacent to bridges differ from other pavement drainage inlets in three respects:

- The economies achieved by system design are often not possible because a series of inlets is not used.
- A closed storm drainage system is often not available to receive the intercepted flow; alternatively, it is usually discharged into open chutes or pipe downdrains which terminate at the toe of the fill slope.
- Total or near total interception is sometimes necessary to limit the bypass flow from running onto a bridge deck.

Figure 7.21 shows a pipe downdrain that confines the flow and cannot cause erosion along the sides. Pipes can be covered to reduce or eliminate interference with maintenance activities on the fill slopes. Open chutes are often damaged by erosion from water splashing over the sides of the chute due to oscillation in the flow and from spill over the sides at bends in the chute.

High velocity flow discharged from downdrains and chutes may cause erosion when unmitigated. Strategies for minimizing erosion include:

- Extending the discharge point to where erosion would not be problematic.
- Providing well-graded gravel or rock to protect soils.
- Installing an elbow or a “tee” at the end of the downdrains to redirect the flow.
- Providing other types of energy dissipators. (HEC-14 provides detailed information on energy dissipator design (FHWA 2005)).

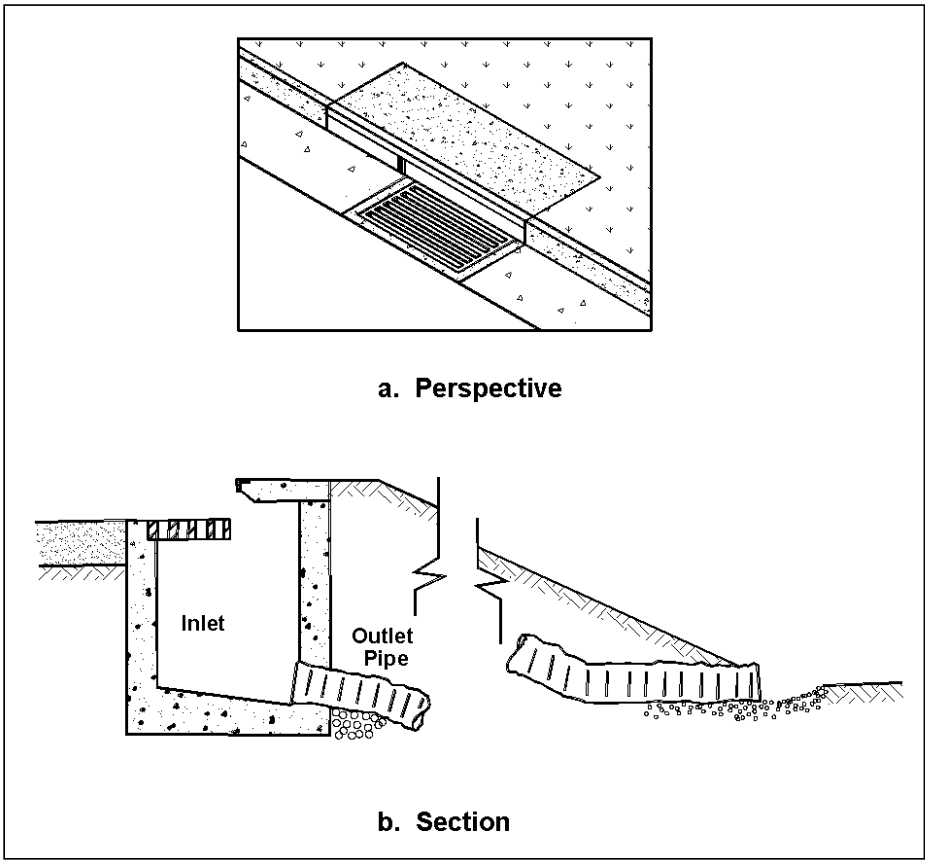

Figure 7.21. Embankment inlet and downdrain.

Section 7.2.1 provides tools and approaches for designing embankment inlets on grade. However, to prevent erosion caused by bypass flow, embankment inlets may necessitate higher or near total interception efficiencies than many typical applications can achieve. Grate inlets intercept little more than the flow conveyed by the gutter width occupied by the grate. Options to increase interception include:

- Combination inlets with the length of curb opening upstream of the grate (sweeper configuration) sufficient to reduce spread in the gutter to the width of the grate.
- Depressed curb openings.
- Extra width grate inlets with the width based on the design spread.
- Slotted inlets with a length based on the design spread.

Chapter 8 - Storm Drain Structures

Storm drain structures provide the connections between the ground surface and the storm drain system and between storm drain conduits. These structures include inlets, access holes, and junction chambers. Other miscellaneous storm drain components include transitions, flow splitters, siphons, and flap gates.

Most State DOTs develop their own design standards for commonly used structures resulting in variations in the design details of even the simplest storm drain structures. Recognizing that design details vary, this chapter describes common features and functions of storm drain structures.

8.1 Inlet Structures

Inlet structures, sometimes referred to as catch basins, allow surface water to enter the storm drainage system. Inlet structures also provide access points for cleaning and inspection.

8.1.1 Configuration and Materials

Figure 8.1 illustrates several typical box-shaped inlet structures including a standard drop inlet, inlet with a sump, curb inlet, and combination inlet. Chapter 7 covers the hydraulic design of surface inlets.

The inlet illustrated in Figure 8.1b captures surface flow in a similar way to a drop inlet but also has a sump that retains sediment and debris transported by stormwater into the storm drainage system. To be effective and to avoid becoming an odor and mosquito nuisance, inlet sumps involve periodic cleaning. However, in areas with site constraints that place storm drains on relatively flat slopes, and where the DOT follows a strict maintenance plan, inlets can serve to collect sediment and debris. Chapter 11 discusses storm drain inlets designed specifically to remove sediment, oil, and debris.

DOTs most commonly use cast-in-place concrete and pre-cast concrete for inlet construction.

8.1.2 Location

Chapter 7 describes inlet spacing based on where they are needed to capture surface flow. Below ground, designers locate inlet structures at the upstream end and at intermediate points along a storm drain line. Designers generally use an iterative process to locate inlet structures to produce an economical and hydraulically effective system.

8.2 Access Holes

Access holes provide convenient access to the storm drainage system for inspection and maintenance. Access holes also serve as flow junctions and can provide ventilation and pressure relief for storm drainage systems. An access hole provides pressure relief if its access door is not water-tight and allows water to escape to the surface when the hydraulic grade line (HGL) reaches the surface. Designers may select water-tight access doors to prevent escape when warranted.

Access holes do not accept surface flows as inlets do. Where access and surface water interception are desirable, designer use inlets rather than access holes.

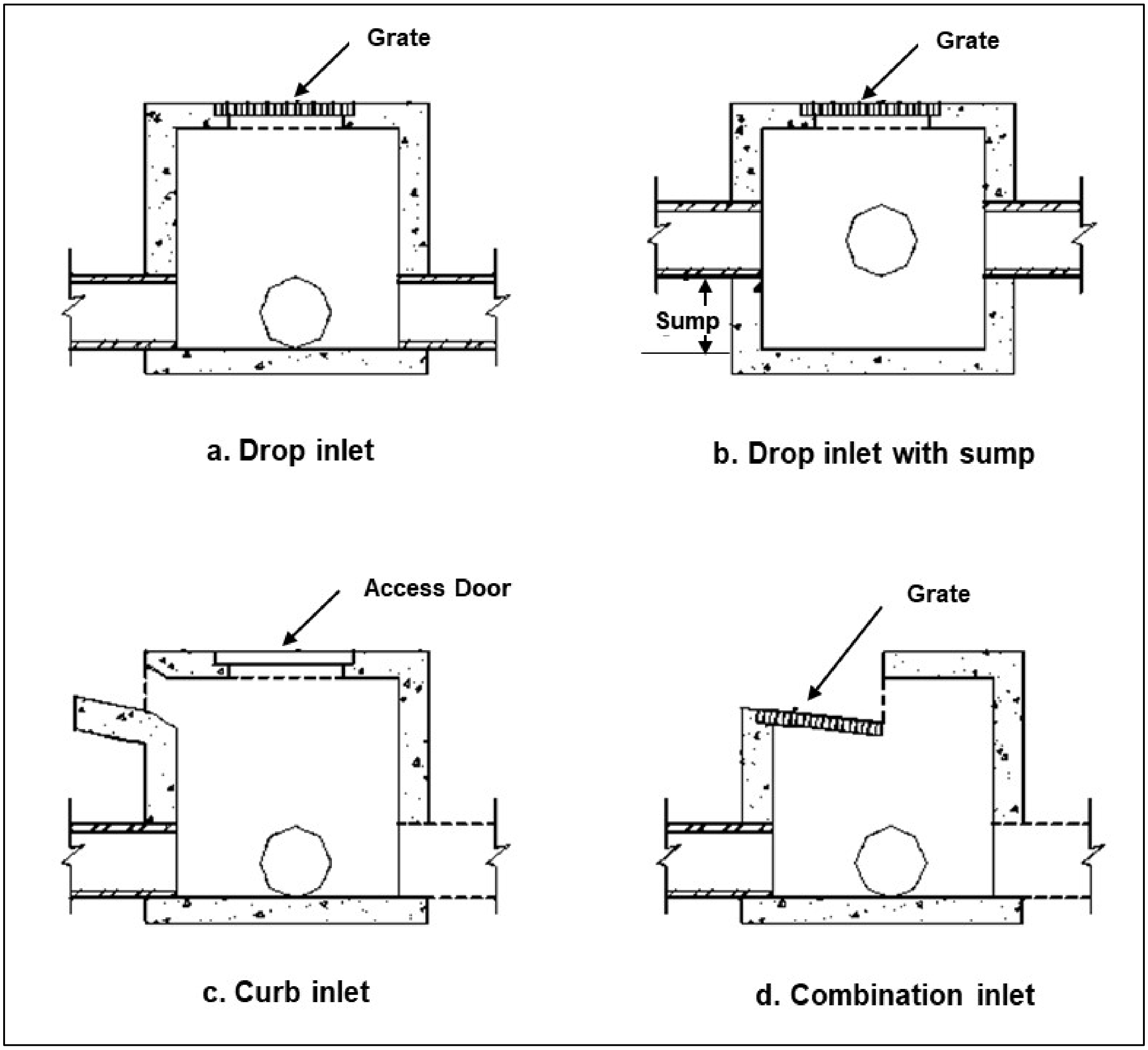

Figure 8.1. Inlet structures (elevation view).

8.2.1 Configuration and Materials

Designers use many configurations but orient the access hole so that workers can safely enter it while facing traffic if traffic exists. Access steps provide a means of convenient access and comply with applicable safety requirements. Using corrosion resistant materials for the steps (e.g., steps coated with neoprene or epoxy, or steps fabricated from rust-resistant material such as stainless steel or aluminum coated with bituminous paint), enhances safety and longevity.

Some access hole configurations do not include steps. Rather, maintenance personnel supply their own ladders to avoid safety issues associated with rust-damaged steps and to restrict access.

Typically, the access shaft (cone) provides a minimum horizontal clear opening of 24 inches. Most access holes are circular with the inside dimension of the bottom chamber being sufficient to perform inspection and cleaning operations without difficulty. Bottom chambers typically include a minimum inside diameter of 4 ft with a 5 ft inner diameter access hole being used with larger diameter connecting storm drain pipes.

In some cases, a smaller access shaft aligns concentrically with the bottom chamber. Figure 8.2a displays a constant diameter bottom chamber up to a conical section a short distance below the top. In other design configurations, the access shaft and bottom chamber align to provide a vertical series of steps for easier access. Figure 8.2b illustrates a practice that uses an eccentric cone for the access shaft.

Figure 8.2c shows an option that maintains the bottom chamber diameter to a height sufficient for adequate working space. This design tapers to 3 ft for the access shaft. The frame rests on the broad base of the access shaft. Because of the inward leaning angle of the steps in the access shaft, designers typically limit these configurations to bottom chambers 3 ft in diameter or less.

Figure 8.2d illustrates a design that minimizes the access shaft height and features a removable flat precast concrete slab that facilitates addressing more extensive maintenance needs. Designers prefer these tangent alignments for access holes with bottom chamber diameters 4 ft or greater.

The size of the bottom chamber limits the size of storm drain conduits that can connect to it. For larger storm drain conduits that are not readily accommodated by typical access hole structure configurations, designers could choose a vertical riser connected to the storm drain pipe with a "tee" unit as illustrated in Figure 8.3.

The configurations in Figure 8.2 represent variations of channels and benching at the bottom of the access hole. Many access holes do not include benching, which designers refer to as a "no benching" configuration. Flow channels provide a smooth, continuous path for the flow, reducing turbulence and, therefore, energy losses in the access hole. The bench elevates the bottom of the access hole on either side of the flow channel further increasing the hydraulic efficiency of the access hole. Because of the added cost associated with benching, designers use it when the HGL is relatively flat and there is no appreciable head available. Chapter 9 discusses energy losses and benching in greater detail.

Access hole frames and covers provide adequate strength to support superimposed loads, provide a fit between cover and frame, facilitate opening while providing resistance to unauthorized opening (especially from children), and prevent blowouts. To differentiate storm drain access holes from other underground utility access such as for sanitary sewers and communication conduits, good practice includes the words "STORM DRAIN" or equivalent cast into the top surface of the covers. Blowouts may occur during a flood event when the HGL in the access hole rises above the ground surface with sufficient pressure to move the cover from its normal position on the frame. To prevent blowouts, designers can provide openings to release surcharged flow or secure the cover in the frame with bolts or another type of locking mechanism.

Designers most commonly use pre-cast concrete and cast-in-place concrete when selecting materials for access holes. In most areas, the availability and competitive cost of pre-cast concrete access holes make them popular. They may include cast-in-place steps at the desired locations and special transition sections to reduce the diameter of the access hole at the top to accommodate the frame and cover. The transition sections are usually eccentric with one side vertical to accommodate access steps.

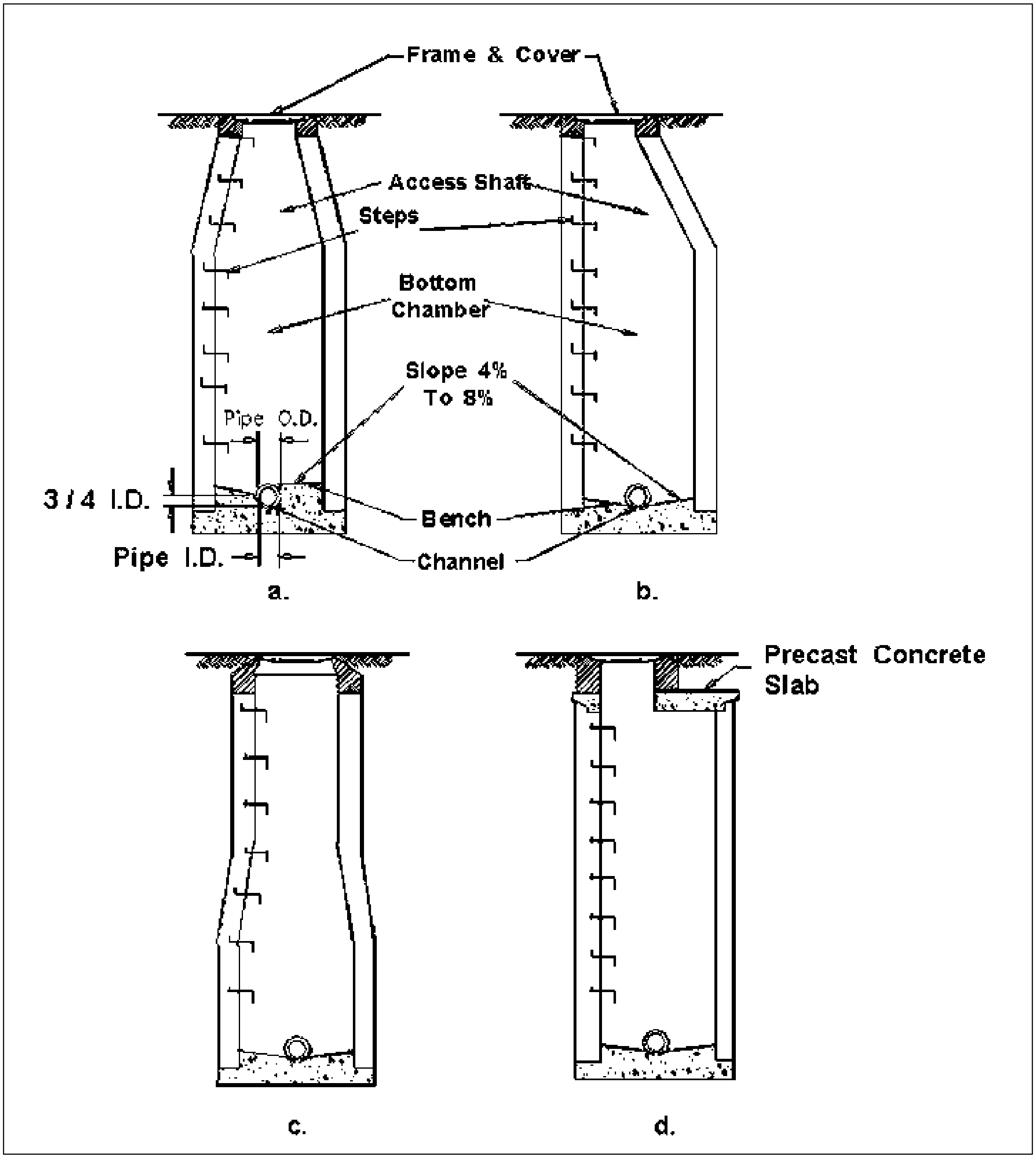

Figure 8.2. Typical access hole configurations.

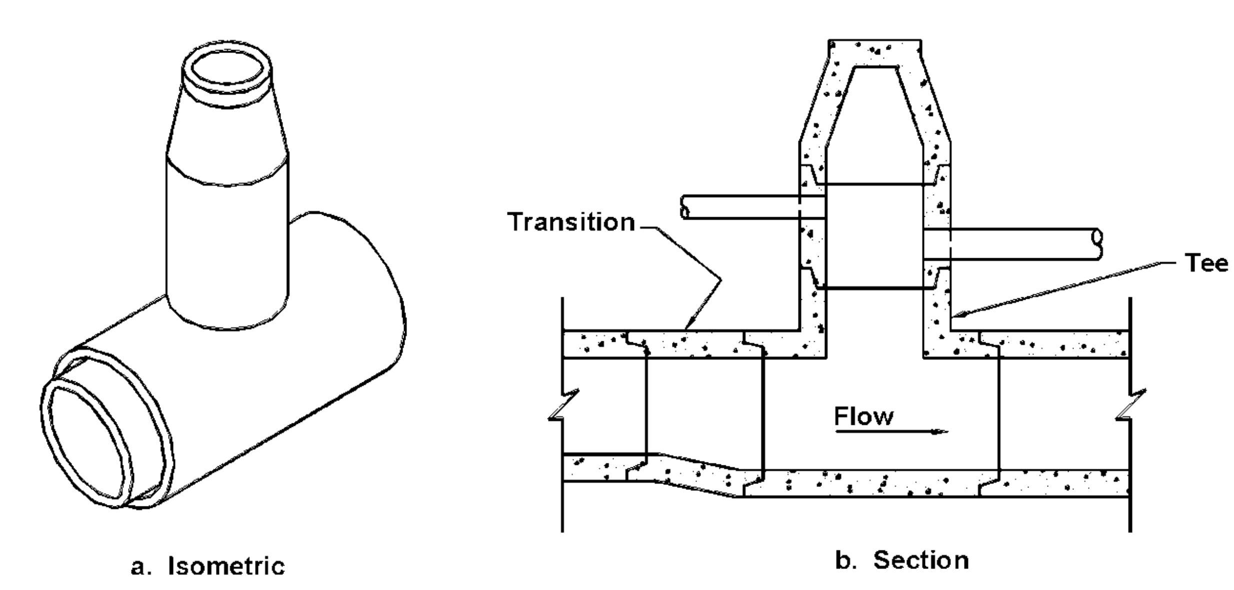

Figure 8.3. “Tee” access hole for large storm drains.

8.2.2 Depth, Location, and Spacing

Access hole depth, location, and spacing depend on design criteria related to hydraulic effectiveness, structural integrity, and maintenance. Chapter 9 (Storm Drain Conduits) describes many of these criteria and how they affect storm drain system design.

The storm drain profile and surface topography contribute to access hole depth. Typically, access hole depths range from 5 to 13 ft. In some circumstances, for example, to avoid other utilities, designers may specify access hole depths outside this range.

Irregular surface topography sometimes results in shallow access holes. When the depth to the invert is only 2 to 3 ft, all maintenance operations can be conducted from the surface. However, because maintenance activities are not comfortable from the surface, even at shallow depths, designers specify the same access hole widths used for bottom chambers of greater depths (4 to 5 ft). To enable a worker to stand in the access hole for maintenance operations, designers will include a large cover with a 2.5 to 3.0 ft opening. Access hole dimensions typically conform to applicable design standards which comply with pertinent safety requirements.

Structurally, access holes withstand soil pressure loads that increase with depth. In addition, access holes that extend below the water table withstand hydrostatic pressure and prevent excessive seepage. Since long portable ladders for deep access holes would be cumbersome and could be dangerous, designers provide access with either steps or built-in ladders that conform to applicable design standards and safety requirements.

Access hole location and spacing criteria consider maintenance access and equipment limitations. Although these criteria vary from jurisdiction to jurisdiction, designers often place access holes where:

- Two or more storm drains converge.
- Pipe sizes change.
- A change in horizontal alignment occurs.
- A change in vertical alignment occurs.

In addition, designers locate access holes at intermediate points along straight runs of storm drain in accordance with applicable spacing criteria. Table 8.1 shows example spacing criteria, but designers use the criteria from the jurisdiction within which they are working.

Table 8.1. Example access hole spacing criteria (AASHTO 2000).

Pipe Size (in)	Suggested Maximum Spacing (ft)
12 – 24	300
27 – 36	400
42 – 54	500
60 and up	1000

8.3 Junction Chambers

A junction chamber is an underground chamber used to join two or more large storm drain conduits. Designers commonly use this type of structure where storm drains are larger than the size that can be accommodated by standard access holes and may be rectangular, circular, or irregular in shape. Unlike access holes, junction chambers do not typically extend to the ground surface and can be completely buried. However, designers often include riser structures to provide surface access or to intercept surface runoff. Where junction chambers are used as access points for the storm drain system, designers follow the same criteria appropriate for access holes as discussed in Section 8.2.2.

To minimize flow turbulence and, therefore energy losses, in junction chambers, designers can include flow channels and benches in the bottom to guide flow through the chamber. Chapter 9 describes the use of benching and energy loss computations in more detail.

Designers commonly use pre-cast concrete and cast-in-place concrete for junction chamber construction materials. Storm drains constructed of corrugated metal may have junction chambers made of the same material.

8.4 Other Structural Components

In addition to inlet structures, access holes, and junction chambers, designers employ other structural components to connect elements of a storm drain system or to serve purposes not needed in many systems. These include transitions, flow splitters, siphons, and flap gates.

8.4.1 Transitions

In storm drainage systems, transitions from one pipe size to another typically occur in access holes or junction chambers. Where maintenance access is not needed, designers use transitions to avoid obstructions, to join different pipe sizes, and to join different conduit shapes. Figure 8.4 illustrates a transition where a rectangular pipe transition is used to avoid an obstruction. Figure 8.3 illustrates use of a transition upstream of tee type access holes.

Because abrupt transitions increase turbulence and energy loss, designers provide smooth, gradual transitions to minimize head losses whenever feasible. For example, when the flow velocity is less than 20 ft/s, designers typically use a 5:1 to 10:1 transition ratio for both expansion and contraction in the straight wall configuration shown in Figure 8.4a. For higher velocities, more gradual transition ratios of 10:1 to 20:1 reduce energy losses further.

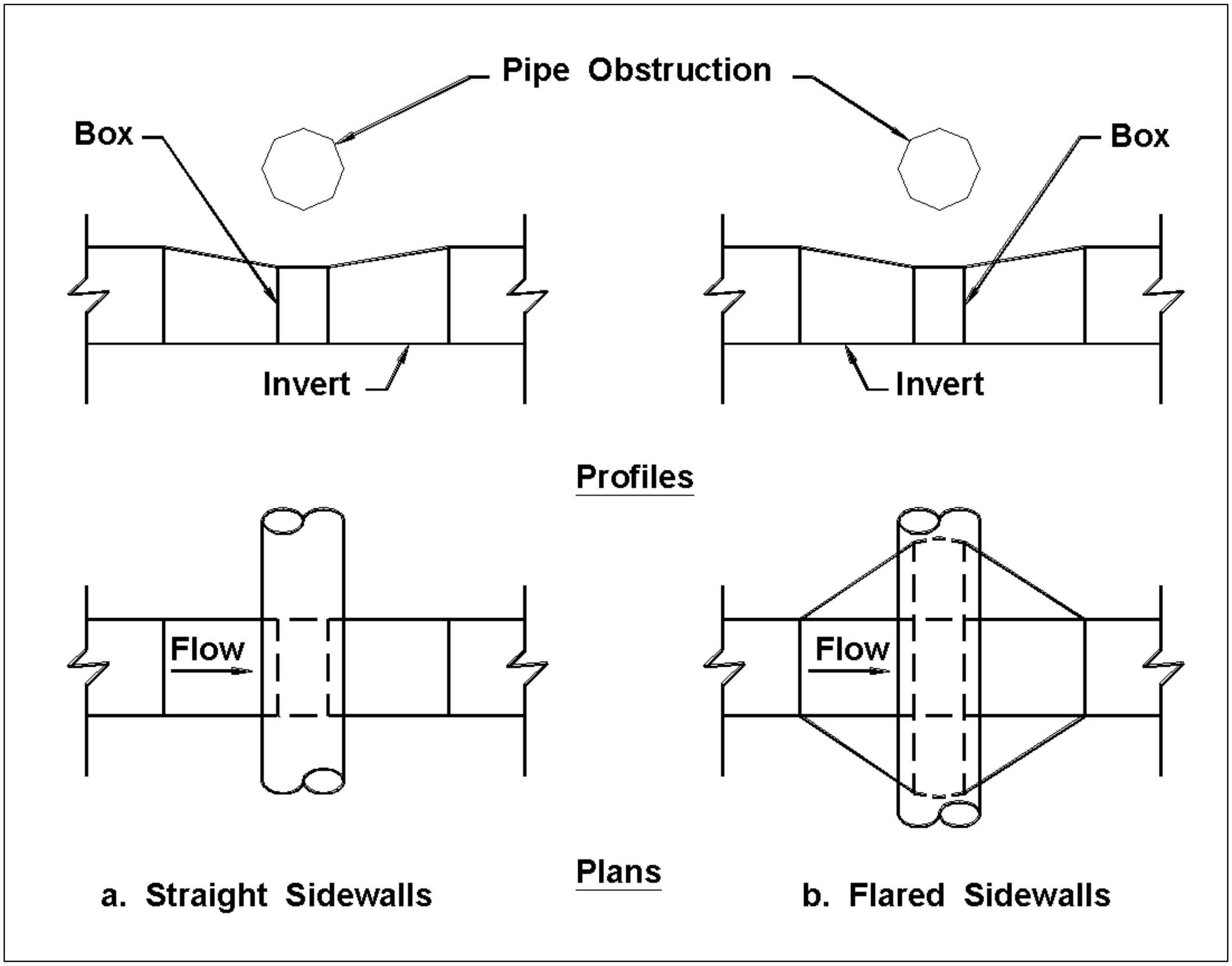

Figure 8.4. Transitions to avoid obstruction.

8.4.2 Flow Splitters

A flow splitter divides flow in an incoming storm drain conduit into two or more outgoing conduits for applications where high flows are diverted away from locations with limited capacity, e.g., water quality devices. As with other structures, designers consider how to minimize the unavoidable energy loss at the point of flow division and within the structure. Deflectors can guide flow through the structure and reduce energy losses. Where possible, designers avoid regions of flow velocity reduction that can cause deposition of material suspended in the stormwater flow.

Designers also seek to avoid capturing debris within the structure. If one of the outgoing conduits is smaller than the incoming conduit carrying debris, the debris can be captured by the outgoing conduit. Because of sediment deposition and debris capture, flow splitters can become maintenance intensive. Although flow splitters can be designed without maintenance access like junction chambers, designers generally provide for maintenance access.

8.4.3 Inverted Siphons

An inverted siphon or depressed pipe carries flow under an obstruction such as a utility conduit, stream, or depressed highway minimizing the energy loss. Figure 8.5 depicts a twin-barrel inverted siphon carrying flow under a river. Inverted siphons can consist of single or multiple barrels; however, the American Association of State Highway and Transportation Officials (AASHTO)suggests a minimum of two barrels (AASHTO 2014). Multiple barrels allow one barrel to operate for lower flows with additional barrels becoming active with higher flows. Regardless of the number of barrels, the lowered section of the inverted siphon does not drain by gravity when the flow stops, and the designer may wish to consider means for draining this section after each storm.

Designers can avoid sediment deposits in the lowered section by designing for sufficiently high velocities over a range of flows to flush any deposits. Designers can also facilitate flushing of sediment deposits by limiting the slope of the rising portion of the lowered section to a maximum of 15 percent or jurisdictionally specified value. Designers may include a sump in the inlet chamber to collect sediment prior to entering the siphon.

To minimize energy losses, debris capture, and sedimentation, designers avoid sharp bends and maintain a constant conduit section throughout the lowered section. Because inverted siphons are generally not maintenance free, designers plan for cleaning and maintenance.

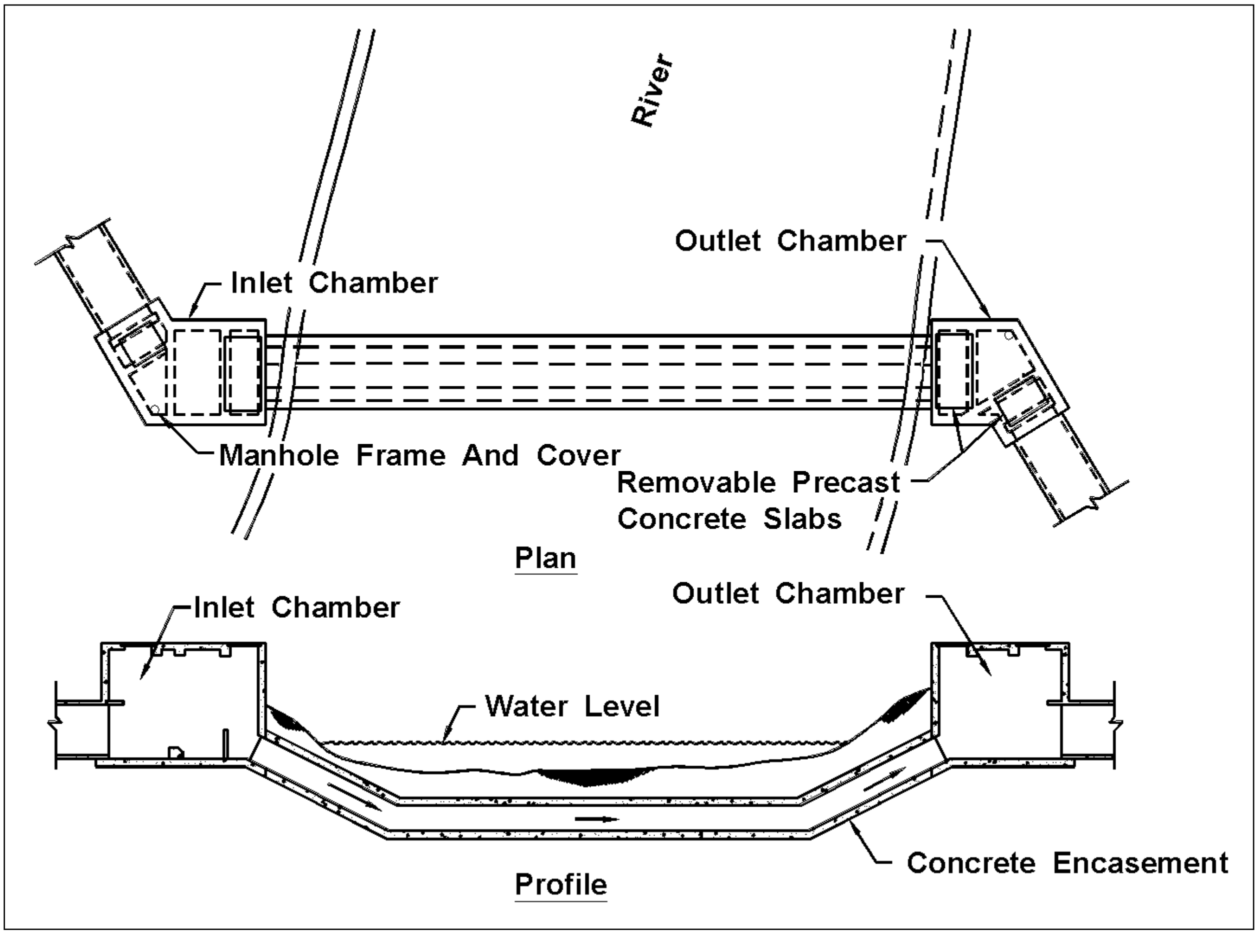

Figure 8.5. Twin-barrel inverted siphon.

8.4.4 Flap Gates

Designers use flap gates to prevent back-flooding of a drainage system outlet in the presence of high tides or high stages in receiving waters. During rainstorms, properly functioning flap gates open in response to the hydrostatic pressure from the stormwater in the conduit allowing discharge to the receiving waters. With high receiving water levels, the hydrostatic pressure from the receiving water keeps the flap gate in a closed position for the purpose of preventing water from entering the storm drain system. When rainstorms and high receiving water levels occur simultaneously, the dominant hydrostatic force, combined with the weight of the flap gate, determines whether the flap gate opens or closes. To avoid storm drain backups, the designer considers the probability and consequences of the situation where the receiving water hydrostatic force dominates and restricts discharge during a rainstorm.

Sediment, organic materials, and trash can impair the functioning of outlet conduits with flap gates by reducing the conveyance of the conduit. The reduction of flow velocity behind a closed or partially open flap gate may also cause sediment deposition in the storm drain near the outlet. Organic materials and trash from the storm drain system or the receiving waters can collect between the flap and seat preventing full closure of the flap gate. In addition, where a flap gate is mounted on a pipe projecting into a stream, the designer considers how to protect the conduit and flap gate from damage by woody material or ice during high flows. Flap gate installations depend on regular inspection and removal of accumulated sediment, organic materials, and trash to serve their intended function.

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Chapter 9 - Storm Drain Conduits

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

9.1 Hydraulics of Storm Drainage Systems

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

9.1.1 Flow Type Assumptions

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

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

9.1.2 Open Channel and Pressure Flow

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

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

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

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

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

9.1.3 Hydraulic Capacity

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

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

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

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

where:

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

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

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

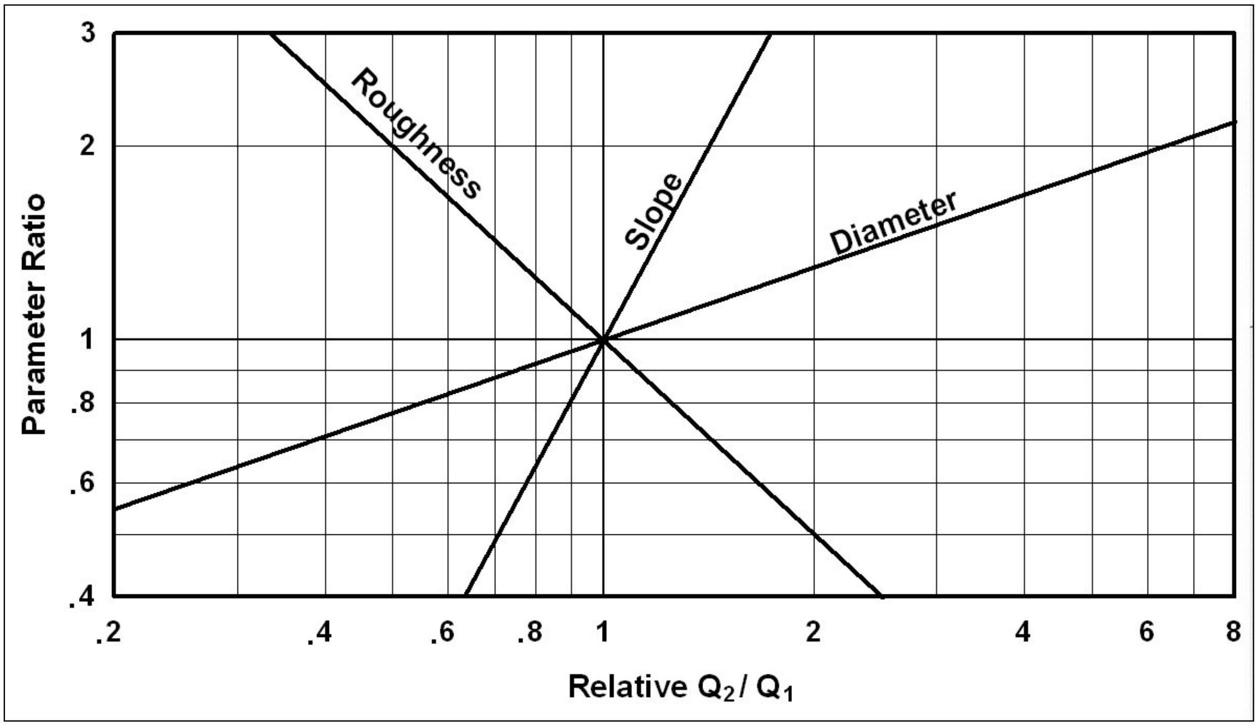


Figure 9.1. Storm drain capacity sensitivity.

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

Economies in Pipe Quantities

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

For circular conduits:

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

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

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

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

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

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

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

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

Example 9.1: Pipe size alternative and capacity.

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

Given:

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

S_o = 0.015 ft/ft (m/m)

n = 0.013 for concrete, 0.017 for corrugated metal

Step 1. Estimate the concrete circular pipe diameter.

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

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

Use D = 21-inch diameter standard pipe size.

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

Using equation 9.2, calculate the CMP diameter.

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

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

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

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

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

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

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

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

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

9.1.4 Energy Grade Line/Hydraulic Grade Line

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

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

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

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

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

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

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

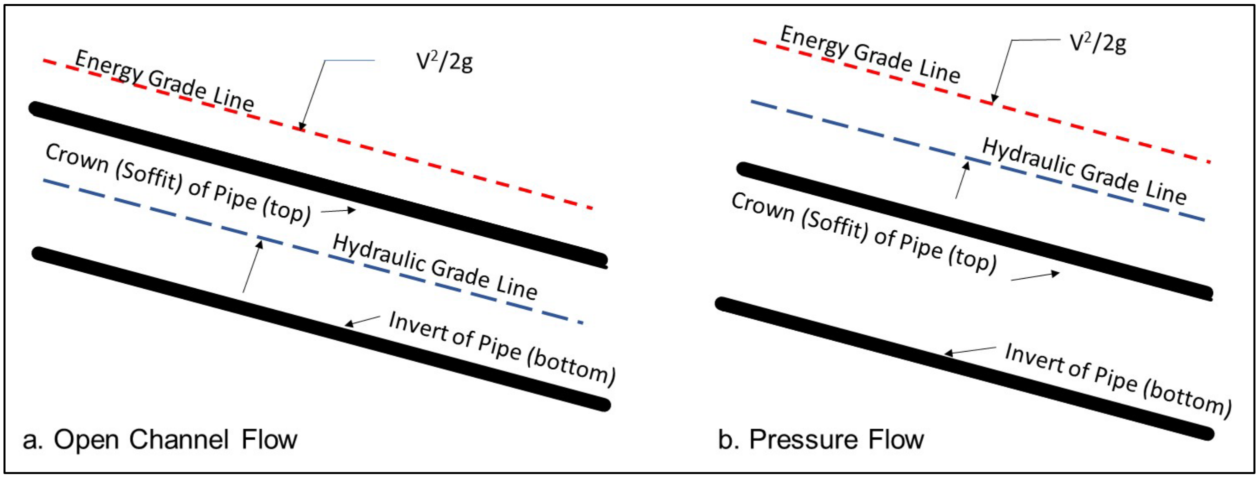


Figure 9.2. Hydraulic and energy grade lines in pipe flow.

9.1.5 Storm Drain Outfalls

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

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

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

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

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

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

9.1.6 Energy Losses

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

9.1.6.1 Pipe Friction Losses

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

$$h_f = S_f L \tag{9.3}$$

where:

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

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

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

where:

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

9.1.6.2 Exit Losses

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

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

where:

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

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

9.1.6.3 Bend Losses

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

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

where:

Δ	=	Angle of bend, degrees

9.1.6.4 Transition Losses

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

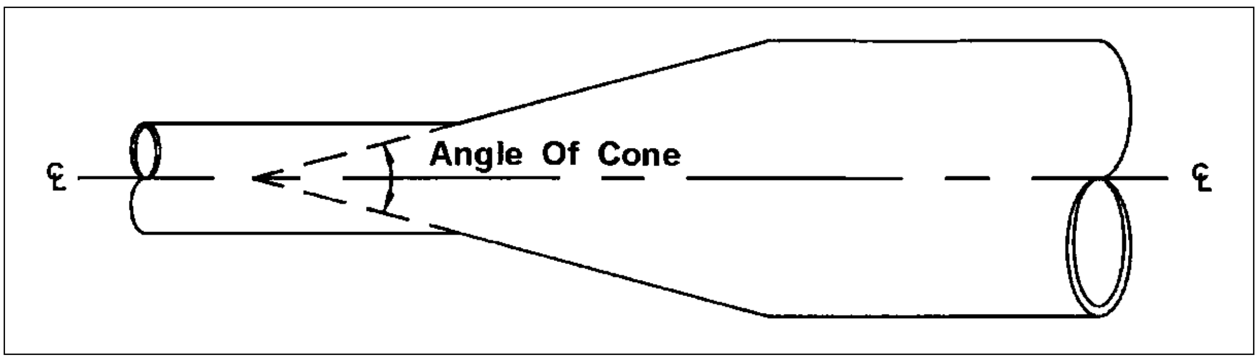


Figure 9.3. Angle of cone for pipe diameter changes.

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

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

where:

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

Table 9.3 presents typical values of K_e for gradual expansions.

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

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

where:

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

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

D_2/D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	.86	1.02	1.06	1.04	1.00

Debris and Clogging

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

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

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

9.1.6.5 Junction Losses

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

$$H_j = \frac{(Q_oV_o) - (Q_iV_i) - (Q_lV_l\cos\theta_j)}{0.5g(A_o + A_i)} + h_i - h_o \quad (9.9)$$

where:

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

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

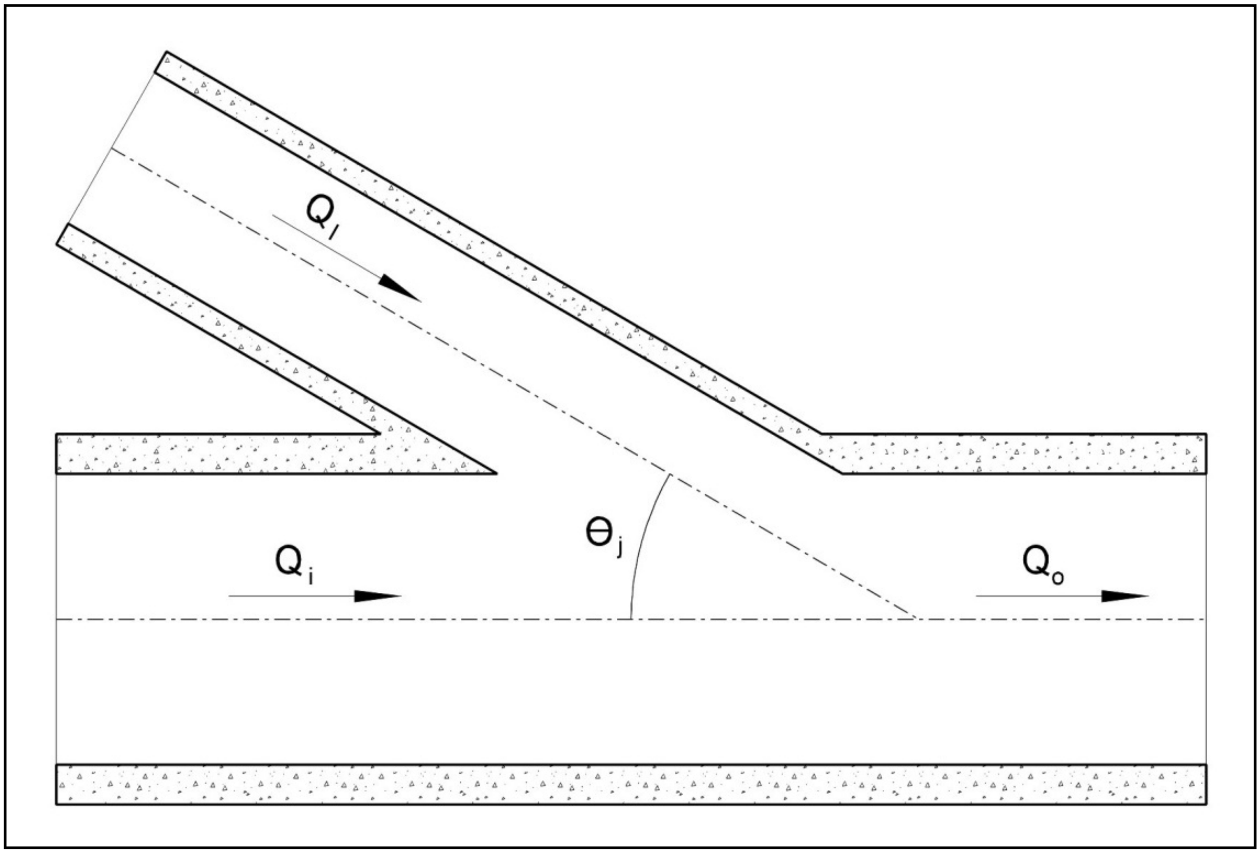


Figure 9.4. Interior angle definition for pipe junctions.

9.1.6.6 Approximate Method for Inlet and Access Hole Energy Loss

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

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

where:

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

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

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

Table 9.4. Head loss coefficients.

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

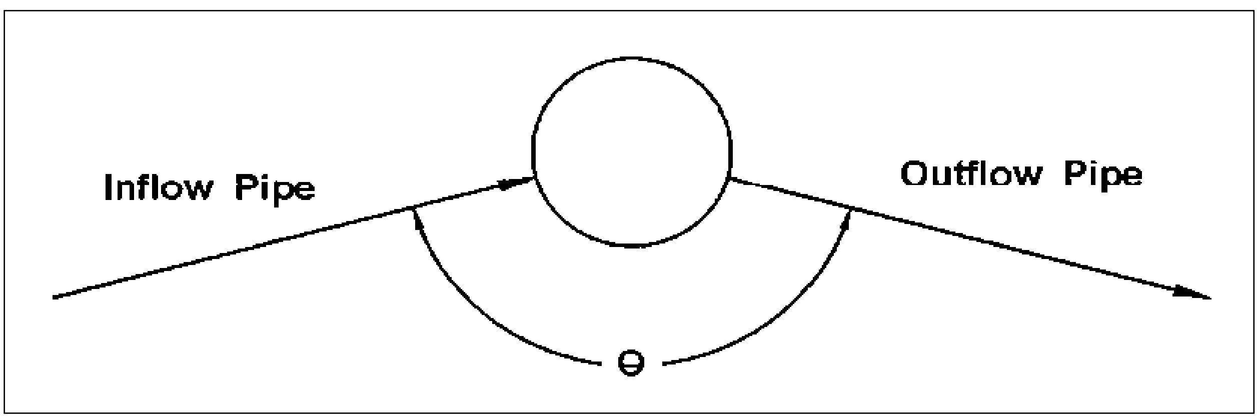


Figure 9.5. Interior angle for access holes.

9.1.6.7 FHWA Inlet and Access Hole Energy Loss

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

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

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

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

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

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

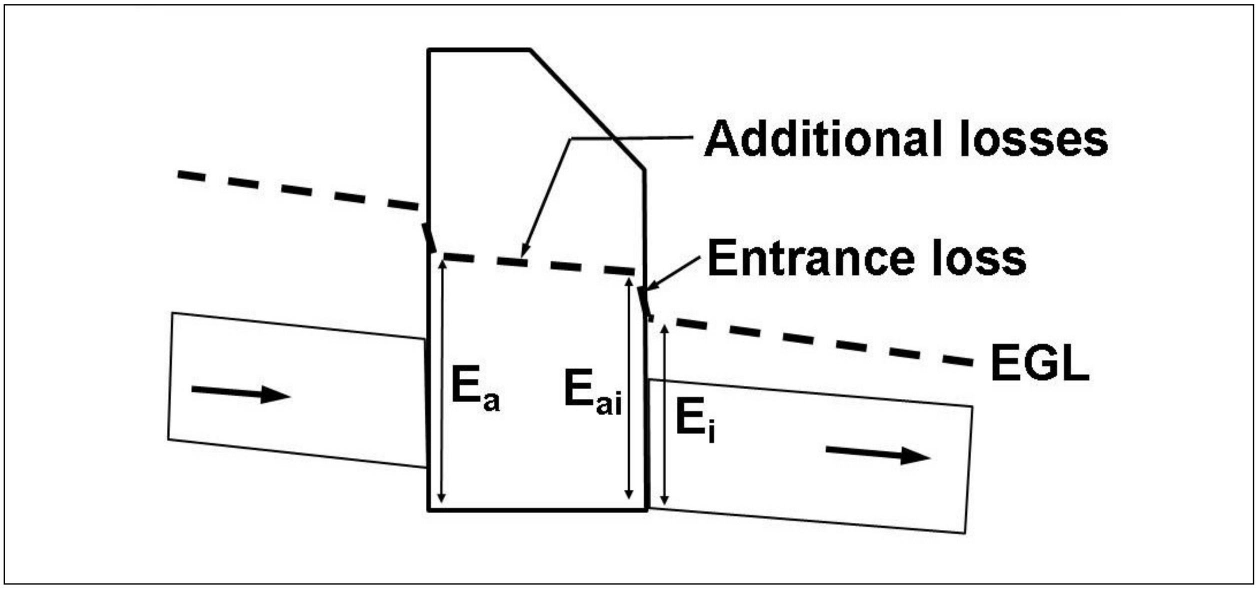


Figure 9.6. Access hole energy level definitions.

9.1.6.7.1 Determining Initial Access Hole Energy Level

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

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

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

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

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

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

where:

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

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

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

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

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

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

where:

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

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

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

where:

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

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

where:

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

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

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

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

where:

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

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

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

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

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

9.1.6.7.2 Adjusting for Benching, Angled Inflow, and Plunging Inflow

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

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

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

$$E_a = E_{ai} + H_B + H_\theta + H_P \quad (9.19)$$

where:

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

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

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

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

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

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

where:

C_B = Energy loss coefficient for benching.

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

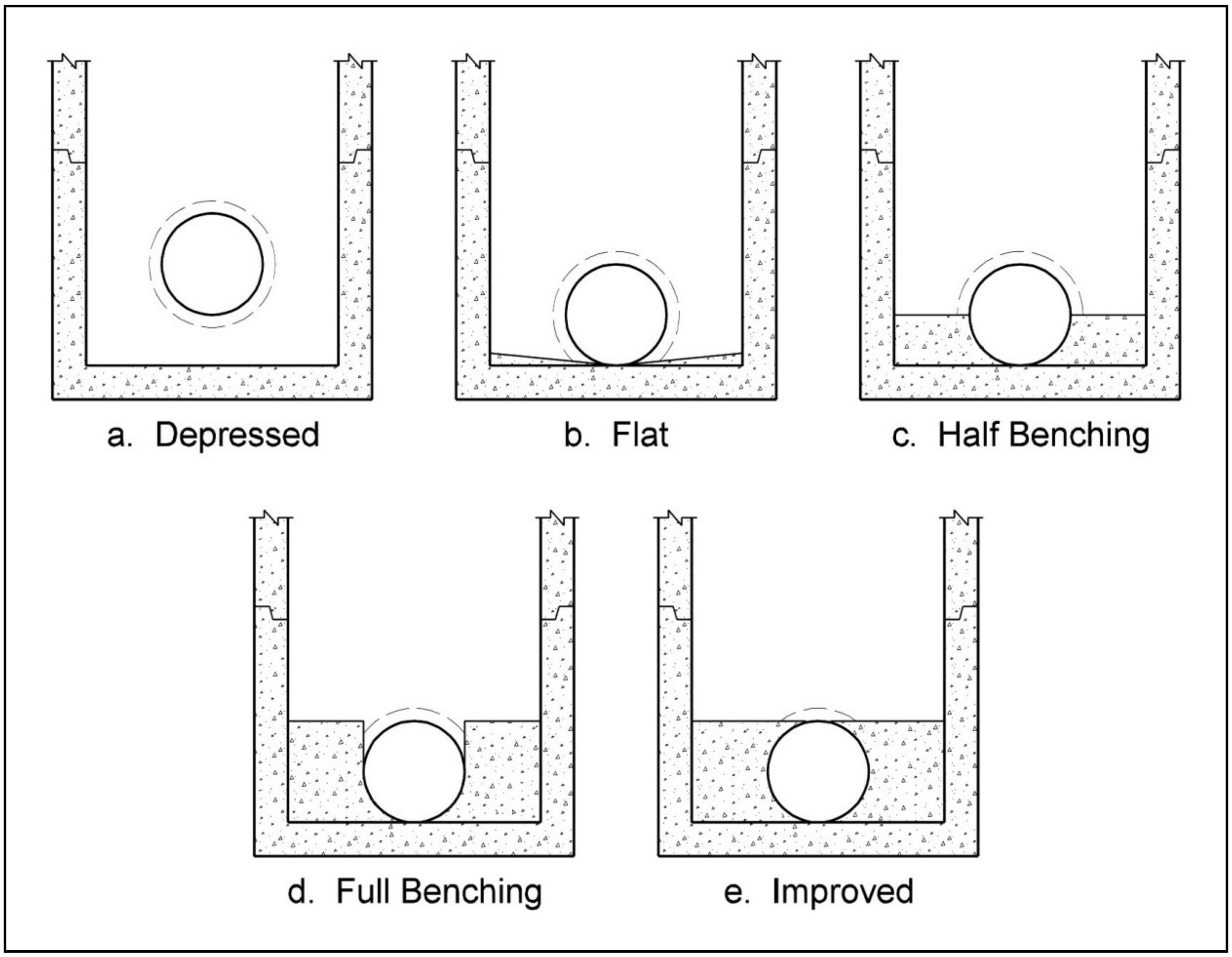


Figure 9.7. Access hole benching methods.

Table 9.5. Values for the coefficient, C_B.

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

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

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

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

where:

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

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

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

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

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

where:

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

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

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

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

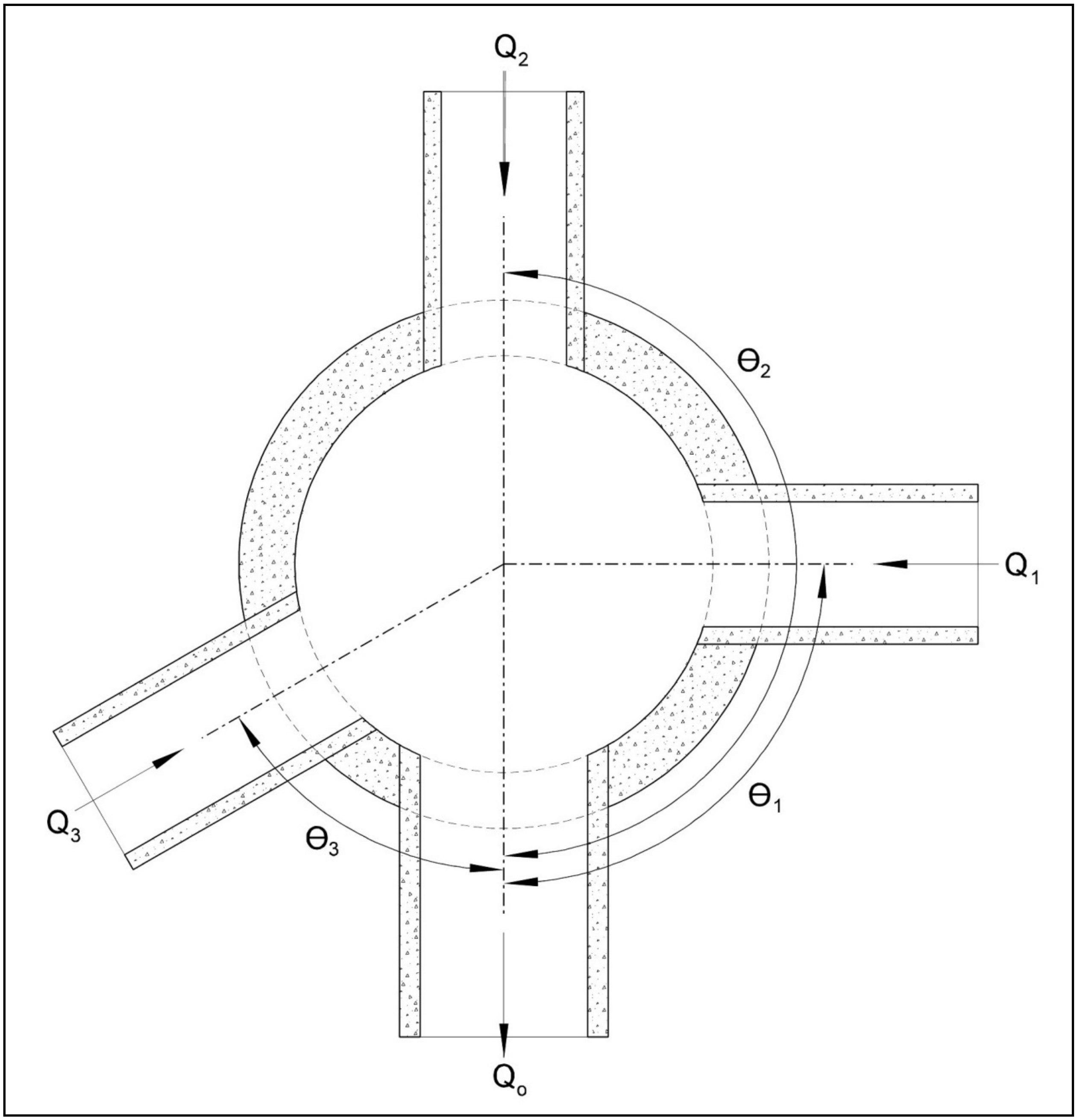


Figure 9.8. Access hole angled inflow definition.

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

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

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

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

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

The additional plunging inflow energy loss is given by:

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

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

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

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

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

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

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

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

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

9.1.6.7.3 Calculating Inflow Pipe Exit Losses

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

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

where:

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

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

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

where:

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

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

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

9.2 Design Considerations

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

9.2.1 Design Storm Frequency

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

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

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

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

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

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

9.2.2 Time of Concentration and Discharge

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

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

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

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

Design Conditions

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

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

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

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

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

where:

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

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

9.2.3 Minimum Velocity and Grades

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

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

where:

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

Conduit Slopes and Velocity

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

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

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

9.2.4 Cover, Location, and Alignment

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

Depth of Cover Under the Traveled Way

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

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

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

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

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

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

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

9.2.5 Maintenance

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

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

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

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

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

9.3 Preliminary Design

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

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

The designer compiles the following initial design information:

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

Step 2. Determine the hydrologic parameters to each inlet.

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

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

- Inlet time of concentration.

Step 3. Size and locate the pipes.

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

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

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

9.4 Hydraulic and Energy Grade Line Evaluation

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

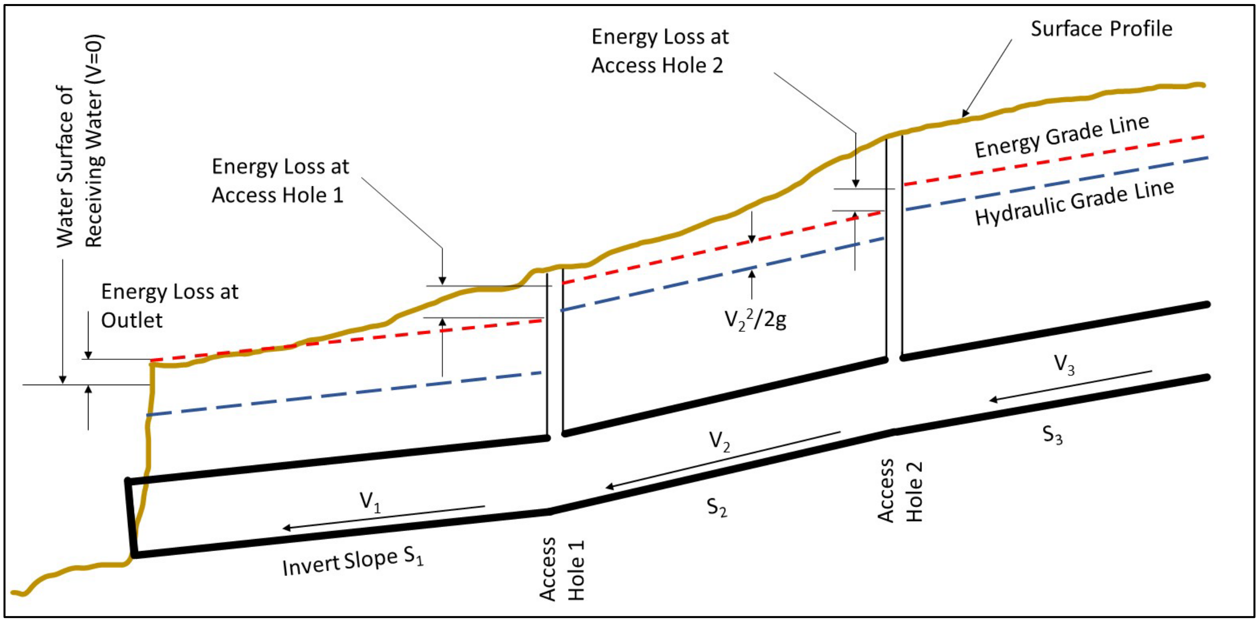


Figure 9.9. Energy and hydraulic grade line illustration.

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

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

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

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

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

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

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

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

Table 9.6. Case conditions and downstream EGL estimates.

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

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

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

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

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

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

Step 7. Calculate EGL and HGL in a structure.

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

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

Iterative Solvers in Modern Spreadsheets

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

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

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

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

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

Step 9. Examine the results.

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

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

Example 9.2: Storm drain design.

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

Given: Table 9.8 summarizes the rainfall data.

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

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

Minimum cover is 3 ft (0.91 m).

Table 9.8. Intensity-duration data for example.

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

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

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

Step 2. Determine the hydrologic parameters for each inlet.

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

Table 9.9. Drainage area information for example.

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

Step 3. Size and locate the pipes.

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

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

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

A = 0.64 ac

C = 0.73

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

From Table 9.8:

I = 7.1 in/h

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

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

Estimate pipe diameter assuming full flow using equation 9.2:

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

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

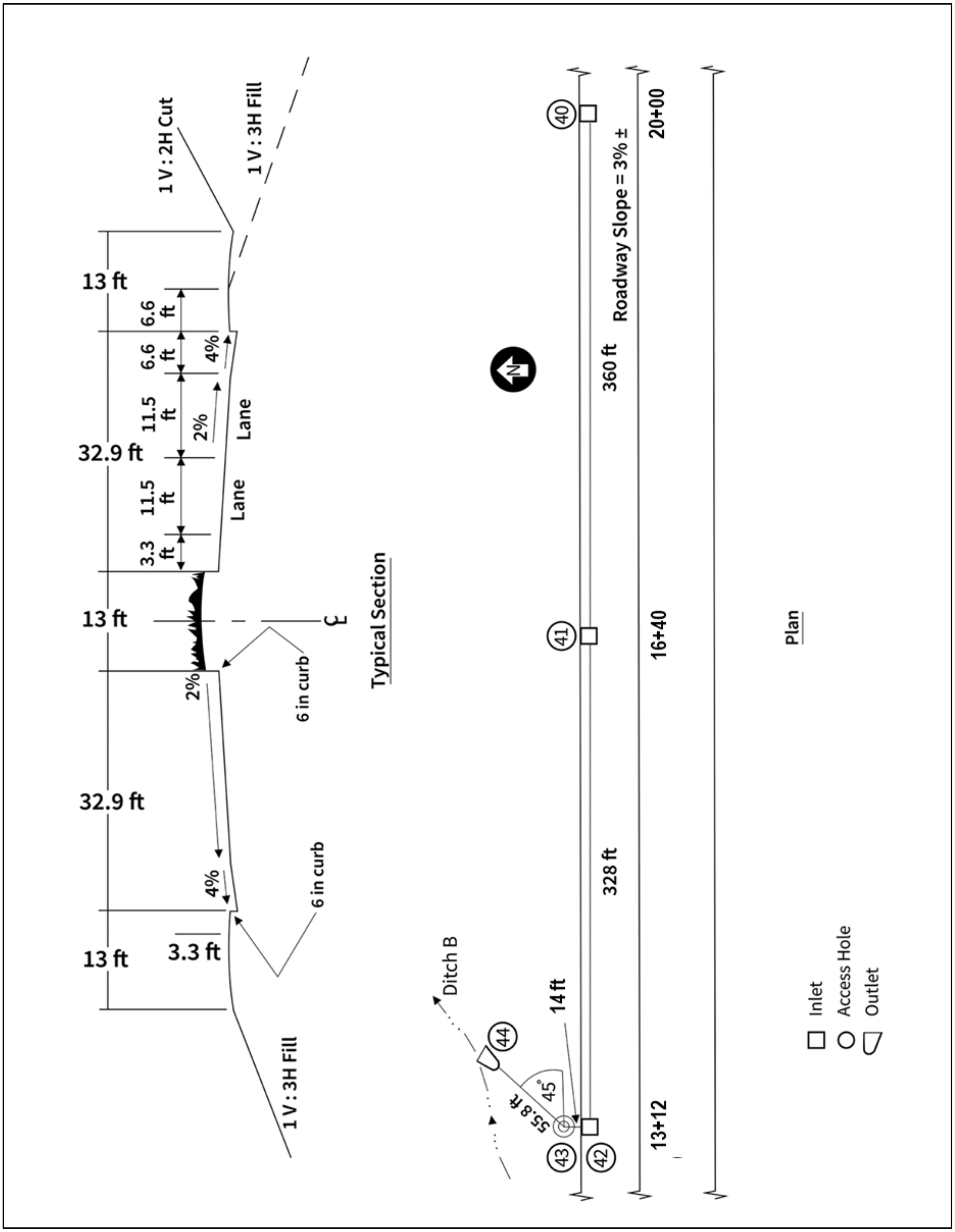


Figure 9.10. Roadway plan and section for example.

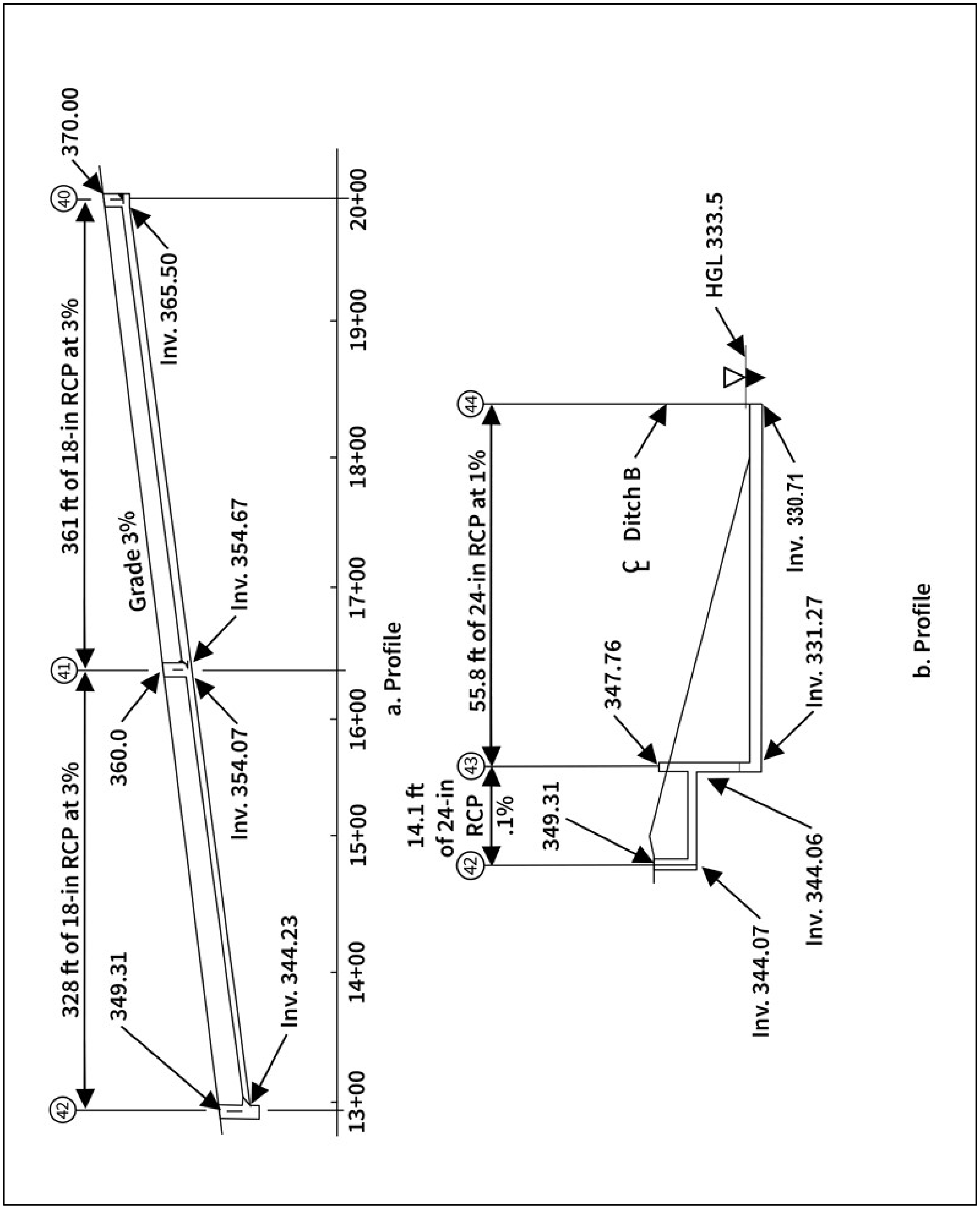


Figure 9.11. Storm drain profiles for example.

Calculate full pipe capacity using equation 9.2:

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

Calculate full pipe velocity using equation 9.1:

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

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

> 0.75 V_f if Q is less than or equal to 0.5 Q_f
>
> 1.1V_f if Q is greater than 0.5 Q_f

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

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

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

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

Calculate the upstream invert elevation:

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

Calculate the downstream invert elevation:

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

Validate that pipe has adequate cover at the downstream end:

From Figure 9.11, the ground elevation is 360.0 ft.

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

Pipe Wall Thickness and Cover

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

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

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

A = 0.64 + 0.35 = 0.99 ac

C = 0.73

Inlet time of concentration = 2 min

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

From Table 9.8:

I = 7.1 in/h

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

$D = D_{min} = 1.5$ ft

$V = 8.7$ ft/s

Travel time = 0.6 min, use 1 min

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

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

Calculate the upstream invert:

$Z_{u/s} = 354.67 – 0.6 = 354.07$ ft

Calculate the downstream invert:

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

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

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

$A = 0.64 + 0.35 + 0.32 = 1.31$ ac

$C = 0.73$

Inlet time of concentration = 2 min

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

From Table 9.8:

$I = 7.1$ in/h

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

$D = 1.96$ ft = use 2.0 ft

$V = 2.6$ ft/s

Travel time = 0.09 min, use 0 min

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

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

Calculate the upstream invert:

$Z_{u/s} = 344.23 – 0.16 = 344.07$ ft

Calculate the downstream invert:

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

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

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

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

$V = 6.1$ ft/s

Travel time = 0.15 min, use 0 min

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

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

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

Calculate the upstream invert:

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

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

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

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

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

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

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

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

BOC_o = 330.71 ft

TOC_o = 330.71 + 2.0 ft = 332.71 ft

Determine the applicable case from Table 9.6:

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

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

Estimate the energy loss exiting the outfall pipe:

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

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

Exit loss for an outfall:

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

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

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

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

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

Pipe length, L = 55.8 ft

BOC_i = 331.27 ft

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

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

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

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

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

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

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

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

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

Step 7. Calculate EGL and HGL at structure 43.

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

Compute E_{aio} using equations 9.14 and 9.15:

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

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

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

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

Compute E_{ais} using equation 9.17:

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

Compute E_{aiu} using equation 9.18:

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

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

Compute E_a:

For benching:

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

For angled inflow:

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

For plunging flow:

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

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

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

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

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

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

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

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

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

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

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

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

D = 2.0 ft

$Q = 6.75\ ft^3/s$

$L = 14.1\ ft$

$BOC_o = 344.06\ ft$

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

Determine the applicable case from Table 9.6:

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

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

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

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

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

$Q/Q_f = 6.75 / 7.12 = 0.95$

$y_n = 1.56\ ft$

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

$y_c = 0.80\ ft$

$H_o = 0.0$

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

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

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

$BOC_i = 344.07\ ft$

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

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

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

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

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

Total pipe loss = 0.01 ft

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

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

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

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

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

$y_n = 1.56\ ft$

$y_c = 0.8\ ft$

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

Step 7. Calculate EGL and HGL at structure 42.

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

Compute E_{aio} using equations 9.14 and 9.15:

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

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

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

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

Compute E_{ais} using equation 9.17:

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

Compute E_{aiu} using equation 9.18:

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

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

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

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

Plunging flow:

z_k = 349.31 ft - 344.07 ft = 5.24

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

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

Check to ensure net energy:

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

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

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

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

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

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

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

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

D = 1.5 ft

Q = 5.1 ft^3/s

L = 328 ft

BOC_o = 344.23 ft

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

Determine the applicable case from Table 9.6:

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

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

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

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

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

$EGL_o = EGL_a + H_o = 345.81$ ft + 0.05 ft = 345.86 ft

$HGL_o = EGL_o – V^2/2g = 345.86$ ft – 0.13 ft = 345.73 ft

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

$BOC_i = 354.07$ ft

$TOC_i = BOC_i + D = 354.07$ ft + 1.5 ft = 355.57 ft

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

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

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

Total pipe loss = 0.78 ft

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

$HGL_i = EGL_i - V^2/2g = 346.64$ ft - 0.13 ft = 346.51 ft

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

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

Estimate y_n and y_c:

$Q/Q_f = 5.1 / 18.1 = 0.28$

$V/Vf = 0.86$

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

$y_n / D = 0.37$

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

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

$y_c = 0.87$ ft

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

$HGL_i = y_n + BOC_i = 0.56$ ft + 354.07 ft = 354.63 ft

$EGL_i = HGL_i + V^2/2g = 354.63$ ft + 1.22 ft = 355.85 ft

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

Step 7. Calculate EGL and HGL at structure 41.

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

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

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

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

Compute E_{ais} using equation 9.17:

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

Compute E_{aiu} using equation 9.18:

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

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

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

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

$z_k = 360.00$ ft - 354.07 ft $= 5.93$

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

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

Check to ensure net energy:

Is $E_{ai} > E_i$?

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

$E_a = E_i = 1.78$ ft

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

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

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

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

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

D = 1.5 ft

Q = 3.3 ft^3/s

L = 361.0 ft

$BOC_o = 354.67$ ft

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

Determine the applicable case from Table 9.6:

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

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

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

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

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

$Q/Q_f = 3.3 / 18.1 = 0.18$

$V/V_f = 0.74$

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

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

$y_n/d_f = 0.30$

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

$y_c = 0.67$ ft

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

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

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

$A_f / A = 0.84$

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

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

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

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

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

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

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

BOC_i = 365.50 ft

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

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

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

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

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

Step 7. Calculate EGL and HGL at structure 40.

Structure 40 is a terminal inlet with no incoming pipe.

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

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

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

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

Compute E_{ais} using equation 9.17:

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

Compute E_{aiu} using equation 9.18:

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

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

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

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

Plunging flow from the inlet:

z_k = 370.0 ft - 365.50 ft = 4.50 ft

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

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

Check to ensure net energy:

Is $E_{ai} > E_i$?

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

$E_a = E_i$ = 1.35 ft

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

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

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

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

Step 9. Examine the results.

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

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

Chapter 10 - Detention and Retention

Land development, including road construction, can significantly change runoff characteristics. These activities convert natural pervious areas to impervious areas and alter drainage pathways. Increased imperviousness causes an increased volume of runoff because of reduced infiltration and typically reduce runoff times causing an increase in runoff peaks. In addition, land development generally decreases the natural storage of a watershed by removing trees and vegetation, which reduces the volume of interception storage, and by site grading, which reduces the volume of depression storage.

In urban areas, lined channels, storm drains, and curb and gutter systems often replace natural drainage systems. These constructed systems produce an increase in runoff volume and peak flow, as well as a reduction in the time to peak of the runoff hydrograph. Figure 10.1 illustrates this effect, comparing a predevelopment and post-development runoff hydrograph. As the figure shows, the post-development runoff hydrograph displays a higher and earlier peak runoff. Surface runoff velocities also increase, which can increase surface rill and gully erosion rates. Higher stream velocities may also increase rates of bed-load movement. Designers use detention, retention, or both to mitigate the hydrologic effects of land development and urbanization.

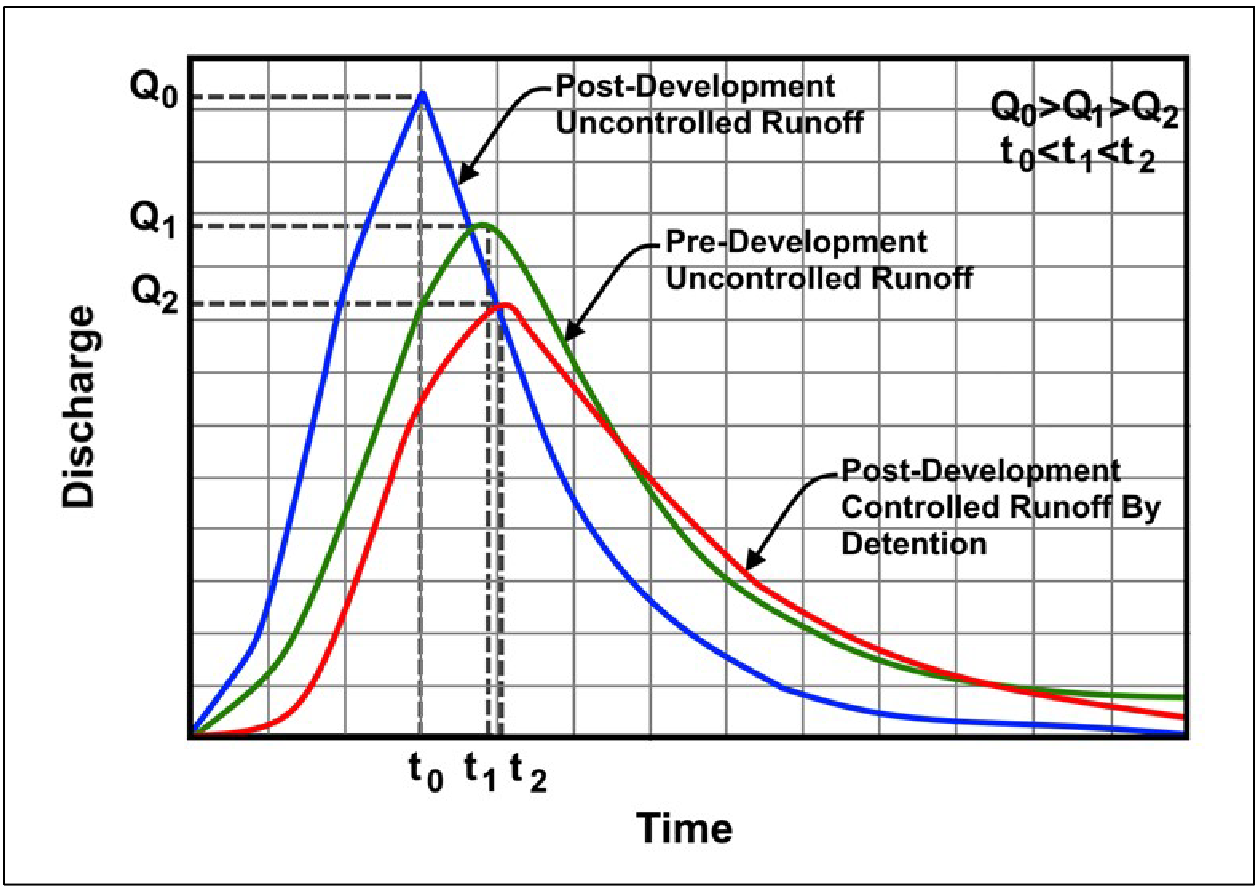


Figure 10.1. Controlled and uncontrolled hydrographs.

10.1 Design Objectives and Challenges

Designers follow applicable design criteria for detention and retention facilities. They also address other important features of these facilities including release timing, safety, and maintenance.

Stormwater Management Ponds

Stormwater quantity and quality control facilities are either detention or retention ponds.

Detention ponds, also known as dry ponds or retarding basins, primarily function to temporarily store a portion of the stormwater runoff volume while providing some water quality benefits.

Retention ponds, also known as wet ponds, have a permanent pool and treat both stormwater runoff quality and quantity.

10.1.1 Design Events

To mitigate the detrimental stormwater runoff effects of land development, designers have several tools that can limit peak flow rates from developed areas to those that occurred prior to development. Designers can use these tools to mitigate one, two, or more design event events, for example the 0.5, 0.1, and 0.01 AEP events. The "post-development controlled runoff by detention" hydrograph in Figure 10.1 illustrates the potential effect of storage in mitigating increased runoff caused by land development.

The storage of stormwater can reduce the frequency and extent of downstream flooding, soil erosion, sedimentation, and water pollution. Designers have also used detention and retention facilities to reduce the costs of large storm drainage systems by reducing the required size for downstream storm drain conveyance systems.

Local government bodies typically establish specific design criteria for peak flow attenuation. Some jurisdictions also require that flow volume be controlled to pre-development levels. Controlling flow volume is only practical when site conditions permit infiltration. To compensate for the increase in flow volume, some jurisdictions require that the peak post-development flow be reduced to below pre-development levels.

Some detention/retention facilities are designed for control of runoff from only a single storm frequency. However, single storm criteria have been found to be rather ineffective since such a design may provide little control of other storms. For example, design for the control of frequent storms (less frequent AEPs) provides little attenuation of lower frequency higher magnitude storm events. Similarly, design for less frequent large storms provides little attenuation for the more frequent smaller storms. Some jurisdictions enforce multiple storm regulatory criteria which dictate that multiple storm frequencies be attenuated in a single design. A common criterion would be to regulate the 0.5, 0.1, and 0.01 AEP events.

10.1.2 Release Timing

The timing of releases from stormwater control facilities can be critical to the proper functioning of overall stormwater systems. As illustrated in Figure 10.1, stormwater quantity control structures reduce the peak flow and increase the duration of the hydrograph at the point of release. However, in some instances this shifting of hydrograph peak times and durations can cause adverse effects downstream.

For example, where the drainage area being controlled is in a downstream portion of a larger watershed, delaying the peak and extending the recession limb of the hydrograph may result in a higher peak on the main channel as illustrated in Figure 10.2. In this example, this can occur if the reduced peak on the controlled tributary watershed is delayed so that it reaches the mainstem at or near the time of the mainstem peak. With multiple detention facilities in a watershed, designers will consider the effects of detention on hydrograph timing downstream to avoid creating flooding problems.

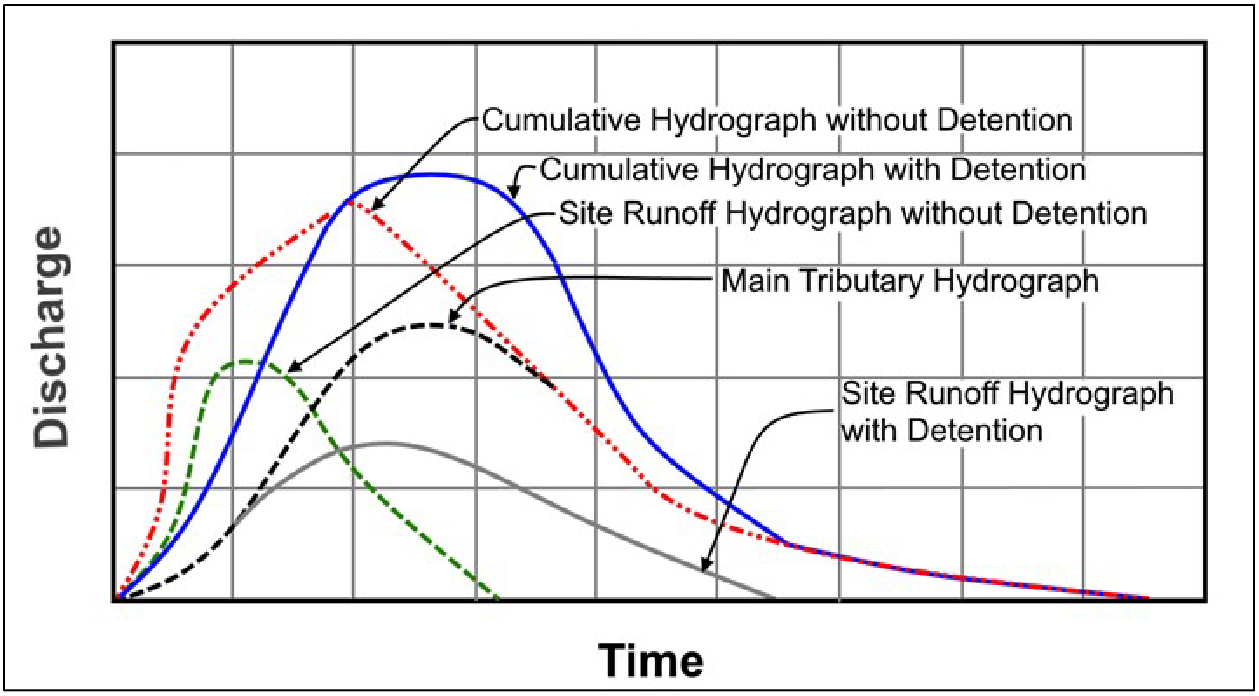


Figure 10.2. Example of cumulative hydrograph with and without detention.

10.1.3 Safety

Detention designers promote safe facilities by preventing public trespass, providing emergency escape aids, and eliminating other hazards. Preventing public trespass recognizes that children and adults may be attracted to the site, regardless of whether the site is intended for their use. For example, the designer may use fences to enclose ponds to limit access. Ideally, designers locate detention basins away from busy streets and intersections.

Designers consider inflow and outflow structures with safety in mind. For example, removable, hydraulically efficient grates and bars may cover the ends of inlet and outlet pipes, particularly if they connect with an underground storm drain system or otherwise present a safety hazard. Safer design of outflow structures would also limit flow velocities at points where people could be drawn into the discharge stream.

Where a detention basin incorporates active recreation areas, designers use mild bottom slopes along the periphery of the basin. Persons who enter a detention pond or basin while stormwater is being discharged may be at risk. The force of the currents may push a person into an outflow structure or may hold a victim under the water where a bottom discharge is used. *Urban Stormwater Management* (APWA 1981) discusses several design precautions intended to improve safety.

10.1.4 Maintenance

Stormwater management facilities depend on proper maintenance to function as intended over time. Depending on the facility type and location, appropriate periodic maintenance may include:

- Scheduled **inspections** may occur for the first few months after construction and on an annual basis thereafter. In addition, inspections during and after major storm events ensure that the inlet and outlet structures continue functioning by either identifying damage or clogging or confirming that none has occurred. They also can identify erosion on embankment side slopes and evidence of soil piping that can degrade the facility.

- **Mowing** at least twice a year discourages woody growth and controls weeds.
- **Sediment, debris, and litter control** at least twice a year maintains functionality and reduces clogging potential. In particular, removing accumulated sediment, debris, and trash around outlet structures prevents clogging of the control device.
- Standing water or soggy conditions within the lower stage of a storage facility can create nuisance conditions such as odors, insects, and weeds. **Nuisance control,** such as providing allowance for positive drainage during design, minimizes these problems.
- Inlet and outlet devices, and standpipe or riser structures deteriorate with time, necessitating **structural repairs and replacement**. The actual life of a structural component will depend on individual site-specific factors, such as soil conditions.

10.2 Storage Facility Types

Designers classify stormwater quantity control facilities as either detention or retention facilities. Detention facilities (dry ponds) store and release or attenuate stormwater runoff by a control structure or other release mechanism. Extended detention usually releases stormwater over a period of days. Retention facilities (wet ponds) maintain a permanent pool and release stormwater via evaporation and infiltration as well as through surface releases using a control structure.

10.2.1 Detention Facilities

Highway and municipal stormwater management plans (SWMPs) most often use the detention concept to limit the peak outflow rate to that which existed from the same watershed before development for a specific range of flood frequencies. Following SWMPs, designers may provide detention storage at one or more locations, either above ground or below ground or both. These locations may exist as impoundments; collection and conveyance facilities; underground tanks; and on-site facilities such as parking lots, pavements, and basins. Detention ponds are the most common type of storage facility used for controlling stormwater runoff peak flows. The majority of these are dry ponds which release all the runoff temporarily detained during a storm.

Designers recommend detention facilities where the facilities will have clear hydrologic, hydraulic, and cost benefits. Additionally, local ordinances may require some detention facilities; in such cases, the governing agency determines appropriate construction which may typically include:

- Design rainfall frequency, intensity, and duration consistent with highway standards and local requirements.
- The outlet structure limits the maximum outflow to allowable release rates. The maximum release rate may be a function of existing or developed runoff rates, downstream channel capacity, potential flooding conditions, and local ordinances.
- Detention facility size, shape, and depth provides sufficient volume to satisfy project storage requirements. Designers determine this by routing the inflow hydrograph through the facility. Section 10.3.1 outlines techniques designers can use to estimate an initial storage volume. Section 10.4 provides a discussion of storage routing techniques.
- An auxiliary outlet that allows overflow potentially resulting from excessive inflow or clogging of the main outlet. This outlet is positioned such that overflows will follow a predetermined route. Preferably, such outflows discharge into open channels, swales, or other approved storage or conveyance features.

- The system releases excess stormwater expeditiously to ensure that the entire storage volume is available for subsequent storms and to minimize hazards. A dry pond, which is a facility with no permanent pool, may need a paved low flow channel to ensure complete removal of water and to aid in nuisance control.
- The facility typically satisfies Federal (see Chapter 2) and State statutes and recognizes local ordinances.
- Provides access for maintenance.
- Avoids being an "attractive nuisance," which may involve fencing and signage.

Figure 10.3 shows a schematic of the cross-section of a detention basin with a single-stage riser. A pool forms behind the retaining structure. The hydrograph of the post-development flood runoff enters the pool at the upper end of the detention basin. Water can be discharged from the pool through a pipe that passes through or around the detention structure. The size of the pipe can serve to limit the outflow rate, thus forming a permanent pool, with the permanent pool elevation changing only through evaporation and infiltration losses.

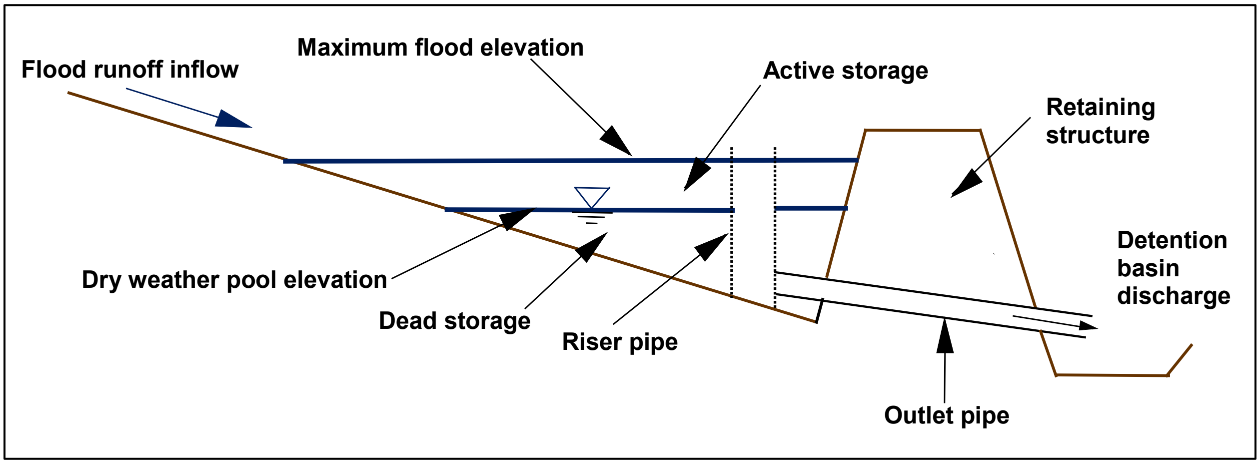


Figure 10.3. Schematic cross-section of a detention basin with a single-stage riser.

Using a permanent pool has several advantages, including water quality control, aesthetic considerations, and wildlife habitat improvement. A permanent pool also increases the total storage volume, involving both a larger retaining structure and a larger commitment of land, both of which increase project cost.

Figure 10.3 does not show several other elements of detention basin design. The designer may determine that the riser inlet should be fitted with both an antivortex device and a trash rack. The antivortex device prevents the formation of a vortex, thus maintaining the hydraulic efficiency of the outlet structure. The trash rack prevents high velocity flows from sucking trash (and people) into the riser.

All detention basins should have an emergency spillway to pass runoff from very large flood events, so the retaining structure is not overtopped and washed out. The elevation of the bottom of the emergency spillway, which will pass high flows around the retaining structure, is above the elevation of the riser outlet, but below the top of the retaining structure. Designers call the zone between the detention or surcharge storage zone.

Various methods exist for the planning and design of stormwater detention facilities. Design involves the simultaneous sizing of the storage volume characteristics and the riser/outlet characteristics. Some stormwater management (SWM) methods ("planning methods") only apply to estimating the volume of storage that would meet the intent of the SWM policies. Designers use other planning methods to determine the characteristics of the outlet facility. Ultimately, the designer selects the final design using a method that simultaneously estimates the volume of storage and the characteristics of the outlet facility.

The simultaneous solution is important because there is a wide array of feasible solutions for any one site and set of design conditions. When designers separately determine storage volume and outlet facility characteristics, they may produce an ineffective, and possibly incorrect, design. In summary, planning methods provide less accuracy and involve less effort than design methods.

10.2.2 Retention Facilities

In addition to stormwater storage, retention facilities (e.g., wet ponds) may be used for water supply, recreation, pollutant removal, aesthetics, and groundwater recharge. As discussed in Chapter 11, infiltration facilities provide significant water quality benefits, and although groundwater recharge is not a primary goal of highway stormwater management, the use of infiltration basins and swales can provide this secondary benefit in tandem with retention facilities.

Designers typically develop retention facilities to provide the dual functions of stormwater quantity and quality control. These facilities may be provided on the surface or buried. The facility may have a permanent pool, which typically provides pollutant control.

Design criteria for retention facilities are the same as those for detention facilities except they may not include provisions for treating the runoff of lower frequency high magnitude storms. However, designers may apply the following additional criteria:

- Provide sufficient depth and volume below the normal pool level for any desired multiple use activity including pollutant removal.
- Include shoreline protection where erosion from wave action is expected.
- Provide that capability to lower the pool elevation or drain the basin for cleaning, shoreline maintenance, and emergency operations.
- Design dikes or dams with a safety factor commensurate with applicable regulations.
- Consider safety benching below the permanent water line at the toe of steep slopes to guard against accidental drowning.
- Size the emergency spillway and storage volume when closely spaced flood events could prevent complete draining between events.
- Complete detailed engineering geological studies to ensure that the facility will function as planned.

The FHWA (1979) provides additional information on underground detention and retention facilities.

10.3 Detention and Retention Design Information

This chapter introduces preliminary design activities for stormwater detention and retention including estimating the storage volume of the stormwater detention need, developing stage-storage relationships, and developing stage-discharge relationships.

10.3.1 Preliminary Storage Volume

Designers estimate a preliminary estimate of storage to accomplish the hydrograph attenuation goals. The following sections present several methods for determining an initial estimate of storage. Because each method provides preliminary estimates only, the designer may apply several of the methods and use engineering judgment to select an initial storage estimate.

10.3.1.1 Loss-of-Natural-Storage Method

The loss-of-natural-storage method estimates volumes based on the volume of lost natural storage. This approach is conservative for detention volume estimates, but is more appropriate for estimating retention volumes:

$$Q_s = Q_a - Q_b \tag{10.1}$$

where:

Q_s	=	Storage needed, inch (mm)
Q_a	=	Runoff depth for post-development watershed condition, inch (mm)
Q_b	=	Runoff depth for pre-development watershed condition, inch (mm)

Designers often refer to the variable Q as a volume even though it has the dimension of a depth. It represents the volume of water at the computed depth spread uniformly over the entire watershed. The designer computes the volume of storage, V_s, by multiplying Q_s by:

$$V_S = \alpha A Q_S \tag{10.2}$$

where:

α	=	Unit conversion constant, 3,630 in CU (10 in SI)
A	=	Drainage area, ac (ha)

The designer can compute the runoff depths Q_a and Q_b of equation 10.1 using any one of several methods. For the Natural Resources Conservation Service (NRCS) method, apply the NRCS runoff equation (SCS 1986) using the post-development and pre-development curve numbers (CNs). Using the Rational Method to estimate peak flows, the runoff depths, Q, is:

$$Q = \alpha \left[\frac{q_p t_c}{A} \right] \tag{10.3}$$

where:

q_p	=	peak flow, ft^3/s (m^3/s)
α	=	Unit conversion constant, 0.0165 in CU (6.0 in SI)
A	=	Drainage area, ac (ha)
t_c	=	Time of concentration, min

Solve equation 10.3 for both the pre- and post-development conditions by using the appropriate values of q_p and t_c. Then compute the runoff depth difference by entering the values into equation 10.1, ultimately using it to compute the volume of storage with equation 10.2.

Example 10.1:	Storage estimate for a detention pond.
Objective:	Estimate the storage needed for a detention pond using the loss-of-natural-storage method.

Given: An existing watershed under development with the following characteristics:

A	=	5.7 ac (2.3 ha)
C	=	0.2 and 0.45 (existing and developed conditions, respectively)

t_c	=	18 and 11 min (existing and developed conditions, respectively)
I	=	3.1 in/h (79 mm/h) and 4.0 in/h (102 mm/h), (existing and developed conditions, respectively), using local intensity-duration-frequency (IDF) curves

Step 1. Calculate the peak flows for the existing and developed conditions.

$q_{pb} = (1/\alpha)\, C_b\, i_b\, A = (0.2)(3.1)(5.7) = 3.5$ ft^3/s

$q_{pa} = (1/\alpha)\, C_a\, i_a\, A = (0.45)(4.0)(5.7) = 10.3$ ft^3/s

Step 2. Estimate the runoff depths.

Using equation 10.3:

$Q_b = \alpha[(q_{pb}\, t_c)/A] = (1/60.5)[3.5(18)/5.7] = 0.18$ inches

$Q_a = \alpha[(q_{pa}\, t_c)/A] = (1/60.5)[10.3(11)/5.7] = 0.33$ inches

Step 3. Compute the depth and volume of storage.

Using equation 10.1:

$Q_S = Q_a - Q_b = 0.33 - 0.18 = 0.15$ inches

Using equation 10.2:

$V_S = \alpha\, AQ_S = 3630\,(5.7)\,(0.15) = 3{,}100$ ft^3

Solution: The detention pond needs 3,100 ft^3 (88 m^3) of storage.

10.3.1.2 Actual Inflow/Estimated Release Method

The actual inflow/estimated release method of estimating detention storage volume uses an inflow hydrograph (post-development) and the target peak flow release for the design event. The method estimates the outflow hydrograph by sketching an assumed outflow curve as shown in Figure 10.4. This limits the peak of the estimated outflow hydrograph such that it does not exceed the desired peak outflow from the detention basin. The shaded area between the inflow and the approximated outflow hydrographs represents the estimated storage needed. To determine the necessary storage, measure or mathematically compute the shaded area using a reasonable time period and appropriate hydrograph ordinates.

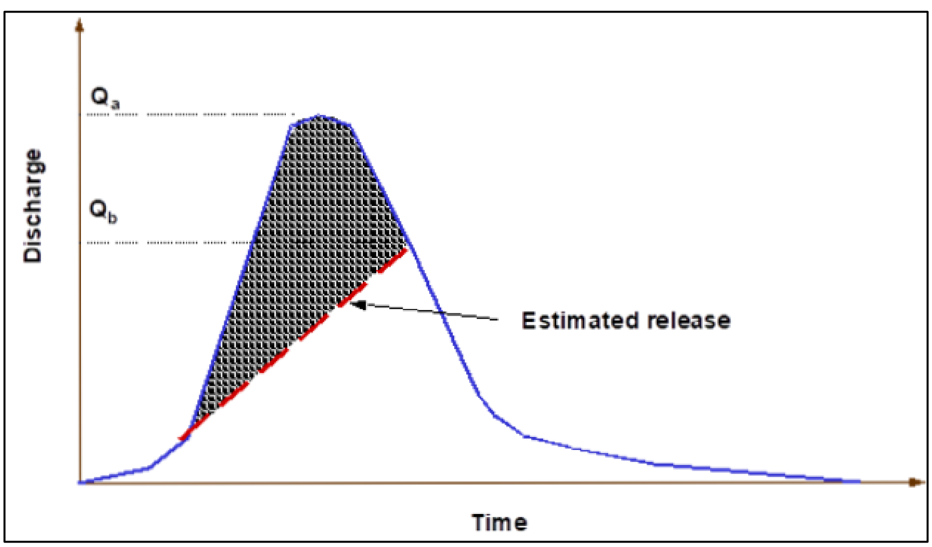


Figure 10.4. Estimating storage using the actual inflow/estimated release of hydrograph method.

10.3.1.3 Rational Method Triangular Hydrograph Method

Designers can obtain a preliminary estimate of the storage volume required for peak flow attenuation from a simplified design procedure that replaces the actual inflow and outflow hydrographs with simplified triangular shapes. This method works best with the Rational Method. Figure 10.5 illustrates the procedure. The area above the outflow hydrograph and inside the inflow hydrograph represents the estimated storage volume:

$$V_S = 0.5\, t_i\, (q_i - q_o) \tag{10.4}$$

where:

V_s	=	Storage volume estimate, ft^3 (m^3)
q_i	=	Peak inflow rate into the basin, ft^3/s (m^3/s)
q_o	=	Peak outflow rate out of the basin, ft^3/s (m^3/s)
t_i	=	Duration of basin inflow, s

The duration of basin inflow equals two times the time of concentration. The triangular hydrograph procedure, originally described by Boyd (1981), compares favorably with more complete design procedures involving reservoir routing.

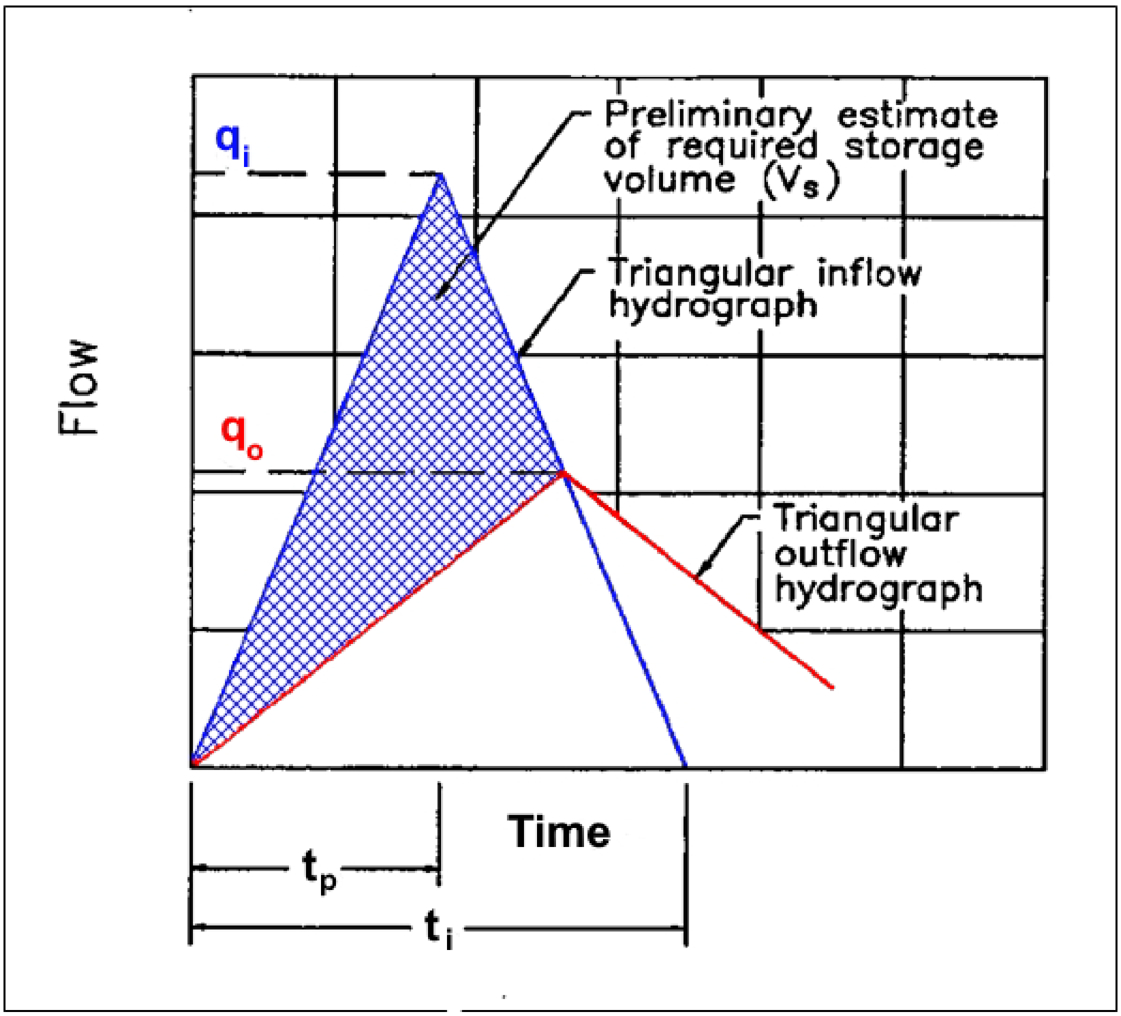


Figure 10.5. Triangular hydrograph method.

10.3.1.4 NRCS TR-55 Procedure

TR-55 (SCS 1986) describes a method for estimating required storage volumes based on peak inflow and outflow rates. The method uses average storage and routing effects observed for a large number of structures. This procedure for estimating storage volume may have errors and, therefore, is only useful for preliminary estimates.

The procedure to estimate required detention storage follows (SCS 1986):

Step 1. Compute discharges.

Determine the peak inflow and outflow discharges q_i and q_o, which correspond to the post- and pre-development flows, respectively.

Step 2. Compute q_o/q_i and R_q.

Calculate the ratio q_o/q_i, which is equal to R_q.

Step 3. Calculate the ratio V_s/V_r for graphical method or compute R_s for arithmetic method.

Calculate R_s:

$$R_S = \frac{Q_S}{Q_a} = C_0 + C_1R_q + C_2R_q^2 + C_3R_q^3 \tag{10.5}$$

The coefficients C_O, C_1, C_2, and C_3 are a function of the NRCS rainfall distribution and are provided in Table 10.1.

Table 10.1. Coefficients for the NRCS detention volume method.

Rainfall Distribution	C_0	C_1	C_2	C_3
I or IA	0.660	-1.76	1.96	-0.730
II or III	0.682	-1.43	1.64	-0.804

Step 4. Determine the storage volume, V_s.

$$V_S = \alpha R_S Q_a A \tag{10.6}$$

where:

α	=	Unit conversion constant, 3,630 in CU (10 in SI)
Q_a	=	Post-development depth of runoff, inches (mm)
A	=	Drainage area, ac (ha)

Example 10.2: Estimation of needed detention pond storage.

Objective: Estimate the needed storage of a detention facility by using the actual inflow hydrograph, Rational Method triangular hydrograph, and NRCS TR-55 methods.

Given: The post-developed (improved conditions) peak flow of 31.1 ft^3/s (0.88 m^3/s) is limited to an outflow rate from the proposed detention facility of 19.4 ft^3/s (0.55 m^3/s). This limiting outflow is a constraint imposed by the downstream receiving water course and is the maximum outflow rate from the drainage area for unimproved conditions. Drainage area, A, equals 43.4 acres.

Step 1. Estimate storage using the actual inflow/estimated release method.

Figure 10.6 illustrates the existing conditions and proposed conditions hydrographs. Assuming the proposed detention facility will produce an outflow hydrograph similar to existing conditions, determine the required detention volume as the area above the existing hydrograph and below the proposed hydrograph. Using the scale on Figure 10.6, 1 inch^2 equals 22,500 ft^3. Measuring the area between the two hydrographs yields an area of 1.5 inches^2, which converts to the following volume:

V_s = (1.5 inches^2) (22,500 ft^3/inch^2) = 33,750 ft^3

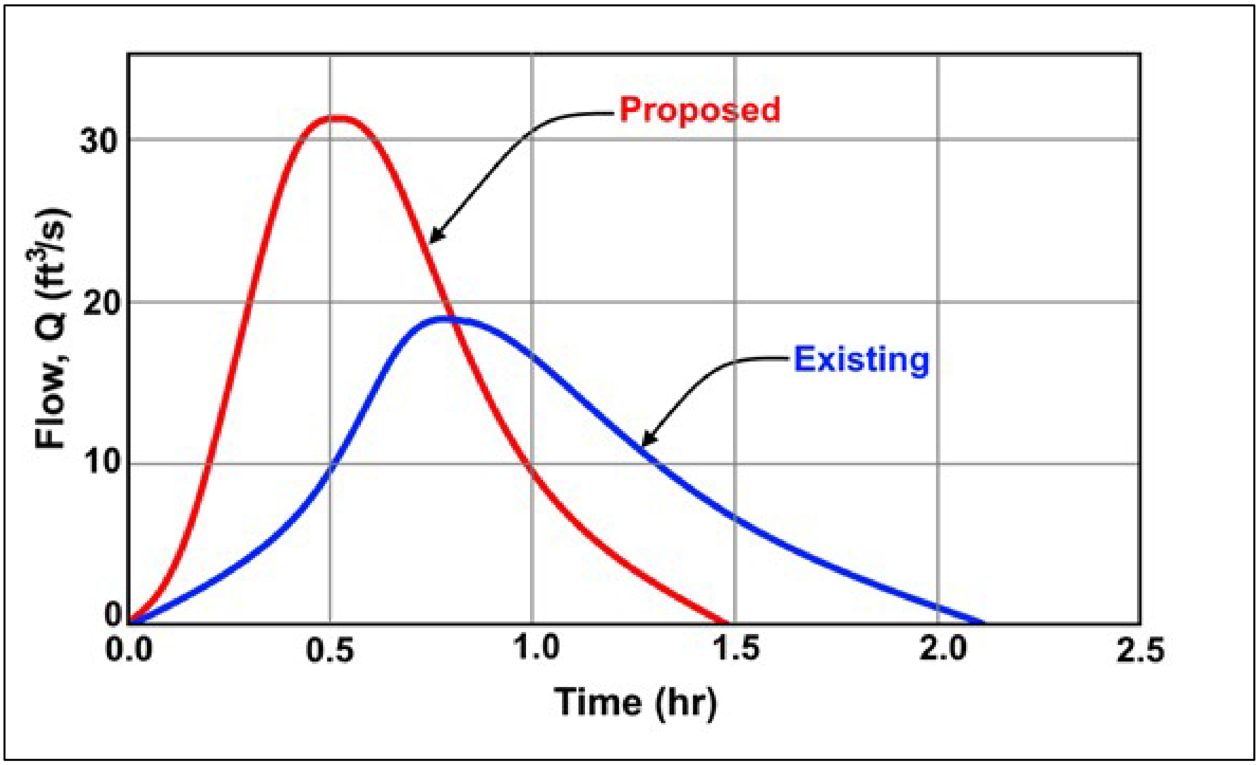


Figure 10.6. Hydrograph for existing and proposed conditions.

Step 2. Calculate storage using the Rational Method Triangular Hydrograph method.

The inflow rate into the detention basin (q_i) is 31.1 ft^3/s. The peak flow rate out of the basin (q_o) is set to be = 19.4 ft^3/s.

Using equation 10.4, compute the initial storage volume as:

$V_s = 0.5\ t_i\ (q_i - q_o)$ = (0.5)(1.43 h x (60 min/h)(60 s/min))(31.1 - 19.4) = 30,116 ft^3

Step 3. Determine storage using the NRCS TR-55 Method. Calculate R_q.

As previously noted, the inflow discharge is 31.1 ft^3/s, and the outflow discharge is set to be 19.4 ft^3/s by local ordinance.

The ratio of basin inflow to basin outflow is:

$q_o / q_i = R_q$ = 19.4 / 31.1 = 0.62

Step 4. Determine V_s / V_r.

With q_o / q_i = 0.62 and a Type II Storm, calculate R_s using equation 10.5 and the coefficients from Table 10.1. Q_a = 0.43 inches.

R_S = $Q_s / Q_a = C_0 + C_1 R_q + C_2 R_q^2 + C_3 R_q^3$

= 0.682 + (-1.43)(0.62) + (1.64)(0.62)^2 + (-0.804) (0.62)^3 = 0.23

Step 5. Calculate the preliminary estimated storage volume (V_S) using the NRCS TR-55 Method and equation 10.6.

$V_s = \alpha R_s Q_a A$ = (3630) (0.23) (0.43) (43.4) = 15,600 ft^3

Solution: The hydrograph and triangular hydrograph methods result in the most consistent and conservative estimates, whereas the NRCS TR-55 method is less conservative.

10.3.2 Stage-Storage Relationship

A stage-storage relationship links the depth of water and storage volume in the storage facility and depends on the topography at the site of the storage structure. Figure 10.7 illustrates a typical stage-storage curve. Calculate the volume of storage by using simple geometric formulas expressed as a function of storage depth. After estimating the required storage as described in the previous section, determine the configuration of the storage basin to develop the stage-storage curve. The following relationships can be used for computing the volumes at specific depths of geometric shapes commonly used for storage facilities.

10.3.2.1 Rectangular Basins

Figure 10.8 provides a schematic of a rectangular basin, a common shape for underground storage. Compute the volume of a rectangular basin by dividing the volume into rectangular and triangular shapes. If the basin is not on a slope, then the geometry will consist only of rectangular shapes.

Volumes of rectangular and triangular shapes, respectively, are:

$$V = LWD \tag{10.7}$$

$$V = 0.5W\left(\frac{D^2}{S}\right) \tag{10.8}$$

where:

V	=	Volume at a specific depth, ft^3 (m^3)
D	=	Depth of ponding for that shape, ft (m)
W	=	Width of basin at base, ft (m)
L	=	Length of basin at base, ft (m)
S	=	Slope of basin, ft/ft (m/m)

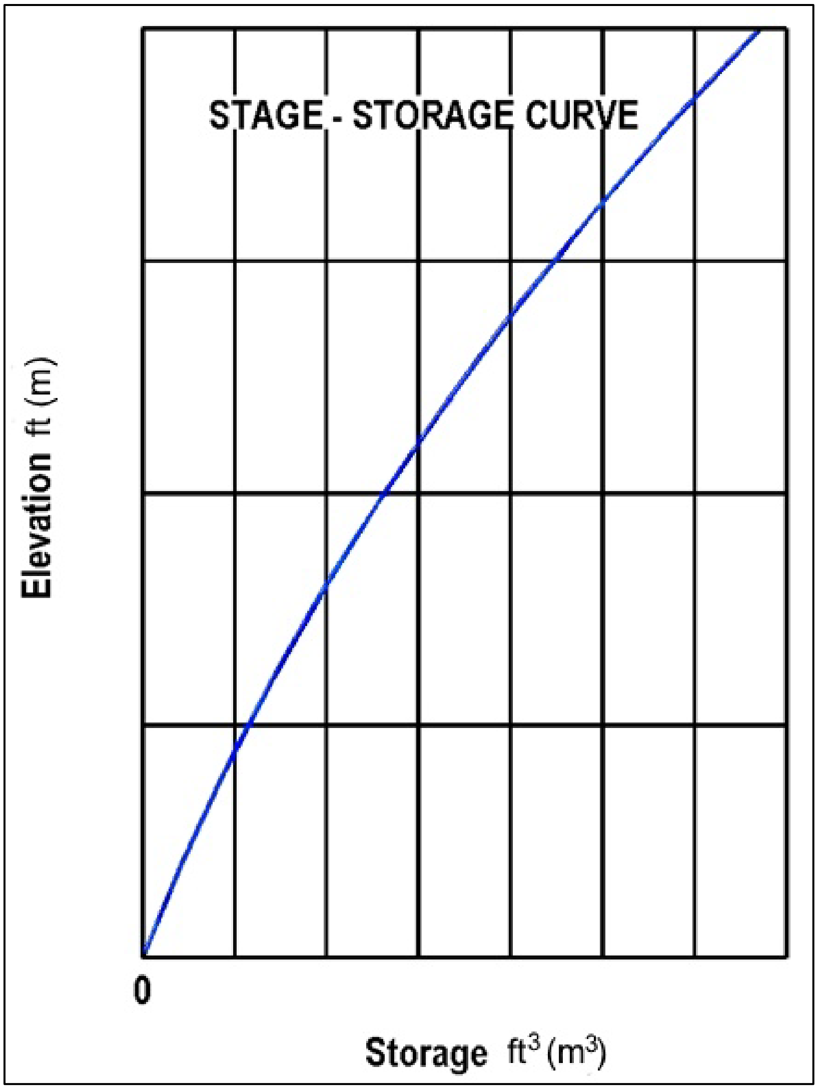


Figure 10.7. Stage-storage curve.

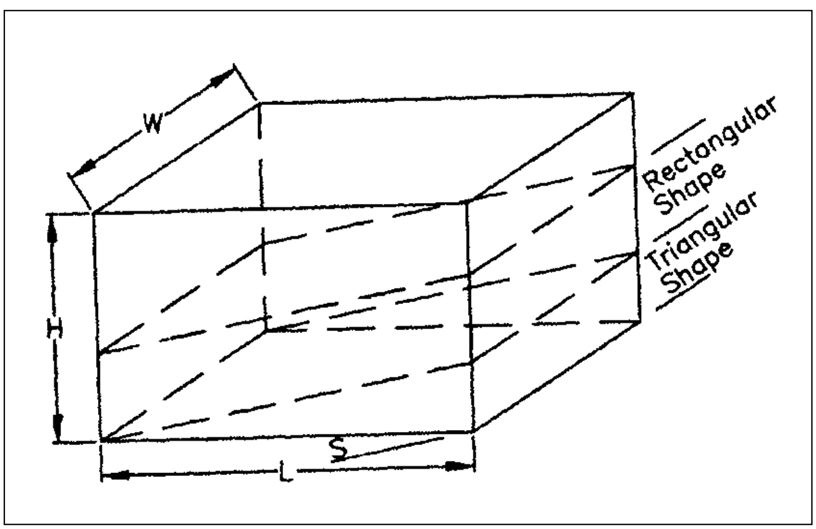


Figure 10.8. Rectangular basin.

A special case, rarely permitted by site, occurs with a horizontal rectangular bottom area and vertical sides. In this case, the storage simply equals the bottom area (i.e., length times width) multiplied by the depth of storage. If the relationship is plotted in a Cartesian axis system with storage as the ordinate and stage as the abscissa, the stage-storage relationship will be a straight line with a slope equal to the surface area of the storage facility.

A variation on the rectangular basin occurs where the bottom of the storage facility is a rectangle (L x W), the longitudinal cross- section is a trapezoid, and the ends are vertical. In this case, the stage-storage relationship is given by:

$$V = \frac{L}{\tan\theta} h^2 + (L\,W)h \tag{10.9}$$

where:

θ	=	Angle of side slopes
h	=	Height above bottom of basin, ft (m)

Graphing equation 10.9 results in a stage-storage relationship with the shape of a second-order polynomial with a zero intercept and a shape that depends on the values of L, W, and θ.

Example 10.3: Storage estimate of rectangular basin.

Objective: Estimate the maximum volume of a rectangular basin.

Given: Consider a rectangular basin:

L	=	656 ft (200 m)
W	=	328 ft (100 m)
Z	=	2

Step 1. Apply equation 10.7, which becomes:

$$S = 2L\,h^2 + (L\,W)h$$

Step 2. Calculate storage using equation 10.9.

Calculate the storage at a depth of 4.9 ft (1.5 m) as follows:

$S = 1{,}312h^2 + 215{,}200h = 1{,}312\,(4.9)^2 + 215{,}200\,(4.9) = 1{,}086{,}000$ ft^3

Solution: The maximum volume of the trapezoidal basin is 1,086,000 ft^3 (30,800 m^3).

10.3.2.2 Trapezoidal Basins

Figure 10.9 schematically represents a trapezoidal basin. Calculate the volume of a trapezoidal basin by dividing the volume into components of triangular and rectangular shape and applying equation 10.10. “Z” in this equation equals the ratio of the horizontal to vertical components of the side slope. For example, if the side slope is 1 to 2 (V:H), “Z” equals 2.

$$V = LWD + (L + W)\;ZD^2 + \left(\frac{4}{3}\right)Z^2 D^3 \tag{10.10}$$

where:

V	=	Volume at a specific depth, ft^3 (m^3)
D	=	Depth of ponding or basin, ft (m)

L	=	Length of basin at base, ft (m)
W	=	Width of basin at base, ft (m)
r	=	Ratio of width to length of basin at the base
Z	=	Side slope factor; ratio of horizontal to vertical components of side slope

Estimate the trial dimensions of a basin by rearranging the equation for volume in terms of basin length:

$$L = \frac{\left\{-ZD(r+1) + \left[(ZD)^2 (r+1)^2 - 5.33(ZD)^2 r + \left(\frac{4rV}{D}\right)\right]^{0.5}\right\}}{2r} \quad (10.11)$$

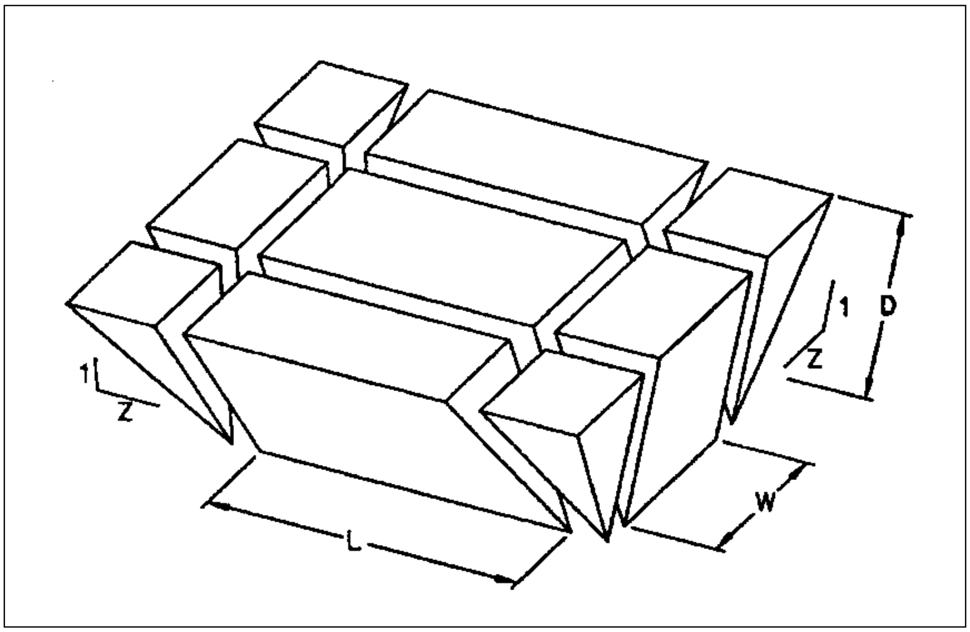


Figure 10.9. Trapezoidal basin.

Example 10.4:	Stage-storage curve of a trapezoidal basin.
Objective:	Find the dimensions of the basin at its base and develop a stage-storage curve for the basin.

Given: Estimated storage volume (V); depth available for storage during a 0.1 AEP event (D); available freeboard (F_B); basin side slopes (Z); and width-to-length ratio of basin (r).

V	=	30,016 ft^3 (850 m^3)
D	=	5.25 ft (1.6 m)
F_B	=	2.0 ft (0.6 m)
Z	=	3 (V:H = 1:3)
r	=	0.5

Develop a stage-storage curve for the basin assuming that the base elevation of the basin equals 32.8 ft (10.0 m), and the crest of the embankment is at 40.0 ft (12.2 m). Determine this crest elevation by adding the 5.25 ft (1.6 m) of available depth plus the 2.0 ft (0.6 m) of freeboard.

Step 1. Establish the basin dimensions.

Substituting the given values in equation 10.11:

$L = \{-Z\,D\,(r + 1) + [(Z\,D)^2\,(r + 1)^2 – 5.33\,(Z\,D)^2\,r + ((4r\,V) / D)]^{0.5}\} / 2r$

$L = \{(-3\,(5.25)\,(0.5 + 1) + [(3\,(5.25))^2\,(0.5+1)^2 – 5.33\,(3\,(5.25))^2\,0.5+((4\,(0.5)\,30016)/5.25)]^{0.5}\}/ 2\,(0.5)$

L = 82.82 ft, use L = 85 ft

W = 0.5 L = 42.5 ft, use W = 43 ft

Therefore, select an 85-ft by 43-ft basin.

Step 2. Develop the stage-storage relationship.

By varying the depth (D) in equation 10.10, develop a stage-storage relationship for the trapezoidal basin. Table 10.2 summarizes the results.

Table 10.2. Stage-storage relationship.

Depth (ft)	Stage (ft)	Storage Volume (ft^3)
0	32.81	0
0.66	33.46	2,567
1.31	34.12	5,425
1.97	34.77	8,774
2.62	35.43	12,455
3.28	36.09	16,548
3.94	36.75	21,074
4.59	37.4	26,052
5.25	38.06	31,503
5.91	38.71	37,413
6.56	39.37	43,906

Solution: Table provides the stage-storage curve.

10.3.2.3 Pipes and Conduits

Figure 10.10 provides a definition sketch for a generalized prism in a pipe or other shape of conduit. To provide for sediment transport through the conduit, designers place the conduit on a slope. The prismoidal formula provides an estimate of storage volume based on estimates of cross-sectional areas at three locations:

$$V = \left(\frac{L}{6}\right)(A_1 + 4M + A_2) \qquad (10.12)$$

where:

V	=	Volume of storage, ft^3 (m^3)
L	=	Length of section, ft (m)
A_1	=	Cross-sectional area of flow at base, ft^2 (m^2)
A_2	=	Cross-sectional area of flow at top, ft^2 (m^2)
M	=	Cross-sectional area of flow at midsection, ft^2 (m^2)

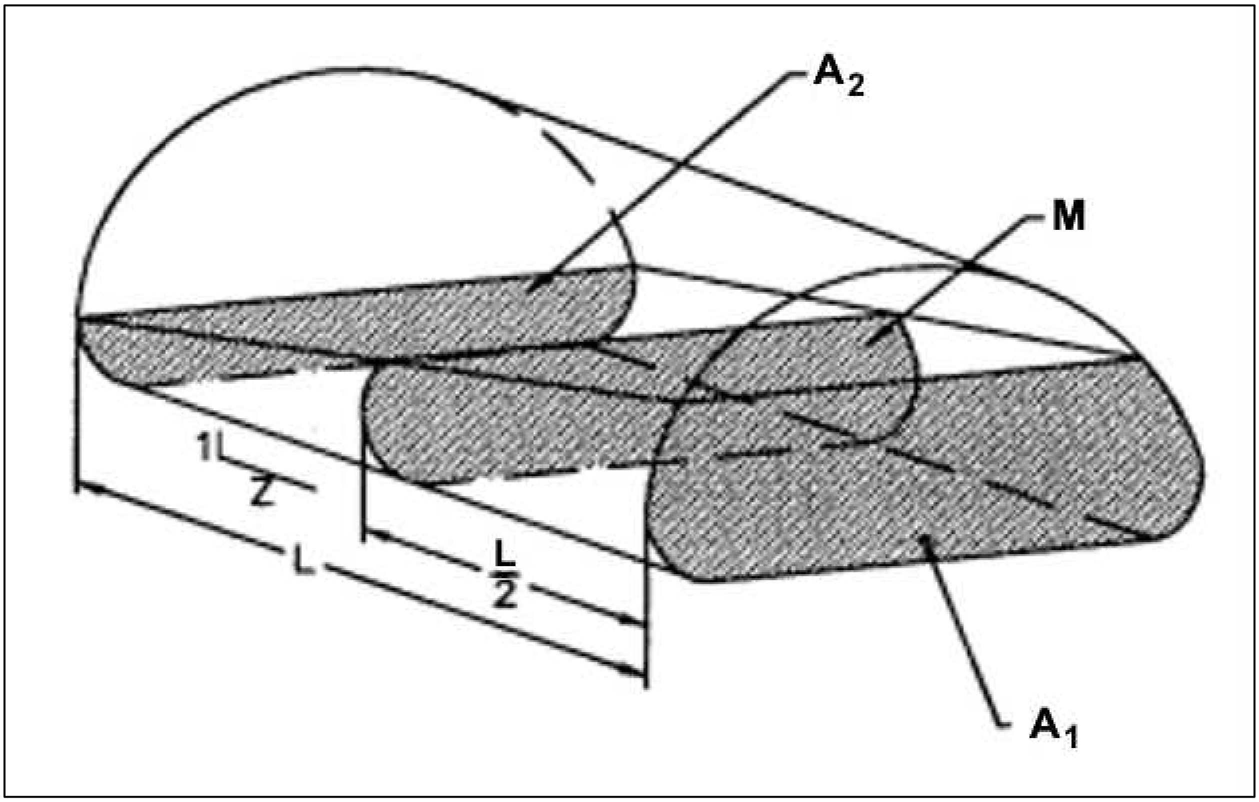


Figure 10.10. Definition sketch for prismoidal formula.

For a circular conduit, as illustrated in Figure 10.11, an exact storage volume results from when $d \leq 2r$:

$$V = \frac{H\,(0.667a^3 \pm cB)}{r \pm c} \tag{10.13}$$

$$a = \left[\left(2r - d\right)d\right]^{0.5} \tag{10.14}$$

$$c = d - r \tag{10.15}$$

$$\alpha = 2\sin^{-1}\left(\frac{a}{r}\right) \tag{10.16}$$

where:

V	=	Volume of storage, ft^3 (m^3)
B	=	Cross-sectional end area at depth d, ft^2 (m^2)
H	=	Wetted pipe length, ft (m)
r	=	Pipe radius, ft (m)

α	=	Angle as described in Figure 10.11, radians
a, c	=	Distances as described in Figure 10.11, ft (m)
d	=	Flow depth in pipe, ft (m)

To assist in the determination of the cross-sectional area of B, the area of the associated circular segment is:

$$A_s = (\alpha - \sin\alpha)\left(\frac{r^2}{2}\right) \tag{10.17}$$

where:

A_s	=	Segment area, ft^2 (m^2)

Compute the wetted area as follows:

For $d \leq r$; $B = A_s$

For $d > r$; $B = A - A_s$

A is the total pipe area. Alternatively, various texts, such as Brater and King (1976), contain tables and charts for use in determining the depths and areas described in the above equations.

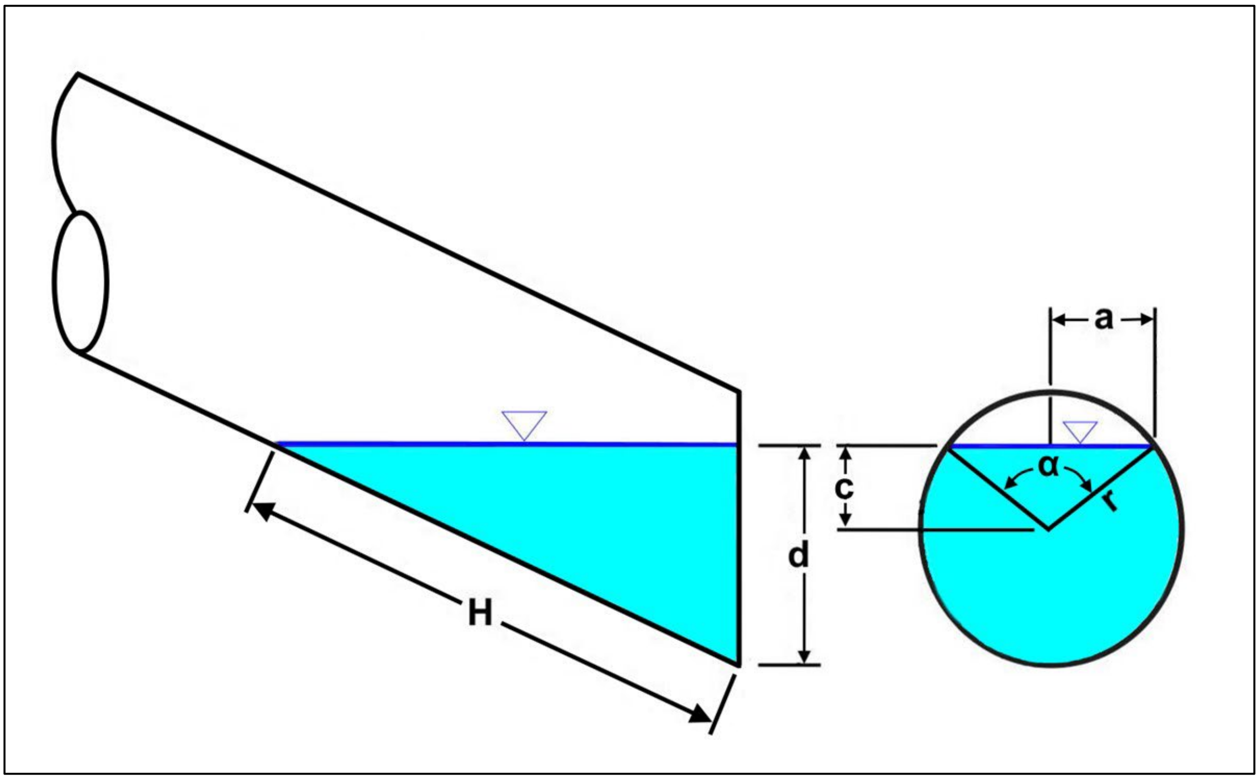


Figure 10.11. Definition sketch for a circular conduit.

Example 10.5: Estimation of pipe storage.

Objective: Develop a stage-storage tabulation between elevations 98 ft (30 m) and 103 ft (31.5 m)

Given: A storm drain pipe having the following properties:

D = 60 inches (1500 mm)
S = 0.01 ft/ft (m/m)
L = 820 ft (250 m)
Invert = 98 ft (30 m)

Step 1. Solve for the volume of storage.

Using equations 10.13 and 10.17:

$V = H\,[(2/3\ a^3 \pm c\,B) / (r \pm c)]$

$A_s = (\alpha - \sin\alpha)\,(r^2 / 2)$

Note that: $B = A_s$ for $d \leq r$

$B = A - A_s$ for $d > r$

Step 2. Tabulate storm drain pipe stage-storage curve.

Table 10.3 summarizes the results.

Table 10.3. Storm drain pipe stage-storage relationship.

d (ft)	a (ft)	c (ft)	H (ft)	alpha (rad)	B (ft^2)	V (ft^3)
0.0	0.00	-2.46	0.0	0.000	0.0	0.0
0.7	1.67	-1.80	66.0	1.495	1.5	41.6
1.3	2.18	-1.15	131.0	2.171	4.1	222.8
2.0	2.41	-0.49	197.0	2.739	7.1	588.8
2.6	2.45	0.16	262.0	3.008	10.3	1159.9
3.3	2.32	0.82	328.0	2.462	13.5	1940.0
3.9	1.97	1.48	394.0	1.855	16.3	2917.3
4.6	1.23	2.13	459.0	1.045	18.5	4061.1
4.9	0.00	2.46	492.0	0.000	19.01	4676.9

Solution: Table provides the tabular pipe stage-storage curve.

10.3.2.4 Irregular Basins

Rectangular and trapezoidal basins often oversimplify common detention and retention basins. However, the concepts used to derive the stage-storage relationship for the simple forms also apply to deriving the stage-storage relationship for an actual site that exhibits irregular shapes. The stage-storage relationship is derived as a discrete function (i.e., a set of points) from contours on topographic mapping.

Designers usually develop the storage volume for irregular basins using a topographic map and the average-end area or conic section formulas. The average-end area formula is expressed as:

$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2}\right] d \tag{10.18}$$

where:

$V_{1,2}$	=	Storage volume between elevations 1 and 2, ft^3 (m^3)
A_1	=	Horizontal area at elevation 1, ft^2 (m^2)
A_2	=	Horizontal area at elevation 2, ft^2 (m^2)
d	=	Change in elevation between points 1 and 2, ft (m)

Generally, the average-end area formula approximates irregular basin storage volume well when calculating volumes with small changes in elevation between respective elevations or when the basin width or length is changing but not both.

The conic section formula approximates irregular basin storage volume more accurately when both length and width of a basin are changing as shown in Figure 10.12. The conic approximation is:

$$V = d\left[\frac{A_1 + (A_1 A_2)^{0.5} + A_2}{3}\right] \tag{10.19}$$

where:

V	=	Volume of frustum of a pyramid, ft^3 (m^3)
A_1	=	Surface area at elevation 1, ft^2 (m^2)
A_2	=	Surface area at elevation 2, ft^2 (m^2)
d	=	Change in elevation between points 1 and 2, ft (m)

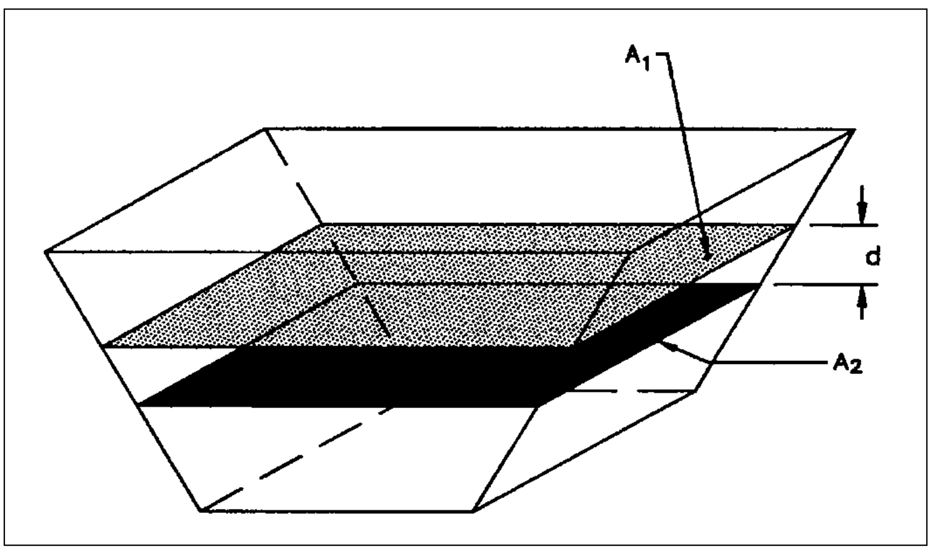


Figure 10.12. Definition sketch for irregular basins.

Example 10.6:	Storage estimate of basin for an irregular basin.
Objective:	Estimate the maximum volume using topography and average area method.

Given: Topographic map for site shown in Figure 10.13.

Step 1. Using average area method, develop a stage-storage curve.

The area bounded by each 1-ft contour line is estimated and the average area within adjacent contours is computed. Using equation 10.9 compute the stage-storage relationship which is summarized in Table 10.4 and plotted in Figure 10.14.

Solution: Based on the contours of the site, the maximum storage is 61,985 ft^3 (1760 m^3).

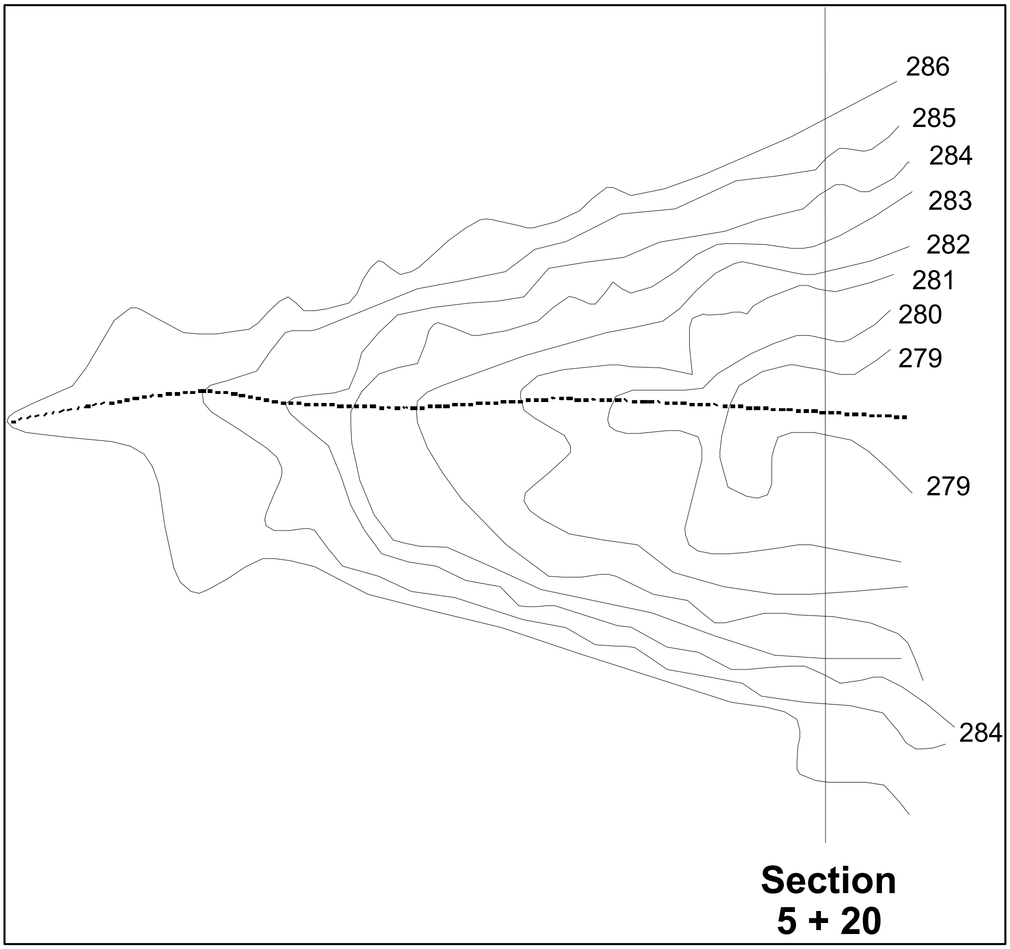


Figure 10.13. Topographic map for deriving stage-storage relationship at site of structure (section 5+20).

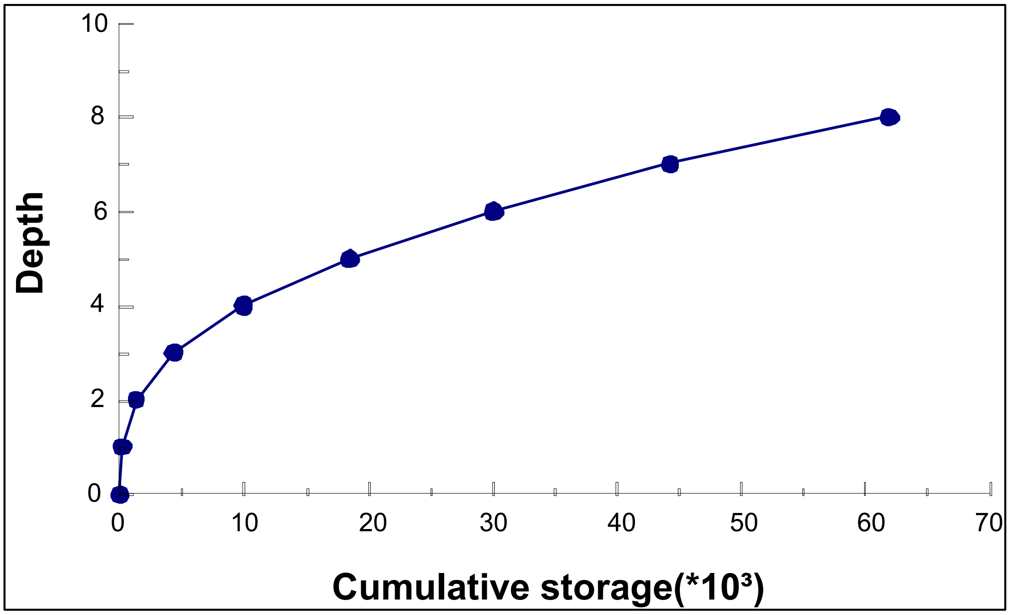


Figure 10.14. Stage-storage relationship.

Table 10.4. Derivation of stage-storage relationship for example irregular basin.

Contour Elevation (ft)	Total Area within Contour Elevation (ft^2)	Average Area (ft^2)	Contour Interval (ft)	Depth (ft)	Change in Storage (ft^2)	Cumulative Storage (ft^3)
278	0			0		0
		245	1		245	
279	490			1		245
		1,115	1		1,115	
280	1,740			2		1,360
		3,015	1		3,015	
281	4,290			3		4,375
		5,585	1		5,585	
282	6,880			4		9,960
		8,600	1		8,600	
283	10,320			5		18,560
		11,575	1		11,575	
284	12,830			6		30,135
		14,165	1		14,165	
285	15,500			7		44,300
		17,685	1		17,685	
286	19,870			8		61,985

Many storage facilities include a permanent pool. In such cases, the elevation of the weir or the bottom of the orifice is set above the elevation of the bottom of the pond. Storage below the outlet elevation is called dead storage. Storage above the outlet elevation is called active storage. Total storage equals the sum of the active and dead storages.

Example 10.7: Dead and active storage.

Objective: Determine dead and active storages.

Given: A storage facility with dead and active storage with the following characteristics.

Δh = 0.25 ft (0.076 m)
A_b = 2,000 ft^2 (186 m^2) (bottom surface)

Step 1. Establish depth increment.

Use a topographic map of the detention facility site to measure areas. Use a depth increment of 0.25 ft for computation.

Step 2. Tabulate stage-active storage relationship.

Table 10.5 can be used to illustrate the development of stage-active storage relationship. Column 2 gives the areas and column 3 gives the average areas. The incremental volumes equal the product of the change in depth, 0.25 ft, and the average area. The total storage at a given depth is the sum of the incremental storages up to and including that depth.

Table 10.5. Computation of stage-active storage relationship for example basin.

(1) Depth h (ft)	(2) Surface Area, A (ft^2)	(3) Average Area, A (ft^2)	(4) Incremental Volume ΔV (ft^3)	(5) Total Volume V (ft^3)	(6) Dead Storage V_d (ft^3)	(7) Active Storage V_a (ft^3)
0	2,000	-	-	0	0	0
-	-	2,100	525.0	-	-	-
0.25	2,200	-	-	525.0	525.0	0
-	-	2,250	562.5	-	-	-
0.50	2,300	-	-	1087.5	1087.5	0
-	-	2,400	600.0	-	-	-
0.75	2,500	-	-	1687.5	1087.5	600.0
-	-	2,650	662.5	-	-	-
1.00	2,800	-	-	2350.0	1087.5	1262.5
-	-	2,900	725.0	-	-	-
1.25	3,000	-	-	3075.0	1087.5	1987.5
-	-	3,050	762.5	-	-	-
1.50	3,100	-	-	3837.5	1087.5	2750.0

Solution: If the outlet facility has a minimum elevation of 0.5 ft (0.15 m), all storage below this elevation is dead storage. The active storage is 0.0 at an elevation of 0.5 ft (0.15 m). The active storage above 0.5 ft (0.15 m) equals the difference between the total storage and the dead storage. The stage (column 1) versus active storage (column 7) would be used when designing a storage facility.

10.3.3 Stage-Discharge Relationship (Performance Curve)

A stage-discharge (performance) curve describes the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has both a principal and an emergency outlet. The principal outlet usually conveys the design flood without allowing flow to enter the emergency outlet for flooding events up to the design flood for the principal outlet. The principal outlet structure typically consists of combinations of pipe culverts, weirs, orifices, or other appropriate hydraulic controls selected to satisfy single or multiple applicable hydrologic criteria. Designers develop composite stage-discharge curves by estimating the cumulative effects of the discharge rating relationships for each component of the outlet structure.

10.3.3.1 Discharge Pipes

Detention facilities typically have discharge pipes as part of the outlet structure and may also have a riser. The design of these pipes can be for either single or multistage discharges. A single step discharge system consists of a single culvert entrance system and is not designed to carry emergency flows. A multi-stage outlet structure adds a riser at the inlet end of the pipe. The design of an inlet structure allows the design discharge to pass through a weir or orifice in the lower levels of the structure and the emergency flows to pass over the top of the structure or through an emergency spillway. The design of the pipe allows it to carry the full range of flows from a drainage area.

For outlets without a riser, the facility design resembles a simple culvert. HDS-5 (FHWA 2012a) outlines appropriate design procedures. For multistage control structures, design of the inlet control structure considers both the design flow and the emergency flows. Designers develop a stage-discharge curve for the full range of flows the structure would experience. Typically, the design flows are orifice flow through whatever shape the designer has chosen while typically the higher flows are weir flow over the top of the control structure. Designers can use the equations in Section 10.3.3.2 for orifices and the equations in Section 10.3.3.3 for weirs. Designers select the pipe to carry all flows considered in the design of the control structure.

In designing a multistage structure, the designer first develops peak flows that must be passed through the facility. Next, the designer selects a pipe that passes the peak flow within the allowable headwater and develops a performance curve for the pipe. Third, the designer develops a stage-discharge curve for the inlet control structure, recognizing that the headwater for the discharge pipe will be the tailwater that needs to be considered in designing the inlet structure. Last, the designer uses the stage-discharge curve in the basin routing procedure.

Example 10.8:	Sizing detention basin outlet pipe.
Objective:	Find the size pipe needed to carry the maximum allowable flow rate from the detention basin.

Given: The maximum head on pipe (H_{max}), inlet invert elevation, length (L), slope (S), roughness (n), and square edge entrance coefficient (K_e), for a corrugated steel discharge pipe as shown in Figure 10.3. The discharge pipe outfall is free (not submerged).

H_{max} = 2.3 ft (0.75 m)

Invert = 32.8 ft (10.0 m)

L = 164 ft (50 m)

S = 0.04 ft/ft (m/m)

n = 0.024

K_e = 0.5

Step 1. Evaluate the pipe for inlet and outlet control.

Using the same discharges from example 10.2, the maximum predeveloped discharge from the watershed is 19.4 ft³/s (0.55 m³/s). Since the discharge pipe can function under inlet or outlet control, the pipe size will be evaluated for both conditions. The larger pipe size will be selected for the final design.

Step 2. Size the pipe for inlet control.

Using HDS-5 (FHWA 2012a) yields the relationship between head on the pipe and the resulting discharge for inlet control. The analysis shows the pipe diameter that will carry the flow under the specified conditions is 30 inches.

Step 3. Size the pipe for outlet control.

Using HDS-5 (FHWA 2012a) yields the relationship between head on the pipe and discharge for barrel (outlet) control. The analysis shows the pipe diameter that will carry the flow under the specified conditions is 27 inches.

Solution: For the design, select pipe diameter equal to 30 inches (750 mm).

10.3.3.2 Orifices

Figure 10.15 shows a schematic of a tank with a hole of area in its bottom. Assuming all losses can be neglected, write Bernoulli's equation between a point on the surface of the pool (point 1) and a point in the cross-section of the orifice (point 2):

$$\frac{P_1}{\gamma}+\frac{V_1^2}{2g}+z_1=\frac{P_2}{\gamma}+\frac{V_2^2}{2g}+z_2 \tag{10.20}$$

where:

P	=	Pressure, lb/ft² (N/m² or Pa)
V	=	Velocity, ft/s (m/s)
γ	=	Specific weight, lb/ft³ (N/m³)
z	=	Height above datum, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s² (9.81 m/s²)

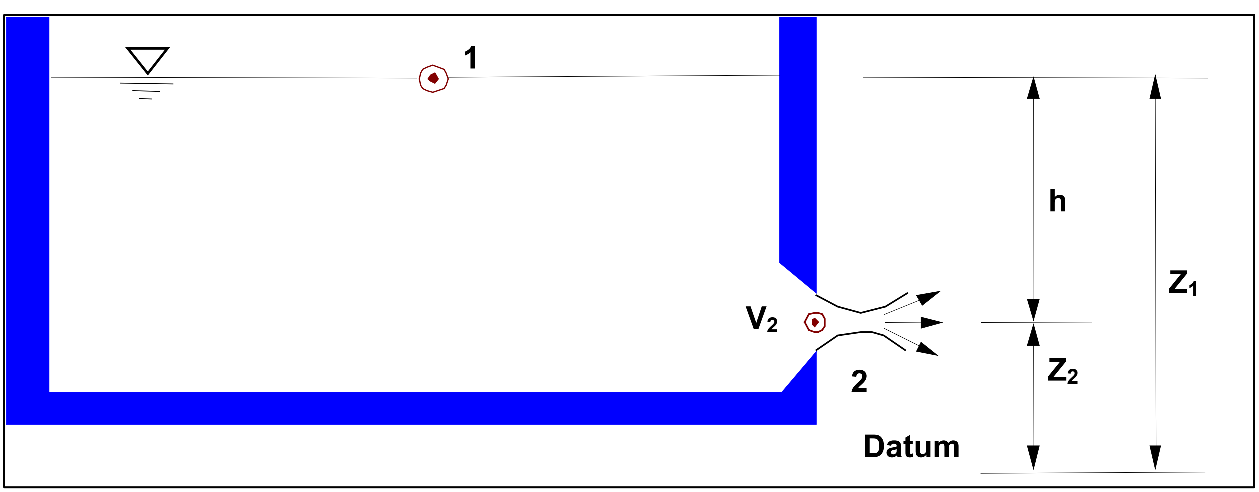


Figure 10.15. Schematic diagram of flow through an orifice.

This can be simplified by assuming: 1) the pressure at both points is atmospheric, therefore $p_1 = p_2$; 2) the surface area of the pool A_1 is very large relative to the area of the orifice A_2, so from the continuity equation V_1 is essentially zero; and 3) $z_1 - z_2 = h$. Equation 10.20 becomes:

$$h = \frac{V_2^2}{2g} \tag{10.21}$$

Solving for V_2 and substituting it into the continuity equation yields:

$$Q = AV = A\sqrt{2gh} \tag{10.22}$$

Equation 10.22 assumes ideal conditions with zero energy losses and atmospheric pressure across the opening of the orifice. It is actually atmospheric at a point below the orifice, where the cross-sectional area of the discharging water is slightly smaller than the area of the orifice. To account for these conditions, the orifice flow equation introduces a discharge coefficient:

$$Q = C_o A_o \sqrt{(2gh_o)} \tag{10.23}$$

where:

Q	=	Orifice flow rate, ft³/s (m³/s)
C_o	=	Discharge coefficient
A_o	=	Area of orifice, ft² (m²)
h_o	=	Effective head on the orifice measured from the centroid of the opening, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s² (9.81 m/s²)

Values of C_o range from 0.5 to 1.0, with a value of 0.6 commonly used, for square-edged, uniform orifice entrance conditions. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, use a value of 0.4.

If the orifice is not horizontal, the depth, h, is usually measured from the center of area of the orifice.

If the orifice discharges as a free outfall, i.e., unsubmerged, then measure the effective head from the centerline of the orifice to the upstream water surface elevation. For a submerged orifice discharge, the effective head equals the difference in elevation of the upstream and downstream water surfaces. Figure 10.16 shows this latter condition of a submerged discharge.

Designers use orifice plates on riser structures to control outflow from a detention pond. As shown in Figure 10.17, orifice plates consist of multiple openings. Compute flow through multiple orifices by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, multiplying the discharge for a single orifice by the number of openings yields the total flow.

Orifice or Culvert

Pipes smaller than 1 ft (0.3 m) in diameter may be analyzed as a submerged orifice when h_o/D is greater than 1.5. Headwater and tailwater effects influence pipes greater than 1 ft (0.3 m) in diameter, which function as discharge pipes, not just as an orifice; analysis accordingly incorporates this design consideration.

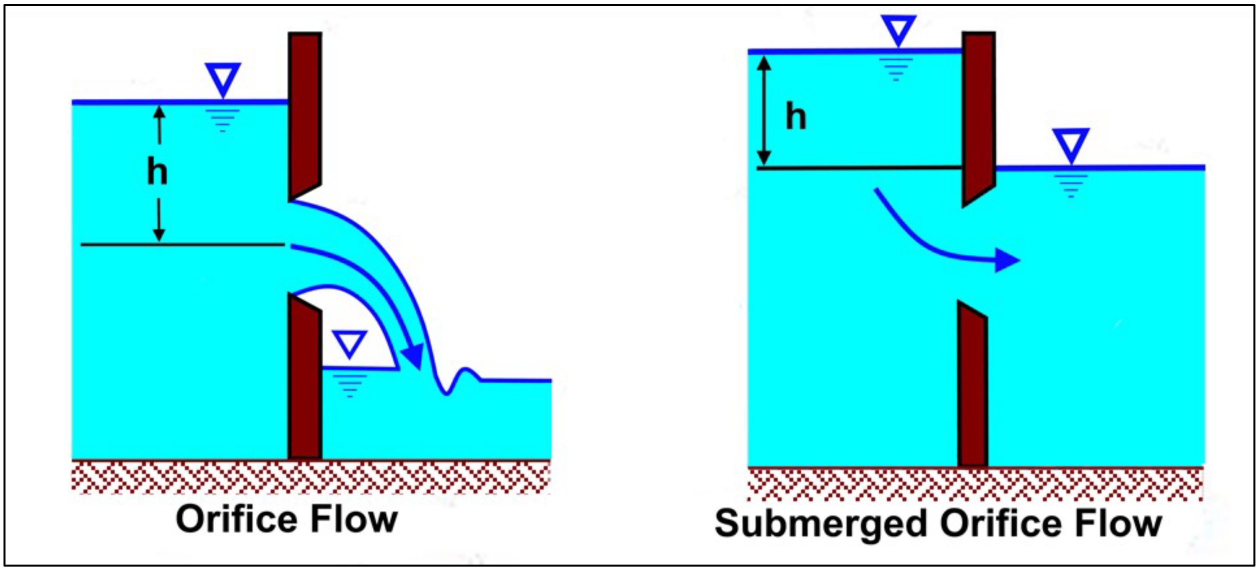


Figure 10.16. Definition sketch for orifice flow submergence.

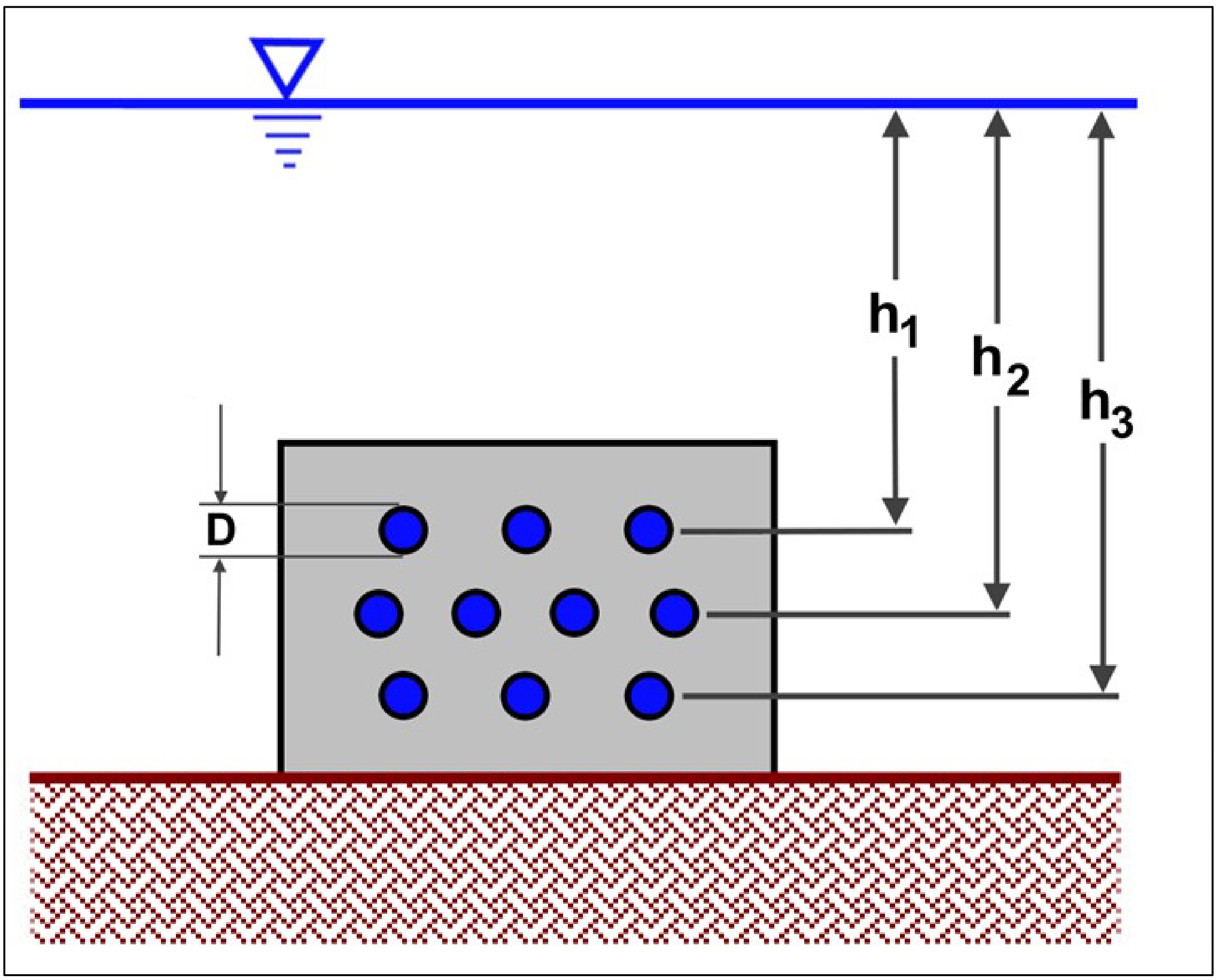


Figure 10.17. Orifice plate with multiple openings.

Example 10.9:	Orifice flow through an orifice plate.
Objective:	Estimate total discharge through an orifice plate with multiple openings.

Given: Given the orifice plate in Figure 10.17 with a free discharge and:

D = 1.0 inch or 0.0833 ft (25 mm)
h_1 = 3.61 ft (1.1 m)
h_2 = 3.94 ft (1.2 m)
h_3 = 4.26 ft (1.3 m)

Step 1. Use a modification of equation 10.23 to calculate the flows for each row of orifices.

Q_1 = (0.6) [(3)(π*0.0833^2 / 4)] [2*32.2*3.61]$^{0.5}$ = 0.15 ft^3/s

Q_2 = (0.6) [(4)(π*0.0833^2 / 4)] [2*32.2*3.94]$^{0.5}$ = 0.21

Q_3 = (0.6) [(3)(π*0.0833^2 / 4)] [2*32.2*4.26]$^{0.5}$ = 0.16

Step 2. Sum the orifice flows for each row of equal heads to arrive at the total flow.

Q_{total} = $Q_1 + Q_2 + Q_3$ = 0.52 ft^3/s

Solution: The flow through the orifice plate equals 0.52 ft^3/s (0.015 m^3/s).

Example 10.10:	Orifice stage-discharge rating curve.
Objective:	Develop the stage-discharge rating curve between 32.80 ft (10 m) and 39.37 ft (12.0 m).

Given: Given the circular orifice in Figure 10.16(a) with:

D = 0.49 ft (0.15 m)
invert = 32.80 ft (10.0 m)
C_o = 0.60

Using equation 10.23 with D = 0.49 ft yields the following relationship between the effective head on the orifice (h_o) and the resulting discharge:

h_o = Depth - D/2

For D = 3.3 ft, h_o = 3.3 - (0.49)/2 = 3.06

Q = C_o A_o [(2g(h_o)]$^{0.5}$ = (0.6)(π*0.49^2 / 4) [2*32.2*3.06]$^{0.5}$ = 1.59 ft^3/s

Table 10.6 summarizes the stage-discharge curve.

Solution: The table summarizes the stage-discharge tabulation for the orifice.

Table 10.6. Stage-discharge orifice flow example.

Depth (ft)	Stage (ft)	Discharge (ft^3/s)
0	32.8	0
0.7	33.5	0.61
1.3	34.1	0.93
2	34.8	1.2
2.6	35.4	1.39
3.3	36.1	1.59
3.9	36.7	1.74
4.6	37.4	1.89
5.2	38.1	2.04
5.9	38.7	2.16
6.6	39.4	2.29

10.3.3.3 Weirs

The following sections provide relationships for sharp-crested, broad-crested, V-notch, and proportional weirs.

10.3.3.3.1 Sharp-Crested Weirs

Consider the cross-section shown in Figure 10.18. Point 1 is located upstream of the weir at a distance where the weir does not influence the flow characteristics. Point 2 is at the weir. The following analysis assumes: (1) ideal flow, (2) frictionless flow, (3) critical flow conditions at the obstruction, and (4) the obstruction has a unit width perpendicular to the direction of flow.

For the critical flow conditions, the following equations describe the hydraulics at the obstruction:

$$F_r = 1 = \frac{V_c}{(gy_c)^{0.5}} \tag{10.24}$$

$$y_c = \left(\frac{q_u^2}{g}\right)^{\frac{1}{3}} \tag{10.25}$$

$$E_c = y_c + \frac{V_c^2}{2g} = \frac{3}{2}y_c \tag{10.26}$$

where:

F_r	=	Froude number
V_c	=	Critical velocity, ft/s (m/s)
y_c	=	Critical depth, ft (m/s)
q_u	=	Discharge rate per unit width, ft^2/s (m^2/s)
E_c	=	Minimum specific energy, ft (m)

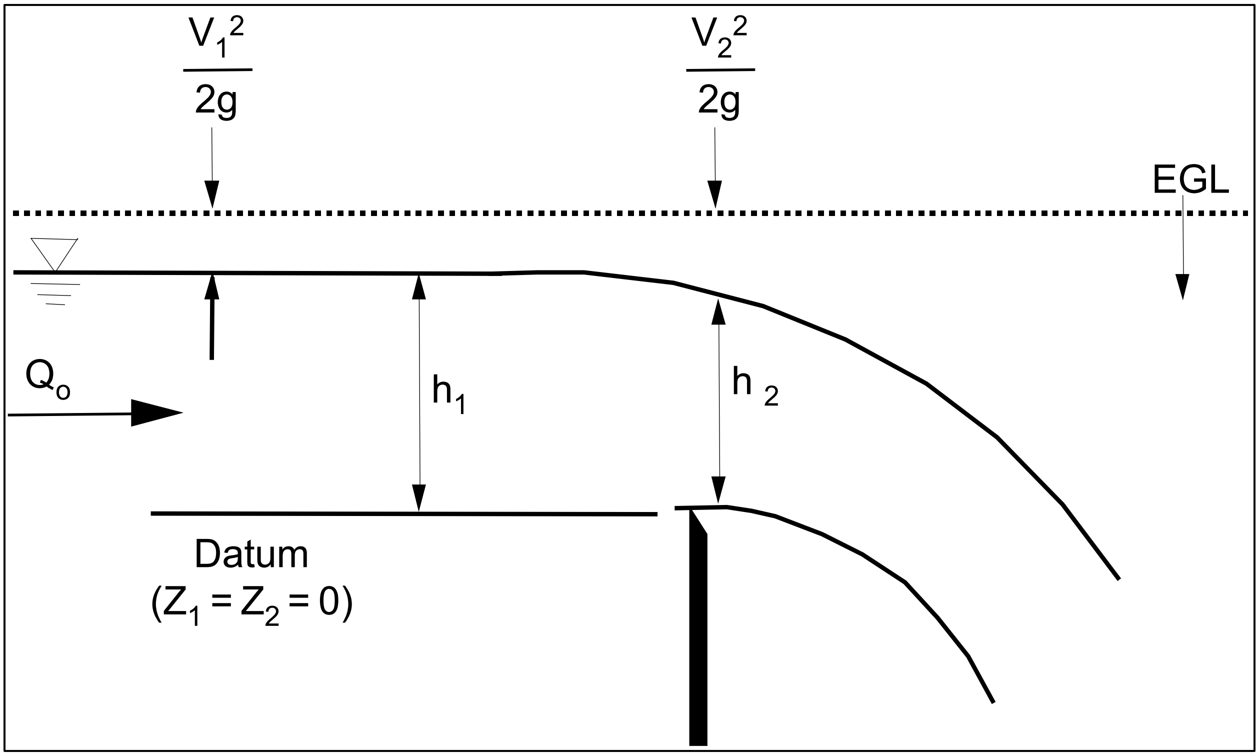


Figure 10.18. Schematic diagram of flow over a sharp-crested weir

Assuming hydrostatic pressure at sections 1 and 2, then $P_i/\gamma = h_i$. Thus, Bernoulli's equation is:

$$h_1 + \frac{V_1^2}{2g} + z_1 = h_2 + \frac{V_2^2}{2g} + z_2 \tag{10.27}$$

By setting the datum at the top of the weir, $z_1 = z_2 = 0$, assuming that the velocity head at section 1 is much smaller than the velocity head at section 2, and recalling that the flow passes through critical depth as it passes over the weir, then $V_2 = V_C$ and $h_2 = y_C$, reducing equation 10.27 to:

$$h_1 = \frac{V_c^2}{2g} + y_c \tag{10.28}$$

The velocity head is $V_c^2/2g = y_c/2$. Defining $h = h_1$, then:

$$h = \frac{y_c}{2} + y_c = \frac{3y_c}{2} \text{ or } y_c = \frac{2}{3}h \tag{10.29}$$

Solving equation 10.25 for q_u, it then follows that:

$$q_u = \left(gy_c^3\right)^{0.5} = \left[g\left(\frac{2}{3}h\right)^{0.5}\right] = \left(\frac{8g}{27}\right)^{0.5} h^{\frac{3}{2}} = \left(\frac{2}{\sqrt{27}}\right)\sqrt{2g}h^{\frac{3}{2}} \tag{10.30}$$

Letting $Q = Lq_u$, the general weir equation is:

$$Q = \left(\frac{2}{\sqrt{27}}\right)\sqrt{2g}\,h^{1.5}\,L \qquad (10.31)$$

where:

Q	=	Discharge over a horizontal weir, ft^3/s (m^3/s)
h	=	Head (depth) of approach flow above the weir, ft (m)
L	=	Weir length, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2).

Weir Coefficient

It is significant to note that many presentations of the weir equation embed the 2g into the coefficient, C_w, making it a dimensioned rather than dimensionless quantity. By keeping the gravity term in the equation, C_w becomes a property of a specific weir type and not dependent on the system of units.

Equation 10.31 represents ideal flow over a weir. Because actual weirs perform less efficiently, engineers modify the equation by the addition of a weir coefficient, C_w The value of C_w depends on the type of weir, head, weir height, and other factors. It also includes the initial quotient in equation 10.31.

$$Q = C_w\sqrt{2g}\,h^{1.5}\,L \qquad (10.32)$$

For the sharp-crested weir of this derivation, values of C_w can range from 0.27 to 0.38. The range reflects the variation in losses from alternative weir/flow configurations. Losses depend on the depth of flow over and approaching the weir, weir length, weir thickness, and weir height. Even laboratory studies have difficulty accurately estimating C_w. Designers can use a value of 0.37 for sharp-crested rectangular weirs where more information is not available.

Equation 10.32 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure 10.19). Designers typically treat flow over the top edge of a riser pipe as flow over a sharp-crested weir with no end constrictions.

Equation 10.33 provides the discharge equation for sharp-crested weirs with end contractions (illustrated in Figure 10.19). As indicated above, the value of the coefficient C_w varies with the ratio h/h_c (see Figure 10.20 for definition of terms). For values of the ratio h/h_c less than 0.3, designers often use a constant C_w of 0.415.

$$Q = C_w\sqrt{2g}\,(L - 0.2\,h)\,h^{1.5} \qquad (10.33)$$

Submergence affects sharp-crested weirs when the tailwater rises above the weir crest elevation, as shown in Figure 10.20. These effects reduce discharge over the weir. The discharge equation for a submerged sharp-crested weir is (Brater and King 1976):

$$Q_s = Q\left(1 - \left(\frac{h_2}{h_1}\right)^{1.5}\right)^{0.385} \qquad (10.34)$$

where:

Q_s	=	Submerged flow, ft^3/s (m^3/s)
Q	=	Unsubmerged weir flow, ft^3/s (m^3/s)

h_1 = Upstream head above crest, ft (m)
h_2 = Downstream head above crest, ft (m)

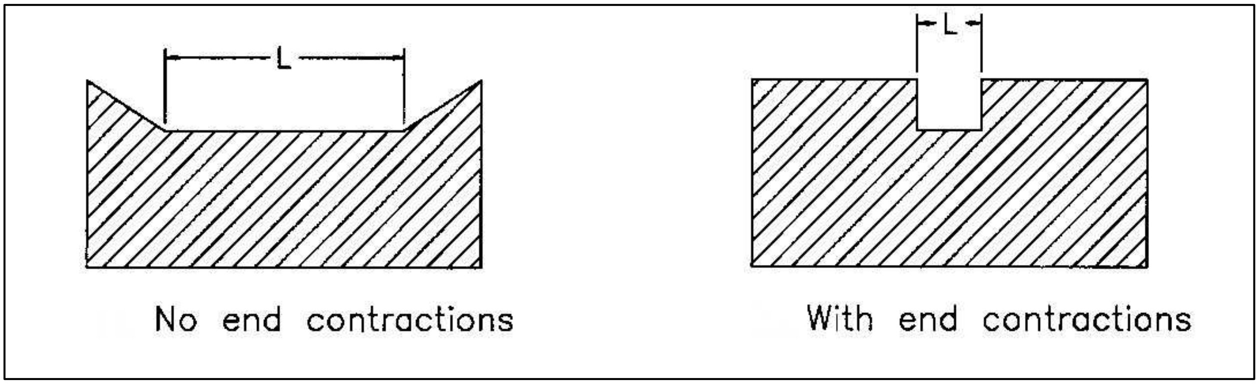


Figure 10.19. Sharp-crested weir contractions.

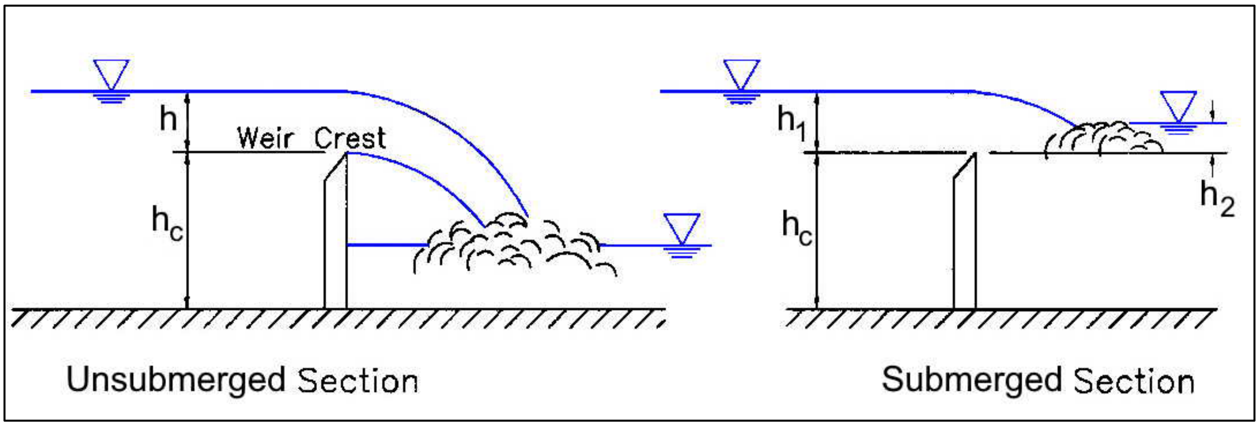


Figure 10.20. Sharp-crested weir submergence.

10.3.3.3.2 Broad-Crested Weir

Equation 10.32 also applies to a broad-crested weir. If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum weir coefficient value of 0.41. For sharp corners on the broad-crested weir, use a minimum value of 0.29. Table 10.7 provides additional information on C_w values as a function of weir crest breadth and head.

10.3.3.3.3 V-Notch Weir

Figure 10.21 illustrates discharge through a v-notch weir. Calculate discharge from (Brater and King 1976):

$$Q = C_w \sqrt{2g}\left[\tan\left(\frac{\theta}{2}\right)\right]h^{2.5} \tag{10.35}$$

where:

Q = Discharge, ft^3/s (m^3/s)
Θ = Angle of v-notch, degrees
h = Head on apex of v-notch, ft (m)
C_w = Weir coefficient for a v-notch weir (typically equal to 0.31)

Table 10.7. Broad-crested weir coefficient values (adapted from Brater and King 1976).

Head * (ft)	Breadth of Weir Crest (ft)										
	0.5	0.75	1	1.5	2	2.5	3	4	5	10	15
0.2	0.35	0.34	0.34	0.33	0.32	0.31	0.30	0.30	0.29	0.31	0.33
0.4	0.36	0.35	0.34	0.33	0.33	0.32	0.32	0.32	0.31	0.32	0.34
0.6	0.38	0.36	0.34	0.33	0.33	0.32	0.33	0.34	0.34	0.34	0.34
0.8	0.41	0.38	0.36	0.33	0.32	0.32	0.33	0.33	0.33	0.34	0.33
1.0	0.41	0.39	0.37	0.34	0.33	0.33	0.33	0.33	0.33	0.33	0.33
1.2	0.41	0.40	0.38	0.36	0.34	0.33	0.33	0.33	0.33	0.34	0.33
1.4	0.41	0.41	0.40	0.36	0.35	0.33	0.33	0.33	0.33	0.33	0.33
1.6	0.41	0.41	0.41	0.38	0.36	0.34	0.33	0.33	0.33	0.33	0.33
1.8	0.41	0.41	0.41	0.38	0.36	0.34	0.33	0.33	0.33	0.33	0.33
2.0	0.41	0.41	0.41	0.38	0.36	0.34	0.34	0.33	0.33	0.33	0.33
2.5	0.41	0.41	0.41	0.41	0.38	0.36	0.35	0.34	0.33	0.33	0.33
3.0	0.41	0.41	0.41	0.41	0.40	0.38	0.36	0.34	0.33	0.33	0.33
3.5	0.41	0.41	0.41	0.41	0.41	0.40	0.37	0.34	0.33	0.33	0.33
4.0	0.41	0.41	0.41	0.41	0.41	0.41	0.38	0.35	0.34	0.33	0.33
4.5	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.36	0.34	0.33	0.33
5.0	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.38	0.35	0.33	0.33
5.5	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.36	0.33	0.33

*Measured a distance at least 2.5 times critical depth upstream of the weir.

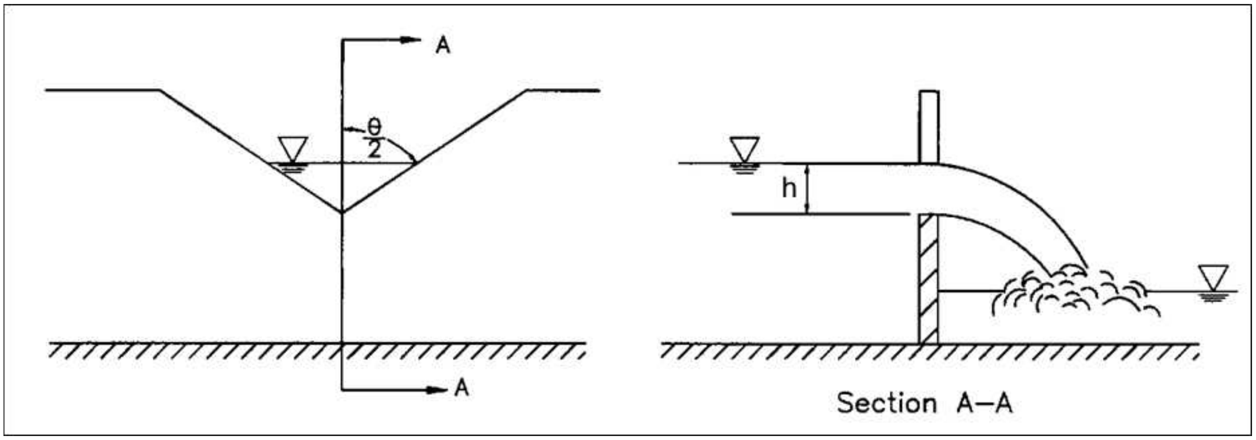


Figure 10.21. V-notch weir.

10.3.3.3.4 Proportional Weir

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir differs from other control devices by having a linear head-discharge relationship. This relationship is achieved by allowing the discharge area to vary nonlinearly with head with a configuration shown in Figure 10.22. The following equation describes the relations between the dimensions of a proportional weir (Sandvik 1985):

$$\frac{x}{b} = 1 - (0.315)\left[\arctan\left(\frac{y}{a}\right)^{0.5}\right] \tag{10.36}$$

Discharge for a proportional weir is (Sandvik 1985):

$$Q = C_w \sqrt{2g}\, a^{0.5}\, b\left(h - \frac{a}{3}\right) \tag{10.37}$$

where:

Q	=	Discharge, m^3/s (ft^3/s)
h	=	Head above horizontal sill, ft (m)
C_w	=	Weir coefficient for a proportional weir (typically equal to 0.62)

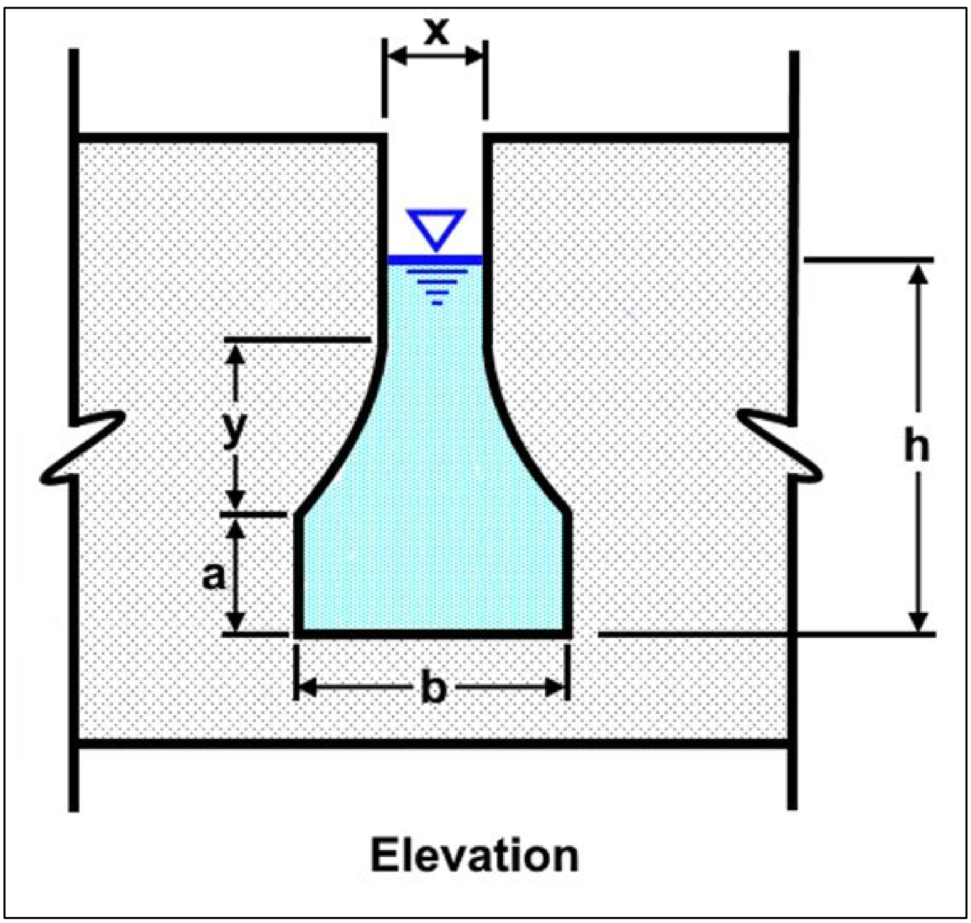


Figure 10.22. Proportional weir.

Example 10.11: Detention pond riser hydraulics.

Objective: Establish a stage-discharge rating curve for a riser pipe for water surface elevations in the pond between 32.8 ft (10.0 m) and 39.4 ft (12.0 m).

Given: The diameter (D), crest elevation (EL), and weir height (H_c) for a riser pipe as shown in Figure 10.23 with the following characteristics:

D = 1.74 ft (0.53 m)

EL = 35.4 ft (10.8 m)

H_c = 2.6 ft (0.8 m)

Since the riser pipe functions as both a weir and an orifice (depending on stage), develop the rating by comparing the stage-discharge produced by both weir and orifice flow.

Step 1. Establish relationship between orifice flow and head.

Using equation 10.23 for orifices with D = 1.74 ft yields the following relationship between the effective head on the orifice (H_o) and the resulting discharge:

$$Q = C_o A_o [(2g(h_o)]^{0.5} = (0.6)(\pi*1.74^2 / 4) [2*32.2*h_o]^{0.5} = (0.6)(\pi\ 1.74^2 /4)(64.4 * h_o)^{0.5}$$

$$= 11.44\ h_o^{0.5}$$

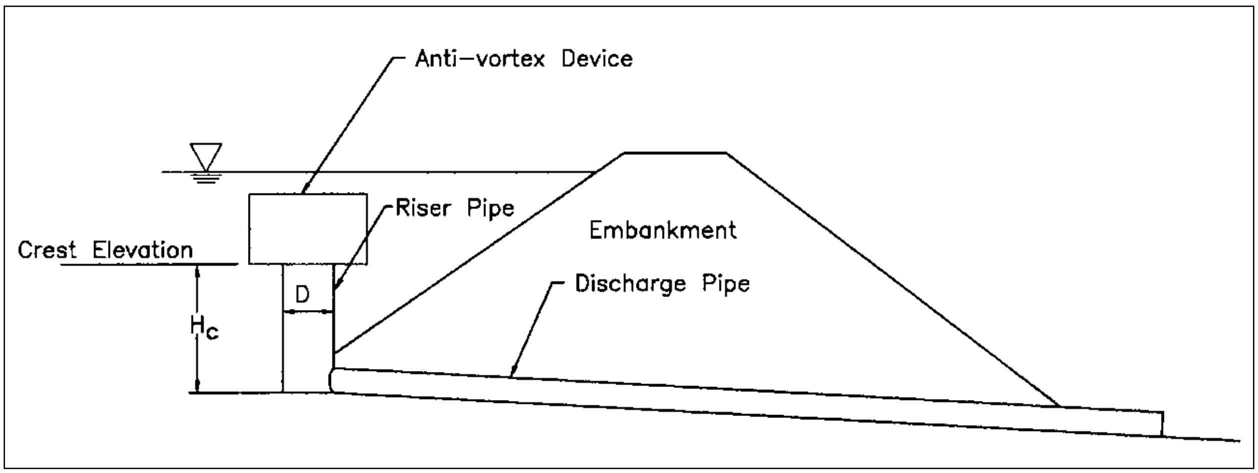


Figure 10.23. Riser pipe configuration for example.

Step 2. Establish relationship between weir flow and head.

Using the dimensionless equation 10.32 for sharp-crested weirs with C_W = 0.41 (h/h_c assumed less than 0.3), and L = pipe circumference = 5.5 ft yields the following relationship between the effective head on the riser (h) and the resulting discharge:

$$Q = C_W (2g)^{0.5} L\ h^{1.5} = (0.41) (64.4)^{0.5} (5.5)\ h^{1.5} = 18.32\ h^{1.5}$$

Step 3. Tabulate stage-discharge relationship.

Using the relationships established in steps 1 and 2, develop the stage-discharge curve, shown in Table 10.8.

Table 10.8. Stage-discharge relationship.

Stage (ft)	Effective Head (ft)	Orifice Flow (ft^3/s)	Weir Flow (ft^3/s)
32.8	0	0	0
35.4	0	0	0
35.8	0.33	6.6	3.5 *
36.1	0.66	9.2 *	9.8
36.7	1.31	13.1 *	27.5
37.4	1.97	15.9 *	50.6
38.1	2.63	18.7 *	77.7
38.7	3.28	20.8 *	108.8
39.4	3.94	22.6 *	143.2

*Designates controlling flow.

Solution: The flow condition (orifice or weir) producing the lowest discharge for a given stage determines the controlling relationship. As illustrated in Table 10.8, at a stage of 35.76 ft (10.9 m) weir flow controls the discharge through the riser. However, at and above a stage of 36.09 ft (11.0 m), orifice flow controls the discharge through the riser.

10.3.3.4 Auxiliary Spillway

An auxiliary (emergency) spillway provides a controlled overflow relief for storm flows exceeding the design discharge for the storage facility. A broad-crested overflow weir, as shown in Figure 10.24, represents a suitable emergency spillway for highway detention facilities. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. The embankment height is typically at an elevation 1 ft to 2 ft above the maximum design storage elevation. It is preferable to have a freeboard of 2 ft minimum. However, for very small impoundments, less than 1- to 2-acre surface area an absolute minimum of 1-foot of freeboard may be acceptable (UDFCD 2001). The USDA-NRCS Engineering Field Handbook (2021) provides design information for auxiliary spillways.

10.3.3.5 Composite Stage-Discharge Relationships

Presented earlier in this chapter, Figure 10.3 shows a schematic of a basin with an outlet structure consisting of a riser and a pipe outlet that together form the outlet structure controlling the discharge from the basin. The riser may include one or more weir or orifice openings. The designer evaluates the outlet structure to develop a composite stage-discharge curve. In addition to determining the diameter of the pipe outlet, the designer also establishes invert elevations of the pipe. For a basin with a permanent pool, both the volume of dead storage and the corresponding elevation of the permanent pool are set.

In the sizing of outlet structures, the designer determines both the required volume of storage and the physical characteristics of the structure including the outlet pipe dimensions; the riser dimensions; orifice and weir dimensions; and the elevations of the pipe outlet, weirs, and orifices. A single-stage riser provides for a single control feature suitable for satisfying a single hydrologic criterion. A multiple-stage riser provides for multiple hydrologic criteria.

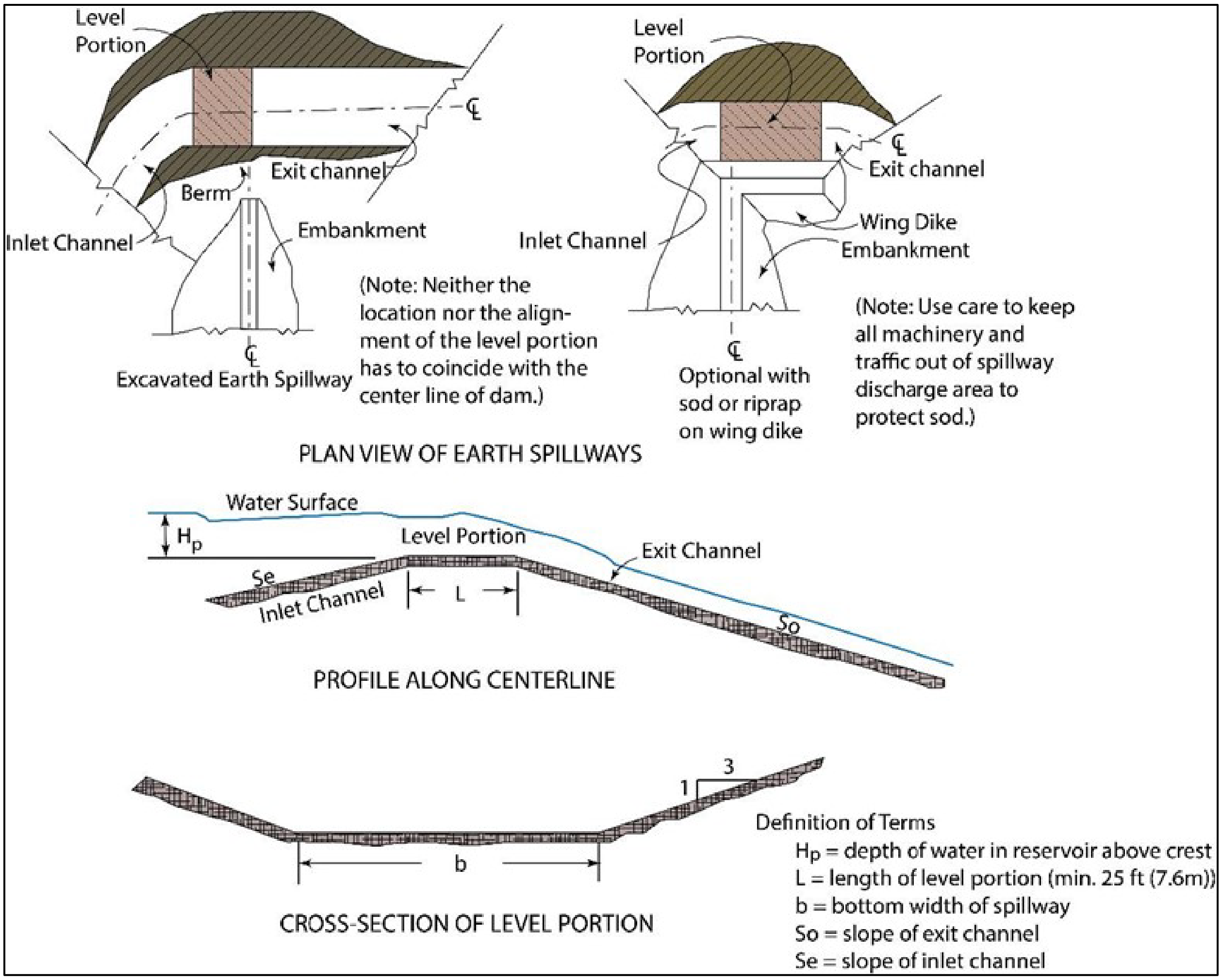


Figure 10.24. Profile and cross-section of excavated earth spillway. Source: USDA-NRCS (2021).

Estimating the characteristics of an outlet structure uses the following inputs:

- Watershed characteristics, including area, pre- and post-development times of concentration, and pre- and post-development curve numbers (assuming NRCS CN procedures are used for abstractions).
- Rainfall depth(s) for the design storm(s).
- Characteristics of the riser and outlet pipe structure, including pipe roughness (n), length, and an initial estimate of the diameter.
- Elevation information, including stage vs. storage values, the wet pond elevation, if applicable, and the elevation of the centerline of the pipe.
- Hydrologic and hydraulic models, including a model for estimating peak flows and runoff depths, a model for estimating the volume of storage as a function of pre- and post-development peak flows, and a model for estimating weir and orifice coefficients, as appropriate.

10.3.3.5.1 Single-Stage Risers

Single-stage risers, as shown in Figure 10.25, provide runoff control for cases where practice specifies one exceedance frequency. The following procedure estimates both the riser characteristics and the volume of storage (adapted from Woodward 1983).

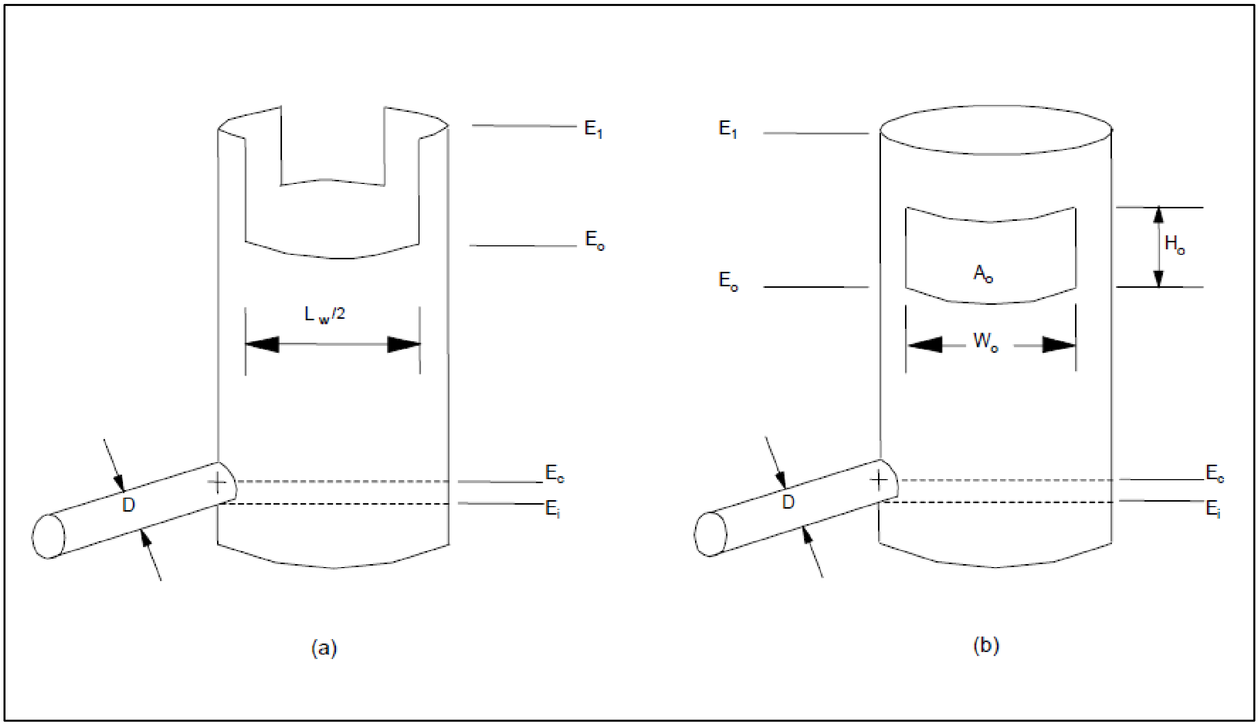


Figure 10.25. Single-stage riser characteristics for (a) weir flow and (b) orifice flow.

Step 1. Design the culvert barrel.

Design the culvert barrel using the procedures of HDS-5 (FHWA 2012a) and set the culvert invert elevation. To simplify the riser design, the designer selects a culvert so that the maximum culvert headwater, which is based on the maximum outlet design discharge, is no higher than the invert of the orifice opening (or weir). If this is not the case, this condition "submerges" the opening forcing the designer into an iterative procedure to establish the stage-discharge relationship.

Step 2. Estimate the preliminary storage volume.

Estimate the preliminary storage volume, V_s, using one of the techniques described in Section 10.3.1.

Step 3. Estimate the dead storage.

Using the elevation E_o of either the weir or the bottom of the orifice, obtain the volume of dead storage V_d from the elevation-storage curve.

Step 4. Estimate the total storage needed.

Compute the total storage:

$$V_t = V_d + V_s \tag{10.38}$$

Step 5. Obtain highest water surface elevation.

Enter the elevation-storage curve with Vt to obtain the water surface elevation, E_I.

Step 6. Size the outlet opening.

For an orifice, determine its characteristics:

- Assume an orifice height, H_o.
- Compute the area of the orifice opening assuming unsubmerged flow, A:

$$A_o = \left(\frac{1}{C_d\sqrt{2g}}\right)\frac{Q_{pb}}{\left(E_1 - E_o - \frac{H_o}{2}\right)^{0.5}} \tag{10.39}$$

where:

A_o	=	Orifice opening area, ft² (m²)
Q_{pb}	=	Discharge through the orifice (pre-development), ft³/s (m³/s)
E_1	=	Water surface elevation upstream of the orifice, ft (m)
E_o	=	Elevation of the bottom of the orifice, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s² (9.81 m/s²)

$H_o/2$ adjusts for head being measured from the center of the orifice.

For a rectangular orifice, compute the width of the orifice opening W_o:

$$W_o = \frac{A_o}{H_o} \tag{10.40}$$

For a weir, determine the weir length for unsubmerged flow:

$$L_w = \left(\frac{1}{\sqrt{2g}C_w}\right)\frac{Q_{pb}}{(E_1 - E_0)^{1.5}} \tag{10.41}$$

Step 7. Verify performance with storage routing.

Verify the design performance using storage routing. (The approximate procedures for estimating storage volume do not account for performance of the outlet structure throughout the passage of the inflow hydrograph and may result in an under- or over-design.)

Example 10.12: Single-stage riser design.

Objective: Reduce the post-development 0.5 AEP peak flow to the pre-development peak flow.

Given: A single-stage riser is being designed for a pond. It must reduce post-development 0.5 AEP peak to the pre-development peak flow. The outlet structure will consist of an outlet pipe culvert and a riser with a rectangular orifice. The pond will include permanent pool elevation for water quality purposes.

A	=	11.2 ac (4.53 ha)
$Q_{2,post}$	=	17.7 ft³/s (0.5 m³/s)
$Q_{2,pre}$	=	2.8 ft³/s (0.08 m³/s)
Invert	=	96.8 ft (29.5 m) (pond bottom)

EL_{pool}	=	98.8 ft (30.1 m) (dead storage)

Step 1. Select culvert size, slope, and material.

First, select and locate a culvert barrel so that the headwater at the culvert entrance under the design discharge does not reach the invert of the orifice forcing it to operate under submerged conditions. Use the procedures of HDS-5 (FHWA 2012a) and consider the site constraints on culvert slope and invert location.

D	=	36 inches (910 mm)
L	=	34.4 ft (10.5 m)
n	=	0.012 (reinforced concrete pipe)

If the selected size is not available, specify the next available larger size. Place the entrance invert at an elevation of 96.1 ft (29.3 m). The designer selects the culvert inlet so that the culvert headwater, i.e., the depth of water inside the riser, does not rise above the lowest riser opening for the design conditions, where possible. The designer also adjusts the culvert invert to be compatible with site conditions.

Step 2. Develop Stage-storage curve.

Section 10.3.2 gives details on constructing a stage-storage relationship. For this example, Table 10.9 gives the stage-storage relationship at the site of the detention structure. Based on the permanent pool elevation, interpolate the dead storage from the stage-storage curve to be 13,400 ft^3.

Step 3. Estimate total storage.

Compute the active storage using one of the methods presented in Section 10.3.1, estimating it at 23,700 ft^3. The total storage equals the sum of the active and dead storages, 37,100 ft^3.

Step 4. Find the maximum stage.

Find the elevation corresponding to the total storage by interpolating the stage-storage curve, which is 101.4 ft.

Step 5. Compute orifice flow.

The orifice invert was established at an elevation of 98.8 ft to create the permanent pool. Assuming an initial orifice height of 0.5 ft, compute the area of the orifice in the riser with equation 10.39 to create an outflow of 2.8 ft^3/s at the assumed stage in the pond:

$$A_o = \{1 / [0.6 (2*32.2)^{0.5}]\} \{2.8 / [101.4 – 98.8 – (0.5 / 2)]^{0.5}\} = 0.379 \text{ ft}^2$$

$$W_o = A_o / D = 0.379 / 0.5 = 0.76 \text{ ft}$$

Solution: One possible orifice is 0.5 ft by 0.76 ft. If the calculated dimensions are not practical construction sizes, conduct an iterative trial process with practical dimensions resulting in the same performance until finding suitable dimensions. It is not usually appropriate to select the next larger available dimensions because this will allow excessive discharge through the orifice. Using storage routing, check the design before finalizing. The diameter of the riser (if it is also circular) usually equals 2 to 3 times the diameter of the outlet culvert.

Table 10.9. Stage-storage curve.

Elevation (ft)	Volume (ft^3)
96.8	0
97.0	1,390
97.5	4,410
98.0	7,620
98.5	11,240
99.0	15,540
99.5	19,850
100.0	24,440
100.5	29,280
101.0	34,120
101.5	39,940
102.0	46,170
102.5	52,860
103.0	60,120
103.5	68,420
104.0	77,990
104.5	88,030

10.3.3.5.2 Multiple-Stage Risers

Designers use multi-stage risers where two or more exceedance frequencies apply. Figure 10.26 depicts a two-stage riser, which is similar to the single-stage riser except that it includes either two weirs or a weir and an orifice. For the weir/orifice combination, the orifice controls the more frequent event, and the weir controls the larger event. Designers refer to the runoff from the smaller and larger events as the low-stage and high-stage events, respectively. Recognizing that the two events will not occur simultaneously, designers control the high-stage event using both the low-stage weir or orifice and the high-stage weir.

The procedure for a multi-stage riser follows the same general steps as for the single-stage riser, except the designer determines both the high-stage weir and low-stage outlet characteristics. The procedure uses the same inputs for sizing a two-stage riser as for a single-stage riser but relies on computing many of the values for both the low-stage and the one or more high-stage events.

Designers can size a two-stage riser for the cases where the low-stage outlet is either a weir or an orifice and the high-stage outlet is a weir using the following steps:

Step 1. Design culvert barrel.

Design the culvert barrel using the same process as for the single-stage riser. For the multi-stage riser, the maximum design discharge corresponds to the highest-stage event.

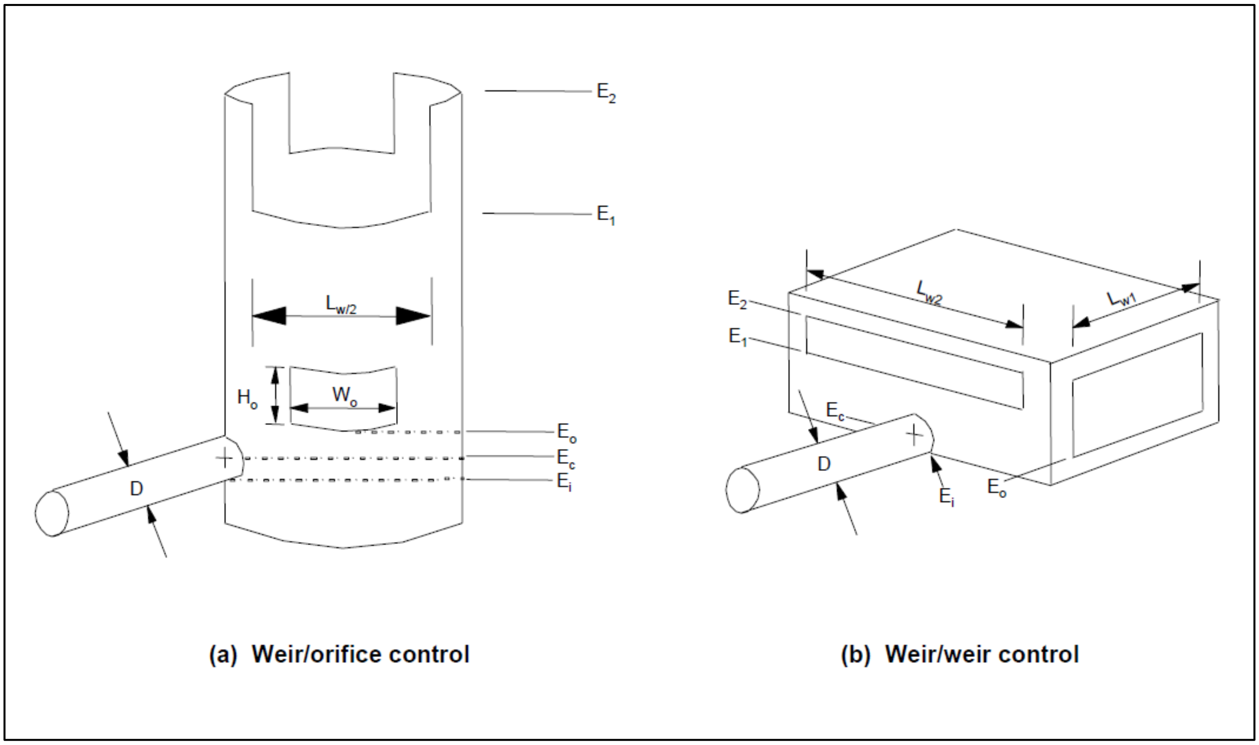


Figure 10.26. Two-stage riser.

Step 2. Design the low-stage opening.

Design the low-stage opening as described for a single-stage riser. If the low-stage opening is a weir of the configuration shown in Figure 10.26(b), it operates independently up to an elevation equal to the invert of the high-stage weir.

Step 3. Estimate the high-stage storage volume.

Estimate the appropriate high-stage storage volume, V_{s2} (Section 10.3.1).

Step 4. Compute the high-stage storage.

Compute the total high-stage storage:

$$V_{t2} = V_d + V_{s2} \tag{10.42}$$

Step 5. Obtain the water surface elevation.

Enter the elevation-storage curve with V_{t2} to obtain the water surface elevation, E_2.

Step 6. Estimate discharge through the low-stage opening.

Estimate the discharge through the low-stage opening during the high-stage event.

If the low-stage outlet is an orifice, the discharge is:

$$q_{o2} = C_{d1}\sqrt{2g}\, A_{o1}\left(E_2 - E_0 - \frac{h_o}{2}\right)^{0.5} \tag{10.43}$$

If the low-stage outlet is a weir, the discharge is:

$$q_{o2} = C_{w1}\sqrt{2g}\, L_{w1}(E_2 - E_0)^{1.5}$$

Step 7. Set high-stage invert.

Set the invert of the high-stage weir equal to E_1 (maximum elevation during the low-stage event).

Step 8. Compute high-stage weir length.

Compute the high-stage weir length:

$$L_{w2} = \left(\frac{1}{\sqrt{2g}\, C_w} \right) \frac{q_{pb2} - q_{o2}}{(E_2 - E_1)^{1.5}} \tag{10.44}$$

Step 9. Analyze additional high-stage openings.

If designing additional high-stage openings, repeat steps 3 through 8 accounting for all other openings that are active during the event.

Step 10. Verify with routing.

Verify the design performance using storage routing to confirm that the outlet structure and storage volume perform as intended.

Table 10.10 summarizes a composite stage-discharge relationship for an outlet control device consisting of a low flow orifice and a riser pipe connected to an outflow pipe. The structure also includes an emergency spillway. Table 10.6, Table 10.8, and emergency spillway computations summarize the individually calculated components. Using the methods in Section 10.3.3.4, the spillway passes 0, 39.55, and 55.8 ft^3/s at elevations of 38.1, 38.7, and 39.4 ft, respectively. The total discharge equals the sum of individual components at each stage.

Figure 10.27 illustrates the same information. Initially, the low flow orifice controls the discharge. At an elevation of 35.4 ft the water surface in the storage facility reaches the top of the riser pipe and begins to flow into the riser. The flow at this point equals a combination of the flows through the orifice and the riser. Orifice flow through the riser controls the riser discharge above a stage of 36.1 ft. At an elevation of 38.0 ft, flow begins to pass over the emergency spillway. Beyond this point, the total discharge from the facility equals the sum of the flows through the low flow orifice, the riser pipe, and the emergency spillway. Additionally, the designer ensures that the outlet pipe from the detention basin is large enough to carry the total flows from the low orifice and the riser section so that the outlet pipe does not limit the flow from the basin.

Table 10.10. Composite stage-discharge tabulation.

Stage (ft)	Low Flow Orifice (ft^3/s)	Riser Orifice Flow (ft^3/s)	Emergency Spillway (ft^3/s)	Total Discharge (ft^3/s)
32.8	0	0	0	0.00
33.5	0.61	0.00	0.00	0.61
34.1	0.93	0.00	0.00	0.93
34.8	1.20	0.00	0.00	1.20
35.4	1.39	0.00	0.00	1.39
36.1	1.59	9.18	0.00	10.77
36.7	1.74	13.07	0.00	14.81
37.4	1.89	15.89	0.00	17.78
38.1	2.04	18.72	0.00	20.76
38.7	2.16	20.84	39.55	62.55
39.4	2.29	22.60	55.80	80.69

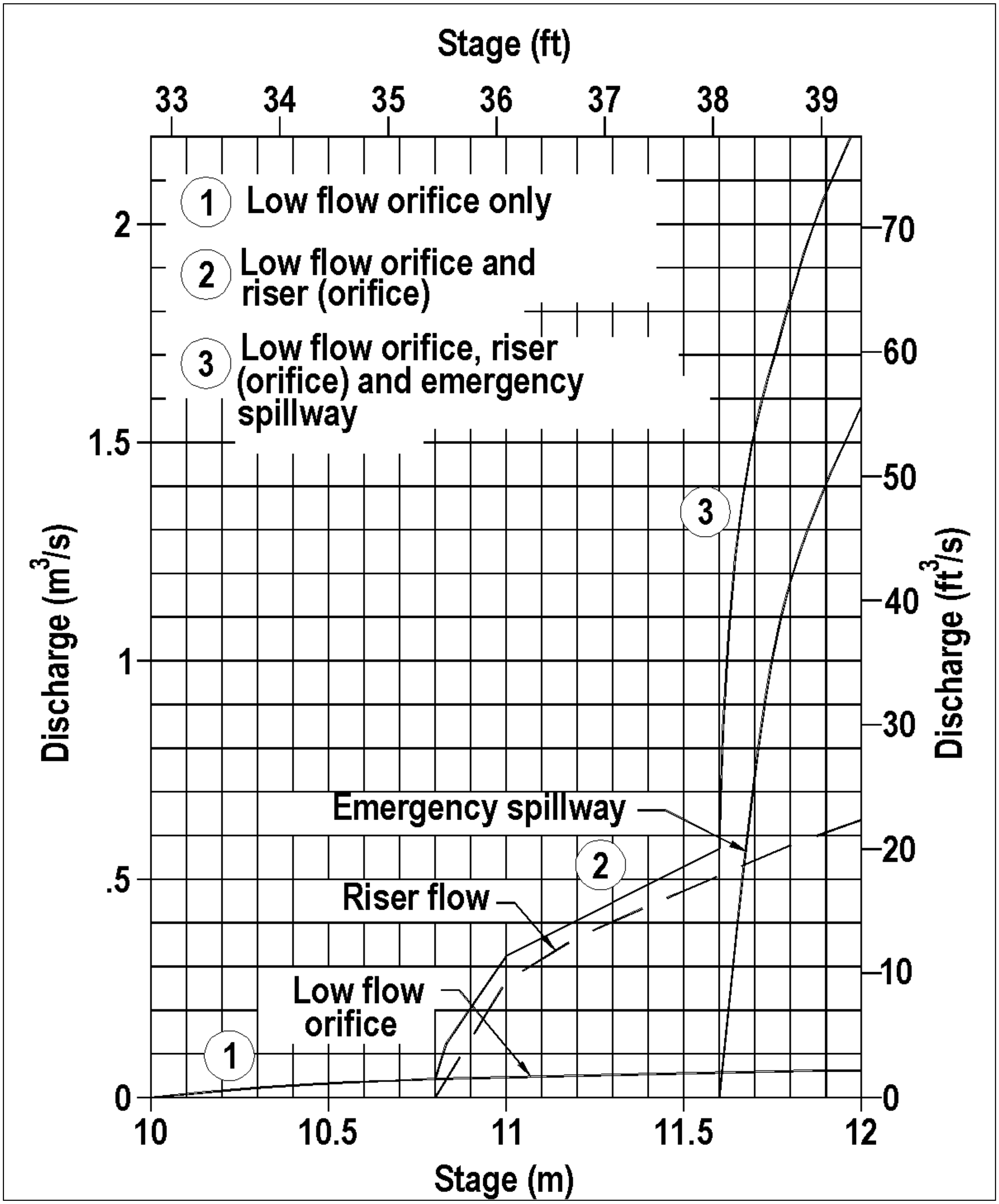


Figure 10.27. Typical composite stage-discharge relationship.

10.4 Water Budgets for Wet Ponds

Designers prepare water budgets for permanent pool facilities to confirm that the pool will be present under the conditions intended by the designer. At a minimum, these typically include average rainfall conditions but may also include alternative wet or dry conditions. A water budget considers all significant inflows and outflows, including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation, and outflow.

10.4.1 Water Budget Components

Designers can compute average annual runoff using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Base infiltration and exfiltration on site-specific soils testing data. Approximate evaporation using the mean monthly pan evaporation or free water surface evaporation data for the area of interest.

Analyzing infiltration involves knowledge of soil permeabilities and hydrogeologic conditions in the vicinity of the basin. Although county soils reports often publish infiltration rates, designers may secure field measurements when site-specific estimates for these parameters are important for the water budget. This is particularly important in karst areas where the hydrogeologic phenomenon controlling infiltration rates may be complex.

HDS-2 (FHWA 2002) provides a detailed discussion on water budgets, specifically as they relate to wetland applications for roadway projects.

Example 10.13: Water budget of a retention pond.

Objective: For average annual conditions, determine if the facility will function as a wet pond with a permanent pool.

Given: A shallow basin with characteristics including average surface area (A_s), bottom area (A_B), watershed area (A), post-development runoff coefficient (C_D), average infiltration rate for soils (F_a), average annual rainfall from rainfall records (P_a), and mean annual evaporation (E_a).

A_s = 3 ac (1.21 ha)
A_B = 2 ac (0.81 ha)
A = 100 ac (40.5 ha)
C_D = 0.3
F_a = 0.1 in/h (2.5 mm/h)
P_a = 50 inches (127 cm)
E_a = 35 inches (89 cm)

Step 1. Compute average annual runoff.

Runoff = C Q_D A (modification of equation 4.1)

Runoff = (0.3) (4.17 ft) (100 ac) (43,560 ft^2/ac) = 5,445,000 ft^3

Step 2. Estimate the average annual evaporation.

Evaporation = E_a A_s = (2.92 ft) (3 ac) (43,560 ft^2/ac) = 381,150 ft^3

Step 3. Estimate the average annual infiltration.

Infiltration = (F_a) (time) (A_B) = (0.01 in/h) (24 h/day) (365 days/yr) (2.0 ac) (43,560 ft^2/ac)

= 6,359,760 ft^3

Step 4. Estimate the runoff (or inflow) less evaporation and infiltration losses.

Assuming no basin outflow and no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:

Net Budget = 5,445,000 – 381,150 – 6,359,760 = -1,295,910 ft^3

Since the average annual losses exceed the average annual rainfall, the proposed facility will not function as a wet pond with a permanent pool. If the facility needs to function with a permanent pool, accomplish that by reducing the pool size.

Step 5. Revise the pool surface area.

Pool surface and bottom areas equal 2.0 ac and 1.0 ac, respectively.

Step 6. Recompute the evaporation and infiltration.

Evaporation = (2.92 inches) (2.0 ac) (43560 ft^2/ac) = 254,100 ft^3

Infiltration = (0.01) (24) (365) (1.0) (43560/12) = 3,179,880 ft^3

Step 7. Estimate revised runoff less evaporation and infiltration losses.

Net Budget = 5,495,000 – 254,100 – 3,179,880 = 2,011,020 ft^3

Solution: The revised facility appears able to function as a wet pond with a permanent pool. However, these calculations use average precipitation, evaporation, and losses. During years of low rainfall, the pool may not persist.

10.4.2 Water Budget for Landlocked Storage

Designers can evaluate watershed areas which drain to central depressions with no positive (gravity-driven) outlet using a mass flow routing procedure to estimate flood elevations. Typical examples include retention basins in karst topography or other areas having high infiltration rates. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrologic phenomenon that depends on good field measurements and a thorough understanding of local conditions. Since outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

Figure 10.28 illustrates a mass routing procedure for the analysis of landlocked retention areas. The step-by-step procedure is:

Step 1. Obtain cumulative rainfall data for the design storm.

If no local criteria are available, AASHTO (2014) suggests a 0.01 AEP, 10-day storm.

Step 2. Calculate the cumulative inflow.

Using the rainfall data from step 1 and an appropriate runoff hydrograph method (see Chapter 4), estimate the cumulative inflow.

Step 3. Develop the basin outflow.

From field measurements of hydraulic conductivity or infiltration, considering worst-case water table conditions, estimate the basin outflow. Establish hydraulic conductivity/infiltration using in situ test methods. Then, plot the mass outflow with a slope corresponding to the worst-case outflow.

Step 4. Draw tangent line.

Draw a line tangent to the mass inflow curve from step 2 having a slope parallel to the mass outflow line from step 3.

Step 5. Locate the point of tangency.

Locate the point of tangency between the mass inflow curve of step 2 and the tangent line drawn for step 4. The distance from this point of tangency and the mass outflow line multiplied by the drainage area represents the maximum storage required for the design runoff.

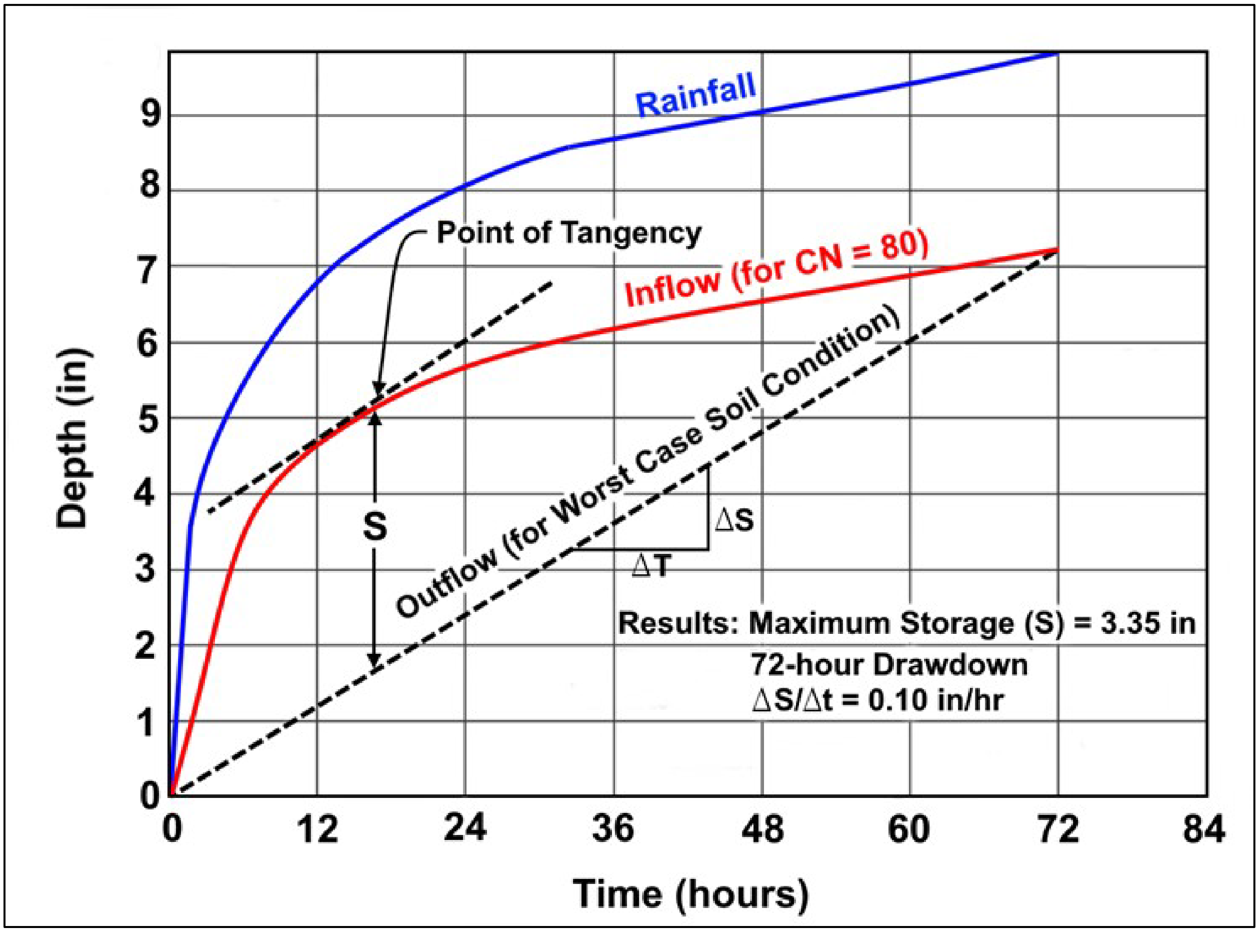


Figure 10.28. Mass routing procedure.

Step 6. Determine the flood elevation.

Estimate the flood elevation associated with the maximum storage volume determined in step 5. Use this flood elevation to evaluate flood protection requirements of the project. Establish the zero-volume elevation as the normal wet season water surface or water table elevation or the pit bottom, whichever is highest.

If runoff from a project area discharges into a drainage system tributary to the landlocked depression, the pre-development discharge requirements for the project may include detention storage facilities.

10.5 Storage Routing

Most commonly, designers route the inflow hydrograph through a detention pond using the storage-Indication or modified Puls method. This method derives from the continuity equation which states that the inflow minus the outflow equals the change in storage. By taking the average of two closely spaced inflows and two closely spaced outflows, the discretized continuity equation is expressed by:

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \tag{10.45}$$

where:

ΔS	=	Change in storage, ft^3 (m^3)
Δt	=	Time interval, min
I	=	Inflow, ft^3 (m^3)
O	=	Outflow, ft^3 (m^3)

Subscripts 1 and 2 refer to the beginning and end of the time interval, respectively. Figure 10.29 illustrates the routing process graphically with the inflow hydrograph as the input to the routing and the outflow hydrograph as the result of the routing. HDS-2 (FHWA 2002) provides a more detailed description of storage routing.

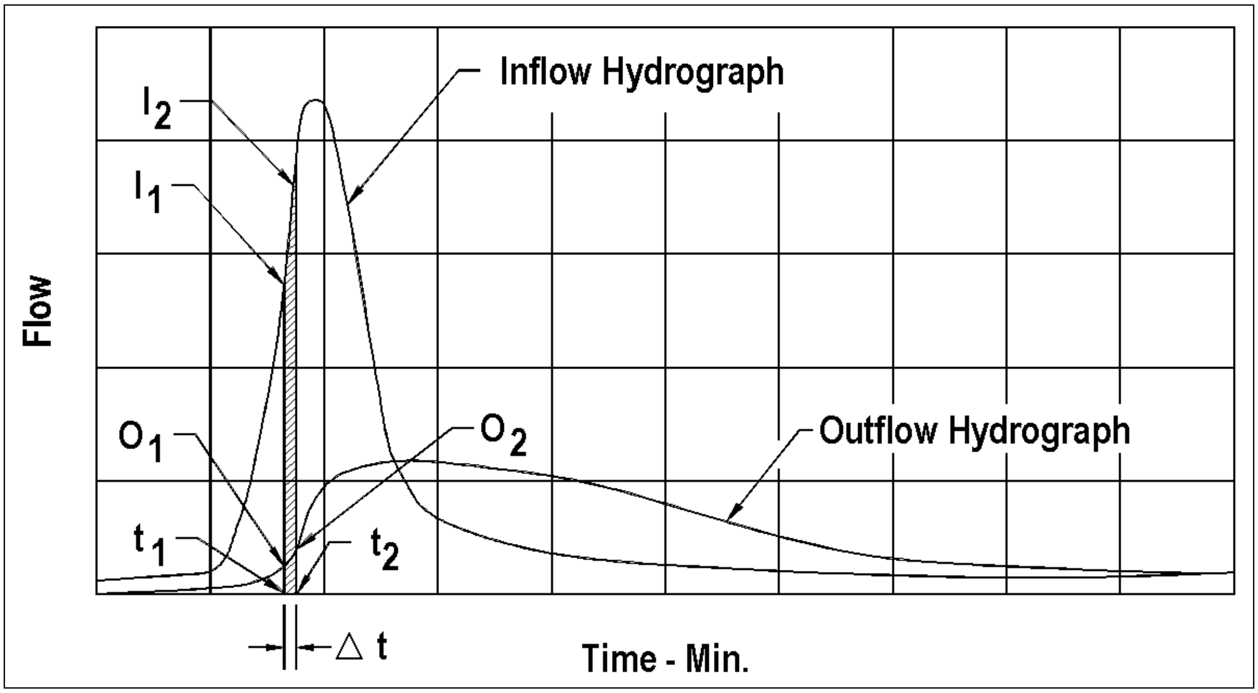


Figure 10.29. Inflow and routed outflow hydrographs.

10.6 Detention Design Procedure

Detention basin design has the hydrologic goals of identifying both the outlet structure characteristics (discharge pipe, weirs, and orifices) and the storage geometry that will limit the computed post-development peak flow out of the detention basin so that it does not exceed the target discharge (commonly the peak flow under pre-development conditions). Both the target discharge and the outlet structure characteristics influence the stage-storage-discharge relationship. Because of this interdependence, detention design uses an iterative procedure. The design procedure begins with some assumptions about the outlet structure characteristics. Then the designer applies the storage routing procedure to determine whether the design meets the target discharge requirement. If the routed discharge exceeds the target discharge, then the designer adjusts the design to reduce the discharge until the target is met. Conversely, if the routed discharge is significantly below the target discharge, the designer may also adjust the design to reduce the needed storage and potentially reduce the cost of the facility.

Four inputs support the design:

- Initial conditions. Establish initial conditions, including setting the time interval Δt, the storm time at which computations end, and the initial outflow O_1 and storage S_1.
- Inflow hydrograph. Compute the design storm inflow hydrograph and the target maximum discharge, q_o, for the design storm. The design storm inflow hydrograph is usually the output from the convolution of a rainfall-excess hyetograph and a unit hydrograph, with the post-development conditions used to compute the rainfall excess. The target discharge is usually the peak flow of the pre-development hydrograph. See Chapter 4.
- Stage-storage relationship. Obtain topographic information and compute the stage-storage relationship. See Section 10.3.2.
- Stage-discharge relationship. Set the riser characteristics (i.e., number of stages, type of outlet, and values of the discharge coefficients). See Section 10.3.3.

The designer routes the inflow hydrograph through the current basin design using the routing procedure introduced in Section 10.5 based on the initial conditions, stage-storage relationship, and stage-discharge relationship. After routing, the designer compares its peak flow of the outflow hydrograph to the target discharge q_o. If it is greater than q_o, then the capacity of the assumed outlet configuration is too large. Thus, the designer will decrease the weir lengths or orifice areas. If the peak flow of the outflow is less than the target discharge q_o, the assumed outlet configuration meets the target, but may be overdesigned. The designer can increase the weir lengths or orifice areas. Any adjustment to the riser structure involves recomputing the stage-discharge relationship followed by rerouting the inflow hydrograph. When the peak outflow approximately equals the target discharge, the assumed outlet facility is a reasonable design.

The designer estimates required storage by the largest value of storage associated with the outflow hydrograph. The designer uses the maximum storage as input to the stage-storage curve to estimate the depth of storage. The designer evaluates the computed design for safety and cost.

Example 10.14: Detention basin design process.

Objective: Design a detention basin storage and outlet structure.

Given: A watershed described as:

A	=	38 acres (15.8 hectares)
AEP	=	0.1

Step 1. Determine the hydrologic goals for the detention basin.

Using an appropriate hydrologic method, the pre-development and post-development peaks are:

q_{pb} = 50 ft^3/s

q_{pb} = 131 ft^3/s

Development within the watershed increased the 0.1 AEP discharge by 162 percent. The detention basin design goal is to reduce the peak flow from 131 ft^3/s to 50 ft^3/s.

Step 2. Establish inflow hydrograph.

Develop an input hydrograph for design. Figure 10.30 displays the inflow hydrograph with a peak of 131 ft^3/s. Use the ordinates on a 0.1-hour increment for a 2-hour period of the 24-hour storm; discharges for the remainder of the 24-hour storm duration are either zero or very small.

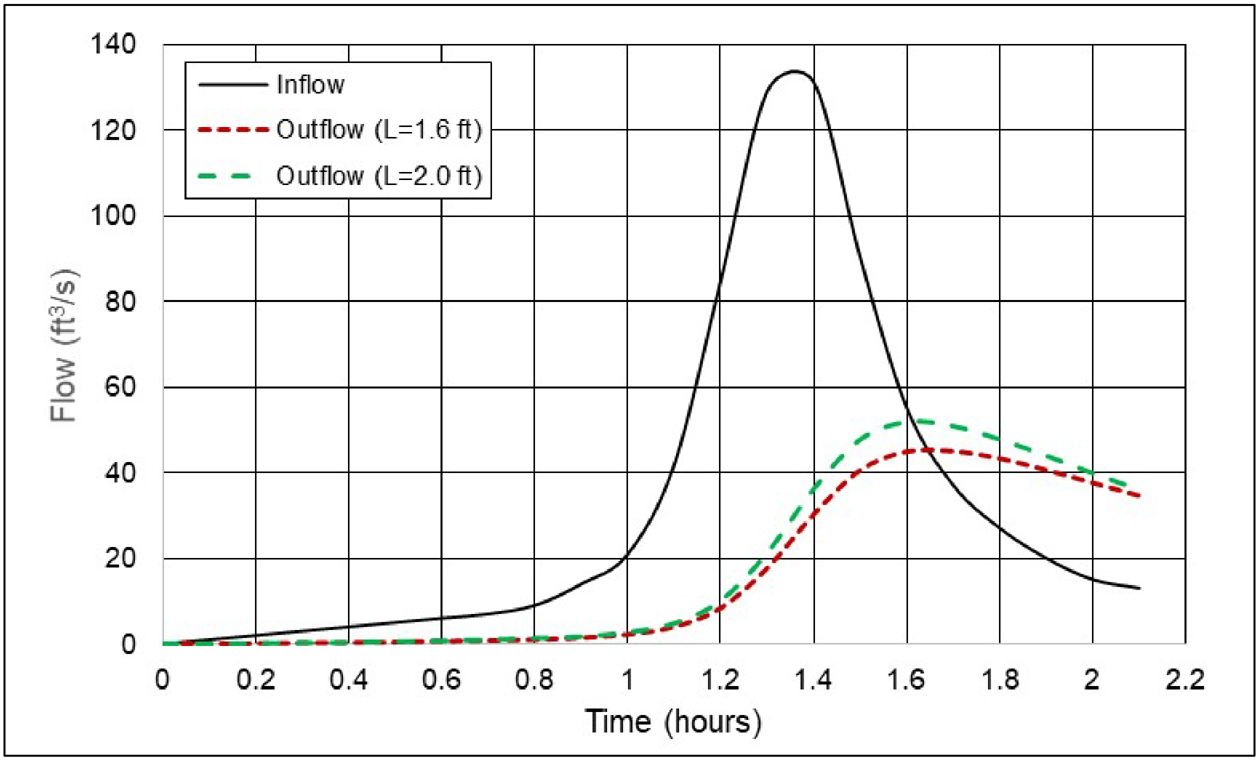


Figure 10.30. Inflow and routed outflow hydrographs for detention basin example.

Step 3. Develop stage-storage curve.

Compute the stage-storage relationship from topographic information. Table 10.11 shows the estimated surface areas at the site from the topography at an increment of 0.5 ft. Multiply the average area (column 3) and the incremental depth of 0.5 ft to yield the incremental storage (column 4). Accumulate the incremental storage values to compute a cumulative storage at each elevation.

Step 4. Develop stage-discharge curve.

The designer selects a one-stage riser with a weir. Make an initial estimate of the weir length using the weir equation, with an assumed depth of 4.9 ft and the target outflow discharge of 50 ft^3/s from step 1:

$$L_w = \left(\frac{1}{\sqrt{2g}\,C_w}\right)\frac{q_o}{h^{1.5}} = \left(\frac{1}{3.09(0.94)}\right)\frac{50}{(4.9)^{1.5}} = 1.6 \text{ ft}$$

Use an initial weir length of 1.6 ft. Figure 10.31 summaries the resulting stage-discharge curve.

Step 5. Perform routing to estimate peak outflow and maximum storage.

Route the inflow hydrograph with the storage-indication method introduced in Section 10.5. Figure 10.30 gives the results when the weir length is 1.6 ft. The peak outflow of 45 ft^3/s occurs at approximately 1.6 hours.

From Figure 10.31, a discharge of 45 ft^3/s occurs when the stage in the pond is approximately 4.5 ft. From Table 10.11, the storage at this stage is approximately 160,000 ft^3.

Table 10.11. Derivation of stage-storage relationship.

(1) Depth, h (ft)	(2) Surface Area, A (ft^2)	(3) Average Area (ft^2)	(4) Incremental Storage (ft^3)	(5) Storage Volume, V_s (ft^3)
0.00	33,500	-	-	0
-	-	33,650	16,825	-
0.50	33,800	-	-	16,825
-	-	34,050	17,025	-
1.00	34,300	-	-	33,850
-	-	34,450	17,225	-
1.50	34,600	-	-	51,075
-	-	34,950	17,475	-
2.00	35,300	-	-	68,550
-	-	35,500	17,750	-
2.50	35,700	-	-	86,300
-	-	35,950	17,975	-
3.00	36,200	-	-	104,275
-	-	36,500	18,250	-
3.50	36,800	-	-	122,525
-	-	37,150	18,575	-
4.00	37,500	-	-	141,100
-	-	37,750	18,875	-
4.50	38,000	-	-	159,975
-	-	38,400	19,200	-
5.00	38,800	-	-	179,175

Step 6. Confirm compliance with design criteria.

Since the computed peak with L = 1.6 ft is less than the allowable peak (50 ft^3/s), reduce the required storage volume by allowing a higher peak flow to leave the basin.

To accomplish this, increase the weir length to 2.0 ft and repeat the computations. Figure 10.31 summarizes the stage-discharge curve for the new weir length. Figure 10.30 shows the routed the inflow hydrograph with a resulting peak flow of 52 ft^3/s. This exceeds the allowable peak of 50 ft^3/s.

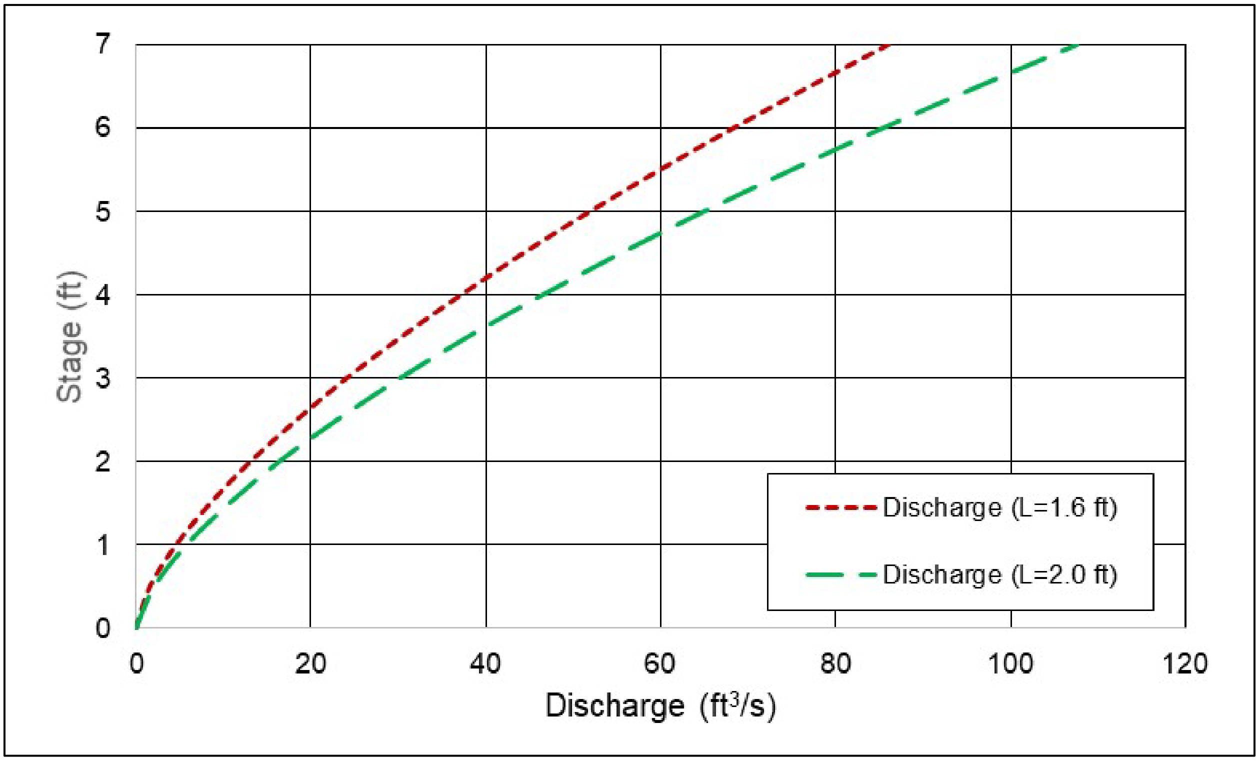


Figure 10.31. Stage-discharge curves for detention basin example with weir lengths of 1.6 and 2.0 ft.

Solution:	It is appropriate to evaluate whether an additional iteration on weir length is worthwhile, especially in the context of available constructed weir lengths. The 1.6 ft (0.5 m) length meets the design requirement; the 2.0 ft (0.61 m) length does not. If an intermediate length is constructible, further analysis may be warranted. If not, select the 1.6 ft (0.5 m) length.

Chapter 11 - Urban Stormwater Quality

This chapter provides an overview of urban stormwater quality practices specific to roadway applications. The purpose of an urban best management practice (BMP) is to mitigate the adverse impacts of development activity. BMPs can provide stormwater control benefits and pollutant removal capabilities. Evaluation of the available BMP options considers site-specific conditions and overall watershed management objectives. Often, requirements for water quality practices derive from those for the National Pollution Discharge Elimination System (NPDES) Program under the Clean Water Act as discussed in Chapter 2. [33 U.S.C. § 1342]. Local ordinances and regulations vary and may not require water quality practices in specific project locations. Early coordination with project environmental specialists assists in the recognition of water quality requirements and can facilitate identification and design of effective BMPs.

11.1 BMP Alternatives and Selection

Engineers consider several factors to determine the suitability of a particular BMP: physical conditions at the site, the watershed area, stormwater quantity objectives, and water quality objectives. Malhotra and Normann (1994) developed a matrix of site selection criteria for several BMPs and outlined site selection restrictions for each BMP. The Watershed-Based Stormwater Mitigation Toolbox (WBSMT), a spreadsheet-based planning level tool, uses nationally available geographic information systems (GIS) data to help State Departments of Transportation (DOTs) identify and prioritize potential stormwater runoff mitigation opportunities (NASEM 2017).

Schueler (1987) provided a comparative analysis of pollutant removal for various BMP designs. Generally, BMPs provide high pollutant removal for non-soluble particulate pollutants, such as suspended sediment and trace metals. BMPs typically achieve much lower rates for soluble pollutants, such as phosphorus and nitrogen.

Best Management Practices

In 1977, the Clean Water Act (CWA) introduced the concept and term, “best management practices” (BMPs) as an approach to controlling water pollution. BMPs manage stormwater by mitigating:

- Quantity – attenuate urbanized peak flows and store runoff volumes.
- Quality – reduce pollutant loads.
- Source – prevent or reduce the introduction of pollutants to stormwater with nonstructural measures.

An important parameter BMP designers consider is the runoff volume treated, often called the first flush volume or the water quality volume (WQ_V). This initial flush of runoff carries the most significant non-point pollutant loads. Methods to quantify the first flush or WQ_V vary. Most commonly, engineers estimate first flush volume as:

- The first 0.5 inch of runoff of impervious area.
- The first 0.5 inch of runoff of the entire catchment area.
- The first 1.0 inch of rainfall resulting in runoff from the entire catchment area.

In general terms, the greater the volume treated, the better the pollutant removal efficiency. However, treating volumes in excess of 1.0 inch of catchment area results in only minor improvements in pollutant removal efficiency (Schueler 1987).

In addition to Malhotra and Normann (1994) and Schueler (1987), researchers have created additional resources addressing BMP selection, performance, and evaluation of alternatives:

- *Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring* (FHWA 2000).
- *State of the Practice in Data Collection and Performance Measurement* (FHWA 2014a).
- *Stochastic Empirical Loading and Dilution Model (SELDM)* for evaluating the adverse effects of runoff from highway projects (Granato 2013).

11.2 Pollutant Loads

Estimated pollutant loadings for both pre- and post-development scenarios indicate the impact of highway development activities in a watershed. Several methods and models employ algorithms for pollutant loading estimation. The aptly named, empirical Simple Method applies to sites of less than 1 mi^2 (Schueler 1987). To yield an average annual loading estimate, L, the following equation multiplies an average pollutant concentration by the annual runoff.

$$L = c\,R\,C\,A \tag{11.1}$$

where:

L	=	Average annual loading, lb (chemical constituents) or billion colonies (bacteria)
c	=	Unit conversion factor, 0.226 (chemical) or 103 (bacteria)
R	=	Annual runoff (inch)
C	=	Pollutant concentration (mg/l) for chemical constituents
	=	Bacteria concentration (1,000/ml) for bacteria
A	=	Area (ac)

The FHWA developed a computer model that characterizes stormwater runoff pollutant loads from highways and predicts impacts to receiving water, specifically lakes and streams. The four-volume FHWA report *Pollutant Loadings and Impacts from Highway Stormwater Runoff* (Driscoll 1990) contains more detail on the estimating procedures.

More recently, the FHWA, in cooperation with the U.S. Geological Survey (USGS), developed the highway runoff database (HRDB) (FHWA 2009) and the SELDM to estimate and simulate stormflow volumes, concentrations, and loads of highway and urban runoff constituents (Granato 2013, USGS 2020). HRDB and SELDM provide data, tools, and techniques to help transform complex scientific data into meaningful information about the risk of adverse effects of runoff on receiving waters, the potential need for mitigation measures, and the potential effectiveness of such management measures for reducing these risks (Granato 2013, Granato 2014, Granato and Jones 2019).

Several other stormwater management software applications and tools can generate pollutant loads and the fate and transport of the pollutants:

- Stormwater Management Model (SWMM) (USEPA 2020).
- Storage, Treatment, Overflow, Runoff Model (STORM) (USACE 1977).
- Hydrologic Simulation Program, Fortran (HSPF) (USEPA 2002).
- Spreadsheet Tool for Estimating Pollutant Loads (STEPL) (USEPA 2018).

11.3 Structural BMPs

Structural BMPs consist of stormwater management facilities, created by moving earth, planting vegetation, or construction, or a combination of these elements. Storage and infiltration BMPs represent the two categories of structural BMPs.

11.3.1 Storage BMPs

Storage BMPs include extended detention dry ponds or wet ponds. They function primarily to store stormwater runoff and remove pollutants by promoting settling.

11.3.1.1 Extended Detention Dry Ponds

Following a storm event, extended detention dry ponds temporarily store a portion of stormwater runoff in depressed basins. These facilities typically store water for up to 48 hours following a storm by means of a hydraulic control structure to restrict outlet discharge. The extended detention of the stormwater provides an opportunity for urban pollutants carried by the flow to settle out. The water quality benefits of a detention dry pond increase by extending the detention time. If the pond retains stormwater for 24 hours or more, it can remove as much as 90 percent of particulates. However, extended detention only slightly reduces levels of soluble phosphorus and nitrogen found in urban runoff. Extended detention dry ponds typically do not have a permanent water pool between storm events.

Figure 11.1 shows the plan and profile views of an extended detention facility. In addition to the storage area, such facilities may include a stabilized low flow channel, an extended detention control device (riser with hood), and an emergency spillway.

Extended detention dry ponds reduce the frequency of erosive floods downstream, depending on the quantity of stormwater detained and the time over which stormwater is released. A cost-effective BMP, extended detention rarely involves construction costs more than 10 percent above those for conventional dry ponds that have shorter detention times and are used as a flood control device.

Extended detention dry ponds benefits may include creation of local wetland and wildlife habitat, limited protection of downstream aquatic habitat, and recreational use opportunities in the infrequently inundated portion of the pond. Negative impacts may include occasional nuisance and aesthetic problems in the inundated portion of the pond (e.g., odor, debris, and weeds); moderate to high routine maintenance requirements; and the eventual need for sediment removal. Extended detention generally applies to most new development situations and presents an attractive option for retrofitting existing dry and wet ponds.

11.3.1.2 Wet Ponds

A wet pond, or retention pond, serves the dual purpose of controlling the volume of stormwater runoff and treating the runoff for pollutant removal. Wet ponds store a permanent pool during dry weather. Hydraulic outlet devices designed to discharge flows at various elevations and peak flow rates release overflow from wet ponds. Figure 11.2 shows a plan and profile view of a typical wet pond and its components.

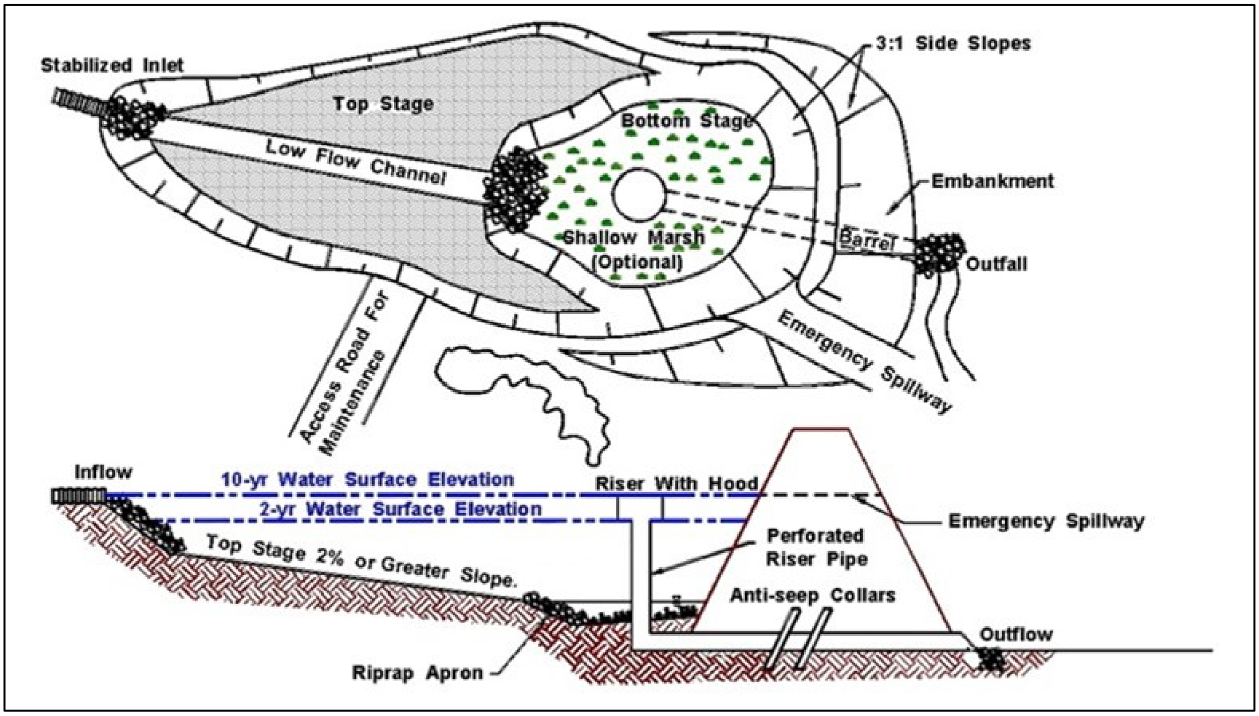


Figure 11.1. Extended detention pond.

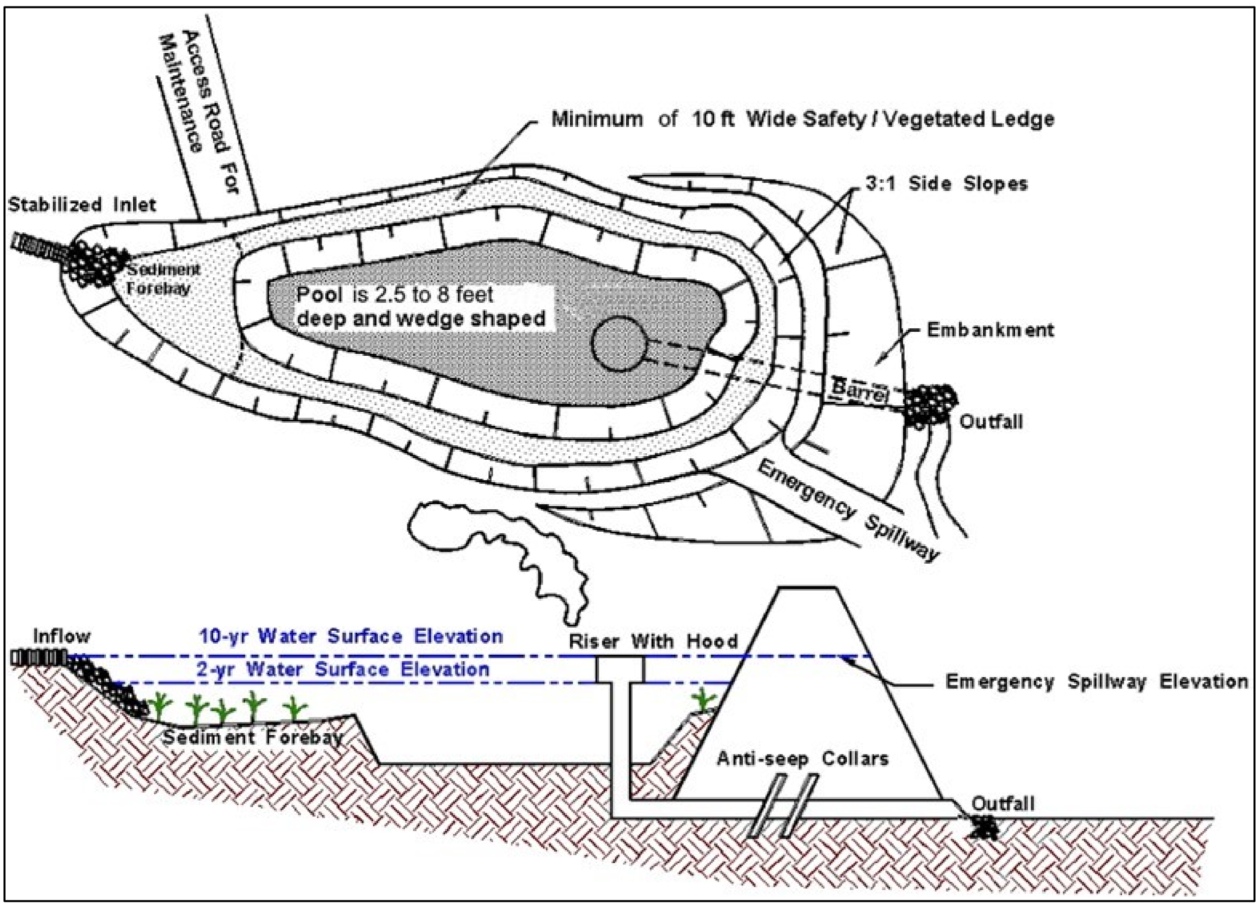


Figure 11.2. Typical wet pond schematic.

Wet ponds represent an effective water quality BMP. If properly sized and maintained, wet ponds can achieve a high removal rate of sediment, biological oxygen demand, organic nutrients, and trace metals. Biological processes within the pond also remove soluble nutrients (nitrate and ortho-phosphorus) that contribute to nutrient enrichment (eutrophication). Wet ponds are most cost-effective in larger, more intensively developed sites. Positive impacts of wet ponds can include creation of local wildlife habitat; higher property values; recreation; and landscape amenities. Negative impacts can include possible upstream and downstream habitat degradation; downstream sediment imbalance; potential safety hazards; occasional nuisance problems (e.g., odor, algae, and debris); and the eventual need for costly sediment removal.

11.3.2 Infiltration BMPs

Infiltration BMPs function primarily to remove pollutants by percolating stormwater runoff through soil strata. Common applications of infiltration BMPs include infiltration/exfiltration trenches, infiltration basins, and sand filters.

11.3.2.1 Infiltration/Exfiltration Trenches

Infiltration trenches are shallow excavations which have been backfilled with a coarse stone media. The trench forms an underground reservoir which collects runoff and either exfiltrates it to the subsoil or diverts it to an outflow facility. The trenches primarily serve as a BMP providing moderate to high removal of fine particulates and soluble pollutants, but they also serve to reduce peak flows. Infiltration trenches only work with permeable soils and where the seasonal groundwater table is below the bottom of the trench. Figure 11.3 shows an example of surface trench design (Schueler 1987). Engineers frequently use infiltration trenches for highway median strips and parking lot “islands” (depressions between two lots or adjacent sides of one lot). The components of an infiltration trench can include backfill material, observation wells, permeable filter, overflow pipes, emergency overflow berms, and a vegetated buffer strip.

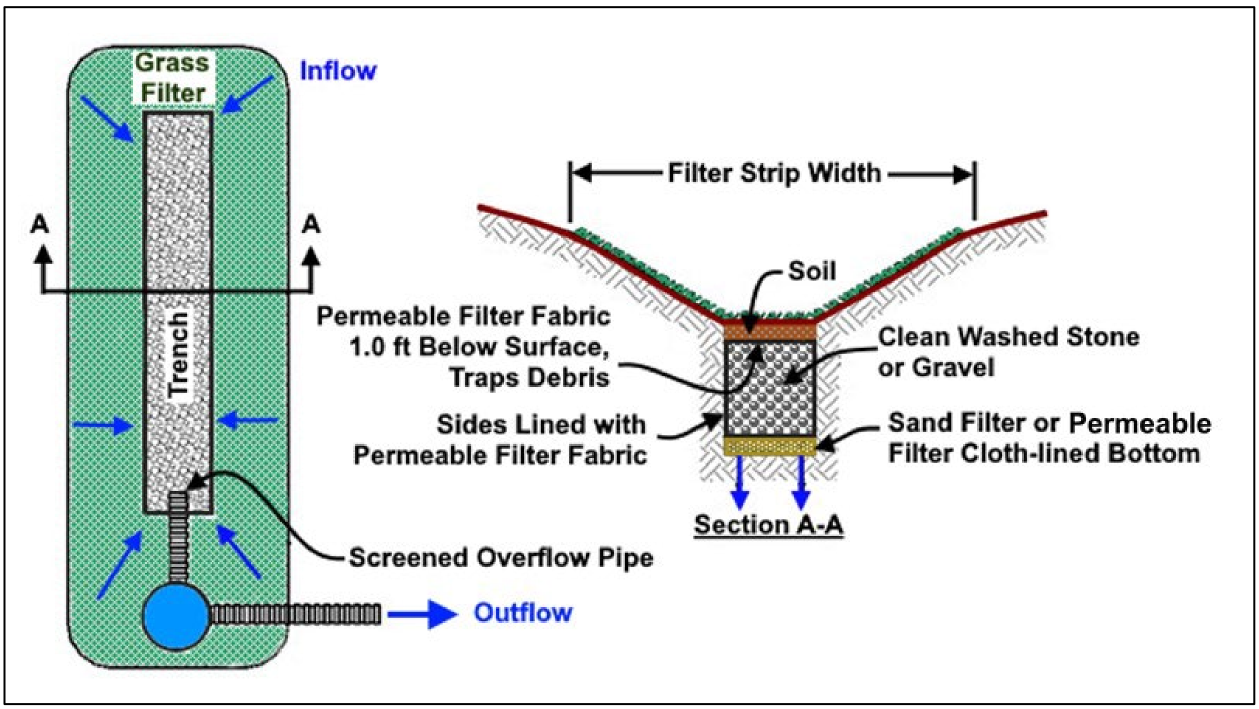


Figure 11.3. Median strip trench design.

Advantages of infiltration trenches include preservation of the natural groundwater recharge capabilities of a site and the relative ease of fitting them into the margins, perimeters, and other unused areas of a development site. Few other BMPs offer pollutant removal on small sites or infill developments.

The disadvantages associated with infiltration trenches include practical difficulties in keeping sediment out of the structure during site construction (particularly if development occurs in phases), the need for careful construction of the trench and regular maintenance thereafter, and a possible risk of groundwater contamination. Frequent clogging involves routine maintenance, which has a cost impact. Designers may wish to examine the limitations, risks, and benefits of infiltration BMPs in the context of the built and natural environments (e.g., surface water, groundwater, soils, and infrastructure) (NASEM 2019).

There are three basic trench systems: complete exfiltration, partial exfiltration, and water quality exfiltration. In a **complete exfiltration system**, water can exit the trench only by passing through the stone reservoir and into the underlying soils (i.e., it includes no positive pipe outlet from the trench). As a result, the stone reservoir must be large enough to accommodate the entire expected design runoff volume, less any runoff volume lost via exfiltration during the storm. The complete exfiltration system provides total peak flow, volume, and water quality control for all rainfall events less than or equal to the design storm. To handle any excess runoff from storms greater than the design storm, a rudimentary overflow channel, such as a shallow berm or dike, may be included.

A **partial exfiltration system** offers an alternative where complete reliance on exfiltration to dispose of runoff may not be possible or prudent. For example, designers may be concerned about the long-term permeability of the underlying soils, downstream seepage, or clogging at the interface between the filter fabric and subsoil. To address this, a partial exfiltration system includes a perforated pipe to collect water and direct it to an outlet.

If the designer locates the perforated pipe as an underdrain at the bottom of the trench, the underdrain will collect and convey a high percentage of the water that has not yet exfiltrated from the trench. As a result, these designs may only act as a short-term underground detention system. Together, the low exfiltration rates and short residence times can result in poor pollutant removal and hydrologic control.

Engineers can improve the performance of partial exfiltration systems during smaller storms by including a perforated pipe near the top of the trench. In this design, runoff will not exit the trench through the pipe until it rises to the level of the outlet pipe. Storms with less volume than the design storm may never fill the trench to this level and will be subject to complete exfiltration.

In either design, engineers can analyze the passage of the inflow hydrograph through the trench with hydrograph routing procedures to determine the appropriate sizing of the trench. Due to storage and timing effects, partial exfiltration trenches will be smaller in size than full exfiltration trenches serving the same site.

For a **Water Quality Exfiltration System**, engineers design the storage volume of a water quality trench to receive only the first flush of runoff volume during a storm. As discussed in Section 11.1, engineers quantify first flush volume variously according to local practice. The trench does not treat the remaining runoff volume, and instead conveys it to a conventional detention or retention facility downstream. Engineers estimate the water quality volume using:

$$WQ_V = Q\,A = (R_V\,P)A \tag{11.2}$$

where:

WQ_v	=	Water quality volume
Q	=	Depth of runoff, inch (mm)
R_V	=	Volumetric runoff coefficient
P	=	Rainfall depth, inch (mm)
A	=	Drainage area, ac (ha)

While water quality exfiltration systems do not typically satisfy stormwater storage goals, they may result in smaller, less costly facilities downstream. The smaller size and area requirements of water quality exfiltration systems allows considerable flexibility in their placement within a development site, an important factor for "tight" sites. Additionally, if for some reason the water quality trench fails, a downstream stormwater management facility may still adequately control stormwater.

11.3.2.2 Infiltration Basins

An infiltration basin impounds stormwater flow and gradually exfiltrates it through the floor of an excavated area. Infiltration basins have a similar appearance and construction to conventional dry ponds. However, the detained runoff exfiltrates though permeable soils beneath the basin, removing both fine and soluble pollutants. Engineers may design infiltration basins as combined exfiltration/detention facilities or as simple infiltration basins. They can be adapted to provide stormwater management functions by attenuating peak flows from large design storms and can serve drainage areas up to 50 ac. Figure 11.4 shows a plan and profile schematic of an infiltration basin and its components.

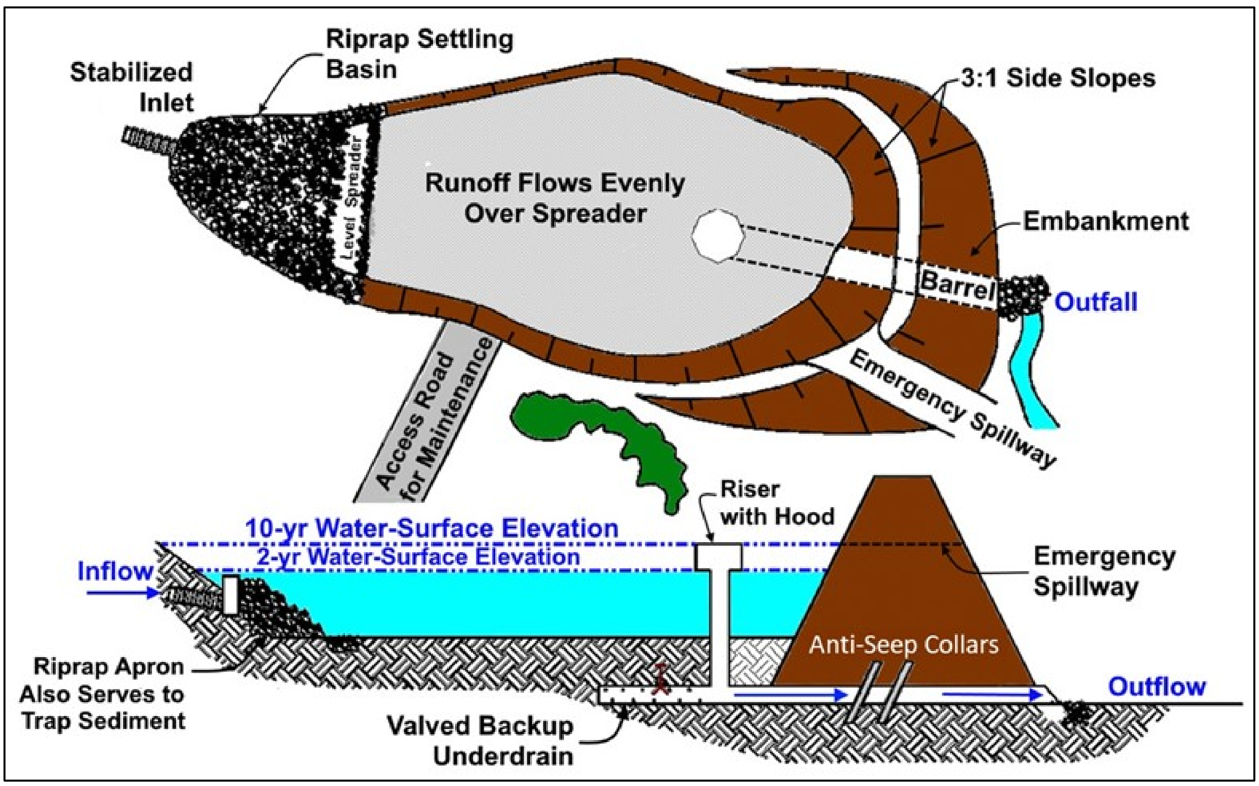


Figure 11.4. Infiltration basin schematic.

Infiltration basins present a feasible option where soils are permeable, and where the water table and bedrock are situated well below the soil surface. They have similar construction costs and maintenance requirements to those for conventional dry ponds. Experience to date indicates that infiltration basins have one of the highest failure rates of any BMP from plugging of the permeable soils, emphasizing the importance of regular inspection for standing water.

Advantages of infiltration basins include:

- Preserving the natural water balance of the site.
- Serving larger developments.
- Usefulness as sediment basins during the construction phase.
- Reasonable cost-effectiveness in comparison with other BMPs.

Disadvantages of infiltration basins include:

- High rate of failure due to unsuitable soils.
- Need for frequent maintenance.
- Frequent nuisance problems (e.g., odors, mosquitoes, soggy ground).

11.3.2.3 Sand Filters

Sand filters provide stormwater treatment for first flush runoff. Runoff filters through a sand bed before returning to a stream or channel. Engineers generally use sand filters in urban areas and for groundwater protection where infiltration into soils is not feasible. Alternative designs of sand filters use a top layer of peat or some form of grass cover through which runoff passes before being strained through the sand layer. This combination of layers increases pollutant removal. Effective BMPs such as bioretention and rain gardens include sand filters as a component (USEPA 2021).

Engineers use a variety of sand filter designs. Figure 11.5 and Figure 11.6 present examples of the two general types of filter systems. Figure 11.5 shows a cross-section schematic of a sand filter compartment (City of Alexandria 1992), and Figure 11.6 shows a cross-section schematic of a peat-sand filter (Galli 1990).

Sand filters have the advantage of being adaptable. They can be used on areas with thin soils, high evaporation rates, low soil infiltration rates, and limited space. Sand filters also have high removal rates for sediment and trace metals and have a very low failure rate. Disadvantages associated with sand filters include frequent maintenance to ensure proper operation, unattractive surfaces, and odor problems.

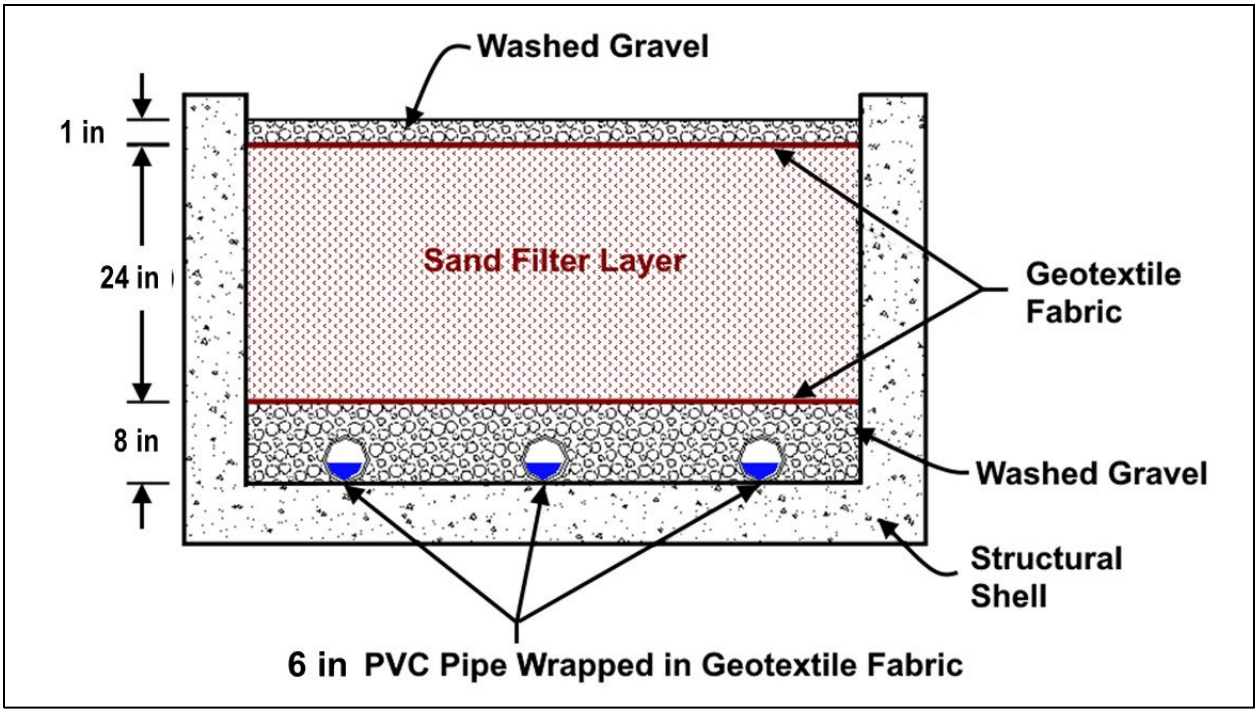


Figure 11.5. Cross-section schematic of sand filter compartment.

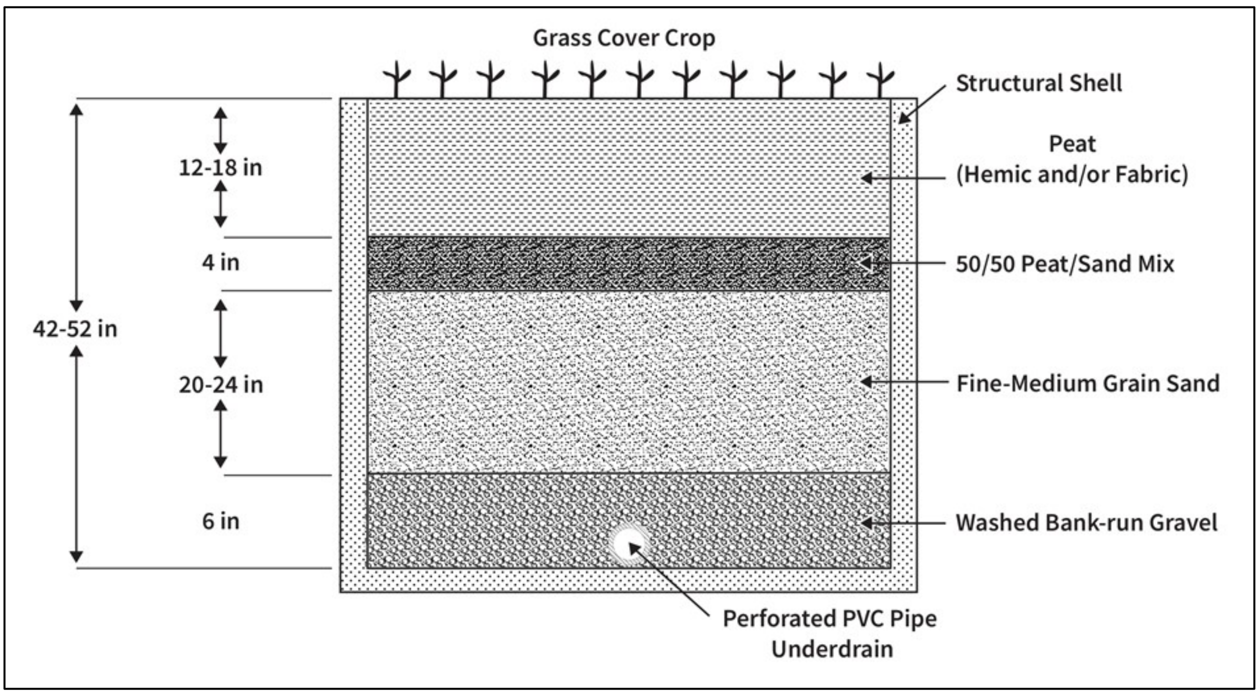


Figure 11.6. Cross-section schematic of peat-sand filter.

11.4 Green Infrastructure

Green infrastructure involves stormwater management designed to capture rainwater near where it falls, largely by slowing runoff and promoting infiltration in ways that mimic natural runoff and infiltration. Examples of green infrastructure include green roofs, rain gardens, grass paver parking lots, infiltration trenches, permeable pavements, bioswales, planter boxes and rainwater harvesting.

Gray Infrastructure

In contrast to green infrastructure, gray infrastructure includes approaches for stormwater collection, storage, and conveyance designed to move rainwater away from impervious surfaces, such as roadways, parking lots and rooftops, as quickly as possible. It may include curbs, gutters, drains, piping, and collection systems.

Green infrastructure represents a resilient nature-based solution that manages wet weather impacts and provide ecological, economic, and social benefits to the affected community. The International Union for Conservation of Nature (IUCN 2020) describes "nature-based solutions" (NBS) as "actions to protect, sustainably manage, and restore natural or modified ecosystems that address societal challenges effectively and adaptively, simultaneously providing for human well-being and biodiversity benefits."

Green infrastructure may also be called natural infrastructure. Section 11103 of BIL added a definition of natural infrastructure under Section 101 of Title 23 of U.S. Code as follows:

> The term "natural infrastructure" means infrastructure that uses, restores, or emulates natural ecological processes and —
>
> (A) is created through the action of natural physical, geological, biological, and chemical processes over time;
>
> (B) is created by human design, engineering, and construction to emulate or act in concert with natural processes; or
>
> (C) involves the use of plants, soils, and other natural features, including through the creation, restoration, or preservation of vegetated areas using materials appropriate to the region to manage stormwater and runoff, to attenuate flooding and storm surges, and for other related purposes.

Executive Order 13690 "Establishing a Federal Flood Risk Management Standard and a Process for Further Soliciting and Considering Stakeholder Input" also promotes such NBS and natural infrastructure by requiring agencies, where possible, to use natural systems, ecosystem processes, and nature-based approaches when developing alternatives for consideration.'' (80 FR 13690 (Jan. 30, 2015), revoked by EO 13807 (Aug. 15, 2017), but reinstated by EO 14030 (May 20, 2021)). Green infrastructure (NBS and natural infrastructure) is important to consider as FHWA and others seek to ensure the transportation network is resilient in the face of the risk associated with climate change.

11.4.1 Low Impact Development

Low impact development (LID) is a decentralized source and treatment control strategy for stormwater management (NASEM 2006). LID establishes a stormwater management system to prevent pollution resulting from development and urbanization. It mimics natural processes to maintain the hydrological processes of infiltration, interception, and evapotranspiration that existed before development. LID strategies come in many forms including:

- Bioretention area (rain garden): Retains, infiltrates, and filters runoff and pollutants using a shallow surface depression usually planted with native vegetation. See Figure 11.7.
- Bioretention swales: Also referred to as bioswales or vegetated swales, consist of typically parabolic or trapezoidal depressions that use bioretention soil media and vegetation to promote infiltration, water retention, and sedimentation and pollutant removal. See Figure 11.8.
- Stormwater curb extensions: Also called stormwater bump outs, extend the curb into the roadway to reduce traffic speed and capture stormwater runoff from roadways and sidewalks.
- Stormwater planters: Consist of narrow, flat-bottomed landscape areas, typically of rectangular shape with vertical walls.
- Stormwater tree systems (i.e., pits and trenches): Intercept and capture stormwater using a tree or shrub, bioretention soil media, and a gravel reservoir.
- Infiltration trenches: Excavated linear areas filled with layers of stone and sand wrapped in geotextile fabric. See Figure 11.9.
- Subsurface infiltration and detention practices: Underground storage that holds runoff and allows infiltration.
- Permeable pavements: Includes porous asphalt (pavement) and pavers that allow runoff to infiltrate through void space instead of becoming surface runoff. See Figure 11.10.

Figure 11.7. Green infrastructure practice: bioretention (rain garden). Source: Roger Kilgore.

Figure 11.8. Green infrastructure practice: bioswale (USEPA 2021).

Figure 11.9. Green infrastructure practice: infiltration trench (USEPA 2021).

Figure 11.10. Green infrastructure practice: permeable pavers (USEPA 2021).

Table 11.1 summarizes the relative effectiveness (based on pollutant removal efficiencies) of properly maintained green infrastructure practices for various water quality constituents that roadway runoff typically produces in high concentrations.

Table 11.1. Pollutant removal efficiencies of green infrastructure practices (USEPA 2021).

Green Infrastructure Practice	Pollutant Removal Efficiency *						
	Total Suspended Solids	Total Nitrogen	Total Phosphorus	Fecal Coliform	Total Zinc	Total Copper	Total Lead
Bioretention (Rain Garden)	●	○	●	–	●	–	⦿
Bioswale	⦿	○	○	○	–	–	–
Stormwater Curb Extension	●	○	●	–	●	–	⦿
Stormwater Planter	●	○	●	–	●	–	⦿
Street Trees	●	⦿	⦿	●	⦿	⦿	⦿
Infiltration Trench	●	○	●	●	●	–	–
Subsurface Infiltration and Detention	●	⦿	⦿	●	●	●	●
Permeable Pavement	●	–	⦿	–	●	⦿	⦿
Permeable Friction Course	●	–	–	–	●	⦿	●

*○ = 0 – 30%; ⦿ = 31 – 65%; ● = >65%; – = no data.

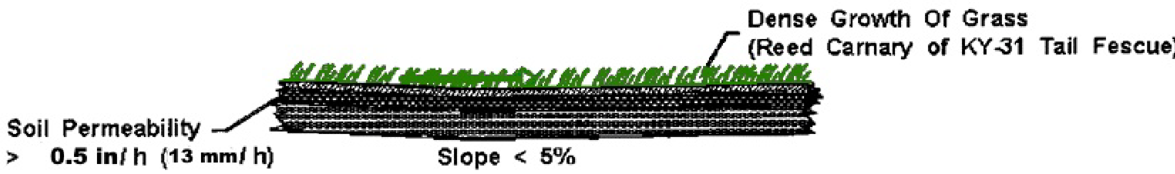
11.4.2 Grassed Swales

Engineers typically use grassed swales in developments and highway medians as an alternative to curb and gutter drainage systems. Swales have a limited capacity to accept runoff from large design storms, and often lead into storm drain inlets to prevent large, concentrated flows from gullying/eroding the swale. HEC-15 (FHWA 2005) provides information for the design of grassed swales.

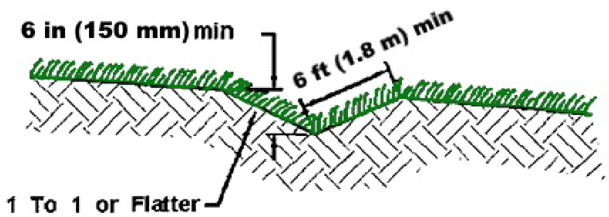
Grassed swales sometimes incorporate check dams and level spreaders. Level spreaders consist of excavated depressions running perpendicular across the swale. Engineers incorporate level spreaders and check dams into a swale design to reduce overland runoff velocities. Figure 11.11 shows a schematic of a grassed swale level spreader and check dam. Swales with check dams placed across the flow path can provide some stormwater management for small design storms by infiltration and flow attenuation. In most cases, however, engineers combine swales with other BMPs downstream to meet stormwater management requirements.

Swales can filter out particulate pollutants under certain site conditions. However, swales generally cannot remove soluble pollutants, such as nutrients. In some cases, trace metals leached from culverts and nutrients leached from lawn fertilization may increase the export of soluble pollutants. Grassed swales usually cost less than the curb and gutter.

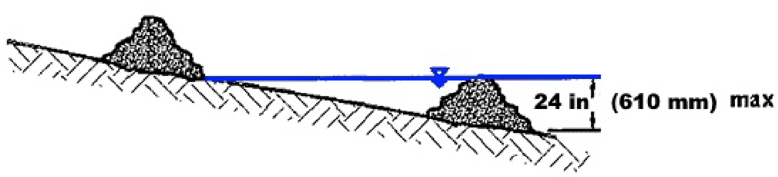
In addition to being conveyors of stormwater, grassed swales can also function as biofilters in the management of the quality of stormwater runoff from roads. Swales designed to increase hydraulic residence to promote biofiltration are known as biofiltration swales as discussed in the previous section. Biofiltration swales take advantage of filtration, infiltration, adsorption, and biological uptakes as runoff flows over and through vegetation. Removal of pollutants by a biofiltration swale depends on the time that water remains in the swale, or the hydraulic residence time, and the extent of its contact with vegetation and soil surfaces. The Washington State Department of Transportation *Highway Runoff Manual* contains information on biofiltration swales (WSDOT 2019).



a. Grassy Swale – Slows runoff; filters sediment; roots bind surface of soil and enhance infiltration; grass and leaves protect soil surface from rainfall impact; and litter layer improves porosity.



b. Level Spreader – Redistributes concentrated flow to sheet flow, increases infiltration.



c. Check Dam – Reduces erosive velocities.

Figure 11.11. Schematic of grassed swale level spreader and check dam.

11.4.3 Filter Strips

Filter strips have many similarities to grassed swales, but only accept overland sheet flow. To function, runoff from an adjacent impervious area must be evenly distributed across the filter strips. Runoff has a strong tendency to concentrate and form a channel, effectively "short-circuiting" the filter strip so that it does not perform as designed.

A properly functioning filter strip has: 1) some sort of level spreading device; 2) dense vegetation with a mix of erosion resistant plant species that effectively bind the soil; 3) grading to a uniform, even, and relatively low slope; and 4) length at least equal to the contributing runoff area. HEC-15 has information on permissible shear stresses (erosion resistance) of various types of vegetation (FHWA 2005). Filter strips built using HEC-15 can remove a high percentage of particulate pollutants. Little research exists on the capability of filter strips in removing soluble pollutants. Filter strips cost relatively little to establish and almost nothing if preserved before site development. A creatively landscaped filter strip can become a valuable community amenity, providing wildlife habitat, screening, and stream protection. Engineers also commonly use grass filter strips to protect surface infiltration trenches from clogging by sediment.

Filter strips do not provide storage or infiltration to effectively reduce peak flows. Typically, filter strips make up one part of an integrated stormwater management system. Thus, the strips can lower runoff velocity (and, consequently, the watershed time of concentration), slightly reduce both runoff volume and watershed imperviousness, and contribute to groundwater recharge. Filter strips also provide important benefits of preserving the riparian zone and stabilizing streambanks.

11.4.4 Constructed Wetlands

Wetlands can efficiently remove pollutants from highway and urban runoff. Engineers often use wetlands or shallow marshes in conjunction with other BMPs to achieve maximum pollutant removal. Studies on wetlands concluded that detention basins and wetlands appear to function comparably well in removing monitored pollutants (Strecker et al. 1992), but for some indicators wetlands perform better (NASEM 2017). An effective design of a wetland as a water quality measure would include the creation of a detention basin upstream of the wetland. The detention basin provides an area where heavy particulate matter can settle, thus minimizing disturbance of the wetland soils and vegetation.

Frequently, engineers design wetlands for BMP purposes in conjunction with wet pond sites, provided that the runoff passing through the vegetation does not dislodge the aquatic vegetation (AASHTO 2007). Vegetation systems may not be effective where the water's edge is extremely unstable or heavily used. Additionally, in flood-prone areas, the alteration of the hydraulic characteristics of the watercourse may cause some types of marsh vegetation to be ineffective as a BMP.

11.5 Ultra-Urban BMPs

Densely developed areas with very limited right-of-way, termed "ultra-urban" environments, present a unique challenge for the use of traditional treatment BMPs given the lack of available surface area. The term "ultra-urban BMPs" generally describes the use of treatment BMPs installed underground and resulting in small footprints (NASEM 2012). These methods capture runoff contaminants before they reach surface water and groundwater. They apply particularly well to retrofitting urban areas, as well as to new urban development.

These engineered devices are typically structural and are made on a production line in a factory. Engineers may design them to handle a range of pollutant and water quantity conditions and, because they have small footprints they may be dropped into the urban infrastructure or integrated into the streetscape of both private and public sector property. Others may be installed beneath parking lots and garages or on rooftops. Engineers design still others to remove pollutants before they are flushed into urban runoff collection systems.

Pre-cast storm drain inlets, or "water quality inlets," remove sediment, oil and grease, and large particulates from runoff originating from paved surfaces before it reaches storm drainage systems or infiltration BMPs. Water quality inlets typically serve highway storm drainage facilities adjacent to commercial sites generating large amounts of vehicle wastes, such as gas stations, vehicle repair facilities, and loading areas. They may pretreat runoff before it enters an underground filter system.

These inlets can include a three-chamber underground retention system designed to settle out grit and absorbed hydrocarbons as shown in Figure 11.12: a sediment trapping chamber, an oil separation chamber, and the final chamber attached to the outlet. The sediment trapping chamber settles out grit and sediment and traps floating debris in a permanent pool. An orifice protected by a trash rack connects this chamber to the oil separation chamber. The oil separation chamber also maintains a permanent pool of water. An inverted elbow connects the separation chamber to the third chamber.

Advantages of water quality inlets include compatibility with the storm drain network, easy to access, able to pretreat runoff before it enters infiltration BMPs, and unobtrusive. Disadvantages include their limited stormwater and pollutant removal capabilities, frequent cleaning (which cannot always be assured), the possible difficulties in disposing of accumulated sediments, and cost.

Other water quality ultra-urban applications, include filter inserts, hydrodynamic devices, and simple sumps. Bag or basket type filter inserts have small openings to allow low flows to seep through and larger flows to overflow without causing backwater to the inlet. Hydrodynamic devices typically include baffles, vortex mechanisms, or other settling components, or a combination of these elements. These devices separate sediment and pollutants from stormwater and commonly are inserted between inlets and storm drain pipes. Sumps placed at the bottom of access holes and below the storm drain pipe flow lines allow sediment and debris to deposit while releasing stormwater through weep holes.

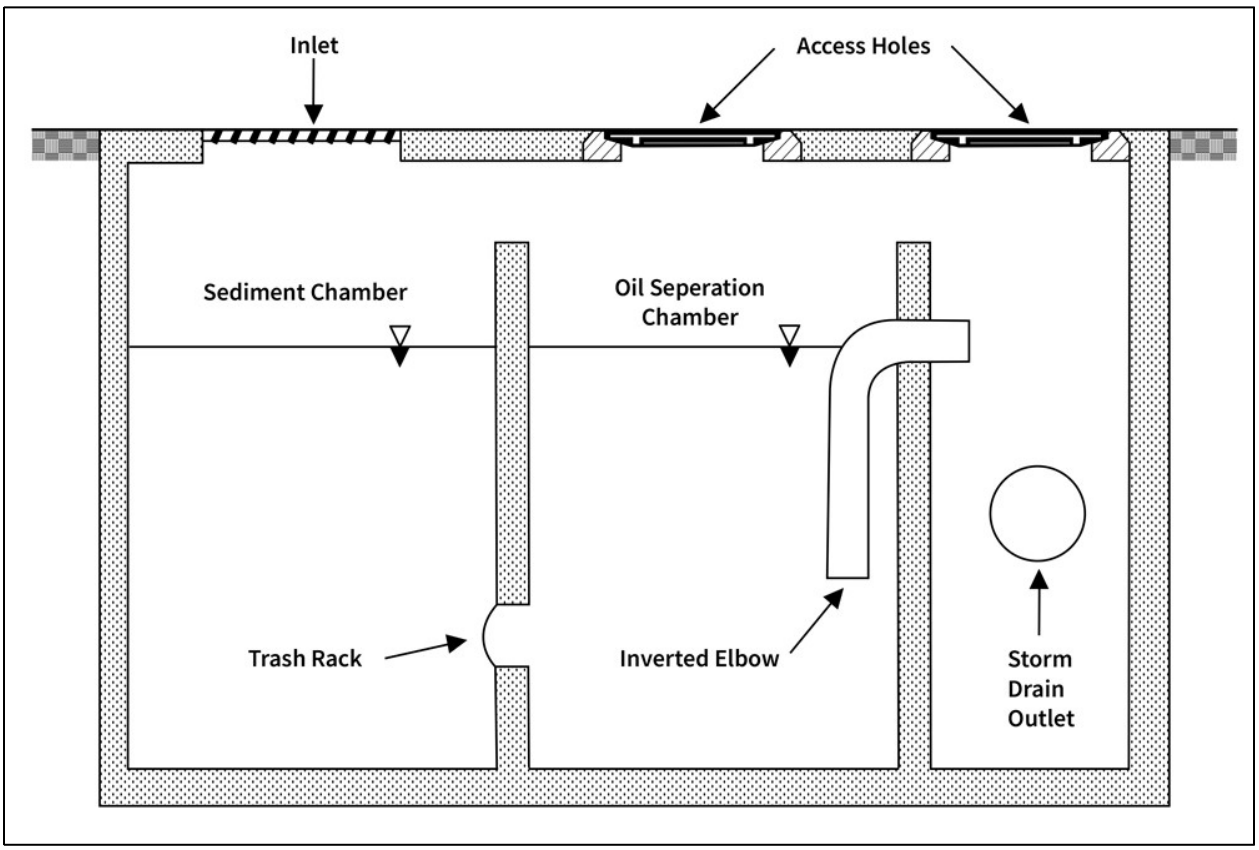


Figure 11.12. Cross-section detail of a typical oil/grit separator.

11.6 Non-Structural BMPs

Drainage engineers and planners commonly use non-structural BMPs in conjunction with structural BMPs, particularly as a means of pre-treating runoff before retention, detention, storage, or discharge. Non-structural BMPs do not involve grading or construction, and therefore cost significantly less than structural controls. In contrast to structural BMPs, which treat or remove pollutants, non-structural BMPs focus primarily on the avoidance of pollution. Governmental agencies and other organizations typically implement non-structural BMPs by establishing ordinances and administrative policies, and through watershed planning. Some of the most common non-structural BMPs are (NASEM 2014):

- **Storm drain cleaning**: Removal of sediment and debris from storm drain pipes and inlets. Has a minor effect on water quality improvement because most pollutants, notably nutrients, tend to pass through the drainage inlets and catch basins. This BMP most efficiently removes suspended solids.
- **Street sweeping**: Removal of sediment from paved surfaces. Has modest water quality benefits, in particular in the removal of sediment, debris, and trash/litter.
- **Efficient landscaping practices**: Minimizing or eliminating the use of pollutants in landscaping (e.g., fertilizers) and avoiding excessive irrigation.
- **Trash management practices**: Minimizing public littering and minimizing windblown trash from vehicles and landfills.
- **Elimination of groundwater inflow to storm drains**: Use of watertight joints and the elevation of pipes above the groundwater table. Storm drains that are below the groundwater table can perennially convey low flows with concentrations of pollutants.
- **Slope and channel stabilization**: Vegetating, lining, or reconfiguring embankment slopes and channels to reduce erosion.
- **Winter maintenance:** Proper use of deicing chemicals and abrasives to alleviate winter conditions and post-winter cleanup.
- **Irrigation runoff reduction practices**: Maintain landscaped areas and reduce overwatering of landscapes to mitigate excess runoff and associated high concentrations of pollutants, typically during the dry weather season.

Chapter 12 - Pump Stations

Stormwater pump stations move stormwater from highway sections where elevation and topography prohibit gravity flow. Compared with gravity drainage, pump stations involve high life-cycle costs and present several potential design and operational challenges discussed in this chapter. Therefore, designers only consider stormwater pump stations where gravity flow systems are not feasible. Gravity alternatives to pump stations include deep conduit trenches, tunnels, siphons, and groundwater recharge basins, although recharge basins are often aesthetically unpleasing and can create maintenance problems. This chapter provides on overview of stormwater pump stations for highway applications. The FHWA publication *Highway Stormwater Pump Station Design* (HEC-24) provides more in-depth information (FHWA 2001). The Hydraulic Institute also prepared several publications which provide information for the successful design of pump stations (Hydraulic Institute n.d., AASHTO 2014).

Design of stormwater pump stations can differ from design of pump stations for other applications. For example, stormwater management requirements from an applicable jurisdiction may limit the maximum discharge from stormwater pump stations. Designers often meet such requirements by providing additional storage as discussed in Section 12.4.

Unusual Features

Pump stations are some of the more unusual and complex features that may be encountered in highway drainage design. Their design involves knowledge of electrical, mechanical, and control systems as well as building design and construction that may be unfamiliar to most highway and drainage designers. The drainage designer will likely call on designers of other disciplines to assist with the complete design of a pump station.

Designers make many stormwater pump station design decisions based on engineering judgment and experience. To enhance cost effectiveness, the designer may compare annual or life-cycle costs of alternatives. The decision to install a pump station involves long-term commitment of funds and personnel. This chapter provides information to minimize construction, operation, and maintenance costs of highway stormwater pump stations, while remaining consistent with the functional goals for the pump stations.

12.1 Pump Station Types and Pumps

Designers choose from several stormwater pump station types and numerous pump types. Selection of both depends on the operational requirements of the pump station, the number and size of pumps, hydraulic conditions, and frequency of operation.

12.1.1 Station Types

Designers can categorize pump stations as wet-pit or dry-pit. In **wet-pit stations**, the pumps are submerged in a **wet well** or sump, with the motors and the controls located overhead. With this design, stormwater arriving in the wet well is pumped vertically from the well through a “riser” pipe. Commonly, the motor connects to the pump by a drive shaft located in the center of the riser pipe.

Another type of wet-pit design involves using submersible pumps. A submersible pump commonly involves less maintenance because it does not use a long drive shaft. Submersible pumps also allow for convenient maintenance in wet-pit stations because the pumps may be removed relatively easily. Submersible pumps come in many sizes and have many applications. Rail systems are available which allow removal of pumps without entering the wet well.

Dry-pit stations consist of two separate elements: a storage box or wet well and a dry well. Stormwater arrives in the wet well, which is connected to the dry well by a horizontal suction pipe. The dry well contains the stormwater pumps. Designers often use radial flow pumps in this configuration. Either motors mounted in the dry well or drive shafts with overhead motors may provide power.

Since dry-pit stations cost more than wet-pit stations, designers most often use wet-pit stations. The hazards associated with pumping stormwater usually do not warrant the added expense of dry-pit stations, and available space within the highway right-of-way may be a limiting factor. However, dry-pit stations offer some advantages, including ease of access for repair and maintenance, the protection of equipment from fire and explosion, and adaptability for storage volume.

For both wet-pit and dry-pit stations, the station depth influences the cost and functionality of the pump station. Engineers minimize station depth with designs that use only the depth that will allow pump submergence and hydraulically necessary clearance below the inlet invert. HEC-24 (FHWA 2001) provides additional information on station types.

12.1.2 Pump Types

One or more pumps provide the capacity to move water from a lower to higher elevation for discharge. The most common stormwater pump types are rotary pumps of the **axial flow**, **radial flow,** or **mixed flow** types.

Axial flow pumps move water in the direction along the axis of rotation of the pump. The impeller of an axial flow pump usually looks like the propeller on a boat or a ship. These pumps operate in open water rather than within a confined space. Axial flow pumps perform best where they can move large volumes of fluid against relatively low head.

Axial flow pumps lift the water up a vertical riser pipe; water flows parallel to the pump axis and drive shaft. Designers commonly use axial flow pumps for low head, high discharge applications. Axial flow pumps do not handle debris particularly well because the propellers may be damaged if they strike a relatively large, hard object. Also, fibrous material will wrap itself around the propellers.

Modern Pumps and Controls

Many pumps have synchronous, AC electric motors and simple on/off switches. Modern technology allows the consideration of variable-speed pump motors with computer control in place of the traditional type. The pumps themselves still fall within the same classifications (axial, radial, or mixed-flow), but variable motor speed may allow pumping rate to vary with inflow rate and needed outflow rate, controlled by a computer, based on sensor information.

Radial flow pumps take water into the pump casing in the direction of the pump's axis of rotation. The impeller then changes the direction of the water's movement by "flinging" the water outward from the inlet direction, perpendicular to the impeller's axis of rotation, and imparting rotational motion in the direction of the impeller's rotation. The pump case is scroll-shaped, and water leaves the pump perpendicular to, and offset from, the impeller's axis of rotation, with greatly increased energy head.

Radial flow pumps use centrifugal force to increase head and move water up the riser pipe. They will perform in any range of head and discharge but perform best for high head applications. Some forms of radial flow pumps handle debris quite well. A single vane, open configured impeller handles debris best because it provides the least interference with the passage of debris through the pump. The debris handling capability decreases the number of vanes, since the size of the openings decreases.

Mixed flow pumps represent a physical transition from axial flow to radial flow and have some attributes of each. Unlike with an axial flow pump, inside a mixed flow pump, the water flow direction changes from along the impeller's axis of rotation to some angle away from that axis. But, unlike in a radial flow pump, the change in direction is not perpendicular to the axis of rotation. As in a radial flow pump, the impeller "flings" the water outward and adds energy, but the pump case then redirects the water back along the axis of rotation.

Getting the Right Pump

The procurement process in some jurisdictions may not allow the designer to specify a particular brand, type, or style of pump or pump motor for reasons of fairness and competition. The designer of a pump station may find it necessary to specify certain aspects of pump performance within acceptable ranges, allowing contractors bidding on projects to select equipment within that range, with the agency retaining the right of approval of shop drawings after bidding and award of a contract and before purchase of the equipment. For such reasons, the final configuration and equipment of a pump station may differ from the initial design.

Very often, mixed flow pumps are multi-stage. This design stacks together several impellers inside of several cases and drives them by a common shaft. Water passes through the impellers progressively, with each stage imparting more energy to the water. The impellers of a mixed flow pump can be designed to shed and pass debris better than an axial flow pump. Mixed flow pumps work best for intermediate head and discharge applications. Because they are easily configurable for multiple stages, and because of the physical configuration of mixed flow pumps, most submersible pumps are of this type.

All pumps can use motors or engines housed overhead or in a dry well, or submersible motors located in a wet well. Submersible pumps frequently provide the advantages of simplified design, construction, and maintenance and, therefore, lower associated cost. Designers rarely use anything other than a constant speed, single suction pump.

12.1.3 Pump Selection and Sizing

Designers select pumps by establishing criteria, characterizing operating requirements, and then selecting a combination that meets the design criteria. They consider cost, reliability, and operating and maintenance requirements. Because stormwater pump stations have relatively short annual operating periods, initial cost usually influences selection more than operating costs. Designers typically minimize initial pump costs by providing as much storage as possible.

The designer can obtain an approximate range of pump and motor sizes for consideration by reviewing the design of existing pump stations, pumps, and their performance curves. Pump manufacturer information provides further information on the performance characteristics of available pumps. The designer can narrow down pump type and size by considering pump **specific speed** because each pump type performs best in certain ranges of specific speed.

12.1.3.1 System Curve

Designers select and size pumps by referring to the system requirements expressed in the form of a system curve. The system curve relates the head required of the pump station as a function of discharge as shown in Figure 12.1. Since changes in head influence pump performance, designers calculate the head required as accurately as possible including all "minor losses" attributed to valves and bends. Designers can minimize these various head losses by carefully selecting discharge line size and other components such as check valves and gate valves.

Designers select the discharge pipe size by considering the manufactured pump outlet size, either matching the outlet size or, to reduce the loss in the line, by using a pipe larger than the outlet. Generally, this approach allows the designer to identify a reasonable compromise in balancing cost but would involve inclusion of an expansion loss in the calculations.

The static head represents the vertical lift required of the pump station, that is, the difference between the head at the pump station outlet and inlet. It varies depending on the water levels in the storage at the inlet and may also vary if the outlet water surface elevations fluctuate.

The total head required of the pump station combines static head, friction head, velocity head, and minor losses (through fittings, values, pipe expansions, pipe contractions, etc.). This quantity is called total dynamic head (TDH). It is dynamic because, except for static head, TDH increases with flow. Designers compute TDH as:

$$TDH = H_s + H_f + H_v + H_l \tag{12.1}$$

where:

TDH	=	Total dynamic head, ft (m)
H_s	=	Static head, ft (m)
H_f	=	Friction head (loss), ft (m)
H_v	=	Velocity head, ft (m)
H_l	=	Losses through fittings, valves, etc., ft (m)

Figure 12.1 displays a system curve which determines the energy required to pump any flow through the discharge system. It is especially critical for the analysis of a discharge system with a force main. When overlaid with pump performance curves (provided by the manufacturer), it will yield the pump operating range.

12.1.3.2 Pump Performance Curve

A pump performance curve expresses the capabilities of the pump in terms of both discharge and TDH, as Figure 12.1 shows. The designer selects multiple pumps for a given station to operate together to deliver the design flow (Q) at a TDH computed to correspond with the design water level. Because pumps must operate over a range of water levels, the quantity delivered will vary between the lowest level and the highest level of the range.

Typically, the designer specifies the conditions for the TDH expected over the full operating range of the pump with emphasis on at least three points on the pump performance curve: near the highest head, at the design head, and at the lowest head. Manufacturers always provide a curve of TDH versus pump capacity for every pump. When running, the pump will pump the discharge associated with the actual TDH on the curve. The designer can develop an understanding of the pumping conditions (head, discharge, efficiency, horsepower, etc.) throughout the full range of head under which the pump will operate by studying the performance curves for various pumps.

Pump efficiency, also provided by the manufacturer, influences pump selection. Designers select a pump to operate with the best efficiency at its design point, which corresponds to the design water level of the station. The efficiency of a stormwater pump at its design point will vary, depending on the pump type. HEC-24 (FHWA 2001) provides additional information on pump performance curves.

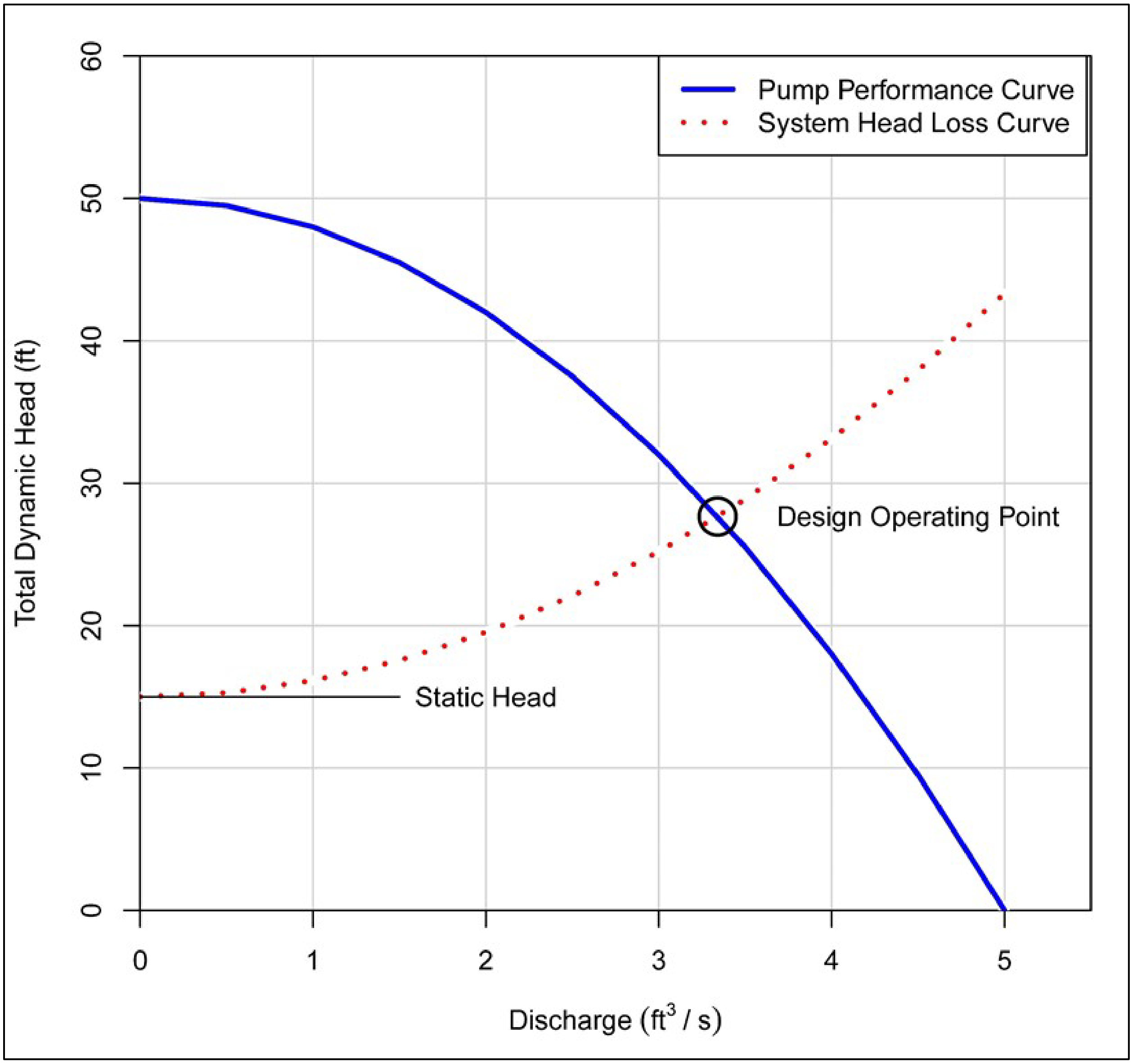


Figure 12.1. System and pump curves.

12.1.4 Number of Pumps

System requirements determine the number of pumps needed. However, two to three pumps generally represent the recommended minimum. When pumping a small total discharge and where the area draining to the station has little chance of increasing substantially, designers typically prefer to use a two-pump station. Designers may consider oversizing the pumps to compensate, in part, for a pump failure. The two-pump system could have pumps designed to pump from 66 to 100 percent of the required discharge, and the three-pump system could be designed so that each pump would pump 50 percent of the design flow. Designers can use the damage resulting from the loss of one pump as a basis for deciding the size and numbers of the pumps.

Limitations on power unit size as well as practical limitations governing operation and maintenance determine the upper limit of pump size. The minimum number of pumps used may increase due to these limitations.

Using pumps of equal size and type has the benefit of enabling all pumps to be freely alternated into service thereby distributing the load between pumps more evenly. Providing an automatic

Equipment Certification and Testing

Testing, certification, and acceptance of equipment is an important element of pump station development. Witnessing equipment testing at the manufacturer's lab or at a suitable facility is ideal, but not always practical. As an alternative, the manufacturer can provide certified test results to the owner. It is good practice to include in the contract specifications the requirement for acceptance testing by the owner, when possible, to ensure proper operation of the completed pump station. Generally, the testing happens in the presence of the owner's representative. If the representative waives the right to observe the test, the manufacture can provide the owner a written report, signed and sealed by a licensed Professional Engineer or person with other credentials deemed appropriate by the owner, to give assurance that the pump equipment meets all performance and reliability requirements. Any component which fails should be repaired and retested.

alternation system for each pump station allows this approach. This system would automatically rotate the lead and lag pump after each pump cycle so that each pump in turn would become the lead pump. This equalizes wear and reduces needed cycling storage. It also simplifies scheduling maintenance and allows pump parts to be interchangeable. Providing hour and start meters aids in scheduling needed maintenance.

12.1.5 Inlet Water Depth and Net Positive Suction Head

Net positive suction head required is the head above vapor pressure head required to ensure that cavitation does not occur at the impeller. The hydrodynamic phenomenon of cavitation can cause substantial damage to pumps. Cavitation results when the depth of water above the pump inlet (submergence) is too low. Insufficient water depth can also allow vortex formation at the inlet, which reduces pump efficiency.

More precisely, cavitation may occur when the net positive suction head (NPSH), which is a function of water depth, is lower than the required NPSH for the pump. NPSH is the minimum pressure under which fluid will enter the eye of the impeller. It varies significantly with pump type and speed and with ambient atmospheric pressure (a function of altitude). This dimension is provided by the pump manufacturer and is determined by laboratory testing. The available NPSH should be calculated and compared to the manufacturer's requirement.

12.2 Pump Station Components

In addition to the pumps and pits, stormwater pump stations include many other components described briefly in this section. HEC-24 provides additional information on these and other pump station components (FHWA 2001).

12.2.1 Water-Level Sensors

Engineers design stormwater pump stations to operate automatically, without human intervention. They rely on water-level sensors to activate the pumps, making those sensors a vital component of the control system. Available sensor types include float switches, electronic probes, ultrasonic devices, mercury switches, and air pressure switches.

The location or setting of sensors inside of the wet well or storage box controls the starting and stopping of pump motors. Their function is critical because pump motors or engines must not start more frequently than an allowable number of times per hour (i.e., the minimum cycle time) to avoid damage. Designers can prolong motor life by providing sufficient storage volume between the pump start and stop elevations to achieve the minimum cycle time requirement.

12.2.2 Power

Designers choose from several types of power for a pump station. Most commonly, they choose electric motors, which involve the least maintenance and oversight. In some cases, designers choose fuel-driven (gasoline, diesel, or natural gas) engines. When selecting fuel-driven engines, considerations include reliable storage with minimal chance of leakage of liquid fuels and fuel perishability. Fuel-driven engines involve periodic maintenance but must start and run reliably without human oversight. Designers select the type of power that best meets the needs of the project based on an estimate of future energy considerations and overall station reliability. Developing a comparative cost analysis of alternatives helps make this decision. However, when readily available, electric power usually costs the least while being the most reliable choice. Getting input from the maintenance engineer will aid the designer in the selection process. The designer will also benefit from remembering that the same conditions necessitating the pump station—rainstorms—are also the conditions under which electric systems may experience service outages.

Because of the tendency for outages to occur during storms, designers generally consider provisions for backup power. However, if they deem the consequences of failure acceptable, they may choose not to provide backup power. Generally, designers make the decision to provide backup power on economics, serviceability, and safety. For electric motors, two independent electrical feeds from the electric utility with an automatic transfer switch may provide sufficient reliability and affordability when backup power is required.

For extensive depressed freeway systems involving several electric motor-driven stations, mobile generators represent another potential source of backup power. Maintenance staff can store a trailer mounted generator at any one of the pump stations, moving the generator to the affected station in case of power outage.

12.2.3 Discharge System

Designers will want to keep discharge piping as simple as possible. Pump systems that lift the stormwater vertically and discharge it through individual lines to a gravity storm drain as quickly as possible represent a preferred design. Because frozen discharge pipes could damage pumps, designers also consider frost depth when deciding the depth of discharge piping.

Depending on topography, designs may use long discharge lines to pump stormwater to a higher elevation. For efficiency, such a design may combine the lines from individual pump stations into a force main or mains. For such cases, designers provide check valves on the individual lines to prevent stormwater from flowing back into the wet well and restarting the pumps or prolonging their operation time. Check valves are preferably located in horizontal lines. To provide for continued operation during periods of repair, etc., designers include gate valves in each pump discharge line. To determine the most efficient length and type of discharge piping and fittings such as manifolds designers perform a cost analysis. Designers keep the number of valves to a minimum to reduce cost, maintenance, and head loss through the system. Because water remaining in the pipe can develop corrosive and hazardous anaerobic conditions and become a nuisance, designers include some provision for draining or other removal of water stored in the force main (that will not drain by gravity) after a pumping event.

12.2.4 Flap Gates and Valving

Designs use flap gates and various valve types to provide controls and connections for a stormwater pump station. **Flap gates** restrict water from flowing back into the discharge pipe and discourage entry into the outfall line. Because flap gates are usually not watertight, designers set the elevation of the discharge pipe above the normal water levels in the receiving channel. If the design uses flap gates, check valves may not be necessary.

Check valves are watertight; designers use them to prevent backflow on force mains which could store enough water to restart the pumps if backflow were to flow into the wet well or storage box. By preventing backflow, they prevent pump direction reversal and resulting motor rotation, which can cause electrical overloads and tripped circuit breakers. Designers use check valves on manifolds to prevent return flow from perpetuating pump operation. To prevent water hammer in the pipes, designers use spring-assisted silent or "non-slam" configuration check valves or otherwise pay careful attention to design and installation. These include swing, ball, dashpot, and electric.

Gate valves are a shut-off device used on force mains to allow for pump or valve removal. Designers should not use valves in pump stations to throttle flow. Instead, they should be either entirely open or entirely closed.

Air/Vacuum valves allow trapped air to escape the discharge piping when pumping begins and prevent vacuum damage to the discharge piping when pumping stops. They are especially important with large diameter pipe. If the pump discharge is open to the atmosphere, an air-vacuum release valve is not necessary. Designers use combination air release valves at high points in force mains to evacuate trapped air and to allow entry of air during system drainage.

12.2.5 Trash Racks and Grit Chambers

Designers can provide trash racks at the entrance to the wet well if they anticipate large debris. For stormwater pumping stations, simple inclined steel bar screens are adequate. Constructing the screens in standardized modules facilitates removal for maintenance and replacement if damaged. If the screen is relatively small, designers typically provide an emergency overflow to protect against clogging and subsequent surcharging of the collection system. Screening large debris at surface inlets may effectively minimize the need for trash racks. Excluding debris at the surface, and thereby preventing entry into the system, facilitates maintenance and improves hygiene.

If designers anticipate substantial amounts of sediment, they may provide an easily accessible grit chamber to capture settleable solids. This will reduce wear on the pump impellers and cases and reduce the need for regular removal of sediment from the wet well. The optimal design provides convenient access to the grit chamber and removal of sediment by mechanical means (e.g., backhoe tractor or vacuum truck), rather than manual removal.

12.2.6 Monitoring Systems and Maintenance

Pump stations are vulnerable to a wide range of operational problems from malfunction of the equipment to loss of electrical power. Designers traditionally use monitoring systems such as onsite warning lights and remote alarms for pump stations to help minimize such failures and their consequences. The expanding use of Intelligent Transportation System (ITS) elements such as video roadway surveillance, electronic changeable message signs, and active monitoring of urban roadways enhances the possibilities for pump station oversight and monitoring. Cellular and other wireless communications allow regular exchange of information with highway features such as traffic signals; monitoring the status, performance, and maintenance of pump stations is no different. The pump station can transmit operating functions to a central control unit, ITS monitoring office, or maintenance office, allowing the central control unit to initiate corrective actions immediately in case of malfunction. This approach allows effective monitoring of such functions as power, pump operations, unauthorized entry, explosive fumes, and high water levels. A regular schedule of maintenance conducted by trained, experienced personnel help assure the proper pump station functioning.

The ease of acquisition of information from remote locations allowed by ITS and modern communications presents opportunities not available in the past. Designers may consider including electronic weather monitoring equipment such as temperature and rainfall measurements at pump stations, along with electronic records of operation (start/stop times, water level in wet wells, inflow, and outflow data) over several years or the lifespan of a pump station. Such data can prove invaluable in improving future design, maintenance, and operation of pump stations, as well as in providing real-time data for active traffic management via ITS, active maintenance management, and emergency incident management. Pump stations provide a logical setting for such data collection and transmission equipment.

Since major storm events occur infrequently, the DOT can develop a comprehensive, preventive maintenance, inspection, and recertification program for maintaining and testing the equipment so that it will function properly when needed. Inclusion of instruments such as hour meters and number-of-starts meters on each pump will help schedule maintenance. Soliciting input from maintenance forces will allow designers to improve each new generation of stations.

Periodically testing equipment, instrumentation, and auxiliary features (hatches, doors, etc.) will help ensure proper operation and condition. DOTs will also wish to schedule relatively frequent inspection of the facility for vandalism, deterioration, weather damage, vehicular damage, and unauthorized entry or occupation. In some areas, vegetation, roots, insects, or other creatures can create entry or maintenance problems. Fire ants, in particular, can damage electronic components. Bats, raccoons, skunks, snakes, and other animals can create disease or safety hazards and damage components.

Safety First

For the safety of operation and maintenance, designers review all elements of the pump station. Ladders, stairwells, and other access points facilitate use by maintenance personnel. Designers also ensure adequate space for the operation and maintenance of all equipment, paying particular attention to guarding moving components such as drive shafts and providing proper and reliable lighting. In some cases, air testing equipment can be available so maintenance personnel can check for clean air before entering. Proper ventilation is essential.

Pump stations will likely be classified as a confined space resulting in appropriate access requirements and safety equipment. Designers ensure pump stations are secure from entry by unauthorized persons, providing as few windows as possible.

12.3 Site Planning and Hydrology

Effective stormwater pump station design starts with evaluation of the site and the site hydrology. This section describes stormwater pump station location, site hydrology, and the stormwater collection system that drains to the site.

12.3.1 Location

Practical considerations usually allow designers to locate pump stations near the low point in the highway drainage system they serve. An adjacent frontage road or overpass can often provide easy access to the station. If possible, locating the station and access road on high ground will allow access if the highway becomes flooded. Soil borings made during the selection of the site will reveal the allowable bearing capacity of the soil and identify any potential problems.

Considering architectural and landscaping issues in the location phase of the design process will allow aboveground stations to blend into the surrounding community or fit with the theme of past and future projects. Foregrounding aesthetic, decorative, and community-relevant aspects of highway projects has become commonplace in recent decades, and pump stations can fit into such schemes. Relevant pump station location and design considerations include:

- Providing architecturally pleasing modern pump stations with a minimal cost increase.
- Using screening walls to hide exterior equipment and break up the lines of the building.
- Adding landscaping and plantings to improve the overall appearance of the site.
- Determining if it is necessary or desirable to place the station entirely underground.
- Accommodating maintenance requirements by providing unobtrusive parking and work areas adjacent to the station without encouraging their use by the public.

Consider Construction Experience

Construction methods impact the cost of the pump station. The more a pump station operates, the smaller the fraction the construction cost is of life-cycle cost. With a stormwater pump station, which operates only when needed (during wet weather) operating costs may be insignificant compared to construction costs. One construction option includes caisson construction, in which the station is usually circular, and construction is open-pit construction. Soil conditions are important in selecting the most cost-effective alternative.

Feedback from construction personnel on any problems encountered can improve future designs. "As-built" drawings document any changes. Personnel knowledgeable and experienced with such equipment conduct construction inspections of pump stations.

Hazardous Materials Spills in the Highway Corridor

Pump stations and pumping equipment may be vulnerable to hazard materials spills (of gasoline, other fuels, oils, corrosive chemicals, pesticides, and other hazardous cargo) and associated fire hazards. Commonly, designers have provided a closed conduit system leading directly from the highway to the pump station without any open forebay to intercept hazardous fluids or vent off volatile gases. A closed system depends on a gas-tight seal between the pump pit and the motor room in the pump station. A safer design isolates the pump station from the main collection system and the effect of hazardous spills by placing the storage facility upstream of the station. This may be an open forebay or a closed box (with sufficient grating at each end for ventilation) below or adjacent to the highway pavement.

12.3.2 Hydrology

For traffic safety and to avoid flood hazards, engineers usually design pump stations serving major controlled-access thoroughfares and arterial streets to accommodate a 0.02 AEP event (AASHTO 2014). To determine the extent of flooding and the associated risk, designers also validate the drainage system performance for the 0.01 AEP event. Keeping the drainage area contributing to the station small reduces the size of the pumping station and minimizes negative impacts if the pumping station malfunctions. For the same reasons, designers anticipate future development that could contribute to increases in flow to the pumping station.

Consider the feasibility of providing storage, in addition to that which exists in the wet well, at all pump station sites. For most highway pump stations, the high discharges associated with the inflow hydrograph occur over a relatively short time window. Additional storage, above or below ground, may greatly reduce the peak pumping rate. Designers can use an economic analysis to estimate the optimum combination of storage and pumping capacity. However, once constructed, adding pumping capacity typically involves many additional costs compared to relatively low-cost storage volume. Chapter 10 describes procedures for storage routing.

12.3.3 Collection Systems

Local topography and efforts to minimize depth and associated construction costs often restrict storm drains leading to pumping stations to mild grades. A grade resulting in velocities of around 3 ft/s in the pipe while flowing full typically avoids siltation problems in the collection system. Minimum pipe cover, construction clearance, or local head requirements will usually govern the depth of the uppermost inlets. Designers often use baffles to ensure that inflow to the pump well distributes inflow equally to all pumps. The Hydraulic Institute provides information for pump station layout (Hydraulic Institute n.d.).

Collector lines preferably terminate at a forebay or storage box structure, or they may discharge directly into the station. Under the latter condition, designers calculate and carefully check the capacity of the collectors and the storage volume inside of them to provide adequate cycling time for the pumps. A minimum grade of 2 percent may prevent or control siltation problems in storage units.

Storm drainage systems tributary to pump stations can be quite extensive and costly. For some pump stations, the storage volume within the collection piping itself may be significant. Designers may consider storage volume within the system, especially near the pump station, when designing the collection system.

To prevent large objects from entering the system and possibly damaging the pumps, designers typically use debris screens. Screening at the surface facilitates screen maintenance and debris removal, however debris screening may occur either at the surface or inside the wet well/storage system. Design considers the level, accessibility, and convenience of maintenance and inspection when selecting debris trapping devices.

12.4 Storage and Mass Curve Routing

Stormwater pump stations route stormwater inflows at a low point to a higher discharge point, using storage to provide for effective pump station operation. When determining the volume of storage for a pump station, designers strive to achieve a balance between pump rates and storage volume; as available storage increases, required pump size decreases. Designers use an iterative procedure in conjunction with economic estimation to balance storage volumes and pump sizes. Because of the cost associated with pump station construction and ongoing maintenance and operations, designers consider several viable alternatives as they strive to optimize life-cycle cost/benefit. This allows comparisons of life-cycle costs and sensitivity to uncertainties of both physical and financial constraints.

During a stormwater event, pump station operations generally include the following series of events:

1. As stormwater flows to the pump station, the wet well or storage box stores the water, and the water level rises to an elevation which activates the first pump.
2. If the inflow rate exceeds the pump rate, the water level will continue to rise until it causes the second pump to start. If not, the water level will diminish until the pump stops.
3. This process continues sequentially for each pump until either the inflow rate subsides, or enough pumps operate to equal or exceed the inflow.
4. After the pumping rate exceeds the inflow rate, the stage in the station recedes until reaching the pump stop elevations (sequentially), eventually stopping all pumps.
5. If inflow continues at a rate lower than the output of one pump, the water level will again rise in the wet well until a pump starts.
6. The static condition after the end of an event has a water level somewhere between the lowest elevation for NPSH and the first pump start elevation.

Evaluation of the relationship between pump station storage and pumping rate involves developing an inflow mass curve and routing the mass curve through the pump station. The following sections outline these elements of the design.

12.4.1 Storage

Storage attenuates the incoming flow and reduces the demands on pump station operation. Using the inflow hydrograph and pump-system curves, designers can try various levels of pump capacity to determine the corresponding required total storage. Designers estimate the required storage volume by comparing the inflow hydrograph to the controlling pump discharge rate. Stormwater management limitations, the capacity of the receiving system, desirable pump size, or available storage may set this controlling pump discharge rate.

Figure 12.2 depicts an inflow hydrograph and an assumed peak pumping rate. Since the inflow hydrograph peak is greater than the pumping rate, the needed storage volume is the shaded area above the last pump turn-on point. Allowing more of the design storm to collect in a storage facility enables use of a smaller pump station, with anticipated cost benefits.

The location of most highway related pump stations near either short underpasses or long depressed sections, often makes aboveground storage impracticable. Designers ensure that water originating outside of the depressed areas does not enter the depressed areas to avoid pumping additional water. Enlarging the collection system or constructing underground storage represent the simplest forms of storage for such depressed situations. State DOTs typically construct these under the roadway area or in the median, so they rarely involve additional right-of-way.

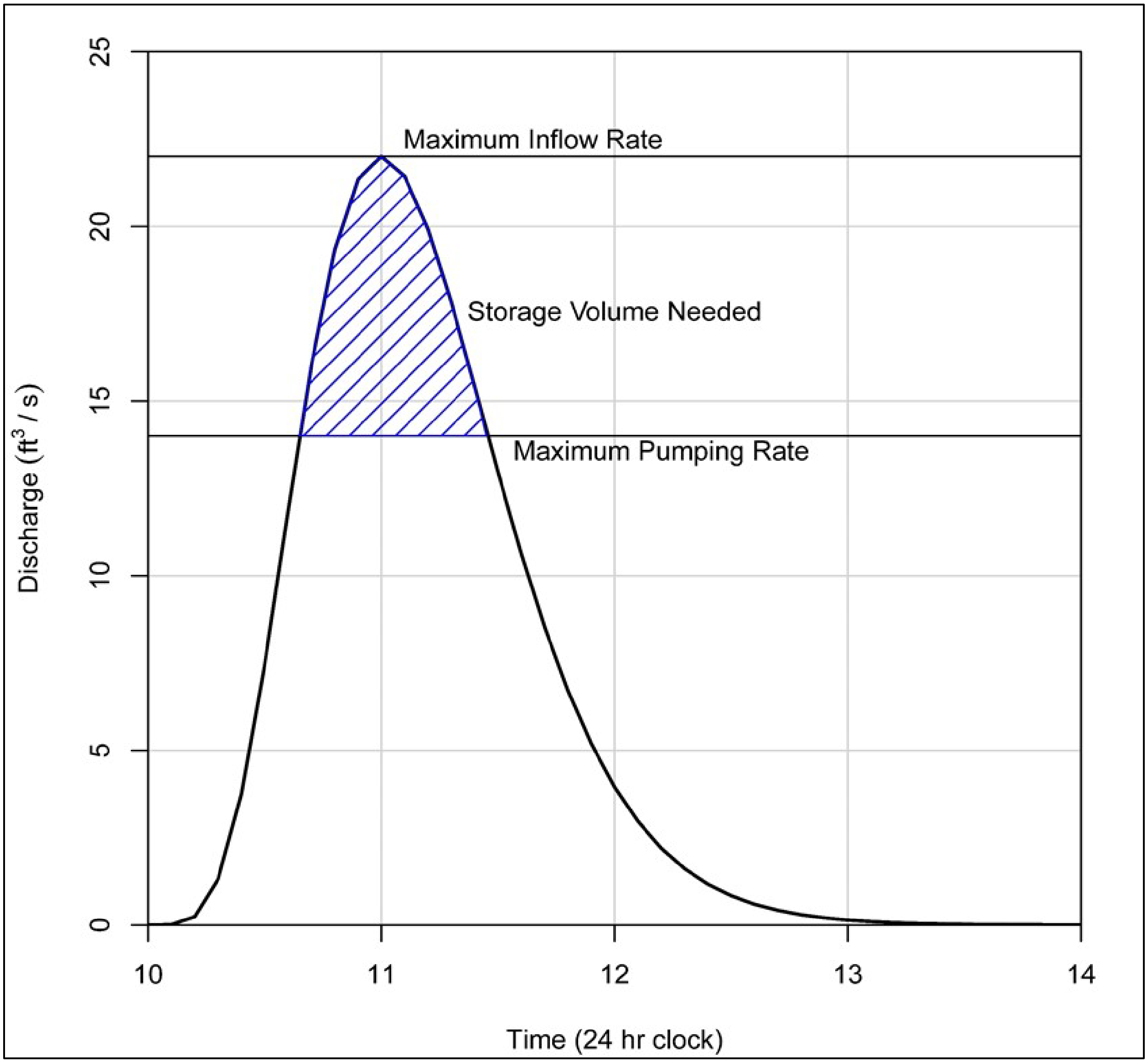


Figure 12.2. Estimated required storage from inflow hydrograph.

12.4.2 Inflow Mass Curve

Designers develop the inflow mass curve by dividing the inflow hydrograph into uniform time increments, computing the inflow volume over each time step, and summing the inflow volumes to obtain a cumulative inflow volume. They then plot this cumulative inflow volume against time to produce the inflow mass curve as shown in Figure 12.3.

12.4.3 Mass Curve Routing

Designers evaluate the relationship between pump station storage and pumping rates using the mass inflow curve in a tabular, computerized form (spreadsheet) or a graphical mass curve routing procedure. Designers can find spreadsheet applications for this procedure online; alternatively, individual designers or agencies can develop them relatively easily. Designers will want to pay close attention to the time steps on the mass inflow curve; sometimes, achieving sufficiently short time steps will depend on curve fitting or polynomial interpolation.

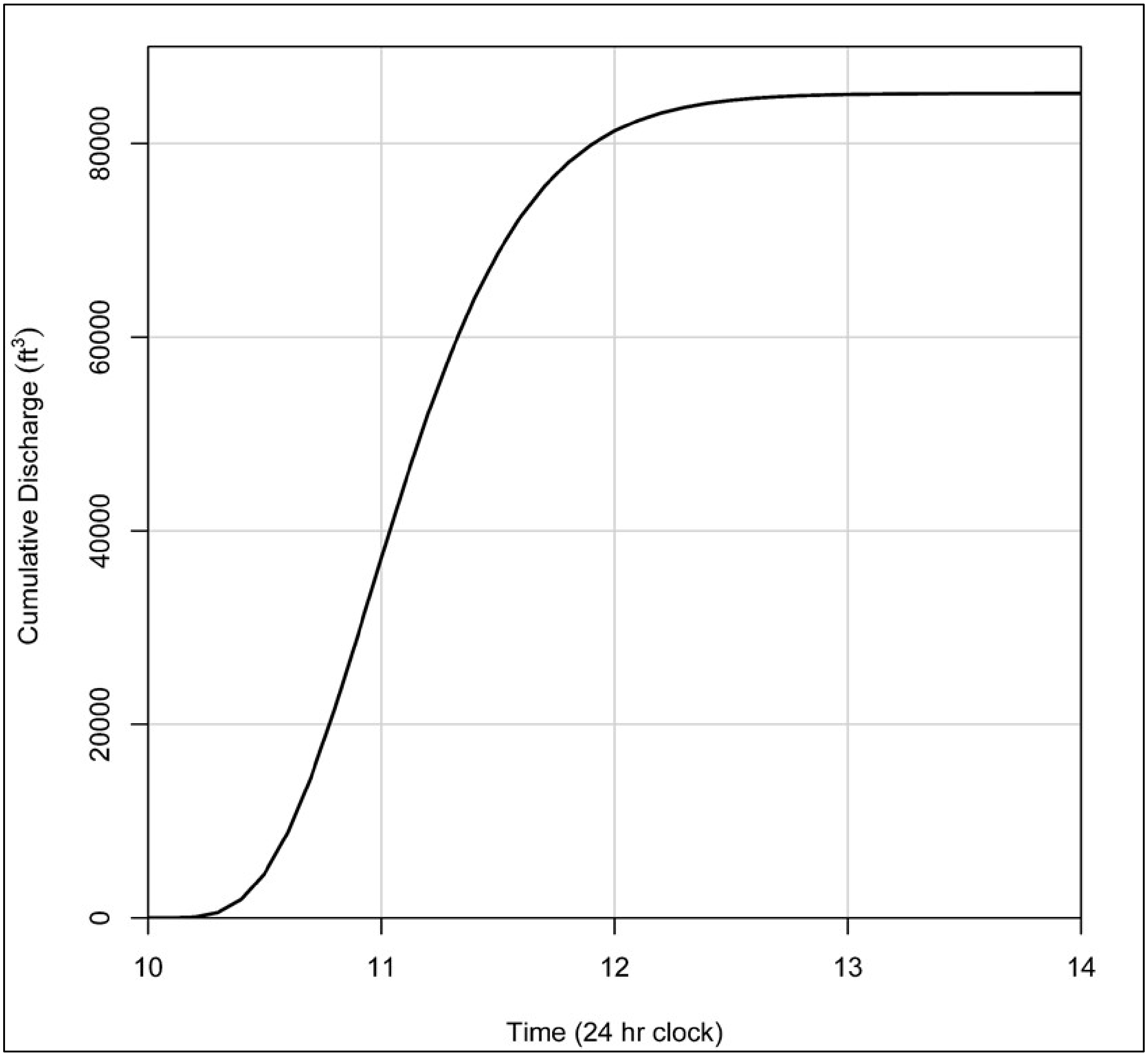


Figure 12.3. Mass inflow curve.

Downstream capacity considerations, limits imposed by local jurisdictions, or other criteria determine an initial maximum pump discharge. With the inflow mass curve and an assigned pumping rate, the designer can determine required storage by various trials of the routing procedure.

Designers use three pieces of information for mass curve routing:

- An inflow hydrograph (Figure 12.2) (from hydrologic evaluation).
- A stage-storage curve (Figure 12.4) (from physical geometry of the storage features).
- A stage-discharge curve (Figure 12.5) (from the pump curve and start/stop elevations).

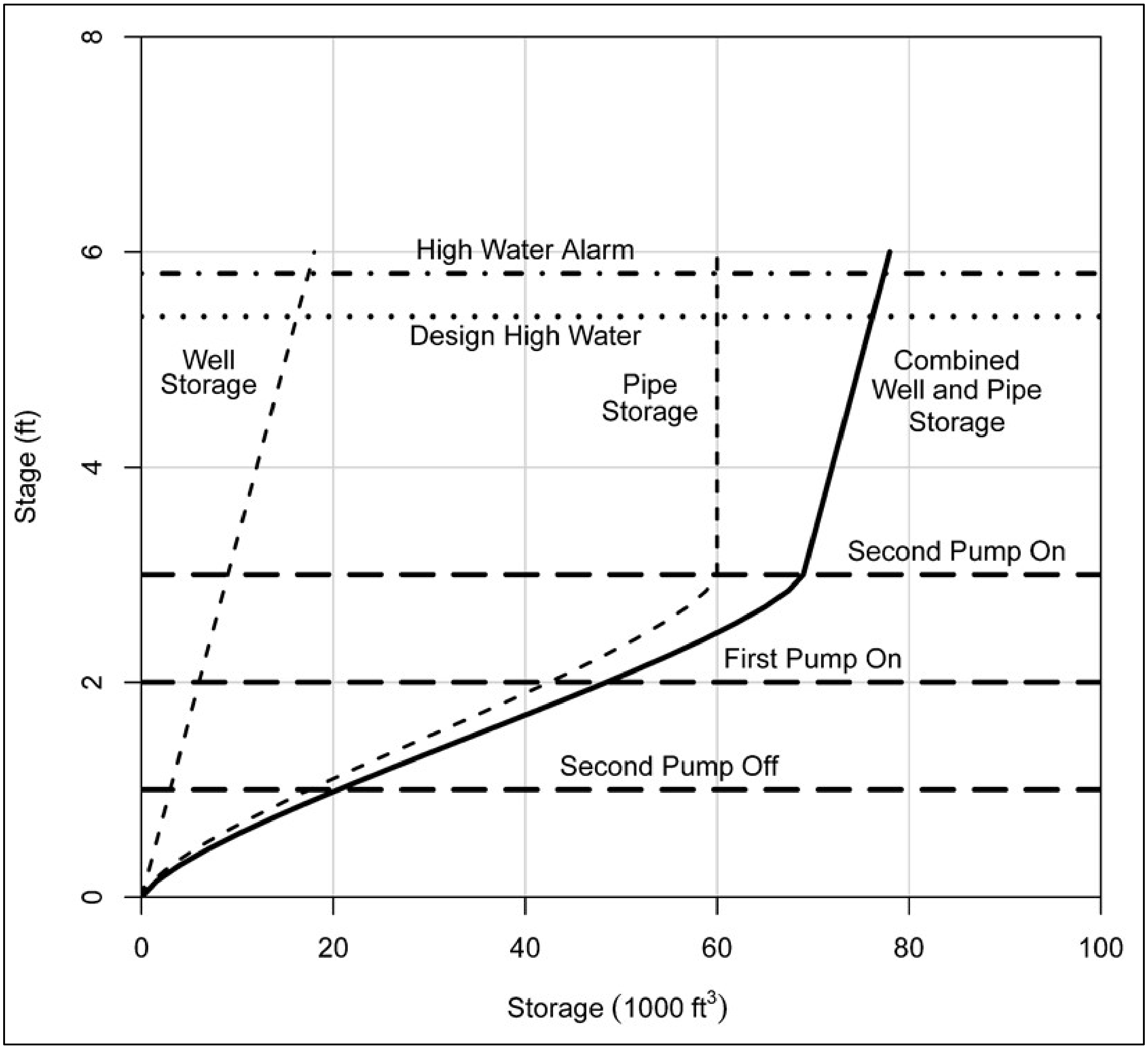


Figure 12.4. Stage-storage curve.

Using this information, designers develop a mass curve routing diagram as shown in Figure 12.6. The letters in the figure note the following sequence of events:

1. The first pump starts at point A and pump at a rate represented by the slope of the line between A and B.
2. At point B, the storage empties and the pump turns off.
3. At point C the start volume has accumulated again, and the lead pump turns on.
4. At Point D the storage has filled to the elevation where the second pump turns on. Since this depicts a two-pump system, both pumps will operate along the pump curve from D to E.
5. Point E represents the elevation where the second pump turns off. At point F, the storage has been emptied and the lead pump turns off.

The vertical lines on Figure 12.6 represent the total volume stored at any given time, such as when a pump starts or stops. The maximum vertical distance between the inflow mass curve and the pump discharge curve represents the amount of storage needed for that set of conditions. The pump start elevations tie directly to the storage volume at that elevation. The designer tries different start elevations iteratively to find a set of start elevations that minimizes the storage needed.

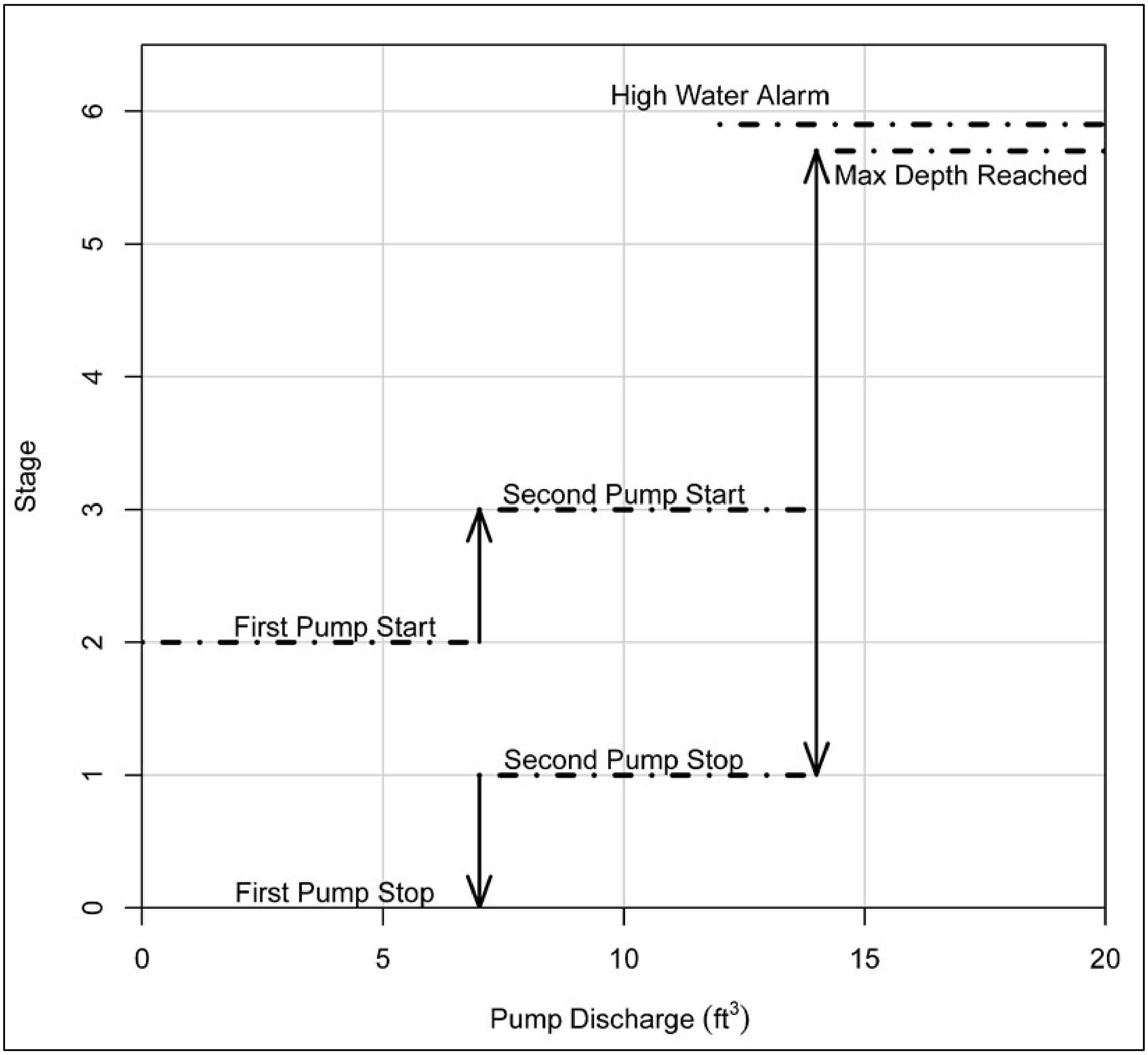


Figure 12.5. Stage-discharge curve.

Designers can use a spreadsheet to perform the calculations. With reasonably short time steps, they can try many different combinations of start/stop elevations. They can try different pump performance curves the same way. HEC-24 provides a detailed example of stormwater pump station design and describes additional design, operations, and maintenance aspects.

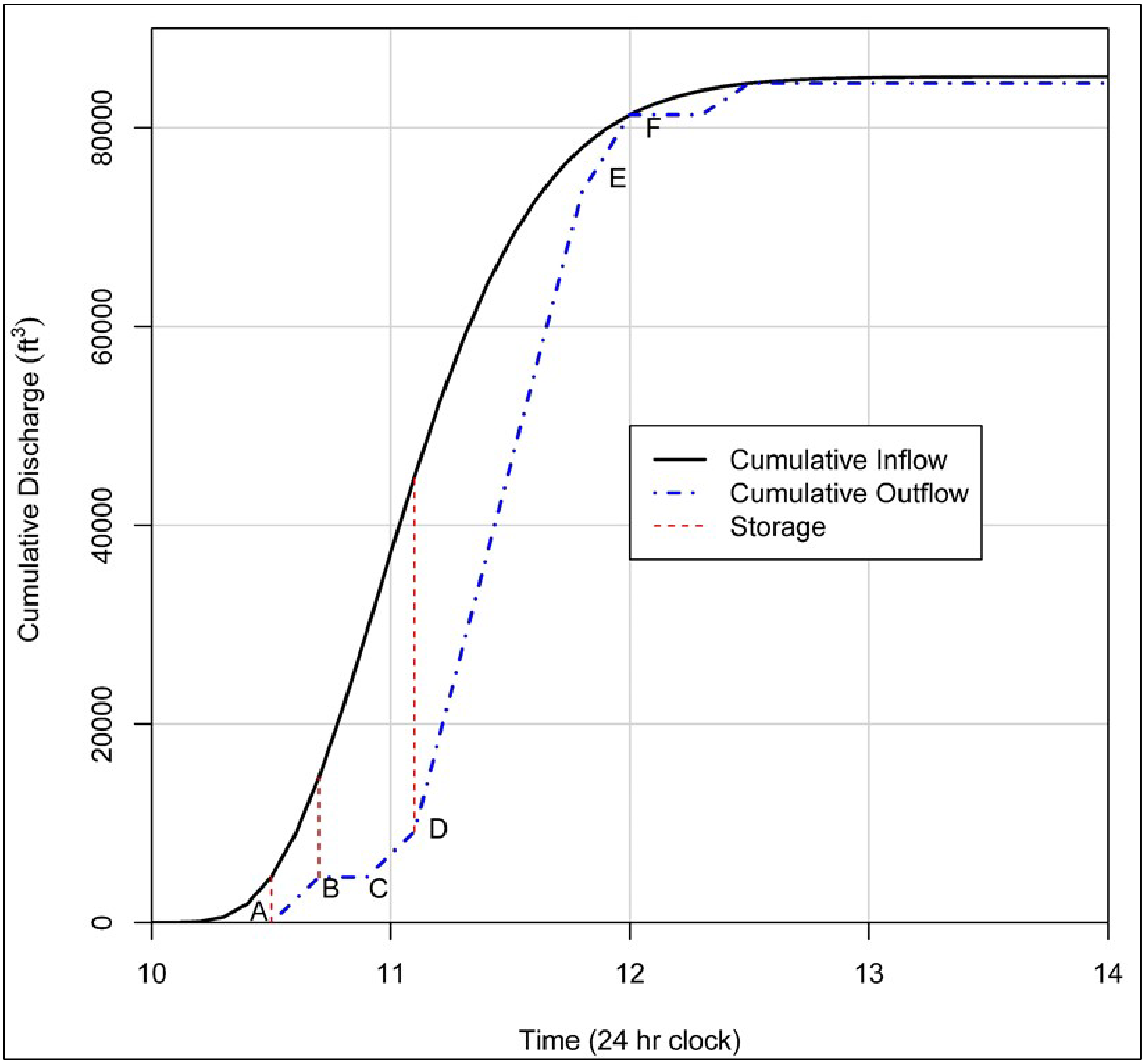


Figure 12.6. Mass curve routing diagram.

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Appendix A - Units

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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Appendix B - Spread-Discharge for Parabolic Sections

A parabolic cross-section can be described by the equation:

$$y = ax - bx^2 \qquad \text{(B.1)}$$

where:

a	=	2H/B
b	=	H/B2
H	=	Crown height, ft (m)
B	=	Half width, ft (m)

Figure B.1 shows the relationships between a, b, crown height, H, and half width, B.

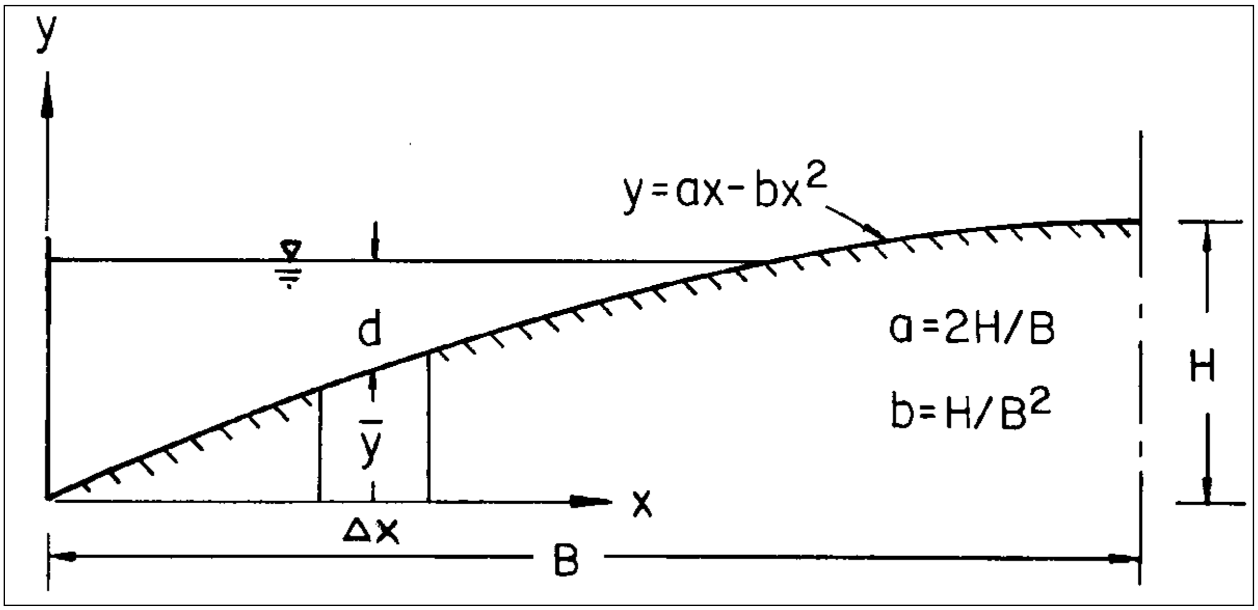

Figure B.1. Properties of a parabolic curve.

To determine total gutter flow, divide the cross-section into segments of equal width and compute the discharge for each segment by Manning's equation. The parabola can be approximated with 2 ft chords. The total discharge is the sum of the discharges in all segments.

The crown height, H, and half width, B, vary from one design to another. Since discharge is directly related to the configuration of the cross-section, discharge-depth (or spread) relationships developed for one configuration do not apply for roadways of other configurations, therefore, the designer develops relationships for each roadway configuration.

The following procedure illustrates the development of a conveyance curve for a parabolic pavement section with a half width, B = 23.9 ft and a crown height, H = 0.48 ft. The Manning's n = 0.016. Table B.1 summarizes the results of the procedure for spreads of 2, 4, and 6 ft.

Step 1. Choose the width of segment, Δx.

Choose the segment width for which the vertical rise will be computed. In this case, select 2 ft and record this in column 1 of Table B.1.

Step 2. Compute the vertical rise.

For H = 0.48 ft and B = 24 ft, equation B.1 becomes:

$y = 0.04x - 0.00083x^2$

Compute the vertical rise for each segment width and enter it into column 2.

Step 3. Compute the mean rise, y_a, of each segment.

Compute the mean rise of each segment and record in column 3.

Step 4. Compute the conveyance from the first segment width.

Depth of flow at the curb, d, for a given spread, T, is equal to the vertical rise, y, shown in column 2. The average flow depth for any segment is equal to depth at the curb for the spread minus the mean rise in that segment. For example, depth at curb for a 2 ft spread is equal to 0.0767 ft. The mean rise in the segment is equal to 0.0384 ft. Therefore, average flow depth in the segment, d = 0.0767 - 0.0384 = 0.0383 ft. Record in column 4.

Compute depth to the five-thirds power and record in column 5.

The sum of d to the five-thirds is 0.0043 as shown in the table.

$Q/S^{0.5} = K = (K_u\ (\Delta x)\ d^{5/3})\ /\ n$

Where K is conveyance and depth, d, is used as an approximation for hydraulic radius, R. K_u is 1.49 for English units.

For this segment:

$K = [1.49\ (\Delta x)\ d^{5/3}]\ /\ n = (1.49\ (2)\ (0.0043))/0.016 = 0.8\ ft^3/s$.

Step 5. Compute the conveyance from the first and second segment widths.

Compute average depth of flow in the next 2-foot segment and enter it into column 6. Average flow depth in the first 2-foot segment nearest the curb is equal to the depth at the curb minus the average rise in the segment.

$d = y - y_a = 0.1467 - 0.0384 = 0.1083$ ft

Similarly, the average flow depth in the second 2-foot segment away from the curb is:

d = 0.1467 - 0.1117 = 0.0350 ft

The sum of d to the five-thirds is 0.0281 as shown in the table.

For the first two segments:

$K = [1.49\ (\Delta x)\ d^{5/3}]\ /\ n = (1.49\ (2)\ (0.0281))/0.016 = 5.23\ ft^3/s$.

Step 6. Repeat for all additional segment widths.

Columns 8 and 9 are computed in the same manner. For T = 6 ft, K = 14.27 ft^3/s.

Repeat the same analysis for spreads up to the half-section width. The computation is limited for to depths less than or equal to the curb height.

Table B.1. Conveyance computations, parabolic street section.

Dis-tance From Curb	Vertical Rise y	Ave. Rise Y_a	T = 2 ft		T = 4 ft		T = 6 ft	
			Ave. Flow Depth (d)	$d^{5/3}$	Ave. Flow Depth (d)	$d^{5/3}$	Ave. Flow Depth (d)	$d^{5/3}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0	0.0384	0.0383	0.0043	0.1083	0.0244	0.1716	0.0527
2	0.0767	0.1117	-	-	0.0350	0.0037	0.0983	0.0208
4	0.1467	0.1784	-	-	-	-	0.0316	0.0031
6	0.2100	0.2384	-	-	-	-	-	-
8	0.2667	0.2917	-	-	-	-	-	-
10	0.3167	0.3384	-	-	-	-	-	-
12	0.3600	0.3784	-	-	-	-	-	-
14	0.3967	0.4118	-	-	-	-	-	-
16	0.4268	0.4385	-	-	-	-	-	-
18	0.4501	0.4585	-	-	-	-	-	-
20	0.4668	0.4718	-	-	-	-	-	-
22	0.4768	0.4784	-	-	-	-	-	-
24	0.4800	-	-	-	-	-	-	-
Sum	-	-	-	0.0043	-	0.0281	-	0.0766

Table B.2 summarizes the results of the analyses for spreads of 8 to 24 ft, which are also plotted in Figure B.2. For a given spread or flow depth at the curb, the conveyance can be read from the figure and the discharge computed from $Q = KS^{0.5}$. For a given discharge and longitudinal slope, the flow depth or spread can be read directly from the figure by first computing the conveyance, $K = Q/S^{0.5}$, and using this value to enter the figure.

As shown in the figure, for a conveyance of 30 ft^3/s (based on Q = 3 ft^3/s and S = 0.01 ft/ft) the water depth at the curve equals 0.275 ft. Using the spread curve at that depth yields a spread of 8.4 ft for the given parabolic curve parameters (H = 0.48 ft and B = 24 ft)

Table B.2. Conveyance and spread for a parabolic street section.

Quantity	T (ft)								
	8	10	12	14	16	18	20	22	24
d (ft)	0.267	0.317	0.360	0.397	0.427	0.450	0.467	0.477	0.480
K (ft^3/s)	27.53	44.71	64.45	85.26	105.54	123.63	137.98	147.26	150.5

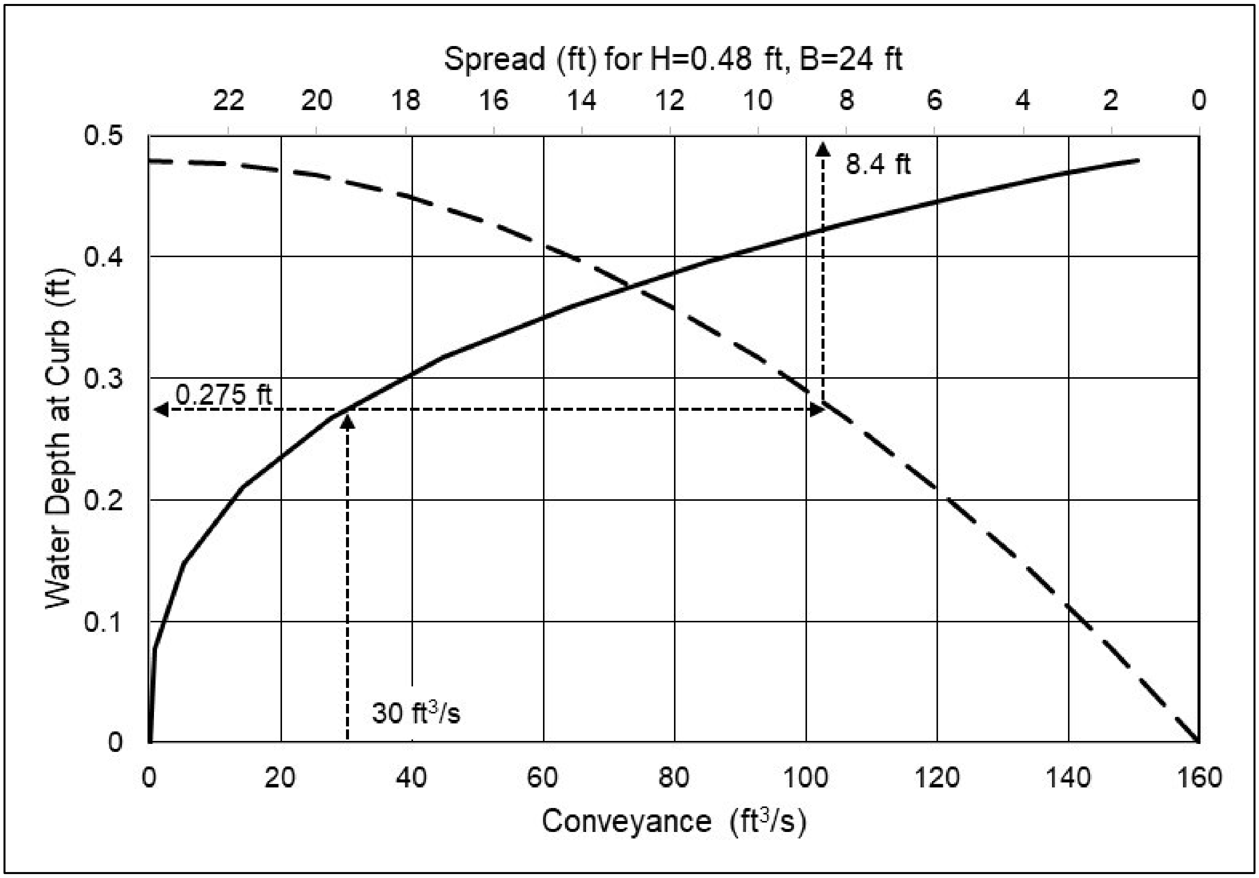

Figure B.2. Conveyance curve for a parabolic cross-section with example application.

Appendix C - Mean Velocity in a Triangular Channel

Flow time in curbed gutters is one component of the time of concentration for the contributing drainage area to the inlet. Velocity in a triangular gutter varies with the flow rate, and the flow rate varies with distance along the gutter, i.e., both the velocity and flow rate in the gutter are spatially varied. Figure C.1 illustrates the concept used to develop average velocity in a reach of channel.

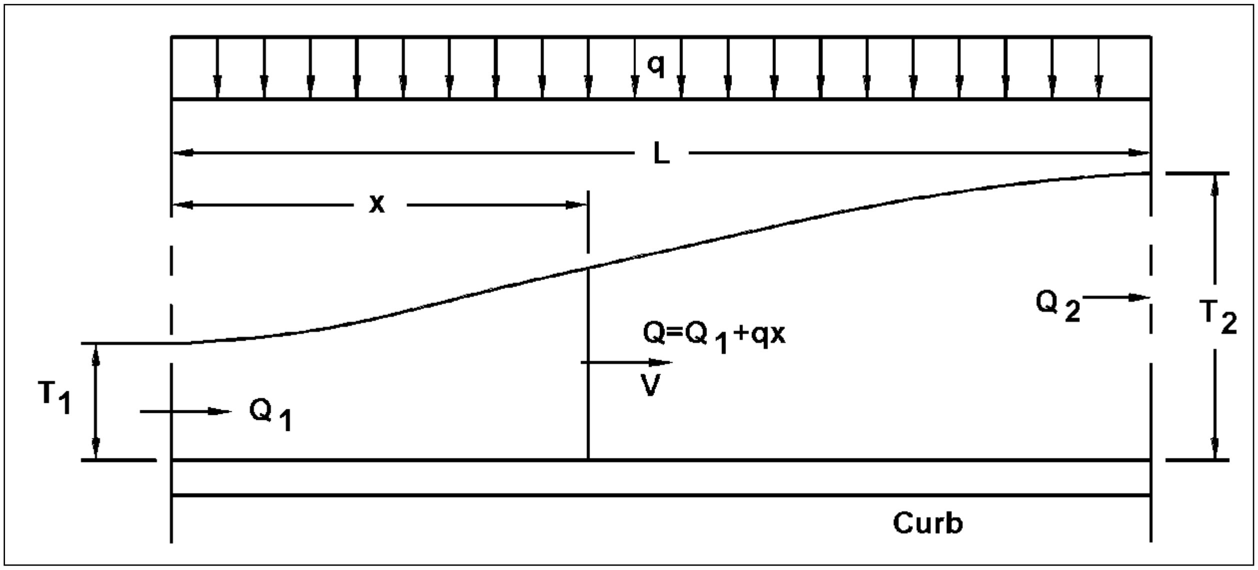

Figure C.1. Conceptual sketch of spatially varied gutter flow.

Time of flow can be estimated by use of an average velocity obtained by integration of the Manning's equation for a triangular channel with respect to time. The assumption of this solution is that the flow rate in the gutter varies uniformly from Q_1 at the beginning of the section to Q_2 at the inlet. Using the gutter equation (equation 5.2), Q is:

$$Q = \left(\frac{K_u}{n}\right) S_L^{0.5}\, S_x^{1.67}\, T^{2.67} = K_1\, T^{2.67} \tag{C.1}$$

With $K_1 = \left(\frac{K_u}{n}\right) S_L^{0.5}\, S_x^{1.67}$

where:

Q	=	Gutter flow, ft^3/s (m^3/s)
n	=	Manning's roughness coefficient
S_L	=	Gutter longitudinal slope, ft/ft (m/m)
S_x	=	Gutter transverse scope, ft/ft (m/m)
T	=	Spread, ft (m)
K_u	=	Unit conversion constant, 0.56 in CU (0.376 in SI)

Dividing the gutter flow by the gutter cross-section area, V becomes:

$$V = \frac{Q}{\left(\left(T^2 S_x\right)/2\right)} = \left(\frac{2K_u}{n}\right) S_L^{0.5} S_x^{0.67} T^{0.67} = K_2 T^{0.67} \qquad \text{(C.2)}$$

With $K_2 = \left(\frac{2K_u}{n}\right) S_L^{0.5} S_x^{0.67}$

where:

V	=	Gutter velocity, ft/s (m/s)
n	=	Manning's roughness coefficient
S_L	=	Gutter longitudinal slope, ft/ft (m/m)
S_x	=	Gutter transverse scope, ft/ft (m/m)
T	=	Spread, ft (m)
K_u	=	Unit conversion constant, 0.56 in CU (0.376 SI)

From equation C.1:

$$T^{0.67} = \left(\frac{Q}{K_1}\right)^{0.25} \qquad \text{(C.3)}$$

Substituting equation C.3 into equation C.2 with V = dx/dt results in:

$$\frac{dx}{Q^{0.25}} = \left(\frac{K_2}{K_1^{0.25}}\right) dt \qquad \text{(C.4)}$$

where:

dx	=	Change in longitudinal distance, ft (m)
dt	=	Change in time, s

Here, $Q = Q_1 + qx$ and therefore $dQ = qdx$. Combining these with equation C.4 and performing the integration results in:

$$t = \frac{4}{3}\left(Q_2^{0.75} - Q_1^{0.75}\right)\left(\frac{K_1^{0.25}}{qK_2}\right) \qquad \text{(C.5)}$$

The average velocity, V_a is:

$$V_a = \frac{L}{t} = \frac{3}{4}\left(\frac{qK_2}{K_1^{0.25}}\right)\left(\frac{L}{Q_2^{0.75} - Q_1^{0.75}}\right) \qquad \text{(C.6)}$$

Substituting $L = (Q_2 - Q_1)/q$ and $Q = K_1 T^{2.67}$, V_a becomes:

$$V_a = \left(\frac{3}{4}\right) K_2 \left(\frac{T_2^{2.67} - T_1^{2.67}}{T_2^2 - T_1^2}\right) \qquad \text{(C.7)}$$

Defining a gutter geometry parameter as:

$$K_G = (S_L^{0.5} S_X^{0.67}) / n \tag{C.8}$$

Then substituting results in:

$$V_a = K_u K_G \left(\frac{T_2^{2.67} - T_1^{2.67}}{T_2^2 - T_1^2} \right) \tag{C.9}$$

where:

V_a	=	Average velocity in the gutter section between T_1 and T_2 locations, ft/s (m/s)
T_1	=	Upstream spread, ft (m)
T_2	=	Downstream spread, ft (m)
K_G	=	Gutter geometry parameter
K_u	=	Unit conversion constant, 0.840 in CU (0.564 in SI)