Drainage Design Policy Manual

4/3/2025
Revision 1.1
Atlanta, GA 30308

This document was developed as part of the continuing effort to provide guidance within the Georgia Department of Transportation in fulfilling its mission to provide a safe, efficient, and sustainable transportation system through dedicated teamwork and responsible leadership supporting economic development, environmental sensitivity and improved quality of life. This document is not intended to establish policy within the Department, but to provide guidance in adhering to the policies of the Department.

Your comments, suggestions, and ideas for improvements are welcomed.

Please send comments to:

State Design Policy Engineer

Georgia Department of Transportation

One Georgia Center

600 W. Peachtree Street, 26th Floor

Atlanta, Georgia 30308

DISCLAIMER

Revision Summary

Revision Number	Revision Date	Revision Summary
1.0	11/15/24	New manual
1.1	4/3/25	Definitions – Updated definitions Chapter 1 – revised state and local agency coordination guidelines Chapter 2 – Updated subheading 2.2.1 and Table 2.1 title Chapter 3 – Changed design discharge criteria to design storm frequencies Chapters 4, 5, 6 – update to the policies for ditch grading and handling of structures within wall foundations Chapter 7 – culvert bends policy clarification Chapter 8 – bridge freeboard requirements clarifications Other minor corrections and updates; Updated figures in Chapters 8 and 9

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List of Effective Chapters

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Acronyms and Definitions

Acronyms

AASHTO - American Association of State Highway Transportation Officials

ADM - AASHTO Drainage Manual

ADT - Average Daily Traffic

AOP - Aquatic Organism Passage

ASTM - American Society for Testing and Materials

BMP - Best Management Practice

CFR - Code of Federal Regulations

CLOMR - Conditional Letter of Map Revision

CN - Curve Number

CWA - Clean Water Act

DNR - Department of Natural Resources

DV - Design Variance

DV_1 - First-Degree Design Variance

DV_2 - Second-Degree Design Variance

EGL - Energy Grade Line

EPD - Environmental Protection Division (Georgia DNR)

EPM - Environmental Procedures Manual

FEMA - Federal Emergency Management Agency

FHWA - Federal Highway Administration

FFPR - Final Field Plan Review

FIRM - Flood Insurance Rate Map

FIS - Flood Insurance Studies

GADNR - Georgia Department of Natural Resources

GDOT - Georgia Department of Transportation

GIS - Geographic Information System

GSMM - Georgia Stormwater Management Manual

GSWCC - Georgia Soil and Water Conservation Commission

HDS - Hydraulic Design Series

HEC - (1) Hydraulic Engineering Circular (USDOT/FHWA); (2) Hydrologic Engineering Center (USACE)

HGL - Hydraulic Grade Line

HMS - Hydraulic Modeling Software

HSG - Hydrologic Soil Group

HW - Headwater

HWd - Headwater Depth

IDF - Intensity-Duration-Frequency

ITB - Invitation to Bid

LIBP – Low Impact Bridge Program

LOCBR – Local Bridge

LOMR - Letter of Map Revision

LRFD - Load and Resistance Factor Design

MS4 - Municipal Separate Storm Sewer Systems

MSE - Mechanically Stabilized Embankment

NCHRP - National Cooperative Highway Research Program

NEPA - National Environmental Protection Act

NFIP - National Flood Insurance Program

NOAA - National Oceanic and Atmospheric Administration

NOI - Notice of Intent

NPDES - National Pollutant Discharge Elimination System

NRCS - Natural Resources Conservation Service

NSS - National Streamflow Statistics

NWP - Nationwide Permit

O&M - Operation and Maintenance

OCGA - Official Code of Georgia Annotated

ODPS - GDOT Office of Design Policy and Support

OES - GDOT Office of Environmental Services

OGFC - Open Graded Friction Course

OMAT - GDOT Office of Materials and Testing

OPD - GDOT Office of Program Delivery

PE - Professional Engineer

PCN - Pre-Construction Notification

PDP - Plan Development Process

PFPR - Preliminary Field Plan Review

PI - Point of Intersection

PM - Project Manager (GDOT Office of Program Delivery)

PPG - Plan Presentation Guide

PVC - Polyvinyl Chloride

QA - Quality Assurance

QC - Quality Control

RAS - River Analysis System

RCP - Reinforced Concrete Pipe

RCUT - Restricted Crossing U-turn

ROW - Right-of-Way

SCS - Soil Conservation Service

SDE - Survey Data Engineer

SFLB – State Funded Local Bridge

SMS - Surface-Water Modeling System

TRM - Turf Reinforcement Mat

TVA - Tennessee Valley Authority

TW - Tailwater

USACE - United States Army Corps of Engineers

USBR - U.S. Bureau of Reclamation

USC - United States Code

USCG - United States Coast Guard

USCS - Unified Soil Classification System

USDA - United States Department of Agriculture

USFS - United States Forest Service

USGS - United States Geological Survey

USFWS - United States Fish and Wildlife Service

VPD - Vehicles per Day

WMS - Watershed Modeling System

Definition of Terms

Abutment – A substructure supporting the end of a superstructure that retains or supports the approach embankment.

Abutment Scour – Erosion of streambed material at and around the abutment

Access Hole –Structure in closed stormwater system which allows access for maintenance and inspection. See manhole.

Aggradation – General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.

Antecedent Moisture – Water stored in the watershed prior to the start of a rainfall event.

Approach Cross Section – Upstream cross section where flow is fully effective, just upstream of contraction reach. See “Cross Section” for the general definition.

Attenuation – In flood control: to temporarily hold back or store stormwater to control the rate of discharge. Also, see Detention.

Backwater – Backwater is defined as the increase of water surface elevation induced upstream from a bridge, culvert, dike, dam, another stream at a higher stage, or other similar structures; or as conditions that obstruct or constrict a channel relative to the elevation occurring under natural channel and floodplain conditions.

Bankfull – The channel condition in a stream where the water level has risen to the point where it completely fills its channel and is on the verge of spilling over into the floodplain.

Bankfull Width – The width of the channel in bankfull condition at a representative cross section.

Barrier Slots – A slot formed in the base of a cast in place concrete bridge barrier so as to act as a deck drain.

Base Flood Elevation (BFE) – The flood elevation having a one percent chance of being equaled or exceeded in any given year. This is the FEMA regulatory standard also referred to as the 100-year flood.

Bent – A substructure unit supporting the superstructure. It is usually a frame consisting of a cap, columns, and footings

Berm – A raised strip of land or a narrow ledge, typically used to control water flow, stabilize slopes, or provide a barrier (general); A shelf or level region in front of the cap on an end bent (specific to bridges).

Bridge – A structure crossing over another feature (stream, river, roadway, railway, etc...) including substructure, superstructure, and approaches.

Bridge Culvert – A culvert or a combination of culverts with a total span of 20 ft. or more. See “Culvert” for the definition of a culvert structure in general.

Bypass – Diverted flow around a structure, reach or feature.

Cast-in-place Structure– Refers to a concrete structure which is poured and cured in its final location in the field.

Catchment – An area confined by drainage divides, often having only one outlet for discharge

Channel – The bed and banks that confine the surface flow of a stream.

Channel Banks – The sides/boundary of channel that confines the flow of water.

Channelized Flow – Concentrated flow contained within an open channel

Channel Lining – Materials along channel side to resist erosive forces of flow. Types include grass, turf reinforcement matting (TRM), rock riprap, and concrete.

Check Storm – Secondary design criteria to analyze performance of drainage design under more intense storm events.

Circular Drain – A round hole formed into a concrete bridge deck at the gutter line so as to act as a deck drain. Holes are typically formed using PVC pipes, which can be extended below the deck to deliver flow to a drainage system or extended below certain girder types to prevent corrosion.

Clear Zone – An unobstructed, traversable roadside area that allows a driver to stop safely or regain control of a vehicle that has left the roadway.

Closed Conduit – Enclosed conveyance structures including circular, or elliptical pipes, box structures.

Combination Inlet – A high-capacity inlet that combines a curb-opening inlet and a grate inlet side by side

Confluence – The junction of two or more streams.

Contraction – The effect of channel or bridge constriction on flow streamlines.

Contraction Scour – A component of scour that results from a contraction of the flow area at the bridge which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.

Coping – The concrete between the top of the beam and the bottom of the slab. The depth of coping varies along the length of the beam to account for differences between roadway grade along the beam and the grade of the top of the beam.

Corrective Effective Hydraulic Model – An updated copy of the hydraulic model used in the effective FIS with inputs corrected from errors or outdated terrain or other data. Also known as the existing conditions hydraulic model.

Countermeasures – A measure intended to prevent, delay, or reduce the severity of hydraulic problems, most typically erosion.

Critical Flow – A specific flow behavior that occurs when the specific energy is the minimum for a given discharge in regular channel cross sections.

Cross Drain – Any pipe or box culvert crossing from one side of the road to the other.

Cross Section – Diagram or drawing cut across a surface that illustrates the overbanks, channel, and water surface.

Cross Slope – The percent of elevation change along the cross section of the road surface.

Crown Elevation – The high point elevation of a roadway pipe or structure.

Culvert – A box or pipe under the roadway that allows water to flow from one side to the other. See Cross Drains.

Curb-Opening Inlet – Drainage structure along curb with opening to capture stormwater flows.

Curve Number – A dimensionless site-specific runoff parameter developed by the (former) Soil Conservation Service (now Natural Resources Conservation Service) to empirically estimate rainfall excess; it accounts for infiltration losses and initial abstractions.

Dam – Structure built across a watercourse for impounding and controlling the flow of water.

Deck Drains – Any opening, formed for fabricated, in a bridge deck for the purpose of removing water from the bridge surface.

Deck Drainage System – A system inclusive of the deck drains, deck drain connections, and the troughs or pipes that convey water away from the bridge super and substructures.

Degradation – The lowering of land or bottom elevation. In stream stability assessment, the lowering occurs through natural erosion of sediment without sufficient incoming sediment to replenish.

Delineation – The process of identifying the boundary of a watershed or drainage basin.

Department – The Georgia Department of Transportation

Depth of Cover – Distance between the top of the subgrade soil and the outside surface of the pipe or drainage structure.

Design Flow – The discharge that is selected as the basis for the design or evaluation of a hydraulic structure including a hydraulic design flood, scour design flood, and scour design check flood.

Design Headwater – The maximum height of stormwater a drainage structure is designed to handle.

Design Standard – Criteria having recognized and usually permanent values which are established formally as a model or requirement.

Design Storm – Minimum storm frequency event used to meet drainage design criteria.

Design Variance – A design condition that does not meet GDOT policy. A Design Variance requires specific agency approval. See Chapter 2 of this manual for more information.

Detention – To temporarily hold back or store stormwater to control the rate of discharge. Normally, the term “Wet Detention” is associated with water quality treatment. Sometimes the term is used for flood control attenuation.

Diversion Structure – A diversion structure removes and redirects water from its normal course within a channel or closed system.

Drainage Area/Drainage Basin – An area confined by drainage divides, often having only one outlet for discharge (catchment, watershed).

Drainage Easement – A legal right granted to GDOT to use a portion of a property owner’s land for a specific purpose to manage stormwater or other types of drainage. A temporary drainage easement is limited in duration, while permanent easement is established to be in effect indefinitely.

Drainage Pavement Course – A type of pavement surface such as Open Graded Friction Course, (OGFC), which has specialized characteristics such as higher permeability and friction.

Drop Structure – Drainage structure where there is an elevation difference between inflow and outflow inverts. Often used in steep terrain and to reduce velocity and energy within flow.

Downstream – The direction towards the mouth of a river or stream, the direction in which water flows.

Downstream Bounding Cross Section – A cross section downstream of a hydraulic structure typically located at the downstream toe of the roadway embankment.

Duplicate Effective Hydraulic Model – A copy of the hydraulic model used in the effective FIS with identical inputs on latest software/equipment.

Embankment – A structure made of earthen materials used to raise and support the roadway from the surrounding ground.

Endrolls – Embankment at end of bridge that extends from the end of the abutment wingwall down to where it meets existing grade.

Energy Dissipators – BMPs used to reduce energy, velocity, and erosive forces at the downstream end of an outlet.

Engineering Judgement – Design decisions made by qualified design professionals on items which fall outside of standard criteria and guidelines. These decisions should be based on knowledge and experience which prioritize safety and minimization of adverse impacts.

Ephemeral Stream – A stream that has flowing water only during, and for a short duration after, precipitation events. Ephemeral stream beds are located above the water table year-round, and groundwater is not a source of water for the stream. Runoff from rainfall is the primary source of water for stream flow.

Existing Conditions Hydraulic Model –Hydraulic model with inputs used from data under current, exiting conditions. In FEMA projects where an effective FIS exists, it can also be referred to as the corrective effective hydraulic model.

Expansion – Zone downstream of a constricted hydraulic opening where flow returns to its natural, fully expanded conditions.

FEMA Zone A – Areas with a 1% annual chance of flooding where detailed analyses are not performed, and no depths or base flood elevations are shown within these zones.

FEMA Zone AE – The base floodplain where base flood elevations are provided.

Flanking Inlets – Drainage structures placed at grade to facilitate or aid in the drainage of structures at the low point of a sag curve.

Flood profile – A graphical representation of water surface elevation along a stream, river, or waterbody for a given flood year event.

Flood stage—The stage at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured.

Floodplain – A flat or nearly flat land adjacent a stream or river that stretches from the banks of its channel to the base of the enclosing valley walls and experiences flooding during periods of high discharge

Floodway –The main channel of a river or other watercourse. A FEMA “Regulatory Floodway” included the mani channel and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height.

Freeboard – The vertical clearance between the water surface elevation of a flood and a design feature. Design features can include the low chord of a bridge, channel top of bank, bottom of pavement, subgrade, or shoulder breakpoint.

Froude Number (Fr) – The Fr value is the dimensionless ratio of inertial forces to gravity forces. If Fr values are less than 1, gravity forces dominate, and the open channel is said to be operating in the sub-critical range of flow. If Fr values are greater than 1, inertial forces dominate, and the open channel is said to be operating in the super-critical range of flow.

Full Valley Cross Section – Cross section located just downstream of expansion reach where flow is no longer affected by the structure.

Girder – A large beam made of steel, timber or concrete used in supporting bridges.

Gradient – The slope or incline of a pipe, channel, or surface, which affects the flow of water.

Grated Inlet (Bridge Deck) – A fabricated element cast into a bridge deck at the gutter line so as to act as a deck drain. These elements interrupt a significant enough area of the bridge deck so as to require grating to support vehicular traffic.

Guide Banks – A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening.

Gutter Spread – The horizontal distance of the stormwater flowing down a pavement and gutter section from the face of the gutter to the water’s edge.

Gutter Width – Distance between the edge of traveled way and the face of curb.

Headwater – Elevation of water at upstream end of culvert.

High Water – The elevation of the highest known specific flooding event at a specific location.

Highwater Elevation – The maximum water levels that can occur in a channel or at the upstream face of a bridge or hydraulic opening without contravention of the adopted design criteria. May also be the usual term used to describe the estimated water surface elevation or profile in the stream (or other surface waters) at the project site for the selected design discharge.

Hydraulic Opening – The cross-sectional area beneath a bridge that is available for conveyance of water.

Hydraulics – The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures, and the ground.

Hydraulic Depth – The ratio of the water flow cross section area to top width.

Hydraulic Radius – The cross-sectional area of a stream divided by its wetted perimeter.

Hydrograph – A hydrograph is a relationship of stage, discharge, velocity, or other state of water with respect to time at a certain location.

Hydrologically Remote Point – Point within a specific watershed from which water takes the longest time to reach the watershed outlet.

Hydrology – The science concerned with the occurrence, distribution, and circulation of water.

Hydroplaning – Loss of traction for a vehicle when thin layer of water impedes contact between tire and roadway surface.

Impervious Area – Land use areas where no infiltration or precipitation or runoff can occur.

Industry Standard Design Storm Events – Suite of storm events typically used in drainage design including the 1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 200-year, and 500-year storms.

Infiltration – The process of water entering the upper layers of the soil profile.

Initial Abstraction – The amount of rainfall which does not turn into runoff and is infiltrated, stored, evaporated or otherwise intercepted.

Intermediate Bent – Substructure support unit between the ends of a bridge. See pier.

Intermittent Stream – An intermittent stream has flowing water during certain times of the year, when groundwater provides water for stream flow. During dry periods, intermittent streams may not have flowing water. Runoff from rainfall is a supplemental source of water for stream flow.

Invert Elevation – The lowest point in the channel cross section or at flow control devices such as weirs, culverts, or dams.

Inlet – Drainage structure that captures and conveys stormwater flow.

Junction Box – Drainage structure which combines and potentially redirects inflow pipe(s) and outflow pipes in a closed conveyance system. Junction boxes with access are known as access holes or manholes.

Land Cover/Land Use – Ground cover and use which are characterized by areas hydrologic characteristics.

Level Spreader – A drainage structure used to distribute stormwater runoff evenly across a certain area to prevent erosion and promote infiltration.

Levee – A levee is a manmade barrier built along a stream to reduce flooding in the adjacent floodplain from rising water in the stream.

Local Scour – Removal of material from around specific location, often at piers, abutments, spurs, and embankments caused by an acceleration of flow and resulting vortices induced by obstructions to the flow.

Longitudinal Encroachments –Fill and development parallel to a landscape feature, most often an existing roadway or stream/floodplain.

Longitudinal Pipes – Run of pipe parallel to main length of roadway.

Lateral Pipes – Run of pipe perpendicular or skewed to main length of roadway which intersects with main or trunk line pipe.

Low Chord – Lowest potion of the bridge superstructure.

Main Line – In a stormwater system, the main or trunk line is the run of largest conveyance with auxiliary runs or segments converging into it to form the main line.

Manhole – Accessible junction box, a drainage structure with no surface inlet.

Mannings N Value – Unitless coefficient which represents the roughness or friction applied to the flow through channels or over surfaces.

Natural Conditions Hydraulic Model – Hydraulic model with inputs used from data under current conditions with existing bridge and constructed embankments are removed.

Normal Crown – A symmetrical cross slope of the roadway with an equal inclination from center to the edge of roadway.

No Rise – A study based on technical data that certifies the flood elevation will not increase.

One-Dimensional Hydraulic Model – A physical or mathematical representation of a flow situation using design software and methodologies which analyzes flow as it travels through cross sections.

Open Channel – Streams, rivers, roadside or median ditches which convey water with a free surface.

Outfall – The point where water discharges from a stormwater system, culvert, or ditch.

Outlet – The point where concentrated flow exits a contributing drainage area and in hydrologic design can be referred to as the study point

Outlet Control Structure – Drainage structure used to control the release and direction of flow. Commonly found in BMPs, dams, weirs, and other bypass and detention structures.

Overbank Area – Area within a floodplain that is outside of the channel banks.

Overtopping – The scenario in which the volume of water exceeds the capacity of the drainage system.

Peak Flow – The highest rate of flow in a given storm event.

Pedestrian Bridge – Bridge structure designed for pedestrian travel. Typically includes spans of greater than 20 feet or are located 15 feet above the groundline.

Perennial Stream – A perennial stream has flowing water year-round. The water table is located above the stream bed and groundwater is the primary source of water for stream flow. Runoff from rainfall is a supplemental source of water for stream flow.

Physiographic Region – Areas of land with similar topography, geology, and vegetation.

Pier – A substructure unit supporting each end of a bridge span; also called a bent.

Pier Scour – Localized erosion of streambed material around a pier location.

Piles – A steel, timber or concrete member driven into the ground that provides support for a bridge due to friction with the surrounding soil and end bearing of the tip on a hard subsurface layer.

Pond – Non- or slow-moving water which accumulates a depth over a surface.

Precipitation – Water falling to the ground from clouds or condensate in the air.

Profile Grade – The longitudinal slope corresponding to the vertical alignment of the roadway.

Proposed Conditions Hydraulic Model – Hydraulic model with inputs used from data under proposed conditions including proposed terrain, land use, hydraulic structure, and any roadway embankments, cut or fill.

Rainfall – The quantity of rain falling as water droplets.

Rainfall Intensity – The average rainfall rate for a specific rainfall duration.

Reach – A segment of stream length that is arbitrarily bounded for purposes of study.

Recurrence Interval – The reciprocal of the annual probability of exceedance of a hydrologic event (also return period, exceedance interval).

Representative Cross Section – A stable, straight reach of the stream that typifies the average conditions of the stream or channel's width, depth, and flow characteristics. Representative cross section is determined by the ecologist during environmental resources identification and transmitted to the design team.

Retention – To retain stormwater and prevent any surface water discharge. The retained stormwater is either infiltrated into the ground or evaporated.

Riprap – Layers of large stones or broken rocks used to prevent erosion and provide stability. See type 1 and type 3 riprap.

Right-of-way – An area of land that is acquired or set aside for transportation purposes.

Roadway Profile – The vertical alignment of the roadway.

Run – Individual segment of a pipe or conveyance system.

Runoff – Precipitation remaining after appropriate hydrologic abstractions have been accounted for.

Runoff Coefficient – Empirical parameter used to calculate rainfall excess as a fixed percentage of precipitation; it accounts for interception, surface storage, and infiltration.

Sag – A low point in a vertical curve, can also be referred to as sump.

Sanitary Sewer – Closed conduit system conveying sewage/wastewater apart from runoff. A combined sewer system conveys both stormwater and wastewater/sewage.

Scour – Erosion of streambed material, typically at hydraulic conveyance.

Segment – Individual section of stormwater pipe or conveyance system.

Shallow Concentrated Flow – Runoff traveling over surfaces as small or minor rills, rivulets, or gullies. Typically occurring after sheet flow, and before channelized flow.

Shear Stress – The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually in units of stress, Pa (N/m^2) or (lb/ft^2).

Sheet Flow – Runoff traveling over surfaces as a thin uniform layer. Typically, in the upper reaches of a drainage area, and for short distances, 50 feet or less, with a maximum of 100 feet before becoming shallow concentrated flow, or channelized flow.

Shoring – The use of temporary supports to stabilize a structure or excavation during construction.

Shoulder Breakpoint – The intersection of the shoulder slope and the embankment slope or ditch slope plane.

Slotted Inlet – Drainage inlet structure composed of a continuous slot built into the top of a pipe or conveyance channel that serves to intercept, collect, and transport runoff.

Spillway – Protected channel outlet used to discharge excess water away from a feature. Emergency spillways are often used in dams and BMPs to provide an overflow channel preventing the water level from reaching higher elevations.

Spring Box – A drainage structure used to collect and protect groundwater from a source.

Spring Tide Elevation – The tide elevation that occurs at or near the time of the new or full moon and which rises highest and falls lowest from the mean sea level.

Spur Dike – An earth embankment projecting upstream from bridges with wide flood plains to prevent the abutment from scouring out.

Standard Design Storm Events – The frequency and probability storm events used to meet standard design criteria.

State Waters – State waters are defined in Section 12-7-3(16) of the Georgia Erosion and Sedimentation Act as "any and all rivers, streams, creeks, branches, lakes, reservoirs, ponds, drainage systems, springs, wells and other bodies of surface or subsurface water, natural or artificial, lying within or forming a part of the boundaries of the State, which are not entirely confined and retained completely upon the property of a single individual, partnership, or corporation." State waters are regulated by the GAEPD and may receive buffer protection.

Stream – Also referred to as "natural stream," a stream is a natural channel with its size and shape determined by natural forces. It is usually compound in cross section with a main channel for conveying flows and a floodplain to transport flood flows, unless it is a highly incised channel, in which case no active floodplain exists.

Streambed – The bottom layer of solid material in a stream or channel.

Strom Drain – Drainage inlet structure designed to collect stormwater into a drainage system.

Storm Duration – The length of time associated with a given storm event. Partial duration should be used when determining peak storm intensity.

Stormwater – Water resulting from precipitation flowing over terrain. See Runoff.

Subcritical Flow – The type of flow where the water depth is greater than the critical depth and behaves in a slow or stable way. Froude number less than one

Subgrade – The surface upon which the roadway pavement and shoulders are constructed.

Substructure – Everything in a bridge below the bearing pad. Includes the cap, column, and footings

Superelevation Transitions – The location of rate of change in the cross-slope of the road.

Supercritical Flow – The state of flow where the velocity of the water exceeds the critical velocity and behaves as rapid or unstable flow. Froude number greater than one.

Superstructure – Everything in a bridge above the cap. Includes barrier, deck, beams and bearing pads

Surcharge – Overloading and overtopping of the carrying capacity of a drainage system or structure

Surface Drainage – Drainage structures conveying runoff over the terrain, including in channels, ditches, swales, river, and streams.

Swale – A broad channel for the conveyance and temporary storage of runoff.

Tailwater – The water surface elevation at the downstream end of a hydraulic conveyance.

Thalweg – In hydraulics, the line joining the deepest points along a flow path.

Time of Concentration – The maximum time required for stormwater runoff to flow from a point within the drainage basin to the basin outlet.

Topography – The configuration and features of a terrain including natural and manmade features.

Total Scour – The sum of long-term degradation, general (contraction) scour, and local scour.

Traversable – A structure or feature which can be traveled over or along by a vehicle.

Trunk Line – In a stormwater system, the main or trunk line is the run of largest conveyance with auxiliary runs or segments converging into it to form the main line. See main line.

Two-Dimensional Hydraulic Model – Hydraulic model of water flowing over a terrain surface, as opposed to a one-dimensional model which

Type 1 Riprap – Rock pieces used for channel lining, outlet protection and other uses as defined by GDOT Standard Specification 805 with all pieces smaller than 4.2 ft^3.

Type 3 Riprap – Rock pieces used for channel lining, outlet protection and other uses as defined by GDOT Standard Specification 805 with all pieces smaller than 1.0 ft^3.

Unconstricted Hydraulic Model – Hydraulic model with inputs used from data under current conditions with existing bridge and constructed embankments are removed. See Natural conditions hydraulic model.

Underdrain – For stormwater management facilities; a system of perforated pipes below a pond that are designed to lower the groundwater table to facilitate pond volume recovery, and/or to filter stormwater runoff prior to discharge.

Upstream – The location towards the source of fluid flow.

Upstream Bounding Cross Section – A cross section upstream of a hydraulic structure typically located at the upstream toe of the roadway embankment.

Watershed – An area bounded peripherally by a drainage divide that concentrates runoff to a particular watercourse or body; the catchment's area or drainage basin from which the waters of a stream are drawn.

Water Surface Elevation – The height, in relation to a specific datum, usually above mean sea level, of floods or normal flow of water.

Weir – A flow restriction with a fixed flowline, width, and height; used to control discharge from a stormwater management facility.

Wetted Perimeter – The length of the line of intersection of the channel wetted surface with a cross-sectional plane perpendicular to the direction of flow.

Wingwall – At abutments, a wall along the side of the bridge and integral with the end bent to hold back earth held behind the end of a bridge.

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Chapter 1. Introduction

1.1 Drainage Design Policy Manual Introduction

The *Drainage Design Policy Manual* is an update to the previous versions of GDOT's Manual on *Drainage Design for Highways*. The *Manual on Drainage Design for Highways* was adopted in 1966 with the purpose of bringing uniformity in the design of drainage structures while conforming to accepted policies of the Georgia Department of Transportation (GDOT) and the Federal Highway Administration (FHWA). Major revisions were adopted in 2015, 2008, 1988, and 1975. This *Drainage Design Policy Manual* was developed to incorporate updated design policy and standards in the state and is policy-based and specific to Georgia rather than a how-to guide.

This chapter provides a discussion on the intended use of this manual, drainage design project workflow, and an overview of the manual contents, manual maintenance procedures, and manual acknowledgements.

1.1.1 How to Use This Manual

This manual primarily contains design policies and guidelines in a condensed format for use by the designer. Although the basic concepts of hydrology and hydraulics are introduced in this manual, the designer will be provided references to various publications within each chapter for more detailed guidelines, step-by-step procedures, and additional example problems. This manual is not intended to be a complete guide to all hydrologic or hydraulic problems encountered and it does not provide guidance on complex issues regarding those problems. Each design project is unique, and this manual should not be used as a substitute for good, sound engineering judgment that comes with experience.

The designer is assumed to be knowledgeable in the use of the referenced items. This manual cannot incorporate computer program user manuals or keep current with these programs or the latest drainage-related federal regulations. Designers of GDOT drainage structures should follow the guidelines presented in this manual and reference the appropriate user manual or technical support group for computer program use.

When the designer encounters a situation that is not described in this manual or in the cited references, the GDOT Office of Design Policy and Support or the GDOT project manager should be contacted for assistance. Designers are encouraged to request assistance as soon as questions or problems arise since timely help can often provide a more efficient and effective design process and can lead to the generation of more applicable solutions.

1.1.2 Overview of Manual Contents

The general contents of each chapter are summarized below:

Chapter 1 - Introduction

Chapter 1 introduces and designates the intended use of this drainage manual to set forth drainage design standards and procedures for Georgia Department of Transportation (GDOT) projects. Includes an overview of the manual contents, manual maintenance procedures, and manual acknowledgements.

Chapter 2 – Design Guidelines and Standards

Chapter 2 establishes design standards, and the procedures for meeting standards, or otherwise obtaining design acceptance through either a Design Variance or a Design Deviation.

Chapter 3 – Hydrology

Chapter 3 presents an overview of hydrology methods and best practices and sets the design standards to be used for the calculations for the remainder of the manual.

Chapter 4 - Pavement Drainage

Chapter 4 discusses pavement drainage and includes curb and gutter flow, spread of water on pavement, types of inlets, inlet capacity on grades and in sags, inlet spacing, and flanking inlets.

Chapter 5 – Storm Drain Design

Chapter 5 provides policy on storm drain design, factors related to, and evaluation of the hydraulic grade line and energy grade line.

Chapter 6 – Open Channels and Streams

Chapter 6 provides design policy for open channel water conveyance systems, including ditches and streams. Roadside and median channel analysis and design requirements are included, as well as an introduction to natural stream channel analysis and design.

Chapter 7 – Culverts

Chapter 7 provides standards and procedures for highway culvert or cross drain hydraulic design.

Chapter 8 - Bridge Hydraulics

Chapter 8 provides hydraulic design criteria for all existing and proposed river and tidal bridge sites and for culverts meeting any of the conditions listed in the chapter.

Chapter 9 - Bridge Deck Drainage Systems

Chapter 9 provides the fundamentals of bridge deck drainage design, including pavement slopes and grades, inlet and storm drain design, and gutter spread limits.

1.1.3 Manual Maintenance

The manual is available through the GDOT Design Policies & Guidelines website at http://www.dot.ga.gov/GDOT/pages/DesignManualsGuides.aspx.

It is the designer's responsibility to determine if there are any manual updates by periodically checking the webpage above or by contacting the GDOT Office of Design Policy and Support or the GDOT project manager. If errors are discovered in this manual, please report them to the GDOT Office of Design Policy and Support at the address or e-mail address shown below or to the GDOT Project Manager, so that corrections can be made.

Georgia Department of Transportation

Office of Design Policy and Support

One Georgia Center

600 West Peachtree Street, 26th Floor

Atlanta, Georgia 30308

E-mail: DrainageManual@dot.ga.gov

1.1.4 Acknowledgements

The GDOT Design Policy Manual was developed by a team of individuals from GDOT and the Georgia engineering consultant community. Sincere appreciation is extended to the following persons who collaborated in the preparation of the manual chapters, provided quality control and quality assurance reviews, and offered technical editing:

GDOT:

David Acree

Robert Elam

Sarah Jones

Aneece Louaked

Daniel Pass

Drew Martin

Brad McManus

Toan Nguyen

Gary Pierce

Brian Stanfield

Walter Taylor

Luis Vazquez

Jason Willingham

Consultants:

Tim Dow

Andrew Farmer

John McWhorter

Anthony Prevost

Nick Sopchak

Brent Story

Rachel Westerfield

1.2 Project Requirements

GDOT's policy regarding stormwater design is included in this drainage manual. In addition to drainage and stormwater design criteria found throughout this manual, additional design criteria and procedures can be found at GDOT's Manuals & Guides page online at : https://www.dot.ga.gov/GDOT/pages/DesignManualsGuides.aspx

The GDOT Design Policy Manual is the primary resource for design standards and guidelines, while the GDOT's Plan Development Process (PDP)[(1-5)] manual outlines the project development process from inception through construction award. For requirements concerning construction plans, GDOT has a Plan Presentation Guide (PPG).

1.2.1 Milestones

GDOT projects have a number of milestones including Preliminary Field Plan Review (PFPR) and Final Field Plan Review (FFPR) where the drainage design is presented in detail. Refer to the Plan Development Process Manual[(1-5)] and Flowcharts for all milestones and information on what is required at each project milestone. Considerations for drainage and MS4[(1-9)] requirements during the conceptual stage of a project are important to optimize the design process and minimize project delays and expenses.

1.2.2 Project Documentation

Drainage design documentation varies based on what aspect of stormwater design is being performed and at what review stage the project resides. Drainage design and permitting criteria and calculations are required as part of the Design Data Book as outlined in the PDP[(1-5)]. This documentation will include all of the necessary calculations and applicable models used for drainage design purposes and should include at a minimum the following documentation:

- Hydrologic Analysis and Calculations
- Hydraulic Calculations (including drainage structures design, outlet protection, ditch lining, and gutter spread, as applicable),
- Post-Construction Stormwater Management Report (MS4, Non-MS4, Ecology), as applicable, see Sec 1.3.4 and GDOT Stormwater Design Guide
- FEMA coordination and Hydrology & Hydraulics Report, as applicable, see Sec 1.3.3
- Sediment Storage Calculations (including sediment basins), as applicable.
- Aquatic Organism Passage (AOP), as applicable, see Sec 7.3.8
- Coordination and adherence to Federal, State and Local Agencies, laws and regulations

1.2.3 Project Considerations

Roadway drainage design should maintain or mimic natural drainage patterns. Planning and location studies should consider potential problems resulting from impacts affecting an area's natural drainage, including erosion and sedimentation, flooding, and debris.

District experience and maintenance personnel are valuable sources for identifying potential drainage problems, and district offices should be coordinated with in addition to performing site visits to evaluate and document existing drainage conditions. FEMA flood maps and other desktop research are useful supplemental tools in providing a full picture of flow patterns and existing flood issues. Documenting the history of flooding (or lack of it), presenting the status of existing conditions, and supplementing the information with photographs and descriptions of field conditions are essential due diligence steps to ensure any pre-existing flooding and erosion conditions are noted before construction. Alternative locations or designs should be considered if the design will require frequent and expensive maintenance due to drainage. During roadway construction, channel changes, minor drainage modifications, and revisions in irrigation systems usually carry the assumption of certain maintenance responsibilities by GDOT.

1.3 Agency Coordination and Regulations

This section provides an overview of the relationship between roadway drainage design and the regulatory framework under which roadway projects are permitted and constructed. The GDOT PDP manual and GDOT Environmental Procedures Manual (EPM) [(1-4)] and associated guidebooks should be referenced for more detailed information on the coordination that must take place between GDOT and various federal and state agencies to permit construction of GDOT projects. Additional publications with information regarding the legal framework with regards to stormwater runoff and drainage discharge from roadway systems are:

- AASHTO Highway Drainage Guidelines[(1-1)], chapter 5
- AASHTO Model Drainage Manual[(1-2)], chapter 2

- FHWA's Federal-Aid Policy Guide, 23 CFR 650.115(a), "Procedures for Coordinating Highway Encroachments on Floodplains with the Federal Emergency Management Agency (FEMA)" (1-3)
- A Citizen's Guide to the NEPA

GDOT's mission is to deliver a transportation system focused on innovation, safety, sustainability, and mobility. In keeping with the GDOT mission statement, multiple levels of coordination must take place between GDOT and various federal, state, and local entities as a project progresses from inception through construction. Permits must be acquired in accordance with existing laws and regulations before a project can be approved for construction. Most of the drainage related permitting and agency coordination that is done for roadway projects will involve compliance with regulations that are in place to protect the environment. Drainage design decisions made on a project play a significant role in determining the extent of a project's impacts to environmental resources along the project corridor and therefore also play a role in facilitating a project's progression through agency review and permitting.

The roadway engineer or NEPA analyst and local government sponsors should coordinate with the GDOT Office of Environmental Services to determine which federal requirements apply to a project. Early Coordination is the means by which federal, state, and local agencies, and project stakeholders are informed of a proposed project. Determining the level of NEPA documentation begins with the Early Coordination process. The Avoidance and Minimization Measures Meeting (A3M) is part of the P6 schedule and is held after Resource ID activities are complete in order to identify, avoid and reduce resource impacts. See GDOT PDP(1-5), GDOT EPM(1-4), and GDOT Environmental Procedures Guidebooks for additional environmental procedures and coordination processes. The final decision on the level of documentation is not made until the environmental studies are complete. All environmental documents are subject to Early Coordination.

Active involvement by the engineer and environmental analyst will facilitate inter-agency communication and avoid project delays that may otherwise occur. When there is more than one reviewing or permitting agency involved, the rules and regulations of the more stringent regulator apply. In situations where these agencies or regulators contradict one another, it is the designer's responsibility to initiate steps to resolve the matter, most likely through a joint coordination meeting or similar means.

1.3.1 Federal Laws and Agencies

Federal laws and regulations are administered and regulated through multiple federal agencies and various regulatory authorities. Permits are typically required for proposed impacts to wetlands and other surface waters, impacts to civil works projects, and for construction over navigable waters of the United States. The federal agencies which administer permits for GDOT projects include:

- Federal Emergency Management Agency (FEMA)
  - For Floodplains (See section 1.3.3 for more details)
- US Army Corps of Engineers (USACE)
  - For wetlands,
  - For activities involving discharge of fill impacting waters of the United States (See Section 404 of the CWA for applicable Nationwide General Permits (NWPs)
  - For dams, dikes, or obstructions other than bridges across navigable waterways

- US Coast Guard (USCG)
  - For bridges and causeways across navigable waterways
- US DOT Federal Highway Administration (FHWA)
- US Environmental Protection Agency (EPA)
  - For point source and non-point source discharges and water quality
- US Fish and Wildlife Service (USFWS)
  - For Endangered Species and associated fish and wildlife resources
- US Department of Agriculture (USDA)
- Natural Resources Conservation Service (NRCS)
  - For infrastructure owned by the agency
- Tennessee Valey Authority (TVA)
  - For infrastructure owned by the agency
- National Oceanic and Atmospheric Administration (NOAA)
  - For streams with endangered species in commercial waters

Additionally, Presidential Proclamations and Executive Orders, federal agency regulations/ documents having general applicability and legal effect, documents required to be published by an Act of Congress, and other federal agency documents of public interest are published daily in the Federal Register. The general and permanent rules published in the Federal Register are codified and published annually in the Code of Federal Regulations (CFR) [(1-6)].

Compilations of Federal Statutory Law, revised annually, are available in the United States Code (USC) [(1-7)].

1.3.2 State and Local Laws and Agencies

The following are the primary state and local agencies having jurisdiction over project resources impacted by roadway drainage:

- GA Department of Natural Resources (GADNR)
- GA EPD
  - For NPDES, Stream Buffer Variance, Impaired Streams, and Safe Dams
- GADNR Coastal Resources Division (CRD)
  - For Coastal Areas and Marshes
- GA Soil and Water Conservation Commission (GSWCC)
  - For Erosion and Sediment Control Plans

Drainage design in roadway applications should adhere to all state ordinances. Georgia laws are published in the Official Code of Georgia Annotated (OCGA) [(1-8)].

Cooperative storm drain projects with cities and municipalities may be beneficial where both a mutual economic benefit and a demonstrated need exist. Early coordination with the governmental entities involved is necessary to determine the scope of the project. Each cooperative project may be initiated by a resolution adopted by the governing body of the municipality either (1) requesting the improvements and/or indicating its willingness to share the cost of a state project, or (2) indicating the municipality's intention to make certain improvements and requesting state cost participation in the municipal project.

1.3.3 FEMA and NFIP

GDOT and its consultants coordinate directly with the community and, through them, coordinate with FEMA. The local community with land-use jurisdiction, whether it is a city, county, or state, has the responsibility for enforcing National Flood Insurance Program (NFIP) regulations in that community if the community is participating in the NFIP. Consistency with NFIP standards is a requirement for federal-aid highway actions involving regulatory floodways. The community, by necessity, is the entity that must approve and sign proposals to FEMA for amendments to NFIP ordinances and maps in that community.

For riverine situations, when the Federal Insurance Administration has identified a flood-prone area without a designated floodway, the community must regulate the floodplain until a floodway has been established. Land-disturbing activities such as fill are not permitted within the floodplain in which base flood elevations have not yet been provided, unless the effect of the proposed use, when combined with all other existing and reasonably anticipated uses of a similar nature, will not increase the water surface elevation of the 100-year flood to more than the local ordinance requirement at any point within the community.

For areas with a designated floodway, where floodplain special flood hazards, the 100-year water surface elevations, and floodway data have been provided, the community must adopt a floodplain ordinance. The community must designate the local ordinance requirements for water surface elevation increases and prohibit, within the designated floodway, fill, encroachments, and new construction and substantial improvements of existing structures that would result in any increase in flood heights above the requirement during the occurrence of the 100-year flood discharge. The floodplain ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the NFIP.

For sites located within a FEMA regulatory floodway, the consultant is responsible for sizing a drainage structure that meets the standards and approval of GDOT, the affected community, and FEMA. The consultant provides the necessary forms, floodway and flood profile computer runs, and other supporting documentation as required for approval. All supporting documentation, along with copies of correspondence and approvals from the community and FEMA are provided to GDOT for its records and use. For state-aid projects, where the consultant prepares a hydraulic study for the community, the consultant, at a minimum, provides GDOT with a copy of a Letter of Concurrence from the community and approval from FEMA (if required).

Since longitudinal floodplain and floodway encroachments by new and widened roadways generally have a major effect on the flood elevations of the affected stream, roadway fill and longitudinal encroachments on the floodplain should be avoided.

1.3.3.1 FEMA Guidelines for Bridges and Culverts

All bridges within the state fall into one of the following five categories concerning FEMA involvement. All culverts fall within categories two through five.

1) If the stream has an established regulatory floodway, the structure should be designed so the bridge approaches do not encroach on the regulatory floodway. The bridge superstructure should also clear the floodway elevation. The bridge substructure is considered, in most cases, to be an insignificant encroachment. If the design keeps the bridge approach out of the floodway and the low chord above the floodway elevation, the affected community is sent a

copy of the proposed roadway plans and preliminary bridge layout with a letter stating the proposed construction does not encroach on the regulatory floodway, and a request for a "letter of concurrence" from the community, approving the project. If an exceptionally large pier is proposed in the floodway, especially at a new crossing, it may be necessary to treat the bridge under category 2 or 3 below. Also, if the project is located within a high-risk area as determined by the hydraulic engineer, it may be necessary to treat the bridge under category 2 or 3 below.

2) If the stream has an established regulatory floodway, and encroachment on the regulatory floodway is necessary, the structure should be designed so there will be no change in the base flood elevations, floodway elevations, or floodway widths at any cross section meeting a “No-Rise.” GDOT defines a "No-Rise" as causing no change in the base flood profile, or the floodway profile rounded to the nearest 0.1 foot or in floodway width rounded to the nearest 1 foot for any cross section outside GDOT’s right-of-way. Changes greater than 0.1 foot in the base flood profile or the floodway profile or 1 foot in the floodway width inside the right-of-way are considered integral to the bridge structure and do not affect another property.

   For consultant projects, a signed and sealed "No-Rise" certification by a registered professional engineer is required. If this criterion is met, two original sets of supporting documentation is prepared. One set is for submission to the affected community, requesting a "letter of concurrence" approving the project to be sent to GDOT. One set will be retained in the project file for GDOT's records. An example of a "No-Rise" condition can be shown in either of the two following cases for the floodway elevation:

   a) When the total difference in the calculated floodway elevations at a section is 0.05 foot or less. An example of a "No-Rise" for this condition is a comparison of the elevations 100.98 and 100.93 feet. Once water surface elevations exceed 0.05 foot, then a no-rise condition can no longer be claimed according to GDOT. The designer should note some local communities have more stringent regulations, such as not increasing at all (0.00 feet). A more stringent local requirement would need to be met to achieve a “No-Rise.”
   b) When the calculated floodway elevations are the same after rounding these elevations to the nearest 0.1 foot. An example of this condition is a comparison of the elevations 100.04 and 99.96 feet. Since both these elevations round off to 100.0 feet, this is considered a "No-Rise” condition as defined by GDOT. The designer should again note some local communities have more stringent regulations, such as not increasing at all (0.00 feet) in which case the local requirement would need to be met for a “No-Rise.” The comparison of floodway widths is similar to the above two cases involving the floodway elevation.

3) If the stream has an established regulatory floodway, and an encroachment on the regulatory floodway is necessary, and the “no-rise” criteria of category 2 are not met, then the affected community will need to make arrangements and obtain approval from any affected property owners and from FEMA. Coordination with FEMA will take the form of a Conditional Letter of Map Revision (CLOMR) where FEMA will review the proposed project and its anticipated impacts and will then provide a letter commenting on whether a proposed project, if built as proposed, or proposed hydrology changes would meet minimum National Flood Insurance Program standards. A LOMR is needed to update or revise the effective base flood elevations, floodway widths, and floodway elevations reported in any published FEMA Flood Maps or Flood Insurance Study (FIS) Reports. Revisions such as these often require local funding that

may not be available, further coordination will be required by GDOT and the local community on a project-specific basis to prepare the necessary map revisions.

For consultant projects, the Professional Certification Form required by FEMA is completed, stamped, and signed by a registered professional engineer (see Guidance for Hydrologic & Hydraulic Studies for reference). For cases such as these, GDOT requires three original sets of supporting documentation be prepared. Two sets are for submission to the affected community, requesting a "letter of concurrence" to be sent to FEMA and copied to GDOT. One set will be retained in the project file for GDOT's records. The community's "letter of concurrence" approves the impacts proposed to the floodplain/floodway along with the proposed revision to the base flood elevations, floodway elevations, and floodway widths. It is the responsibility of the designer to adhere to either GDOT design policy or the local floodplain ordinances, if they are determined to be more stringent.

4) **For a bridge crossing in a floodplain that is shown on a FIRM, but does not have a regulatory floodway, the bridge shall be sized to limit the backwater to no more than a 1-foot increase in the existing base flood elevation, unless the local community's ordinances are more stringent, in which case, the local regulation applies. See GDOT Standard Drainage Design Criteria for reference**.
   a) For Zone AE's with BFEs and a 2nd for Zone A's without BFEs that will have GDOT's 1' backwater limitation and concerning coordination: No coordination is required to replace existing structures. Coordination is strongly recommended if backwater is over 1 ft for the proposed for new location.
   b) For high density area where base flood has NOT been established nor a detailed study has been done, local coordination and subsequently our own engineering due diligence will come into play when properties are located within the floodplain despite not having a base flood established.
5) For bridges outside of NFIP communities or NFIP identified flood hazard areas, the bridge should be sized using the GDOT design criteria and requirements (see chapter 8).

1.3.4 EPD NPDES

The National Pollutant Discharge Elimination System (NPDES) permitting authority in Georgia is the EPD, a division of the Georgia Department of Natural Resources (GADNR). The EPD issues individual permits unique to each facility, as well as three general permits that authorize the discharge of stormwater from three distinct types of construction projects that disturb 1 or more acres of land. The three general permits are for (1) Stand-Alone Construction Activity (GAR100001), (2) Infrastructure Construction Sites (GAR100002) and (3) Common Development Construction (GAR100003).

An additional NPDES permit (GAR041000) was issued to from the Georgia EPD authorizing GDOT to discharge stormwater from a municipal separate storm sewer system (MS4) to the waters of the state of Georgia using appropriate stormwater management.

1.3.4.1 MS4 Permit Policy

NPDES Permit GAR041000 covers all new and existing point source discharges of stormwater from a municipal separate storm sewer system (MS4) owned and/or operated by the GDOT to the waters of the state of Georgia. This permit requires GDOT, and subsequently their consultants, to meet specific requirements within an MS4 area. MS4 policies, procedures, permitted area maps and BMP design guidelines can be found in the GDOT Stormwater Design Guide [(1-9)].

1.3.4.2 ESPC Permit Policy

Of the general permits, most GDOT related projects fall under GAR 100002. Applying for a general permit is accomplished by submitting a Notice of Intent (NOI) to the Georgia EPD. The NOI includes the location and description of the construction activity and defines the erosion, sedimentation, and pollution control plan (ESPCP) goals to minimize impacts with BMPs and monitoring. ESPCP should be implemented as designed, verified by any applicable sampling and inspection, and kept current. Typically, ESPCPs for GDOT's construction projects should be in compliance with the State of Georgia NPDES General Permit, the Manual for Erosion and Sediment Control in Georgia (Green Book)[(1-10)], the Georgia Soil and Water Conservation Commission (GSWCC), GDOT's own guidelines, and all other applicable federal and state laws and rules. See the GDOT PPG[(1-11)] for ESPC plan requirements.

1.4 Chapter 1 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2007. Highway Drainage Guidelines.
2. American Association of State Highway and Transportation Officials (AASHTO). 2014. Drainage Manual.
3. Federal Highway Administration (FHWA), Federal-Aid Policy Guide. “Highways.” Title 23 Code of Federal Regulations (CFR).
4. Georgia Department of Transportation (GDOT). 2012. Environmental Procedures Manual. https://www.dot.ga.gov/GDOT/Pages/EnvironmentalProcedures.aspx
5. Georgia Department of Transportation (GDOT). 2024. Plan Development Process (PDP).
6. *The Federal Register*. Code of Federal Regulations. https://www.ecfr.gov/
7. *OLRC Home*. United States Code. http://uscode.house.gov/
8. Official Code of Georgia Annotated. http://www.lexisnexis.com/hottopics/gacode/
9. State of Georgia Department of Transportation Stormwater Design Guide. 2024.
10. Georgia Soil and Water Conservation Commission (GSWCC – Green Book), 2016. "Manual for Erosion and Sediment Control in Georgia," Sixth Edition.
11. Georgia Department of Transportation (GDOT). 2023. Plan Presentation Guide (PPG).

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Chapter 2. Design Guidelines and Standards

2.1 Design Guidelines and Standards Introduction

Design guidelines and standards are defined as the basic principles and goals established by Georgia Department of Transportation (GDOT) to guide and control the design of roadways and related infrastructure in Georgia. Proper drainage is an essential element of transportation facilities as a large portion of roadway and infrastructure construction costs are devoted to the appropriate capacity of hydraulic structures to provide a safe, seamless, and sustainable transportation system that supports Georgia's economy and is sensitive to its citizens and environment.

This manual sets forth drainage design guidelines and standards for GDOT projects as they pertain to roadways, bridges, culverts, and related infrastructure. Flexibility is permitted to encourage independent design tailored to individual situations. When flexibility is applied to a proposed design, and critical criteria do not meet GDOT design policy, additional documentation is required to document the decision-making-process.

Criteria within this manual denoted as "standard" have been identified as required or mandatory practice with departure from the controls requiring agency approval. All other criteria within this manual are considered "guidelines" intended as recommended practice with flexibility allowed if engineering judgment or a study indicates the departure is appropriate.

For proper drainage design, the drainage facility should meet GDOT's criteria coupled with prudent engineering judgement accounting for, but not limited to, the following:

- minimum cost to meet design criteria;
- desired level of hydraulic performance;
- mitigation of impacts to the stream environment;
- safe movement of desired traffic volume under an acceptable level of service; and
- meeting social, economic, and environmental goals.

Unless stated otherwise, the policies in this manual apply to permanent construction of roadways and related infrastructure.

2.1.1 Definitions

The following definitions offer a framework for interpreting policy statements presented in this manual:

Standard: A required criteria or mandatory practice. Criteria denoted as standard are identified by the Department as having substantial importance to the operational and safety performance or maintenance of a roadway. Special agency review and approval, through a Design Variance, will be required before departure from the controls can be retained or incorporated into a design. In this manual, all standard statements are identified and formatted in **bold type**. The GDOT Standard Drainage Design Criteria are presented in **bold type with separation lines above and below the paragraph**. The verb "**shall**" is used to indicate these mandatory requirements.

Guideline: Recommended practice in typical situations. Departures from criteria denoted as guidelines are allowed when an engineering study and judgement determine them to be appropriate. An adequate study, justification, and documentation in the form of Design Deviation prepared by the GDOT office or consultant responsible for the engineering are required. It is recommended for the designer to consider all appropriate sections of the standard design variance template when writing a deviation. Internal deviations are usually recommended by the design group manager and approved by the roadway office head; external deviations are usually recommended by the project engineer and approved by the principal or project manager. Decisions to depart from guidelines are subject to review and scrutiny by GDOT at any time. The verb “should” is used.

GDOT Standard Drainage Design Criteria: GDOT identified the design elements listed below as having substantial importance to the operations and safety of a roadway. Special agency attention should be given to these criteria in the design decision-making process.

1) Design headwater, culvert, and bridge culvert
2) Check headwater, culvert, and bridge culvert
3) Freeboard, Bridge
4) Freeboard, Road Subgrades Adjacent to a Bridge
5) Backwater, Bridge
6) Gutter Spread, Roadways
7) Gutter Spread, Bridges
8) Inlets and Manholes Inside of Travel Lanes
9) Minimum Depth of Cover, Culvert/Storm Drain

The criteria defined by GDOT for each of these design elements are denoted as standard. The decision to use a value not meeting the criteria defined by GDOT in this manual will require prior approval of a Design Variance as defined in Section 2.2.1 of this chapter.

The words defined below and utilized throughout this manual provide additional assistance to a designer for interpreting policy statements found in this manual.

Shall: The use of the word “shall” denotes a required or mandatory condition, or standard, and the designer must make every effort to follow the appropriate design criteria or condition.

Should: The use of the word “should” indicates an advisory condition or guideline. Under this condition, it is recommended, although not mandatory, that the designer follow the appropriate design criteria.

May: The use of the word “may” indicates a permissive condition. Under this condition, the designer is encouraged to use sound engineering judgment.

Where practical: Practical is defined as effective and applicable; appropriate, adaptable, and balanced. The use of the term “where practical” is intended to indicate that the designer may consider economic resource constraints when making a design decision.

2.1.2 Sources of Design Policy and Practice

The following publications were used as references in the preparation of this chapter. The designer should refer to these publications for additional information regarding the legal framework within which stormwater runoff may be discharged from roadway systems to the natural environment. GDOT and FHWA references provide guidance on agency coordination that must take place to secure permits to construct roadways and their associated drainage systems.

- AASHTO Drainage Manual, Chapter 2
- AASHTO Highway Drainage Guidelines

- FHWA's Federal-Aid Policy Guide, 23 CFR 650.115(a), "Procedures for Coordinating Highway Encroachments on Floodplains with the Federal Emergency Management Agency (FEMA)"
- FHWA Publications - Hydraulic Engineering Circulars (HEC) and Hydraulic Design Series (HDS)
- Georgia Stormwater Management Manual, “Blue Book” (GSMM)
- Manual for Erosion and Sediment Control in Georgia, “Green Book” (GSWCC)
- GDOT Design Policy Manual
- GDOT Plan Presentation Guide
- GDOT Bridge and Structures Design Manual

For additional guidance on roadway drainage design and related infrastructure, refer to the most current edition of the publications cited in the References section of this Manual, unless a specific version is noted. The following is a list of sources for those publications.

- American Association of State Highway and Transportation Officials (AASHTO)
- Federal Highway Administration (FHWA)
- Georgia Department of Transportation (GDOT)
- Georgia Soil and Water Conservation Commission (GSWCC)
- National Cooperative Highway Research Program (NCHRP)
- Natural Resources Conservation Service/Soil Conservation Service (NRCS/SCS)
- United States Geological Survey (USGS)

2.2 Variances to Design Standards

2.2.1 Design Drainage Design Criteria

If an engineer cannot meet the control for a GDOT standard criteria, a Design Variance must be approved by the GDOT Chief Engineer or the State Design Policy Engineer. The requirement for a Design Variance is not to impede design flexibility, but instead to document a very important design decision that is well scrutinized by GDOT in a deliberate and thorough manner.

To obtain a Design Variance, a comprehensive study and a formal request must be submitted using the format and procedures outlined in the GDOT Plan Development Process manual (PDP).Table 2.1 outlines Design Variance degree for GDOT Standard Drainage Criteria.

All Design Variances should be submitted to the following email address:

DesignException@dot.ga.gov

Table 2.1 – Variances to Design Standards						
GDOT Standard Drainage Criteria	**Interstate, Freeway, Expressway, C-D Lanes & Ramps**	**State Route**	**Temporary**	**Driveways**	***Off-system**	**Off-system**
Headwater Clearance, Design Storm	DV 2	DV 2	N/A	N/A	DV 2	N/A
Headwater Clearance, Check Storm	DV 1	DV 1	DV 2	N/A	DV 2	N/A
Freeboard, Bridge	DV 1	DV 1	N/A	N/A	DV 3	N/A
Freeboard, Road Subgrades Adjacent to a Bridge	DV 1	DV 1	N/A	N/A	DV 3	N/A
Backwater, Bridge	DW**	DW**	N/A	N/A	DW**	N/A
Gutter Spread, Roadways	DV 1	DV 1	DV 2	N/A	DV 1	N/A
Gutter Spread, Bridges	DV 1	DV 1	DV 3	N/A	DV 1	N/A
Inlets and Manholes Inside of Travel Lanes	DV 2	DV 2	N/A	N/A	DV 2	N/A
Minimum Depth of Cover, Culvert/Storm Drain	DV 2	DV 2	N/A	N/A	DV 2	N/A
* When GDOT personnel resources are involved in design, engineering, ROW acquisition, or construction letting ** Design Waiver (see section 8.3.2 for additional information) 1 – Requires Chief Engineer's approval 2 – Requires State Design Policy Engineer's approval 3 – Requires State Bridge Engineer's approval						

2.2.2 Design Variance for Off-System Roadways

For off-system roadways a formal Design Exception or Design Variance as defined in the GDOT DPM and above in Section 2.2.1 will not be required regardless of whether state or federal funding is involved, with the two exceptions listed below:

Whenever employees of the Department are directly involved in the engineering and design, right-of-way acquisition, and/or construction letting of a project on an off-system roadway, then the normal approval of a Design Variance by the Department's Chief Engineer will be required before any deviation to minimum design standards can be incorporated into the project. This also applies to any of the above work activity being accomplished on behalf of the Department by consulting engineering firms or contractors hired by the Department.

The waiver of a formal Design Variance for off-system roadways does not affect project framework agreements between the Department and Local Governments, including the expectation to observe the design policies defined in the GDOT Design Policy Manual. The Department encourages local governments and their Engineer-of-Record to document all design decisions to demonstrate compliance with accepted engineering principles and the reasons for the decision.

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Chapter 3. Hydrology

3.1 Introduction

Hydrology is the study of the distribution, movement, and management of water in the environment. Hydrologic analysis considers the interactions of rainfall over a watershed and project site area and is used in the estimation of peak flows, volumes, and timing of runoff. Drainage design is dependent on accurate estimates of the peak runoff flows for the design storm event. Errors in the estimates will result in a structure that is either undersized and causes flooding problems or oversized and costs more than necessary. However, hydrologic analysis is only an approximation based on predictions of unknown future storm events. GDOT uses several methods to determine peak runoff flow rates and volumes proven to be reliable for use in design, operation, and maintenance of GDOT's drainage facilities. This chapter provides GDOT's policies and an explanation of these methods. For further information on hydrology and hydrologic analysis techniques, refer to FHWA HDS-2 and other documents referenced throughout this chapter.

3.2 Design Storm Events

Design storm frequency for GDOT roadway drainage facilities is based on achieving a balance between construction cost, maintenance needs, traffic volumes, potential flood hazard to adjacent property, and expected level of service. The design storm frequencies for the various road classifications and types of drainage facilities are located in the design storm frequency section of each specific chapter and represent the minimum design storm events that will achieve this balance. The design storm event is used in drainage design and serves as the basis for many of the criteria found throughout this manual. In addition to the design storm, a check storm is typically analyzed to determine the performance of the drainage design with a more intense storm event.

Additional information on the specific design storm policies can be found in the specific chapters and in Tables 4.1, 5.1, 6.2, 7.1, and 8.1.

3.3 Hydrologic Analysis Considerations

3.3.1 Flood Frequency

Flood frequency is the probability of a certain flow magnitude flood occurring during a given storm event. The design storm frequency for a given storm event is typically referred to by the Annual Recurrence Interval (ARI) which is the reciprocal of the Annual Exceedance Probability (AEP) that a storm event will be equaled or exceeded in a given year. For example, if a storm event has a 4 percent chance of being equaled or exceeded in a year, the storm event will probably be equaled or exceeded on average every 25 years and is referred to as the 25-year storm event. The designer should note that the 25-year storm event will not be equaled or exceeded once every 25 years but has a 4 percent chance of being equaled or exceeded in any given year. Therefore, the 25-year storm event could conceivably occur in consecutive years, or possibly even more frequently. Industry standard design storm events, include the 1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 200-year, and 500-year storms.

3.3.2 Rainfall

Rainfall is the direct cause of runoff, and thus is a critical variable needed for the calculation of runoff, regardless of method. The amount, pattern, spatial distribution, and duration for various frequency rainfall events for Georgia are published in TP40, NOAA Atlas 14, Volume 9 (HYDRO-35), and in individual hydrologic method publications such as TR-55. NOAA Atlas 14 [(3-8)] provides precipitation frequency estimates with confidence intervals for various storm durations and recurrence intervals and can be selected for precipitation depth or intensity. When using NOAA Atlas 14 for precipitation frequency estimates, the partial duration time series should be selected.

Rainfall intensity, I, is the most common statistical variable used in hydrologic calculations and is dependent on storm duration. Rainfall intensity relationships have been developed for most weather stations that record precipitation and have been summarized into intensity-duration-frequency (IDF) curves applicable to a defined region. The designer should determine rainfall intensity, I, for the computed duration and desired frequency by using the nearest established IDF relationship for that location. Rainfall intensities can be determined using data obtained from NOAA Atlas 14[(3-8)], or from local rain gauges. Gauges may be used for more accurate estimation of rainfall. See section 3.4.2 for information on obtaining gauge data, and section 3.5.1.4 for the rain-on-grid analysis method which can be used for rainfall and peak flow analysis with sufficient precipitation data for the project area.

3.3.3 Outlet Location

The outlet is the point where concentrated flow exits a contributing drainage area and in hydrologic design can be referred to as the study point. Identifying the study points for a project is the basis for determining the contributing basins and flows to be calculated in the hydrologic analysis. The study points in roadway applications are often the outfall of a stormwater system, culvert, or ditch.

3.3.4 Watershed Delineation

Once the study point has been identified, the next step in hydrologic analysis is the determination of the contributing drainage area. Drainage area delineation identifies all points flowing toward the outlet or study point. The drainage area can be determined manually or with hydrologic modeling software utilizing data from field surveys, USGS topographic maps, aerial photographs, or geospatial information. Automated basin delineation should only be used following site verification.

Complex watershed delineations, including urban areas, coastal areas, marshes, and other flat or continually changing terrain may require additional delineation techniques not included in this manual. More information and guidance on basin delineation for roadway applications is provided by the GDOT Stormwater (MS4) Management Program, including training classes and guidance documents. See ADW06 Basin Delineation and BMP Selection under Category: Stormwater Permit (MS4) & Special Design Post-Construction Details for more information:

https://www.dot.ga.gov/GDOT/Pages/designmanualsguides.aspx

3.3.5 Types of Flows

Runoff can be categorized into one of several types of flow as it moves over a landscape. Runoff typically begins as overland sheet flow in the upper reaches of the drainage area, as rainfall first interacts with the ground surface and does not infiltrate but instead moves over land. Runoff remains as sheet flow for a short distance, typically 50 feet or less, with a maximum of 100 feet. Sheet flow may transition to shallow concentrated flow, and eventually concentrated or channelized flow further down gradient. Concentrated flow includes flows in gutters, pipes, swales, channels, and streams. BMPs and outlet protection can revert concentrated flow to shallow concentrated flow, and in the instance of level spreaders, concentrated flow may be returned to sheet flow.

3.3.6 Travel Time/Time of Concentration

The time of concentration, Tc, is the time required for stormwater runoff to flow from the most hydrologically remote point of the drainage basin to the basin outlet, where remoteness relates to travel time, not necessarily distance. The time of concentration is a function of the size and shape of the drainage basin, slope of the ground surface, land use, rainfall intensity, and how the runoff is conveyed. In-depth calculation methods are in section 2.6 of FHWA HDS-2[3-10]. A simplified calculation method is to sum the travel times of sheet flow, shallow concentrated flow, and the concentrated flow segments (gutters, swales, channels, etc.).

The GDOT standard practice is to use a time of concentration no shorter than 5 minutes. Additionally, the presence of a drainage pavement course, such as OGFC, does not influence the time of concentration. This criterion is set due to limitations in both the empirical equations, as well as in the IDF curves for short durations, and to ensure consistency and standardization for design practices.

3.3.7 Land Use/Cover

Land use defines the type of ground cover in a particular area. There are separate Rural and Urban Regression Equations which consider differing land uses in the contributing drainage basin. A watershed is typically considered urban if the percent impervious area is greater than 10%. The Rational Method uses the runoff coefficient, C, and the TR-55 Method uses the curve number, CN, to represent the land use characteristics of the contributing basins. Composite land use values are frequently used to simplify peak discharge calculations by approximating the land use as uniform over the drainage area.

3.3.8 Slope

The slope of the ground surface within a drainage area can be determined from field surveys, USGS topographic maps, or geospatial information and represents the gradient over which the runoff is conveyed over and through the drainage area. This slope is used in time of concentration calculations, as well as other hydraulic considerations.

3.3.9 Soil Type

The type of soil and its infiltration characteristics within the drainage area will have an important effect on stormwater runoff. Soil and soil moisture characteristics can be obtained by field classification and testing, from NRCS soil surveys at http://websoilsurvey.nrcs.usda.gov, or geospatial information. Soil infiltration will vary with the magnitude and intensity of the rainfall. The NRCS TR-55 combines soil conditions and land uses in its runoff curve numbers. Runoff curve numbers are dependent on the hydrologic soil group (HSG), which is based on the runoff potential and infiltration rates of the soils. Well-draining soils are classified as HSGs A or B, while poorly draining soils are classified as HSGs C or D.

3.4 Hydrologic Data

3.4.1 Existing Studies and Records

The designer should use published flow records when available and applicable. Flow records are typically collected on larger watersheds and therefore are more commonly used for bridge and large culvert projects. The applicability of gauge data requires engineering judgement with reliability of the source and proximity of the gauge to the project location being the primary factors to consider. Flow data and studies establishing flow rates can be gathered from a variety of agencies, such as:

- Gauge Data (Rain Gauges and Stream Gauges) - USGS, GDOT project, National Weather Service, counties, airports, hurricane data:
    - https://maps.waterdata.usgs.gov/mapper
    - https://www.nhc.noaa.gov/data/tcr
    - http://waterdata.usgs.gov/ga/nwis/nwis
- FEMA FIS Report – FEMA Flood Map Service Center:
    - https://msc.fema.gov/portal/home
- Floodplain Information Reports – USACE Flood Risk Management:
    - https://www.usace.army.mil/Missions/Civil-Works/Flood-Risk-Management/
- Storm Event Database – NOAA Storm Events:
    - https://www.ncdc.noaa.gov/stormevents/
- Quantitative Precipitation Estimates –NOAA Advanced Hydrologic Prediction Service:
    - https://water.weather.gov/precip/
- Local community drainage master plans
- Nearby local projects/site plans/permits completed by other entities
- Historical Flood Records from local newspapers or municipality records

With accurate and location specific data, additional hydrologic analysis may be applicable. Statistical analysis may be used to estimate flows to better apply gauge data to a specific site location. When using statistical analysis, a minimum record of 10 years should be used to provide a reasonable statistical model. See section 3.5.1.4 for rain-on-grid method, and 3.5.1.5 for Flow Distribution Statistical Methods for Gauged Sites.

3.5 Hydrologic Analysis Methods

3.5.1 Design Methods

The type and source of information available for hydrologic analysis vary from site to site. The designer should select the hydrological method consistent with the characteristics of the drainage basin under consideration. See Table 3.1 for more information on hydrologic methods, their limitations, and applicable uses. Additional guidance can be found in FHWA HDS-2[(3-9)], HEC-22[(3-3)] or the Georgia Stormwater Management Manual (GSMM) Volume 2, aka the “Blue Book” [(3-2)] for methods for estimating stormwater runoff.

Table 3.1 - Typical Applications of Acceptable Hydrologic Methods

Application	Hydrologic Methods		
	Rational Method	NRCS TR-55(3-4)	USGS Equations
Water Quality		X	
Channel Protection		X	
Overbank Flood Protection		X	X
Extreme Flood Protection		X	X
Storage Facilities		X	
Outlet Control Structure		X	
Gutter Spread	X		
Storm Drain Pipes	X	X	
Culverts	X	X	X
Bridges		X	X
Small Channels	X	X	X
Natural Channels		X	X
Energy Dissipation	X	X	X

3.5.1.1 Rational Method

The rational method is based on the assumption that rainfall occurs at a constant intensity over the entire basin for a storm duration equal to at least the time of concentration of the basin. The rational method is commonly used for small basins and is a function of the interaction of rainfall intensity (I) falling on a given area (A) of a determined land cover which is represented by the dimensionless runoff coefficient (C). The Rational Method produces a peak rate of runoff at the most downstream point, with the same return period as the used rainfall intensity (I).

- Runoff Coefficient: The runoff coefficient, C, is a function of the land use, ground slope, topography, rainfall infiltration rate into the soil, and other factors. The higher the C value, the higher the runoff rate. Where the drainage area is a composite of several land use types, a weighted runoff coefficient should be calculated. A frequency adjustment factor (fa) should be used to compensate for less frequent storms where runoff losses have a proportionally smaller effect on runoff. This adjustment is applicable to areas of exposed soil or vegetation or for C values less than 0.6. Selecting the runoff coefficient for a drainage area, including the adjustment factor, requires careful engineering judgment by the designer.

*Applicable values for runoff coefficients for a 10-year storm frequency, and values for fa, can be found in Appendix C.

Limitations and Considerations:

- Recommended for use in basins with drainage areas less than 64 acres and is It is most applicable to small, highly impervious areas. For drainage areas over 64 acres,

engineering judgement should be used to determine the applicability of the site for the rational method, with the maximum allowable basin size for the rational method being 200 acres.
- The effects of antecedent conditions are not taken into account. Engineering judgement should be used in the areas with overly saturated or dry soils.
- Infiltration is not considered, resulting in higher runoff values.
- Based on an open and unconfined flow with no consideration for storage.
- Time of concentration values for the drainage area should be less than the duration of peak rainfall intensity.

Designers should refer to section 3.4 in this manual for the collection and estimation of input variables for the rational method.

3.5.1.2 NRCS TR-55 Method

The NRCS TR-55 Graphical Peak Discharge Method[(3-4)] provides simplified procedures to estimate peak discharge and to calculate urbanizing areas based on NRCS (formerly Soil Conservation Service (SCS)) procedures. The SCS peak discharge is calculated using a series of equations to establish a rainfall-runoff relationship and is based on precipitation, retention, and initial abstraction estimates within the basin. Retention and initial abstraction values within a basin are a function of land use, interception, infiltration, depression storage, and antecedent moisture which can be represented by the basin's runoff curve number.

- Curve Number: The curve number, CN, indicates the runoff potential of an area and is based on the combination of soil conditions and land uses within a drainage area. The higher the CN value, the higher the runoff potential. When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis.

  A Curve Number chart can be found in Appendix B which lists curve number values based on cover type, hydrologic condition, and hydrologic soil type.

The Tabular Hydrograph method, also described in the TR-55 documentation, is a less common method that involves using tables to estimate peak discharge. When TR-55 method is mentioned in this manual and in other documentation, it generally refers to the Graphical Peak Discharge method.

The TR-55 method is primarily used for the design of post-construction stormwater BMPs, although it is used for other calculations as well. See the Stormwater Controls Design Manual[(3-1)] for more detailed information on the NRCS TR-55 method as it applies to the design of post-construction stormwater BMPs.

Limitations and Considerations:

- This method assumes homogenous watersheds with one CN in one sub-catchment. If composite CN is calculated, it is assumed that the average land use, soils, and cover characteristics are uniformly applied across the area.
- TR-55 is primarily designed to be used on small watersheds. However, it can be applied to larger basins up to 2,000 acres. Because larger basins are less likely to be hydrologically homogeneous, larger basins should be carefully examined before using this method. As the

drainage area approaches the upper limit of 2,000 acres, other methods may become more preferable.

- The sub-catchment may only have one main stream, or additional streams should have nearly equal time of concentration values.
- Time of concentration values are limited to 0.1 – 10 hours (Graphical Peak Discharge Method).
- The minimum CN value should be 40.

This method is fully described in GSWCC Manual for Erosion and Sediment Control in Georgia, Appendices A-1, and B-1[(3-5)]. Adjustment factors as outlined in appendix A-2 of the manual may also be appropriate. If a higher degree of accuracy is warranted, or if the watershed is large and complex, use computer programs such as NRCS Technical Release 20 (TR-20), the USACE Hydrologic Engineering Center Hydraulic Modeling Software (HEC-HMS), or AQUAVEO Watershed Modeling System (WMS).

3.5.1.3 USGS Equations

The USGS developed regression equations for physiographically similar regions by performing a regression analysis on drainage basin characteristics to determine which were most highly correlated to peak flow rates. The regional regression equations relate peak flow rate for a specific recurrence interval to a particular basin's characteristics. The USGS regression method is used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows for drainage areas greater than 0.1 square miles or 64 acres. USGS Regression Method is the preferred method for areas larger than 160 acres or 0.25 square miles.

The USGS method can be used for most design applications, including the design of storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

USGS reports describe regression equations, which vary in applicability within each of the five hydrologic regions in Georgia. Separate equations are used for basins that are primarily rural and for those that are primarily urban. A watershed is considered urban if its impervious area is 10 percent or greater.

For rural ungauged drainage basins, regression equations are used to determine peak flow rates. The equations are based on watershed and climate characteristics specific to the hydrologic region. To estimate peak flow rates in rural ungauged areas, use the equations provided in the latest version of the USGS publication Magnitude and frequency of floods for rural streams in Georgia, South Carolina, and North Carolina [(3-7)]. These equations may be improved for an ungauged site near a gauged site by using a weighting factor. The gauge weighting method is explained in the current USGS publication.

Regression equations are also available for determining peak flow rates in urban areas and should be used where appropriate. The equations outlined in the latest version of the USGS publication Methods for estimating the magnitude and frequency of floods for urban and small, rural streams in Georgia, South Carolina, and North Carolina [(3-6)] should be used for urban calculations.

In addition to the regression equations, USGS has also published an Excel spreadsheet titled "Application of Methods Spreadsheet" that calculates peak flow rates for both rural and urban conditions. The spreadsheet for rural conditions is located on the USGS website http://pubs.usgs.gov/sir/2009/5043/ and the USGS Excel spreadsheet for urban peak flow calculations is at the following website: http://pubs.usgs.gov/sir/2011/5042/.

For areas that are urbanizing or not clearly rural or urban in land use, peak flows should be computed by both methods and the higher value used. On a nationwide basis, these regional equations have been compiled under the National Streamflow Statistics (NSS) program. The NSS program includes stand-alone computer software available at: http://water.usgs.gov/software/NSS/. The two sets of regression equations are updated periodically: be sure to use the most current equations.

3.5.1.4 Rain-on-Grid Method

Manual approaches to hydrologic computations may be performed with sufficient localized rainfall data. Grid cell representation is an analysis tool used to develop information required to model the watershed. This method breaks down a watershed into constituent grid cells, which represent the hydrological characteristics of the watershed. A composite of the cells can be used to create hydrographs and calibrate rainfall and peak flow calculations. The Rain-on-Grid method requires high-quality GIS data that may not be available for all areas. When considering this methodology, coordinate with GDOT Office of Design Policy & Support. See FHWA HDS-2 for more information.

3.5.1.5 Flow Distribution Statistical Methods for Gauged Sites

Where peak annual stream flows are measured, on gauged sites typically found on streams with large drainage areas or which are located in hydrologically sensitive areas, statistical analysis should be used to estimate the design peak discharge.

The data from gauged sites or sites nearby on the same stream can be statistically fit to a frequency distribution to estimate peak flow rates for storm events with specific recurrence intervals. Various methods of analysis can be used including but not limited to Log-Pearson Type III frequency distribution established in "Guidelines for Determining Flood Flow Frequency"[(3-9)], or manual computation using computer programs such as the Hydrologic Engineering Center's Statistical Software Package (HEC-SSP), or websites such as http://water.usgs.gov/osw/bulletin17b/dl_flow.pdf

3.6 Hydrologic Design Considerations

3.6.1 Hydrologic Analysis Software Programs

Hydrologic analysis can be modeled using a variety of hydrologic software. Hydraulic Toolbox, developed by FHWA, is a stand-alone suite of calculators that performs hydrologic analysis using the Rational Method. Aquaveo's Watershed Modeling System (WMS) supports other hydrologic modeling methods, including Regression Equations and TR-55. Other publicly available options include NRCS's WinTR-20 and WinTR-55, and USACE's HEC-HMS. Proprietary hydrologic software applications may be used by a designer if they utilize commonly accepted hydrologic modeling methodologies and produce verifiable and accurate results. Applicable software for different hydraulic analysis can be found in the respective chapters within this manual and online at https://www.dot.ga.gov/GDOT/pages/designsoftware.aspx.

3.6.2 Complex Hydrologic Analysis

For sites affected by regulation from dams or having other significant storage volume upstream of the project site, the storage should be considered when routing the various floods through the basin. Inflow and outflow hydrographs are used to determine the design discharges. For tidal areas, the storm peak flow rates are determined by tidal computer models, using the downstream boundary conditions (typically stage and time storm surge hydrographs) along with the applicable upland riverine discharge (upland drainage basin).

Refer to these websites for more information concerning tidal areas: http://tbone.biol.sc.edu/tide/ and http://coaps.fsu.edu/~hwinter/hycomtc/

The designer has the option of using TR-55 as a check for areas within the range of 30 to 500 acres. A larger upper range may be used in flat areas. Certain watersheds may lend themselves to the analytical methods presented in TR-55 which may be more appropriate, particularly in the coastal areas and areas with sandy and/or sandy loam soils.

3.6.3 Downstream Analysis

A downstream hydrologic assessment is required for many projects with a post-developed flow increase or to evaluate effects of water quantity control facilities (detention) on peak discharge and timing downstream in the watershed. For maintenance (resurfacing, shoulder paving, bridge rehabilitation, culvert rehabilitation, ITB), safety (guardrail, cable barrier, signal upgrades, sign installation, ITS, single lane roundabouts, RCUTs with no added lanes, other safety projects which add less than 0.25 acres of net new impervious area), pedestrian improvements, and bridge replacement projects (over waterways), a detention report should not be submitted outside of MS4 requirements. For other projects such as reconstruction and/or widening, the designer must evaluate post-development peak flows to determine if increased flows would cause flooding, spill outside of channel banks, overtop a road, result in some other adverse effects on downstream properties or if detention will increase downstream flows. For information on how to perform a downstream analysis for a GDOT project, see the GDOT Stormwater Controls Design Manual.

3.7 Chapter 3 References

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Chapter 4. Pavement Drainage

4.1 Introduction

Good drainage design involves properly balancing technical principles and data with the environment while giving due deliberation to other factors including safety, function, and cost. Effective drainage of highway pavements is essential to the maintenance of highway service levels and to traffic safety. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, and limit visibility due to splash and spray.[(4-2)]

Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity. The evaluation of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface. This chapter presents guidance and design criteria for the design of pavement drainage systems.

Roadway features considered in surface drainage, gutter flow, and inlet capacity calculations include:

- Longitudinal grade, cross slope, and gutter slope
- Pavement width
- Curb and gutter sections
- Pavement texture and/or surface roughness
- Bridge deck drainage
- Median barriers
- Roadside and median ditches

The pavement width, cross slope, longitudinal grade, and texture control the time it takes for stormwater to drain to the gutter section. The gutter cross-section and longitudinal slope control the quantity of flow that can be carried in a gutter section.

4.2 Gutter Spread and Design Storm Frequency

The flow of water in the gutter should be restricted to a depth and corresponding width that will neither obstruct the roadway nor present a hazard to the traveling public. Proper hydraulic design of pavement drainage is dependent on allowable gutter spread calculations for a designated design storm frequency. Two of the more significant and interrelated variables that must be considered in the design of highway pavement drainage are (1) the allowable gutter spread and (2) the frequency of the design storm event

4.2.1 Allowable Gutter Spread

Gutter spread is the perpendicular distance from the face of curb or barrier to the furthest extent of water on the roadway during the design storm (Figures 4.1 and 4.2). It is typical for urban roads to contain a curb to define the outer limit of the gutter and keep the water off the sidewalks. The use of curbs on rural roads is generally not acceptable.

Allowable gutter spread widths vary depending on the roadway classification and design speed as shown in Table 4.1. Gutter spread width shall be limited to the values shown in Table 4.1 and a decision to exceed these widths shall require an in-depth engineering study

and Design Variance as well as approval from the GDOT Chief Engineer. The spread criteria listed is for permanent design and temporary construction conditions. See GDOT Standard Drainage Design Criteria for reference.

Table 4.1 Allowable Gutter Spread

Facility [a] [c]	Design Speed	Design Storm Frequency (yr)	Maximum Encroachment (ft)
Interstates	All Design Speeds	10	No Encroachment[d]
	Sag (all design speeds)	50	
State Routes & Hurricane Evacuation Routes	≤ 45 mph	10	½ Lane Width
	> 45 mph	10	No Encroachment[d]
	Sag (≤ 45 mph)	50	½ Lane Width
	Sag (> 45 mph)	50	No Encroachment[d]
Non-State Routes [b]	≤ 45 mph	10	½ Lane Width
	> 45 mph	10	No Encroachment[d]
	Sag (≤ 45 mph)	10	½ Lane Width
	Sag (> 45 mph)	50	No Encroachment[d]

a) The maximum encroachment is applied to temporary conditions and turn lanes as well.
b) Non-State Routes only: Entire width of bike lane or parking lane may be included in maximum encroachment.
c) In areas where PEM/OGFC is utilized, allowable gutter spread shall be confined to the shoulder at the limit of the PEM/OGFC with zero depth.
See Section 9.2 for allowable gutter spread on bridges.
d) Specifically, no encroachment into the outside travel lanes.

Design temporary drainage for traffic diversions and construction phases to provide drainage in areas where construction activities may divert or trap water, compromising the safety and efficiency of the travel lanes. Give additional attention to expected spread for areas that are (1) flood sensitive, (2) high-speed facilities with posted speed limit greater than 55 mph, or (3) using a low-side barrier wall.

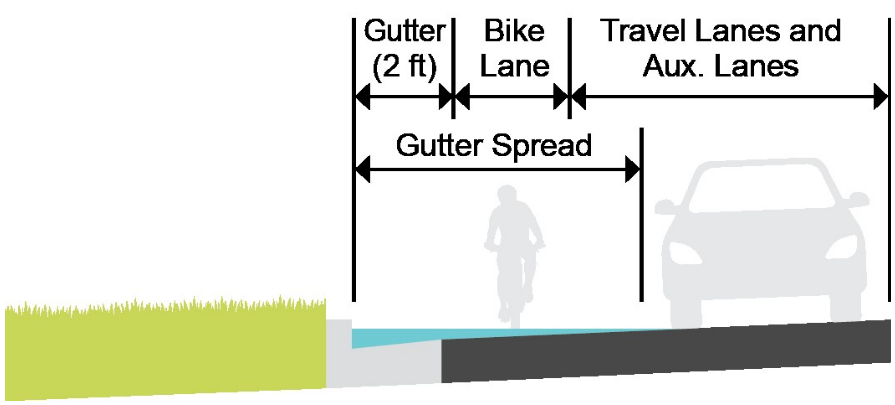

Figure 4.1 – Gutter spread in a typical non-state route urban section for a design storm event

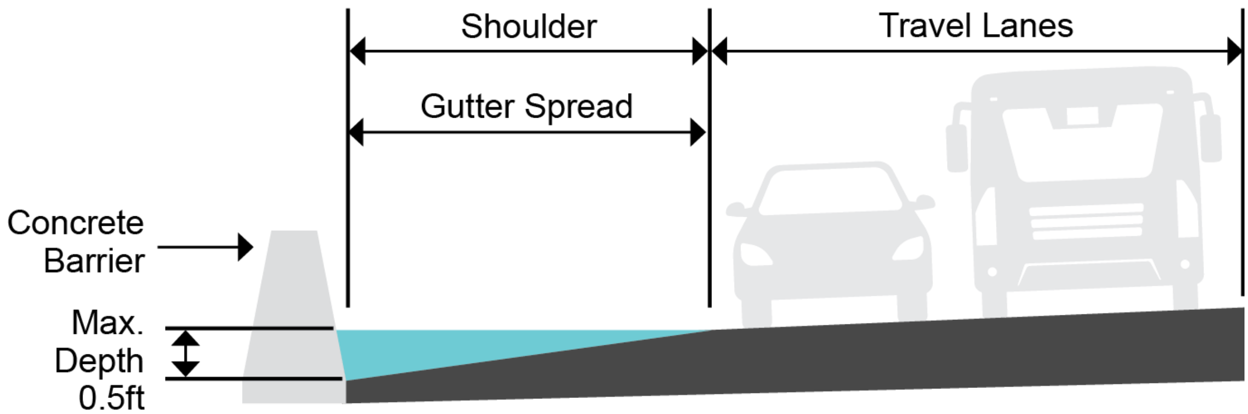

Figure 4.2 – Allowable gutter spread confined to a shoulder on an interstate and state route with a design speed > 45 mph for a design storm event. The preferred maximum depth is 0.5 ft.

4.2.2 Design Storm Frequencies

Inlets and drainage systems at locations, such as depressed sections and underpasses where ponded water can be removed only through the storm drainage system, should be designed to the 50-year frequency storm event so that drainage structures are not hydraulically surcharged.(4-3)

At low points where stormwater can exit the roadway by overtopping the curb without the occurrence of significant ponding, it is typically not necessary to design the drainage system to the 50-year storm event or to evaluate the performance of the system using a check storm event.

The use of less frequent events, such as a 100-year storm, to assess hazards at critical locations is commonly referred to as a check storm or check event. (4-3) A check storm should be used any time runoff could cause flooding during less frequent events. Typically, this occurs in cut sections and/or at flat longitudinal grades where the runoff lacks adequate positive drainage. Inlets should always be evaluated for a check storm in sag vertical curve where ponding to hazardous depths could occur.(4-3)

As illustrated in Figure 4.3 below for a 2-lane roadway (most critical), the criterion for gutter spread is that a minimum of one lane width of traffic should remain open during the check storm event with a maximum depth of water on the pavement (0.5 feet) at sag locations.

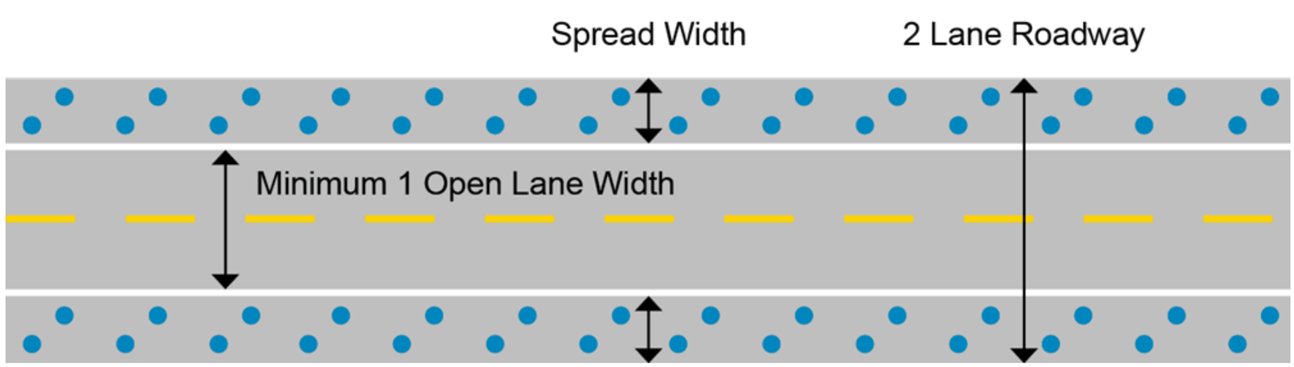

Figure 4.3 – Allowable gutter spread and lane width for check storm event

4.3 Gutter Spread Standard Criteria

The basis for the gutter flow principles discussed in this chapter can be found in Section 4.3 of the FHWA's HEC 22 manual.(4-3)

Gutter flow calculations are necessary to establish a relationship between the total quantity of flow (Q) to the spread of water on the gutter and pavement section. The pavement section includes the travels lanes as well as any shoulders, berms, sidewalks, parking areas or other paved areas adjacent to the roadway. The main components that influence gutter flow are the following:

- longitudinal gutter slope (gutter grade)
- transverse (cross) slopes of the pavement
- roughness characteristics of the gutter and pavement
- inlet spacing

4.3.1 Longitudinal Slope - Gutter Grades

A minimum longitudinal gutter grade of 0.5% is desirable for curbed roadways, but a minimum grade of 0.3% may be used where the paved surface has adequate cross slope.(4-1) Longitudinal grades less than 0.3% may be allowed when mitigation measures such as increased road cross slope or decreased inlet spacing are utilized to meet gutter spread requirements listed in Table 4.1 above.(4-4)

A minimum longitudinal grade of 0.3% should be maintained within approximately 50 ft of the level point on sag and crest vertical curves. This minimum criterion corresponds to a K value of 167 ft per percent change in grade (ft/%). Difficulty with routing the drainage away from the level point on crest vertical curves is typically not experienced when this criterion is met. (4-1)

Special attention to drainage should be exercised when flat sag or crest vertical curves are used (i.e., K value is greater than 167 ft/%). Varying or "rolling" the roadway profile can achieve minimum gutter grades in flat terrain. Varying the cross slope of the travel lanes and/or shoulders is another option to consider for facilitating drainage on a case-by-case basis. (4-1)

4.3.2 Cross Slopes

The design of pavement cross slope is a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort.

Typical practice is to provide a minimum of 2% pavement cross slope for travel lanes, however at certain superelevated sections or superelevation transition areas, the cross slope may be less than 2%. On multi lane roadways, it is recommended that the cross slopes be designed such that they are consecutively higher as the water drains across each lane. In addition, the pavement crown should be located to minimize too many travel lanes sloping to one side. The combination of two practices not only removes the runoff from the lanes faster but also reduces the depth of flow.

Cross slopes on superelevated sections of roadway should typically not exceed 8% due to the hazards associated with snow and ice. However, superelevated cross slopes larger than 8% may be considered on a case-by-case basis for roadways (i.e., loop ramps).

Tip: Whenever possible, low points and high points should coincide with the PI of the horizontal curve. This significantly reduces drainage problems associated with flat cross slopes in superelevation transition areas. Conditions which should be avoided in (1) both high and low points at intersections, (2) low points in cut areas, and (3) low points on bridges and bridge approach slabs.

4.4 Gutter Flow Computations

Refer to Chapter 4 of FHWA's HEC-22 manual for gutter flow and spread computations, utilizing Table 3-4 (4-3) for Manning's n coefficients for pavement. The recommended manning's n value for concrete and asphalt are 0.015 and 0.016, respectively.

The Rational Method (see chapter 3 of this manual) is typically used for inlet design. Selection of design frequency (storm year) should be obtained from the policy in section 4.2.1. Factors that govern flow "Q" are runoff coefficient ("C"), rainfall intensity rate ("i") and contributing drainage area "A." The rainfall intensity rate depends on the design storm frequency and duration time. Duration time is assumed to be equal to the time of concentration. A minimum time of concentration of 5 minutes is to be used to estimate the rainfall intensity rate when the total time of concentration is less than 5 minutes. Refer to Chapter 3 of FHWA's HEC-22 for additional information on runoff coefficients and time of concentration computations. (4-3)

4.4.1 Placement of Inlets on Continuous Grades

Placement and spacing of inlets on continuous roadway grades are dependent upon the gutter spread and geometric controls. Gutter spread is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter geometry. Maximum allowable gutter spread widths are defined in Table 4.1 in section 4.2.1.

Selection of inlet locations on continuous grade may be done using a FHWA's HEC 22 based computer program or by using a hand tabulation method.

Whatever calculation method is chosen, it should be thoroughly documented so the calculations may be easily followed and reproduced by a reviewer.

4.4.2 Low Point and Flanking Structures

At sag points where stormwater cannot escape the roadway and becomes confined behind curbing with no outlet, a minimum of one flanking inlet should be placed on each upstream side of the inlet at the sag point. The purpose of the flanking inlet is to provide a low point relief if the inlet should become completely clogged such that the allowable gutter spread is not exceeded. Where stormwater has the potential to escape the curb, the shoulder slope should be flattened or even reversed at the sag to provide an outlet.

4.5 Inlet Types

Inlets used for the drainage of pavement surfaces can be divided into five major classes. These classes are as follows:

- Curb opening inlets (all GDOT STD 1033 and 1034 series, GDOT STD 9017c)
- Combination inlets (GDOT STD 1010A, GDOT STD 1013, GDOT STD 1019A Type E, GDOT STD 1019B Type V-1)
- Grate inlets (GDOT STD 1010B; GDOT STD 1010C; GDOT STD 1010D; GDOT STD 1019A Types A, B, C and D; GDOT STD 1019B Type V-2; GDOT STD 5001M; GDOT STD 5001N; GDOT DETAIL D-33, D-33B, D-36)
- Slotted drains (GDOT DETAIL D-27)

- Concrete spillways (GDOT STD 9013 Types 1, 2 and 3, GDOT STD 9017J Types, 1, 2, 3 and 4))

Refer to GDOT's Construction Standards and Details for the above listed inlets.

4.5.1 Design of Inlets

The inlets covered in this section are drainage structures used to collect surface water adjacent to curbs or barrier walls where gutter spread must be evaluated and controlled. Several computer modeling software packages are available to perform inlet capacity calculations, such as FHWA's Hydraulic Toolbox, StormCAD, and FlowMaster. GDOT does not specify a particular method be used but does recommend that the results be included on the standard GDOT results form for ease of review. Inlets used for the drainage of highway surfaces can be divided into the following classes:

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

4.5.2 Curb-Opening Inlets

Curb-opening inlets are vertical openings in the curb covered by a top slab. They can convey large quantities of water and debris. They are less susceptible to clogging than slotted drains and grate inlets, and preference should be given to their use in sags. In some cases, grate inlets or combination inlets are preferable in heavily urbanized areas. Concrete top curb inlets should not be used in radii where there is potential for overtracking. For continuous grades above 3%, where bypass may be excessive, grate inlets are normally used.

4.5.3 Combination Inlets

Curb-opening and combination inlets are common. Slotted inlets are also used in combination with grates, located either longitudinally upstream of the grate or transversely adjacent to the grate. Engineering judgment is necessary to determine if the total capacity of the inlet is the sum of the individual components or a portion of each. The longitudinal pavement grade, cross slope, and proximity of the inlets to each other will be deciding factors. Combination inlets are more desirable than grate inlets in sags because they can continue to receive stormwater flow when the grate becomes clogged. For inlets placed in radii that are subject to overtracking, concrete top inlets should not be used. Metal-hooded combination inlets or combination inlets with mountable hoods that are subject to overtracking should be used in radii to prevent crushing.

Capacity Calculations for Combination Inlets

- For combination inlets on grade, the designer should only use the grate component for capacity calculations.
- For combination inlets in a sag location, the designer should use the curb opening and grate components of the inlet for capacity calculations. The designer should use a 50% efficiency factor to account for potential clogging of the inlet.

4.5.4 Grate Inlets

Grate inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Because they are susceptible to clogging with debris, the use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. An example of a minor sag point might be on a side road as it joins a mainline. When grates are used, it is a good engineering practice to assume 50% of the flow intake area is clogged for gutter spread design purposes. A major sag point is a location where water will pond 1 foot or more when the sag inlet is clogged. Special-design (oversized) grate inlets can be utilized at major sag points if sufficient capacity is provided for clogging. In this case, flanking inlets should be used. At low points where significant ponding can occur, such as at underpasses and in sag vertical curves, at least one flanking inlet on each upstream side of the sag inlet may be appropriate. Flanking inlets should be placed on low-gradient approaches to the low point to limit spread within the tolerances given in Table 4.1. It should be assumed that the sag inlet is completely clogged when designing flanking inlets for placement and gutter spread control.

Grates should be bicycle safe, where bicycle or wheelchair traffic is anticipated and structurally designed to handle the appropriate loads when subject to traffic. Because of higher speeds, special consideration is given to limited access highways.

For limited access highways, GDOT STD 5001M should be used on the inside shoulder and adjacent to a concrete barrier where the shoulders are 4.5 feet or wider and bicycles are prohibited.

4.5.5 Slotted Drain Inlets

These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs with flow entering from the side. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets. The two types of slotted inlets in use are the vertical riser type and the vane type. Note that slotted drains are not recommended for use in sags since they are more easily clogged than other inlet types. Slotted drain inlets can be used on curbed or uncurbed sections. Slotted drains may be used to supplement an existing low-point inlet.

4.6 Design Procedures

Design procedures presented in FHWA's HEC-22[(4-3)] for pavement drainage should be followed. When designing pavement drainage, it is recommended to follow the following steps: Firstly, collect and analyze any existing data related to the site. This will help to identify any potential issues that need to be addressed in the design. Secondly, create a preliminary layout for the drainage system, taking into consideration best practices for placement. This can include factors such as the slope of the pavement, the location of existing structures, and the volume of water expected to be drained. Finally, make sure to properly treat any existing drainage structures to ensure they are functioning correctly and can handle the expected volume of water. By following these recommendations, pavement drainage systems can be designed to effectively manage stormwater and prevent flooding or other issues.

4.6.1 Collect and Analyze Existing Data

The following information is recommended for design:

- Existing natural points of concentration and discharge
- Existing drainage systems
- Existing topographic features (contour quad maps)

- Preliminary proposed plans, profiles, cross sections, superelevation
- Determination of runoff (see chapter 3 of this manual)
- Existing pipe data from the Survey Data Engineer (SDE), local authorities, district maintenance office, or other similar source

Use the above collected data to make the following assessments and determinations:

- Determine natural flow patterns of the natural points of concentration and discharge
- Locate existing features, structures, pipes, top elevations, invert elevations, pipe sizes, etc.
- Evaluate existing information on condition and type of existing pipes and structures to determine if any deficiencies exist

Tip: Plotting features on a roll plot will give a better overview than separate plan sheets

4.6.2 Preliminary Layout and Inlet Placement Best Practices

Drainage structure locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass.

Inlets are required whenever the spread on the pavement reaches the limiting design criteria. There are a number of locations where inlets may be necessary with little regard to the contributing drainage area. The following list provides guidance for placement of drainage structures on roadway projects.

Inlets should be placed:

- at all sag locations and low points in the gutter grade.
- on continuous grades to control gutter spread per Table 4.1 in section 4.2.1.
- in locations to minimize sheet flow across the roadway
- immediately upstream of median breaks, entrance/exit ramp gores, crosswalks, and street intersections (i.e., at any location where water could flow onto the travel way).
- immediately upstream of bridges to prevent pavement drainage from flowing onto bridge decks.
- immediately downstream of bridges to intercept bridge deck drainage even where deck drain systems exist.
- within approximately 50 ft upgrade of flat cross slopes in superelevation transition areas.
- immediately upgrade of pedestrian crosswalks.
- on side streets immediately upgrade from intersections.
- in low areas behind curbs, shoulders, or sidewalks.
- in pocketed low points. Pocketed low points commonly occur on driveways where runoff that drained to the roadway prior to construction now drains away from the roadway to the driveway.

Inlet and site-specific placement recommendations:

- Grate inlets should be located outside the through traffic lanes
- Grate inlets must be designed and installed to ensure the safety of bicycles in all situations where used on roadways that allow bicycle travel.
- Grate inlets with a hood should be placed within turning radii that are subject to overtracking. Curb-opening inlets are not to be used in radii. Metal hoods are required in radii to prevent crushing of the inlet top.

- Inlets placed against MSE, or other retaining walls can present structural issues. If practical, avoid placing drainage structures that interfere with the reinforcing or anchoring elements of MSE/anchored retaining walls. If it is necessary to place drainage structures in the reinforcing or anchoring zone, limit the depth of the drainage structure to minimize the impacts. Any drainage structures placed next to MSE walls should be coordinated with the geotechnical and/or structural engineers.
- Drainage inlets for noise walls should be provided to ensure that the runoff does not "pond" behind the wall. Refer to Chapter 6 of the GDOT Design Policy Manual for additional information. [(4-4)]
- Manholes should be placed outside the travel lanes and away from curb ramps.
- Use manholes rather than junction boxes when outside the roadway travel lanes and when site-specific obstacles do not exist in order to provide access.
- Concrete spillways (i.e., GDOT STD 9013) should be used at curb ends when utilizing curb inlets is not feasible.
- Special drainage systems such as trench and slotted drains should be considered and utilized as necessary to control gutter spread within tolerable limits. Trench and slotted drains should be avoided in areas exposed to high volume of traffic or to high-speed traffic.
- Concrete spring boxes (i.e., GDOT STD 9031L) should be used in special circumstances to allow a natural spring or seep to continue to flow after being buried by project grading.
- Roadside channels or inlets should be used to intercept runoff from areas draining toward a highway. This applies to drainage from cut slopes, side streets, and other areas adjacent to and draining toward the mainline pavement.

4.6.3 Existing Drainage Structures

The designer should note existing drainage structures to be removed and replaced in the drainage plans. Existing drainage structures that are to be retained within the project limits should be adjusted or reconstructed to the finish elevation as appropriate. Any existing brick drainage structures that are located in the foundation of a retaining wall must be removed and replaced with cast in place or precast structures. See Chapter 5 for definition of the wall foundation.

4.7 Risks of Inadequate Pavement Drainage

When inadequate pavement drainage occurs, a layer of water may build up between the tires of a vehicle and the road surface, leading to a loss of traction, which is known as hydroplaning. There are several factors that influence the potential for hydroplaning including vehicle speed, vehicle weight, the quality of tire tread, rainfall intensity, pavement surfaces, and the geometry of the roadway or bridge.

During the design of a roadway or bridge, designers should be aware of the potential for build-up of water when one or a combination of the following roadway conditions occur:

- Along horizontal curvature with less than (<) normal crown cross slope, such as transitions in and out of full super elevation;
- Along the peak of a crest vertical curve where longitudinal slope is flat; and
- Along the bottom of a sag vertical curve where the potential for ponding is greater during intense rainfall.

In locations such as this, some typical design measures include:

- Intercept gutter flow before superelevation transitions.
- Limit gutter spread by decreasing inlet spacing.
- Limit ponding depth in sags by increasing the number of inlet structures.

It is important to note that the normal use of design policy, guidelines, and standards adopted by GDOT are sufficient to minimize the potential for hydroplaning on roadways and bridges. Whenever a designer is proposing to retain a non-standard cross-slope where a Design Exception or variance is required, then GDOT may require a hydroplaning study be provided to support the decision. For additional details on hydroplaning calculations see FHWA's HEC-22.

4.8 Chapter 4 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2018. Geometric Design of Highways and Streets, 7th Ed.
2. American Association of State Highway and Transportation Officials (AASHTO). 2014. Model Drainage Manual, 3rd Ed.
3. Brown, S.A., Schall, J.D., Morris, J.L., Doherty, C.L., Stein, S.M., Warner, J.C. 2009, Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22, FHWA-NHI-10-009. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.
4. Georgia Department of Transportation (GDOT). Current Version. Design Policy Manual

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Chapter 5. Storm Drains

5.1 Introduction

A storm drain system associated with transportation facilities consists of the various inlet structures, storm drain pipes, junctions, manholes and other minor structures that are used to collect, convey, and discharge stormwater through pipes to an outlet within and along the highway right-of-way. Storm drain designs should convey and discharge stormwater through a system which minimizes impacts to surrounding properties, developments, utilities, water bodies and environmentally sensitive areas, while improving the safety and maintenance concerns of transportation facilities.

Storm drains are designed after inlet locations and types have been identified. This chapter provides guidance and design criteria for storm drain pipes and drainage structures based on procedures presented in the American Association of State Highway and Transportation Officials (AASHTO) publications, AASHTO Drainage Manual (ADM) [(5-1)], AASHTO Highway Drainage Guidelines[(5-2)] , and the FHWA publication, Urban Drainage Design Manual (HEC-22).[(5-3)]

Prior to starting a storm drain design, the designer should have an understanding of the basic hydrologic and hydraulic concepts and principles of closed conduits, open channels, and the concepts related to their hydraulic performance. Proper storm drain system design includes system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing and hydraulic grade line calculations, to ensure that the system conveys stormwater runoff per the appropriate guidance and design criteria. Chapters 4 and 5 should be used in tandem to ensure storm-drain facilities provide enough combined capacity in the storm drain and the street typical section to convey major-storm runoff through the roadway right of way in a manner which adequately drains the roadway and minimizes potential for flooding and erosion to properties adjacent to the right of way. The most serious effects of an inadequate roadway-drainage system are:

- Damage to adjacent property from water overtopping curb and gutter;
- Risk and delays to the driving public caused by excessive ponding in sag vertical curves, or excessive spread along the roadway;
- Deterioration of pavement structure and subgrade due to saturation caused by frequent and long-duration ponding; and
- Creation of hydroplaning conditions for motorists.

5.1.1 Definition

A storm drain is the portion of the roadway drainage system that receives runoff from inlets and conveys the water through pipes to an outlet where it is then discharged into a channel, water body, or piped system. A storm drain may be a closed-conduit, open channel, or some combination of the two. A section of pipe connecting one inlet or drainage structure to another is termed a "segment" or "run." The storm drain pipe is usually a circular pipe, but can also be a box, elliptical pipe, or other enclosed conduit shape. Drainage structures include inlets (excluding the actual inlet opening), access holes, junction boxes, and other miscellaneous assemblies.

5.2 Storm Drain Capacity and Frequency

Storm drains should be designed to operate without inhibiting the capture efficiency of the inlets by accounting for design peak flow events (design storms). Storm drainage conveyance systems include the single or mixed use of curbs, gutters, storm drains, channels, and culverts.

5.2.1 Storm Drain Capacity

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and roughness coefficient. Storm drain pipes should be sized to flow full or nearly full when carrying the design storm peak flow. Pressure flow should be avoided.

The desired flow regime for the design of a storm drain system should be open channel flow. Pressure flow or surcharging of the drainage system is not desirable, but may be allowed with GDOT Design Policy and Support Office coordination showing the following requirements are met:

- Hydraulic grade line remains below intakes and manhole lids, and
- Watertight joints to withstand pressurized flow are specified.

5.2.2 Design Storm Frequency

The highway classification plays a major role when selecting the appropriate design storm frequency for a storm drain system. The highway classification can be found online on the GDOT State Functional Classification Map.

Table 5.1 provides design storm frequencies for all types of road classifications either on grade or in sags, and traffic volumes.

Table 5.1 — Storm Drain Design Storm Frequency Criteria

Road Classification	Design Storm	
	Outlets Available [1]	Storm Drain is the Only Outlet [1]
Interstate and Freeways	10-year	50-year
Principal and Minor Arterial	10-year	50-year
Minor / Major Collectors	10-year	50-year
Local Streets AADT $\geq$ 400	10-year	50-year
Local Streets AADT < 400	10-year	10-year
Temporary Drainage	10-year	10-year

(1) With a few exceptions, longitudinal and transverse pipes for storm drains are to be designed for the 50-year design storm where the flow has no outlet except through the storm drain system. For example, for an outlet pipe in sag location.

Inlets and drainage systems at locations, such as depressed sections and underpasses where ponded water can be removed only through the storm drainage system, should be designed to the 50-year frequency storm event so that drainage structures are not hydraulically surcharged.[(5-3)] A check storm (e.g., a 100-year storm), should be used to evaluate performance of the storm drainage system per Chapter 4.2.2. If major storm flows cannot be accommodated by the designed drainage system, the size, design storm, and capacity of the system should be reevaluated to mitigate potential flooding hazards to ensure that the drainage system meets allowable gutter spread criteria in Chapter 4.2.1.

5.3 Design Criteria

An effective storm drain design provides capacity to safely convey stormwater in an economical and efficient manner. Storm drain size is determined by the capacity needed to convey design flows by analyzing the HGL while accounting for losses. Other factors to consider when determining the appropriate size and resulting capacity are design velocity and pipe material.

5.3.1 Minimum Pipe Size and Material

The minimum new storm drain pipe size is 18 inches. Specific projects, such as those located in flat terrain, may dictate a minimum pipe size larger than 18 inches to account for sediment accumulation and clogging.

Designers should design all storm drain pipes with the assumption that reinforced concrete pipe will be used on the project, unless project specific conditions dictate the use of other pipe materials. It should be noted that the Contractor may choose to utilize other pipe materials during construction based upon the Pipe Culvert Material Alternatives Chart provided in GDOT Specifications. See Section 5.5 for more information. The Contractor should provide additional storm drain capacity calculations, at his own expense, if a pipe material other than reinforced concrete is selected.

Storm drain pipe should first be designed as concrete in the concept design phase. In later design phases, if a different material is recommended by the designer based on site specific geotechnical, environmental, and regional conditions, OMAT Pipe Culvert Material Alternates chart and the GDOT Construction Standards 1030D1, 2, and 3 should be used to select a suitable alternative material. A full list of GDOT Construction Standards and Details can be accessed via GDOT's R.O.A.D.S. Repository for Online Access to Documentation & Standards. Documentation showing that alternative material meets OMAT Pipe Culvert Material Alternatives chart should be provided in accordance with the Plan Development Process.

Representative values for the Manning's coefficient for various storm drain materials are provided in Appendix A. The values in the table are for new pipes tested in a laboratory.

5.3.2 Minimum Depth of Cover and Clearance

Storm drains should be designed as close to the surface as possible while meeting minimum depth of cover and hydraulic requirements, minimizing excavation costs, and maximizing safety for the construction crew. The pipe depth should be set based on either the minimum depth of the inlet for the pipe size or the minimum cover for the pipe.

The minimum depth of cover for all storm drains shall be 12 inches for concrete or based on Table No. 1 of GA Standard 1030D for other types of material. Depth of cover is measured from the top of the subgrade soil to the outside crown of the pipe. Minimum cover is maintained at all points where a pipe is beneath travel lanes or shoulders. See GDOT Standard Drainage Design Criteria for reference.

The minimum clearance between underground utilities and the exterior surface of storm sewer pipes shall be 6 inches.

Areas near sag points are vulnerable to cover deficiencies since the storm drain system traverses a vertical curve. Cover depths should be checked at regular intervals to ensure the minimum cover requirement is met. An additional drainage structure may be added at the minimum cover location to provide sufficient cover. In addition, a best practice is to show drainage structures at the correct scale on the roadway profile drawings to facilitate checking the minimum cover criteria (refer to chapter 4 for more information on structures).

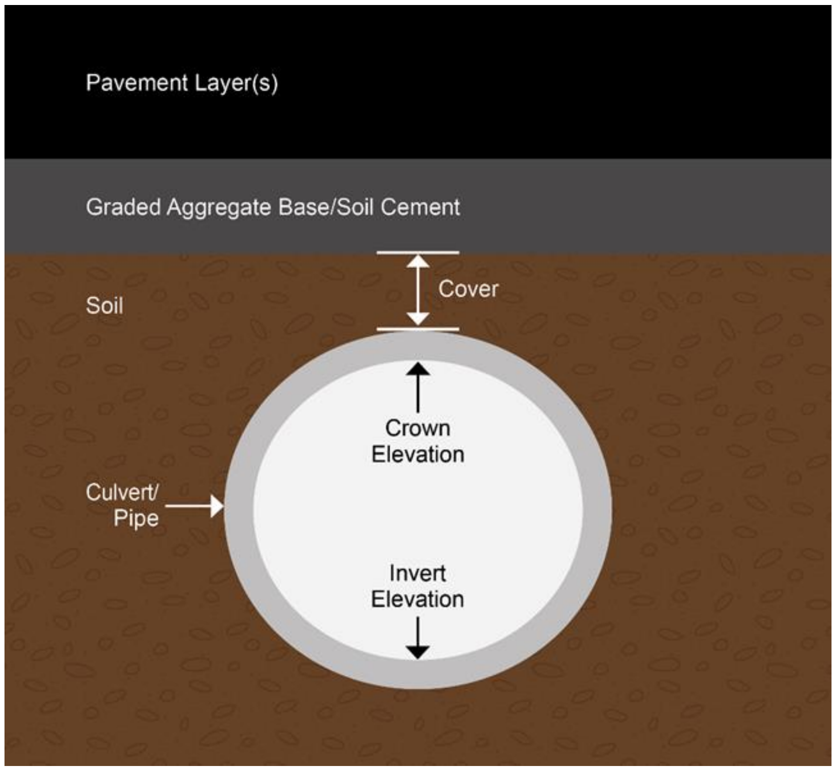

Figure 5.1 – Pipe Cover

Elliptical pipes may be used in lieu of circular pipes where there is inadequate clearance. Elliptical and arch pipes must follow the latest AASHTO and the American Concrete Pipe Association recommendations. The applicable specifications are found in ASTM C507 and AASHTO M207.

Horizontal elliptical concrete pipes are often used when there is limited vertical clearance due to existing structures or for when little cover is available. With the same depth of flow as most other conveyances with equivalent cross-sectional area, horizontal elliptical pipes provide greater flow capacity. Due to the smaller rise, there is a reduction in effective lateral support of a horizontal elliptical pipe compared to a circular pipe. In addition, its span results in greater earth loadings for the same height of cover.

Longitudinal storm drain pipe slope should approximate the road grade. Information on minimum heights, maximum connecting pipe inside diameters and other applicable dimensions for drainage structures are in the GDOT Construction Standards and Details.

The height of fill over the storm drain should also be considered. See GDOT Standard 1030D for pipe diameters, material types, and resulting class of concrete or metal pipe thickness depending on the height of fill. For a varying height of fill, the highest fill value should be used for the entire pipe run.

5.3.3 Minimum and Maximum Velocities

Storm drains should have a design storm velocity no less than 3 ft/s to aid in self-cleaning of the pipe, except in the case of equalizer pipes. For most design situations, the flow velocity at the actual design discharge will be approximately equal to the velocity at full flow. Thus, the full flow velocity may be used to check this criterion.[(5-4)] Refer to FHWA's HEC-22, Chapter 7[(5-3)] for a more in-depth discussion of minimum grades for closed pipes.

Storm drain slopes that cause uniform design storm flow velocities greater than 12 ft/s should be avoided due to the potential for abrasion in concrete pipes. A decision to propose a design with velocities over 12 ft/s should require an in-depth engineering study and a Design Deviation. In steep terrain, elevation differences can be accommodated by using drop structures.[(5-4)] This maximum velocity requirement is based on design calculations for concrete pipe. If the contractor chooses an approved alternate pipe material other than concrete, flow velocities should not exceed the manufacturer's recommendations.

5.3.4 Maximum Structure Spacing and Maintenance Access

Accessible inlets and manholes provide entry to the storm drain system for inspection and cleaning. Accessible inlets are preferred to manholes where the inlets are not precluded by other restrictions and runoff would be collected. Where both accessible inlets and manholes are not appropriate structures, a junction box (GDOT Standard 9031U) may be utilized when absolutely necessary within a storm drain system where a structure/junction is required.

For new projects, inlets and manholes shall be located outside of travel lanes. A decision to place these in travel lanes shall require Office of Design Policy and Support approval. See GDOT Standard Drainage Design Criteria for reference.

The spacing of accessible structures should be in accordance with Table 5.2. See additional information about locating inlets and manholes in section 5.6.3.

Table 5.2 — Accessible Structure Spacing

Size of Pipe, in	Maximum Distance, ft
18 – 24	300
25 – 36	400
37 – 54	500
>54	1,000

5.3.5 Existing Storm Drains

Existing pipe intended use should be noted in the plan set. The designer should determine if a pipe will be removed, replaced, or remain in place and ensure existing pipes within the project limits are in good working order prior to the incorporation in the proposed design. A pipe is deemed in good working order when all design criteria are met.

In some cases, it may be practical to abandon a pipe after it is no longer in use. If abandonment is necessary, the GDOT Design Policy Manual (DPM) Section 12.4.2 guidance should be followed.

5.3.6 Curved Alignment

Curved pipe alignment should only be considered when it is not practical to change the pipe alignment with accessible drainage structures. Long-radius bend sections are available from many suppliers and are a preferred means of changing direction in pipes 48 inches and larger. Using large access holes or manholes solely for changing direction may not be cost effective on pipes that are 48 inches and larger. Short-radius bend sections are also available and can be used if there is not enough room to accommodate a long-radius bend within a storm drain system. Deflecting the joints to obtain the necessary curvature is not desirable. For additional guidance on curved alignment and the use of junction boxes, refer to GDOT Standard 9031u.

5.3.7 Outlet Design

Several aspects of outlet design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

In the design of a storm drain system, it is important to establish the location of the pipe outlets early in the design process. Stormwater runoff conveyed by piping should be transported to a discharge point to which it would have naturally flowed. The outlet point may be a natural river or stream, an existing or proposed ditch, or an existing or proposed storm drain system. Outlet locations should be evaluated for potential increases in erosion and flooding downstream of the outlet. Outlets may need to be located sufficiently upstream of the right-of-way line or receiving stream to allow for adequate energy dissipation and sediment control. See Chapter 6 for more information on outfall protection. Preserve existing drainage patterns as much as practical. If runoff is relocated from one watershed to another, an in-depth engineering study and evaluation of hazard and liability is required along with a Design Deviation.

The outlets are a control point influencing the grade and design of the system. The designer should always strive for a gravity flow system. Pumping stations should be avoided except in extreme circumstances and should never be proposed without consulting GDOT for approval.

The orientation of the pipe outlet is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outlet channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. For all pipe outlets conveying more than 10 cfs or with a tailwater velocity greater than 5ft/s for the 25-year storm, a graded ditch must be provided between the pipe outlet and the nearest existing downstream channel, storm structure, or water resource. In addition, the outlet protection from the pipe to the ditch must comply with D-55B. Channel lining calculations must be performed to determine if lining is necessary beyond the outlet protection. If there is no existing channel, storm structure, or water resource within the ROW, provide a level spreader if appropriate. See Chapter 7 Table 7.3 for information on Pipe Culvert End Treatments at the outlet for storm drains and Section 6.3 for Channel Geometrics and channel lining guidelines.

Stormwater storage and/or energy dissipation may be required to protect an outlet, storm drain outlet, and/or downstream channel or property (including stream banks). An energy dissipator should be used at culvert outlets when outlet velocities become excessive for site conditions as stated above. (See FHWA HEC-14 *Hydraulic Design of Energy Dissipators for Culverts and Channels* [(5-4)] for design of energy dissipators. It also discusses flow transitions from pipe through flared end sections (aprons) and into channels. Chapter 5 of HEC-22 discusses stable channel design procedures and provides example problems for design and evaluation. [(5-3)]

5.3.8 Pipes and Drainage Structures Near Retaining Walls

Longitudinal drainage pipes are not allowed in the foundation of any retaining wall. In this case the foundation is defined as the envelope bounded by a 1:1 slope extending downward from the front and back of the wall footprint. Note that the back of an MSE wall's footprint is 1ft beyond the end of backfill stabilizing devices (straps), which have a length approximately equal to the wall's height, as shown in Figure 5.2.

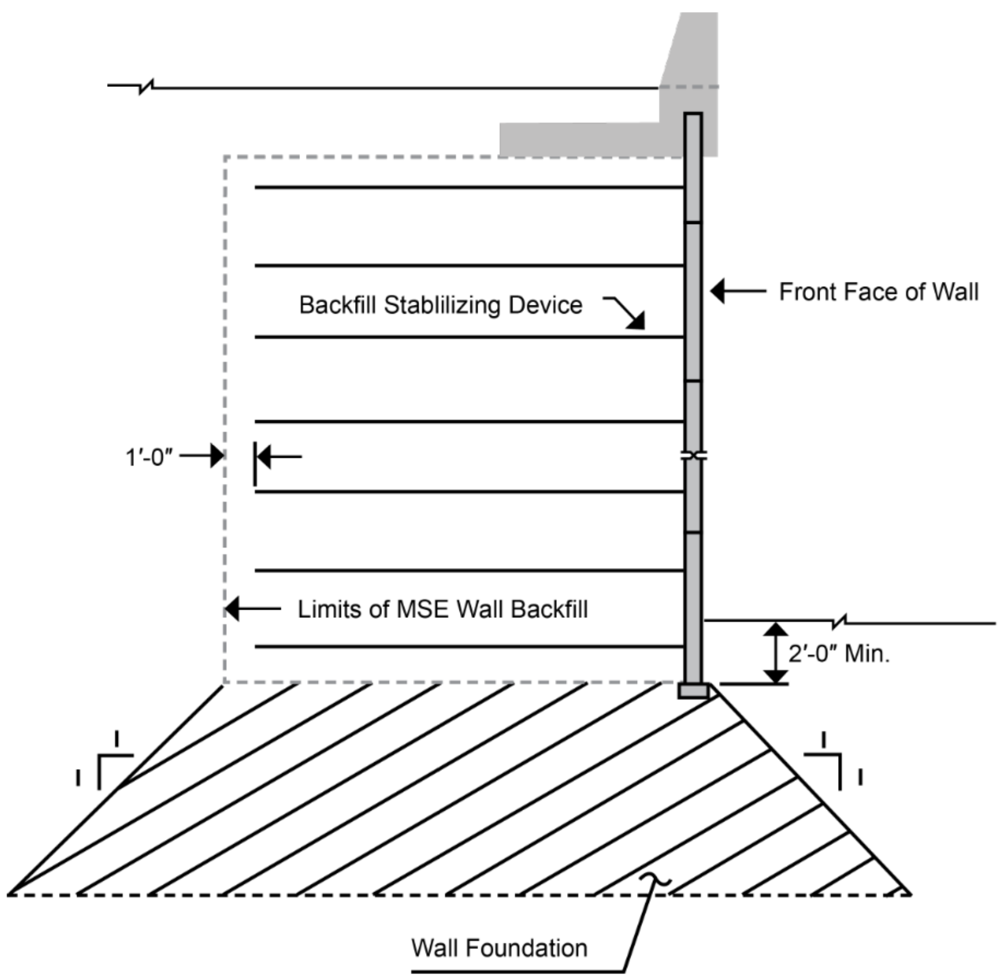

Figure 5.2 – Wall Foundation

Provide a minimum of 1’6” vertical clearance between the bottom of walls and existing or proposed pipes that cross the wall transversely. Condition of existing pipes, drainage structures or culverts should be evaluated for additional loading caused by retaining wall and backfill. Any existing brick drainage structures that are located in the foundation of a retaining wall must be removed and replaced with cast in place or precast structures.

When it is unavoidable for drainage pipes to pass through cast in place retaining wall structures, provide structural details to mitigate the impact of the hole through the wall. This detailing requirement applies to GDOT Standard walls as well as special design retaining walls. Do not design a pipe larger than 36” to pass through a cast in place wall. If multiple pipes are proposed to pass through a wall near one another, the clear distance needed between the pipes for structural stability of the wall should be determined by the structural designer preparing the details for the pipe opening, with consideration of the minimum spacing established in Std 1030D1.

When selecting drainage structures for use behind a standard cast-in-place wall or mechanically stabilized embankment (MSE) wall that has a barrier on its top, a special detail should be created by the designer to ensure that any box and any inlets will not conflict with the wall. The standard drainage details are generally not to be used with walls.

5.3.9 Pipes and Drainage Structures Near MSE Walls

Construction of the MSE wall system around drainage elements adds difficulty and cost to the work, as well as creating long term maintenance challenges. Avoid the placement of drainage systems directly in front of or behind MSE walls, including the entire reinforced backfill volume. Typically, the reinforced backfill of an MSE wall area extends behind the wall face a distance approximately equal to the wall’s height.

When the installation of drainage structures and pipes within and MSE backfill cannot be avoided, the following limitations apply:

- Consider the effect of differential settlement towards potential pipe joint separation before detailing a system near an MSE wall.
- All drainage structures located within the MSE backfill material or within 5ft of the front of the wall must be either precast or cast in place concrete according to Specification 627. Any existing brick drainage structures within an MSE wall’s foundation, as defined in Figure 5.2 that would otherwise be retained during project construction, must be removed and replaced with cast in place or precast structures.
- All pipes installed within the MSE backfill material or within 5ft of the front of the wall must be Class V with pipe joints specified as water tight.
- Longitudinal pipes running parallel to the wall face within the MSE backfill material should be limited to 24-inch inside diameter.
- Longitudinal pipes running parallel to the wall face within the MSE backfill material should be located a minimum of 7 feet away from the MSE wall, measured from the centerline of pipe to face of wall panel. This can be done using a junction box at each drop inlet.

5.4 Hydraulic Parameters and Software

Storm drain system design is a two-phase process that first selects the required surface inlets and then designs the subsurface pipe system to serve the surface inlets. Commonly used software for a storm drainage system simulates the collection of surface drainage and transports the water to an outlet. The list of software below is preferred by GDOT to complete storm drain system design. A full list of Civil Design Software programs approved by GDOT can be accessed via GDOT's R.O.A.D.S. Repository for Online Access to Documentation & Standards. For complex systems, a combination of analysis tools may be required.

- FHWA Hydraulic Toolbox:

  The FHWA Hydraulic Toolbox Program is available for free and is a stand-alone suite of calculators that performs routine hydrologic and hydraulic analysis and design computations. Specific calculators address: Rational Method hydrology, channels, channel linings, weirs, curb and gutter sections, storm drain inlets, detention basins, bridge scour, riprap countermeasures, sediment gradations, and culvert assessments.

- Watershed Modeling System (WMS):

  WMS is available for purchase from Aquaveo, and the software allows for the development of an entire storm drain model, calculates hydraulic computations for surface drainage and closed pipe flow within one comprehensive model.

- OpenFlows StormCAD and CivilStorm:

  StormCAD and CivilStorm are available for purchase from Bentley and the software uses rational method to determine system capacity, performs hydraulic computations for surface drainage and closed pipe flow within one comprehensive model, and includes tools for analysis of circular deck drains in the curb and gutter calculators.

5.4.1 Energy Grade Line (EGL)

The energy grade line is a line representing the sum of the elevation head, velocity head, and pressure head available to a fluid. In an idealized, non-viscous fluid, no losses exist therefore the energy grade line is represented as a horizontal line. In real world applications, losses can be present and are depicted in a line sloped in the direction of flow. Designers should follow guidance provided in FHWA's HEC-22[(5-3)] for energy grade line calculations.

5.4.2 Hydraulic Grade Line (HGL)

The hydraulic grade line is used to assist the designer in evaluating the suitability of a proposed storm drain system by defining the elevation to which water will rise under design conditions. The HGL is determined by subtracting the velocity head from the EGL. A check of the hydraulic grade line should be included with all storm drain design calculations for the design and check storm events to confirm the hydraulic grade line stays within the storm drainage system while accounting for energy losses. The hydraulic grade line check verifies pressure flow (or surge) within the storm drainage system for larger storm events is controlled and released at outlet points where flooding can be prevented. The designer should reference FHWA's HEC-22[(5-3)] for additional information on calculating the hydraulic grade line.

5.4.3 Tailwater Evaluation

The design water surface elevation of the outlet is used to begin the HGL determination. The outlet design water surface elevation must be estimated for the normal operating conditions of the storm sewer design and for alternate scenarios to assess risk of ponding or flooding potential. Determination of the design water surface elevation and alternate discharge scenarios requires knowledge of the watershed conditions and should be evaluated to determine the tailwater elevation. The following should be considered when establishing known tailwater elevations for the design of the storm systems:

Table 5.3 — Tailwater Evaluation

Discharge Scenario	Design Water Surface Elevations
Free Outlet Scenarios	
Outlet discharges freely into a channel or open body of water with a water level equal to or lower than the invert of the pipe	Invert of the pipe at the outlet
Backwater Scenarios	
Lakes	Normal high water
Streams/Rivers	Normal high water
Ditch	Normal depth
Stormwater ponds	Peak stage during the storm drain design event
Note: Backwater scenarios occur when the outlet discharges into a channel or body of water with a higher water level than the invert of the pipe.	

The designer should reference FHWA's HEC-22[(5-3)] for additional information on tailwater conditions.

5.5 Storm Drain Components

Storm drains are comprised of pipes, boxes, or other enclosed conduits to convey the surface water and includes inlet structures (excluding the actual inlet opening), access hole, junction boxes, laterals (or leads), main lines (or trunks), and miscellaneous structures. Storm drain runs are connected by drainage structures (e.g., inlets, access holes, junction boxes). Inlets allow water to enter the storm drain, access holes allow access to storm drains for inspection and maintenance purposes, and junction boxes are underground chambers that join two or more runs together or connect pipes of differing type, size, and/or shape.

5.6 Design Procedures

Design procedures presented in FHWA's HEC-22 [(5-3)] for storm drain design should be followed when designing storm drains. It can be accomplished by using the following steps: Firstly, collect and analyze any existing data related to the site. This will help to identify any potential issues that need to be addressed in the design. Secondly, determine drainage areas and discharges while concurrently preparing a preliminary layout for the drainage system, taking into consideration best practices for placement.

The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed.

The following is a summary of the design procedures for storm drain design:

5.6.1 Collect and analyze existing data
5.6.2 Determine drainage areas and discharges
5.6.3 Preliminary Drainage Layout and Placement Best Practices
5.6.4 Prepare Final Drainage Layout and Documentation

5.6.1 Collect and Analyze Existing Data

The following information is required for design:

- Existing pipe data obtained from the Survey Data Engineer (SDE), local authorities, district maintenance office, or other reputable source
- Existing drainage systems
- Existing topographic features (quadrangle maps or survey generated contours)
- Preliminary proposed plans including profiles, cross sections, superelevation, and other unique project features
- Existing low points
- Existing land use

The data above should be used to:

- Determine the natural flow patterns
- Locate relevant existing features
- Evaluate the integrity of the existing storm drains

5.6.2 Determine Drainage Areas and Discharges

Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system:

- Drainage areas,
- Runoff coefficients,
- Travel time.

The hydrologic method used will typically be the Rational Method and follows the guidance in Chapter 3 (Hydrology). Note that the proposed inlets would need to be placed before determining the resultant drainage areas and discharges and should be completed concurrently due to the placement of the proposed drainage system. Confirm drainage boundaries, flow paths, outlet conditions and other project special design features.

5.6.3 Preliminary Drainage Layout and Placement Best Practices

The design of a storm drainage system evolves as a project develops. The preliminary plans featuring the basic components of the intended design serve as a starting point for the storm drainage design. The preliminary plans will include information, such as roadway cross slopes and profiles, proposed inlet locations, drainage areas, and places along the roadway that exhibit the highest risk if flooding occurs. Having accumulated this information, the designer should prepare a storm drain system plan that delineates the main lines and laterals such that all required inlets are efficiently connected and directed to an outlet. The designer should evaluate the existing conditions information and lay out the storm drain system so that it can be fully sized and designed to handle the appropriate flow conditions. Drainage structures should be spaced to facilitate regular maintenance and placed within GDOT right-of-way. Ensure that adequate access is provided for inspection and cleanout of storm drain systems. See Section 5.3.4 Maintenance Access for more information. The low and high points of the storm drain design should coincide with the points of intersection along the horizontal curves. This significantly reduces drainage issues associated with flat cross slopes in superelevation transition areas. Low points should not be located near flat cross slopes in superelevation transitions, intersections, cut areas, or on bridges and bridge approach slabs.

The following are some typical locations where accessible inlets and manholes should be utilized:

- Location of two or more converging storm drains
- Intermediate points along tangent sections
- Variances in pipe size changes
- Deviations in alignment. If deviations occur and a junction box is needed, skew and other pertinent design details should be in accordance with GDOT Standard 9031u for junction boxes.
- Changes in elevation and grade
- Variations in pipe material

In general, the placement and hydraulic capacity of a storm drainage system should be designed to consider the following best practices to ensure surrounding properties, utilities, and environmental concerns/resources are addressed:

- Traffic interruption due to flooding
- Potential for damage to adjacent property
- Traffic service requirements
- Existing utilities
- Minimization of erosion and sediment accumulation at outlets
- Drainage system design should be coordinated with the proposed staging of large construction projects to maintain an outlet throughout the construction project.

Longitudinal storm drain pipe runs in travel lanes should be avoided. Lateral crossings of travel lanes should be minimized to reduce impacts to traffic during construction and maintenance. Crossing a roadway alignment in a perpendicular manner is preferred. When differing pipe sizes enter and exit a drainage structure, the desired practice is to match the crown elevation of the pipes to improve hydraulic performance. Junctions should be perpendicular if possible (to avoid skewed pipe connections).

The best practice in the design of a proposed drainage system is to keep drainage patterns as close to existing as possible. This includes maintaining identified drainage areas if possible and avoid creating a system that combines site flow and discharges it into one outlet whereas previously it discharged into several outlets or as sheet flow. Also, check existing downstream (and possibly upstream) pipes that are outside of project limits, since projects in post-development condition may adversely affect them and could potentially cause construction issues and delays.

The designer should review and avoid existing utilities when designing the layout of a storm drain system. Gravity systems such as sanitary sewers should be closely checked for conflicts with the storm drains. Pressure fed systems such as water and gas can usually be routed to avoid storm drain systems.

Refer to HEC-22[(5-3)] and Chapter 9 of the AASHTO Highway Drainage Guidelines[(5-2)] for more information regarding best practices in storm drain design process and system planning.

Necessary hydraulic and hydrologic information should be documented in the design data book in accordance with the Plan Development Process manual.

5.6.4 Prepare Final Drainage Layout and Documentation

Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed and evaluated to meet design criteria and guidelines throughout this chapter and in accordance with Section 7.5 Energy Grade Line Evaluation Procedure in HEC-22 [(5-3)].

Drainage layout and documentation should include the temporary drainage design for traffic diversions and construction phases to provide drainage in areas where construction activities may divert or trap water, compromising the safety and efficiency of the travel lanes. Give additional attention to expected spread for areas that are (1) flood sensitive, (2) high-speed facilities with posted speed limit greater than 55 mph, or (3) using a low-side barrier wall.

5.7 Risks of Inadequate Storm Drain Systems

Storm drains convey collected stormwater runoff through the roadway right of way. Storm drains should be designed to minimize flooding and erosion within this right of way and on adjacent properties. Inadequate storm drain systems can cause issues such as:

- Damage to adjacent properties
- Delays to traveling public
- Excessive ponding at low points
- Excessive gutter spread increasing potential for hydroplaning
- Deterioration of pavement structures, subgrade, and the drainage system

5.8 Chapter 5 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2014. AASHTO Drainage Manual, 1st Ed.
2. American Association of State Highway and Transportation Officials (AASHTO). 2007. Highway Drainage Guidelines, 4th Ed.
3. Brown, S.A., Schall, J.D., Morris, J.L., Doherty, C.L., Stein, S.M., Warner, J.C. 2009, Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22, FHWA-NHI-10-009. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.
4. Thompson, P.L., Kilgore, R.T. 2006, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, FHWA-NHI-06-086. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.

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Chapter 6. Channels

6.1 Introduction

The function of open channels is to convey water with a free surface from, though, or around roadway rights-of-way without damage to the highway, open channel, other components of the highway systems or adjacent property. The types of open channels encountered by the designer of transportation facilities include stream channels, roadside ditches, and median ditches and can either be natural, constructed, or a combination of both. Median channels perform the same functions as roadside channels and should be designed using the same criteria.

The primary function of the roadside channel is to collect surface runoff from the highway and areas that drain to the right-of-way and convey the accumulated runoff to acceptable outlet points.

A secondary function of a roadside channel is to drain subsurface water from the base of the roadway to prevent saturation and loss support for the pavement or to provide a positive outlet for subsurface drainage systems such as pipe underdrains.

This chapter provides guidance and design criteria for constructed channels related to transportation facilities and is to be used as a tool to aid the designer when approached with channel design according to GDOT policies and procedures. Natural channels associated with culverts and bridges are not included in this chapter; however, for more information regarding natural channels and streams, assessments of existing stream channels, or guidance on relocating or restoring a stream, the designer should refer to the following technical documents:

- Stream Restoration: A Natural Channel Design Handbook [(6-11)]
- Hydraulic Design Series No. 6 (HDS-6), Highways in the River Environment[(6-12)]
- Hydraulic Engineering Circular No. 20 (HEC-20), Stream Stability at Highway Structures[(6-13)]
- National Engineering Handbook, Part 654 Stream Restoration Design[(6-16)]
- Applied River Morphology[(6-14)]
- Hydraulic Design of Stream Restoration Projects[(6-15)]
- Stream Corridor Restoration: Principles, Processes, and Practices[(6-17)]

Design of open channels should be integrated with the highway geometry and pavement design while considering safety and pavement drainage needs. GDOT policy is to provide a channel configuration that conveys the naturally occurring flow along with the design stormwater runoff through or adjacent to the transportation facility and back to the original course. These ditches should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard (see Section 6.3.2). If necessary, channel linings should be provided to control erosion in ditches. Where design velocities will permit, vegetative linings should be used. Additional resources and guidance for channel design can be found in the AASHTO Drainage Manual Chapter 10 [(6-1)] and AASHTO Highway Drainage Guidelines Chapter 6. [(6-2)]

6.1.1 Constructed Channel Classifications

Median ditches are classified as minor constructed channels that collect sheet flow from the highway pavement or right-of-way and convey flow to an adequate discharge point. Roadside and median ditches typically parallel the highway embankment and are within the right-of-way.

Roadside ditches are classified as major constructed channels that collect drainage from minor channels, pipe systems, and offsite areas, and convey flow to an appropriate discharge point. Major channels are usually trapezoidal in cross section and convey higher flows than minor channels.

Table 6.1 - Federal Guidelines

Agency	Guideline	Description
FHWA	23 CFR 650 Subpart A | Location and Hydraulic Design of Encroachments on Flood Plains	Describes how highway facility designs and/or channel designs that impact channels must satisfy the policies of the FHWA applicable to floodplain management if federal funding is involved.
FEMA	44 CFR 63 | Implementation of Section 1306(c) of the National Flood Insurance Act of 1968	Describes the appropriate sequence and conditions for conformance to FEMA floodway regulations when encroachments and changes are present in the floodway.

6.2 Channel Design Criteria

The design and analysis of constructed channels are influenced by the basic principles of open channel flow. Channel capacity requirements correlate with the discharge associated with the design storm frequency, channel geometry, slope, roughness, and depth. Channel analysis is necessary for the design of a transportation drainage system to assess the following:

- Potential flooding caused by changes in water-surface profile
- Disturbance of the river system upstream or downstream of the highway right-of-way
- Changes in lateral flow distribution
- Changes in velocity or direction of flow
- Need for conveyance and disposal of excess runoff
- Need for channel lining to prevent erosion
- Potential impact of channel water surface elevation on pavement structural integrity.

6.2.1 Channel Capacity

Channels should be designed to provide sufficient capacity to ensure the flows remain within the channel as designed, and that the design highwater elevation remains below the bottom of the subgrade. Additionally, the travel way should not be encroached upon during the 50- year design storm frequency.

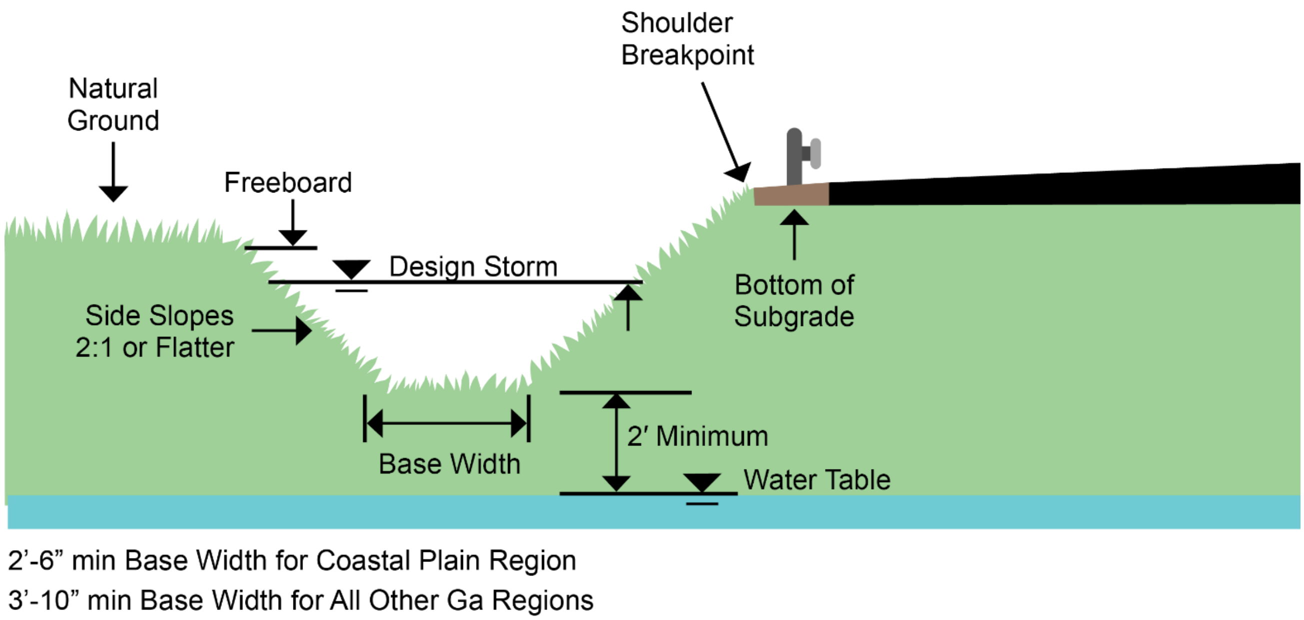

Figure 6.1 – Typical Channel Cross Section

6.2.2 Channel Design Storm Frequency

Median ditches (minor channels) and Roadside ditches (major channels) shall be designed per the design storm frequency outlined in Table 6.2 below. In addition, the highwater elevation shall not encroach the travel lane during the 50-year storm event. A decision to design highwater above the bottom of subgrade shall require a comprehensive study by an engineer and the prior approval of a Design Variance from the GDOT Chief Engineer. See GDOT Standard Drainage Design Criteria for reference.

In situations where the channels may drain slowly or high-water depths may be sustained for several hours, the designer may wish to use a more intense design storm frequency to provide additional capacity to protect the subgrade of the roadway. [(6-2)]

Table 6.2 - Channel Design Storm Frequencies Criteria			
Facility	**Channel Classification**	**Design Storm Frequency**	
		on Grade (Yr)	**in Sag (Yr)**
Interstate/Access Controlled/ State Routes	Median Ditches	10	50
	Roadside Ditches	25	50
Non-State Routes	Median and Roadside Ditches	10	25
Temporary Detours	Median Ditches	2	2
	Roadside Ditches	5	5
All Facilities*	Temporary Channel	2	2
*Temporary channels for phased construction, erosion prevention and sediment control must be designed for a minimum 2-year storm event.			

6.2.3 Freeboard Policy

A designer should provide the minimum freeboard requirements provided in Table 6.3. Freeboard is the vertical distance between the design water surface elevation and the channel top of bank. Additional consideration must be made to ensure freeboard is provided such that the design high water elevation is below the bottom of the roadway subgrade. Where a channel connects with another drainage system element, freeboard required may depend on the other drainage system element.

Table 6.3 - Freeboard Policy	
Condition	**Required Freeboard**
Fill Slope	1 ft
Cut Slope	1 ft or 0.5 ft*
***1 ft of freeboard required above the design flood frequency or 0.5 ft freeboard above the 100-yr event, whichever results in the higher channel top of bank.**	

6.3 Channel Geometrics and Guidelines

GDOT hydraulic design criteria and policies are provided for the proper design of constructed channels to convey floodwaters which avoid hazardous flooding and failure of the highway. The following policies apply to all channel designs:

- For channel designs and/or designs of highway facilities that impact channels see policies of the FHWA applicable to floodplain management.
- Environmental Coordination for USACE permit conditions/regulations for wetland restrictions and stream impacts are to be followed.
- Channels should not be placed within the limits of delineated wetlands or designated floodplains.
- All channels should be evaluated for stability. Flows, velocities, and lining type affect the stability of channels.

Parameters required for the design of roadside and median channels include discharge frequency, channel geometry, vegetation lining type/roughness, freeboard, and shear stress. The use of

Manning's Equation and the Continuity Equation are appropriate for the determination of ditch and channel capacity. The following sections list standard GDOT design criteria that should be used in the design of channels.

6.3.1 Channel Cross Section

The shape of a roadside channel is governed largely by the geometric and safety standards applicable to the project. Roadside channels are typically trapezoidal or V-shaped in cross section and lined with grass or other protective linings, such as riprap.

The channels are to be designed to be traversable if within clear zone and are not protected by barrier. If not traversable, the channels are to be located outside the clear zone or protected by barrier. Roadside channels outside the clear zone can be designed with a trapezoidal cross section with side slopes as steep as 2H:1V. (See GDOT construction Detail D-7 for berm, side, and surface ditches). V- shaped channels should implement 6:1 front and back slopes if within the right-of-way. For more information on slopes, clear zones for roadside ditches, and the definition of traversable slopes, refer to Design Policy Manual and AASHTO's Roadside Design Guide[(6-18)].

Channel side slopes should not exceed the angle of repose of the soil or lining material, or both, and should be 2:1 or flatter in the case of rock riprap lining.

6.3.2 Channel Alignment

Roadside channels should parallel the roadway alignment and be constructed within the right-of-way.

Gradual, smooth changes in alignment as permitted by the right-of-way and terrain are preferred to small radius curves. Alignment changes should occur in channel sections with flatter grades and subcritical flow. Where flow is conveyed around a bend in an open channel, additional freeboard should be provided to account for the superelevation of the water surface at the outside of the bend due to centrifugal forces. For more information on flow in bends see section 5.1.6 of the FHWA's HEC-22.

6.3.3 Channel Grade

The following guidelines should be considered when determining the channel grade:

- Surface ditch grades should be controlled by the topography. Surface ditches should be constructed no less than 6 inches deep and no less than the freeboard depth described in Table 6.3.
- Low points should be draining to roadway channels by way of slope drains down the back slope.
- Channel grades should be a minimum of 0.5%.
- Subcritical flow should be maintained where possible to minimize soil erosion.
- Alignment changes should be kept to a minimum for paved channels with steep slopes and supercritical flow.

6.3.4 Channel Lining

Stable channel design must be achieved by selecting the appropriate material or channel lining to effectively resist the erosive forces of the flow. The most common channel lining protection measures are grass, turf reinforcement matting (TRM), rock riprap, and concrete. All TRM types require permanent grass concurrently with installation. When placing rock and concrete channel lining, consider the clear zone requirements described in the Design Policy Manual.

For proper design of channel protection for roadside channels, design methodology for the evaluation of channel linings is based on shear stress. See Table 6.1 for ditch lining design requirements.

Flexible channel linings, where required, should be designed according to the method of allowable tractive force.

Ranges of permissible shear stresses are given in Table 6.4.

Table 6.4 – Permissible Shear Stress for Channel Lining[1]

Lining Type	Permissible Shear Stress (lb/ft^2) [2]
Type 1: TRM 1	0 – 2
Type 1A: Grass with temporary RECP	0 – 3
Type 2: TRM 2	0 – 4
Type 3: TRM 3	0 – 6
Type 4: TRM 4	0 – 8
Type 5: TRM 5	0 – 10
Type 6: TRM 6	0 – 12
Type 7: Riprap D50 < 0.573 ft (Type 3) [3]	Calculate permissible shear stress for riprap linings using HEC 15
Type 7A: Riprap 0.573< D50 < 1.078 ft (Type 1)[4]	Calculate permissible shear stress for riprap linings using HEC 15
Type 8: Concrete D50 >1.078 ft	

[1] Allowable hydraulic shear stress in the unvegetated condition = 2.0 lb/ft^2
[2] As calculated in accordance with the methods detailed in the FHWA HEC 15
[3] GDOT Type 3 riprap D50 = 0.66 ft.
[4] GDOT Type 1 riprap D50 = 1.20 ft

6.4 Computational References

Open channel flow and fluid mechanics principles are the basis of hydraulic analysis for both natural and constructed channels. The hydraulic analysis of a channel uses a discharge that is conveyed through a passage with known geometry, roughness, and slope to establish a depth and velocity. The designer should properly size a channel based on the preceding variables that achieves the required capacity, freeboard, alignment, erosion resistance.

Two methods are commonly used in hydraulic analysis of open channels. The single-section method (slope-area method) is a simple application of Manning’s equation and is typically adequate for minor drainage channels such as roadside ditches since the analysis assumes steady and uniform flow for short periods and distances, see Section 6.4.1. The step-backwater method or other procedures that consider energy losses in addition to friction are preferred in situations where uniform flow does not occur, see Section 6.4.2.

6.4.1 Roadside and Median Channel Design and Analysis

The design capacity of roadside and median channels is typically determined using the single-section method via the continuity equation and Manning's equation. The continuity equation considers the conservation of mass, and Manning's equation calculates the cross-sectional average flow velocity in the principal direction. Equating the continuity equation and Manning's n allows the designer to determine the channel capacity.

The Manning's Equation is used to compute the mean velocity in an open channel with steady-uniform flow. Refer to Chapter 5 of FHWA's HEC-22 manual for open channel flow computations including the Manning's N equation.

For information on typical cross sections and equations for a particular open geometric configuration, see the "Elements of Channel Sections" figure in the USDA National Engineering Handbook, Section 5. [(6-2)] Manning's n value selection for various types of channels-based stream flow is provided in the FHWA publication, FHWA-TS-84-204, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains." [(6-3)]

In addition to direct application of the Manning's equation to solve for variables of simple trapezoidal channels, another alternative is to use the charts from FHWA document *HDS-3 Design Charts for Open-Channel Flow*. [(6-4)]

The equations should be calibrated with historical high-water marks and/or gaged streamflow data to facilitate accurate representation of local channel conditions. The USGS National Water Information System website offers a source for streamflow characteristics, which can be found here: http://waterdata.usgs.gov/usa/nwis/rt. The following parameters, in order of preference, should be used for calibrations: Manning's n, slope, discharge, and cross section.

6.4.2 Major Channel Design and Analysis

For drainage channels that are long, large, or costly, water surface profiles should be determined by Standard Step Backwater Methods. Complex open channel flow conditions can require multiple iterations to achieve an appropriate design, and computational software to establish water surface profiles can facilitate the design process. In general, there are One-Dimensional and Two-Dimensional flow analysis programs. Both serve as tools in the hydraulic design for proposed constructed channels and the analysis of proposed and existing channels.

6.4.2.1 One-Dimensional Flow Analysis

One-Dimensional flow analysis programs should be used in the design of roadside channels, culvert tailwater channels, and constructed channels with uniform cross sections. One-Dimensional flow analysis tools calculate the steady, uniform flow using Manning's equation combined with the continuity equation.

One-Dimensional analysis can be implemented where channel geometry and variables produce a non-uniform cross section by the use of a series of cross sections placed perpendicular to the downstream flow direction. The step-back analysis procedure deployed through the One-Dimensional computational process via boundary conditions placed at extents of the simulation.

Hydraulic areas of interest can also be easily included in a simulation to represent the existing and proposed conditions.

The FHWA publication, *Introduction to Highway Hydraulics (HDS-4)* [(6-5)] provides the background on flow profile types and the standard step backwater method. HEC-RAS or another acceptable computer program should be used to calculate water surface profiles when the standard step method is required. The standard step backwater method should be used where the following occurs:

- The channel cross section, slope, roughness, or flow is highly irregular
- A structure (culvert, bridge, weir, gate, etc.) affects the water surface profile
- Stream or channel confluences affect the water surface profile
- The slope area method is either not applicable or not sufficiently accurate
- FEMA level stream analysis and floodplain modeling are required

A detailed description of the standard step backwater method for channels with irregular cross sections, such as streams, may be found in the *HEC-RAS Hydraulic Reference Manual*. [(6-6)]

Table 6.5 - One-Dimensional Flow Analysis Software

Software Name	Software Description	Source
HEC-RAS	Utilizes four One-Dimensional River Analysis components for steady water surface profile computation, unsteady flow simulation, moveable boundary sediment transport calculations and water quality analysis. Includes Two-Dimensional analysis functions and GIS capabilities with RAS-Mapper. Allows for enhanced scour calculation ability.	Hydrologic Engineering Center (HEC) Website
FHWA Hydraulic Toolbox	Acts as a group of individual calculators that compute routine hydrologic and hydraulic computations. Equipped for designer to work in project files, run multiple design scenarios, and establish plots for reporting purposes.	FHWA Website
WMS Channel Calculator	A tool in the WMS Hydrologic Modeling Module. Deployed to determine the shear stress for a user input discharge and select an appropriate channel lining.	Aquaveo Website

6.4.2.2 Two-Dimensional Flow Analysis

Two-Dimensional flow analysis is intended for use when a complex flow condition, alignment, or slope are present. Complex conditions that are not adequately represented by a series of cross sections, where flow is confined and sometimes limited by the cross-section extent, are suitable for a Two-Dimensional flow analysis.

A Two-Dimensional model should be considered for major projects with complex flow patterns that one-dimensional models cannot adequately analyze. Situations where Two-Dimensional models should be considered are:

- A primary direction of flow cannot be identified including tidally affected river crossings and crossings of tidal inlets, bays, and estuaries
- If detailed output is needed lateral to primary flow path

- Wide floodplains with multiple channels (conveyance areas), or multiple overflow structures
- Crossings with skewed alignments, or complex geometry such as ineffective flow areas, flow around islands
- Floodplains with significant variations in roughness
- Sites where more accurate flow patterns and velocities are needed to sufficiently design cost-effective countermeasures such as riprap along embankments, walls, and/or abutments to address potential scour in close proximity to bridge piers.
- Where a bridge column and any open drainage channel (ditch, stream, etc.) are located within 50 feet of a wall, perform a 2D flow analysis for the channel to determine the 100-year flood scour potential. Provide countermeasures to address the potential scour. Countermeasures should include armoring solutions.
- High-risk or sensitive locations where losses and liability costs are high

Table 6.6 - Two-Dimensional Flow Analysis Software

Software Name	Software Description	Source
SMS	Utilizes a pre- and post-processor for surface water modeling and design. Deploys a two-dimensional finite element, finite difference, and finite volume analysis. FHWA package SRH-2D includes options for modeling bridge, culvert, and highway drainage scenarios.	Aquaveo Website
HEC-RAS	Allows for one-dimensional (1D), two-dimensional (2D), or combined 1D/2D flow analysis of scenarios. Capable of depicting steady and unsteady flow conditions.	HEC Website

6.5 Channel Protection Materials

Concentrated channel flows cause hydraulic conditions that may lead to erosion and eventual degradation of the channel. Channel protection is required to address these issues and maintain a stable conveyance.

6.5.1 Channel Lining

Channel lining can be categorized as rigid or flexible. The main difference between rigid and flexible linings is the ability to respond to changes in channel geometry. The designer should follow the guidelines established in Hydraulic Engineering Circular No. 15 (HEC-15), Third Edition: Design of Roadside Channels with Flexible Linings.[(6-7)] A computer program entitled *Channel Lining Design* was developed to aid designers in the proper selection of all channel protection materials, which includes the higher shear strength protection materials Types 1 and 3 riprap and concrete. The program calculates the maximum hydraulic shear stress occurring on the channel bottom.

Transitions between channels of dissimilar materials warrants protection from scour and erosion.

Rigid channel linings are useful in flow zones where high shear stress or non-uniform flow conditions exist, such as at transitions in channel shape or at an energy dissipation device. Rigid linings are suitable for implementation where channel width is limited by right-of-way, but enough space is present for a high-capacity channel. Rigid linings are intended for use in permanent, long-lasting installations. Movement in a rigid lining can lead to immediate and catastrophic failure, as well as potential minor failures. Concrete ditch lining is the preferred rigid channel lining, although other rigid channel lining materials may be used if approved by the GDOT Office of Deign Policy on a case-by-case basis. Impermeable membranes should be considered in areas where loss of water from seepage is undesirable.

Rigid lining considerations:

- Foundation stability – soils, subgrade design
- Limiting surface water intrusion – sealing joints
- High water tables or swelling soils that exert an uplift pressure – geotech/soils analysis coordination, under drains, weep holes
- Minimize freeze thaw effects – expansion joints, pavement design

Flexible channel linings allow for movement to respond to channel changes over time improving the sustainability and long-term integrity of the channel. Typically, flexible linings are more cost effective than rigid linings. Flexible linings are limited though in hydraulic performance resisting hydraulic forces. The limitations vary depending on the lining type selected and the site location. Flexible linings can be categorized as long-term, transitional, and temporary. The following list are GDOT accepted flexible linings:

- Long-Term:
  - Vegetative (Permanent Grassing) GDOT Standard Specification Section 700
  - Riprap GDOT Standard Specification Section 805
  - Turf reinforcement matting GDOT Standard Specification Section 711
- Temporary:
  - Vegetative (Temporary Grassing) GDOT Standard Specification Section 163
  - Open-weave textile (degradable) GDOT Standard Specification Section 713
  - Erosion control blankets (degradable) GDOT Standard Specification Section 713

Vegetated lining is defined as a stable grass channel that is densely vegetated and capable of withstanding erosion from stormwater runoff and can carry be used to stabilize channels with low velocities and shear stresses (less than 5 ft/s). Various types of vegetation provide different levels of resistance to shear. See GDOT Standard Specification Section 700. Additional information on vegetal covers, lining types, and their associated retardance classifications and methods for calculating permissible shear stress can be found in HEC 15, Chapter 4.

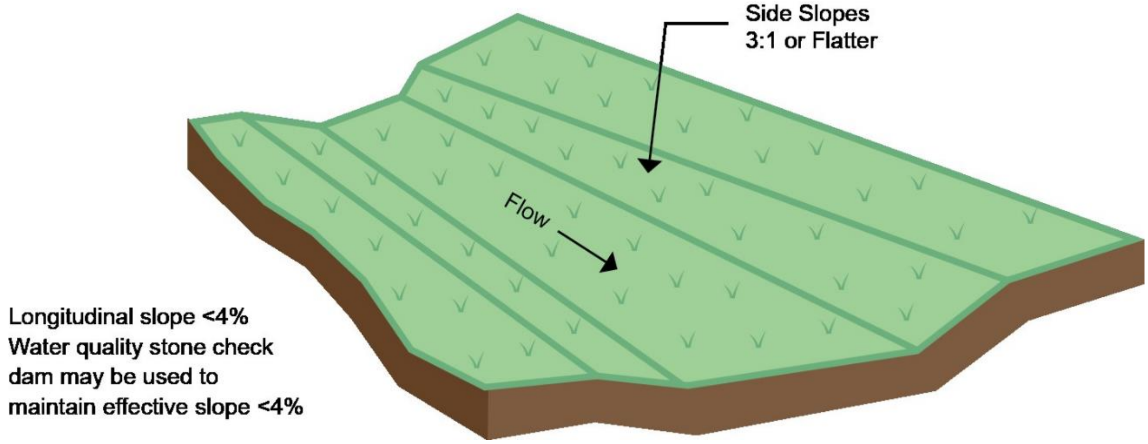

Figure 6.2 – Typical Grass Channel Configuration

Turf Reinforcement Matting (TRM) is a permanent geosynthetic erosion control matting that is used in channels to stabilize the soil while permanent vegetation is rooting, and to provide additional long-term protection. TRM should be used if a vegetated lining is used in channels with velocities greater than or equal to 5 ft/s, but less than 10 ft/s. TRM is categorized based on the permissible shear stress expected in the channel. See GDOT Standard Specification Section 711.

In **Rock Riprap Lining**, dumped riprap or machine placed rock is used in conjunction with a permeable filter blanket to armor and stabilize channels. Riprap provides a higher permissible shear stress than vegetated channels with TRM and should be used in channels with velocities greater than or equal to 5 ft/s but less than 10 ft/s. Historically, design for riprap and concrete channels has been performed not on the basis of a permissible shear stress but on a sizing criterion for riprap determined according to D50 and HEC 15 (6-10) See GDOT Standard Specification Section 805. Riprap should not be used for channel stabilization on slopes greater than 66%.

6.5.2 Outlet Protection

Functional outlets are crucial to mitigating and preventing adverse effects that could be caused from concentrated discharges from one drainage conveyance to another. In order to prevent erosion between drainage conveyance elements, all channels conveying more than 10 cfs or with a tailwater velocity greater than 5ft/s for the 25-year storm should extend to an existing channel, storm structure, or water resource. If there is no existing channel, storm structure, or water resource within the ROW, provide a level spreader if appropriate or another outlet protection device. Common outlet protection devices include:

- Riprap aprons (rock splash pads)
- Flow dispersal trenches
- Gabion or other energy dissipators

The designer should follow the guidelines established in Hydraulic Engineering Circular No. 14 (HEC-14), Third Edition: Hydraulic Design of Energy Dissipators for Culverts and Channels.[(6-7)]

6.5.3 Bank Protection

Stream-bank stabilization should be provided, as a result of any stream disturbance and should include both upstream and downstream banks as well as the local site. The method of stabilization should consider both engineering and environmental benefits. USACE nationwide permit 13 for Bank Stabilization includes additional guidance on bank stabilization best practices.

6.6 Design Procedures

Design procedures for roadside and median channels should follow the design steps provided in the AASHTO Drainage Manual [(6-1)] and HEC-22 [(6-19)]. Channel design generally follows these steps:

6.6.1 Collect and Analyze Existing Data

6.6.2 Determine Drainage Areas and Discharges

6.6.3 Preliminary Drainage Layout and Best Practices

6.6.4 Channel Lining Design

6.6.5 Outlet Protection Design

6.6.6 Bank Protection

6.6.1 Collect and Analyze Existing Data

A roadside plan should be established by collecting available site data, obtaining, or preparing existing and proposed plan and profile layout including the highway, culvert, and bridges. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets. Lay out the proposed roadside channels to minimize diversion flow lengths.

Site data information normally required for design:

- Field measured topography or digital terrain model (DTM)
- USGS/ArcGIS topography for areas outside of project survey boundaries
- Stream profile and cross sections
- Bankfull measurements – environmental survey
- Soil survey and soil erosion index - Geotech
- Determination of the design runoff volume or discharge – Hydrology, see chapter 3
- Drainage basin size and characteristics - Hydrology
- C or CN Factors
- Rainfall intensity - NOAA Atlas 14
- Recorded data from gage stations (if available) - USGS
- Regulatory flood data (FEMA or local community)

The data above should be used to:

- Determine the natural flow patterns
- Locate relevant existing features
- Evaluate the integrity of the existing channels

6.6.2 Determine Drainage Areas and Discharges

An important step in designing channels involves determining accurate drainage areas and corresponding discharges. This foundational process ensures that the design adequately addresses water management along roadways. Refer to Chapter 3 for analyzing hydrology for channel design.

6.6.3 Preliminary Drainage Layout and Best Practices

The designer should evaluate the information obtained in the collection phase and preliminarily lay out the channels so that they may be designed for the required peak discharges.

Obtain or establish cross section data

- Provide roadside channel depth adequate to drain subbase.
- Choose channel side slopes based on geometric design including clear zone, economics (right-of-way for example), soil stability, and access for maintenance. Reference the GDOT Design Policy Manual for more policy-based guidelines.
- Establish bottom width and shape of channel.
- Identify features which may restrict cross section design, e.g., right-of-way constraints, trees or environmentally sensitive areas, utilities, and existing drainage facilities.

Determine channel grades

- Plot initial grades on plan and profile layout, including inlet and outlet considerations. (Slopes in roadside ditch in cuts are usually controlled by highway grades.)
- Provide sufficient grade to minimize ponding and sediment accumulation. The minimum allowable grade is 0.3 percent.
- Consider influence of type of lining on grade.

Check flow capacities and adjust, as necessary

- Compute the design discharge at the downstream end of channel segment.
- Set preliminary values of channel size, roughness coefficient, and slope.
- Determine maximum allowable depth of channel including freeboard.
- Check flow capacity using Manning's equation and the slope area method.
- If capacity is inadequate make adjustments as appropriate, e.g., increase bottom width, make channel side slopes flatter, make channel slope steeper, provide smoother channel lining, and install drop inlets and a parallel storm drain pipe beneath the channel to supplement the channel capacity.
- Provide smooth transitions at changes in channel cross section.

Determine channel lining/protection needed

- Determine design discharge.
- Select a lining and determine the permissible shear stress (lb/ft2).
- Determine maximum depth.
- Determine the area of flow and hydraulic radius corresponding to the depth.
- Calculate the mean boundary shear.
- Estimate the Manning’s n value appropriate for the lining type and solve Manning’s equation to determine maximum discharge for the channel.
- Determine effective shear stress.

Analyze outlet points and downstream effects

- Identify any adverse impacts such as increased flooding or erosion to downstream properties
- Mitigate any adverse impacts

- In order to obtain the optimum roadside channel system design, it may be necessary to perform several trials of the above procedure before a final design is achieved.

Necessary hydraulic and hydrologic information should be documented in the design data book in accordance with Plan Development Process manual.

6.6.4 Channel Lining Design

For proper design of channel protection for roadside channels, the current version of the GDOT Ditch Lining Tool spreadsheet (available at the Department's Design Software webpage), and the Georgia Soil and Water Conservation Commission Manual for Erosion and Sediment Control in Georgia[(6-19)]. Please reference HEC-15 Design of Roadside Channels with Flexible Linings[(6-8)] and FHWA Hydraulic Toolbox to check design.

Stability is the ultimate goal for all channels located within a highway right-of-way or that impact highway facilities. Channel linings should be selected based on calculated shear. To calculate shear, the designer will need to input the channel geometry (width, back and fore slopes, longitudinal slope) flow characteristics (design year discharge and depth of flow), and soil characteristics including soil classification (USCS), Plasticity Index (PI), and 75th percentile soil diameter (D75). While the soil characteristics can normally be found in the project specific Soil Survey Report, this information may not be readily available. The NRCS website http://websoilsurvey.nrcs.usda.gov/ and other soil information sources are additional resources for estimating soil properties, both horizontally and vertically up to depths of 6 or 7 feet.

Velocities in natural channels when flowing at the bankfull discharge or the 25-year frequency discharge in constructed channels are to be used in determining the appropriate channel lining for stabilization of the channel.

6.6.4.1 Grass Channel Lining Design

A stable grass channel is densely vegetated and capable of withstanding erosion from the stormwater runoff. Grass type, density of cover, and stem height are factors that must be estimated to determine the roughness and the shear stress of vegetated linings. Typical grass types are sod, bunch, or mixed. Density of grass cover can be estimated into three categories: Excellent (500 stems/sq-ft), Good (300 stems/sq-ft), or Poor (100 stems/sq-ft). Good, conservative beginning estimates for the vegetative input are bunch grass, good cover density (75-95%), and 0.5-foot stem height.

6.6.4.2 Rigid Channel Design

Movement can impede the efficacy of a rigid channel lining; therefore, the design should consider likelihood that the materials may experience frost heave, slumping, or swelling of the underlying soils which can displace rigid linings, ultimately causing failure.

6.6.4.3 Flexible Channel Design

Flexible channel lining is appropriate for conditions with uniform flow and moderate shear stress (0 to 12 lb/ft^2) where grass linings alone cannot be used typically for design flows with velocities greater than or equal to 5ft/s but less than 10 ft/s. There are several types of TRM available per GDOT Standard Specification Section 711. The designer should use the calculated shear stress range to choose the appropriate TRM type.

Designers should also consider the climate and geographic location when selecting an appropriate long-term, transitional, or temporary application. Methods for analysis include HEC-15 "Design of Roadside Channels with Flexible Linings," and HEC-11. "Design of Riprap Revetment,"

6.6.5 Outlet Protection Design

For all drainage outlets, a rock splash pad should be provided using GDOT Design Detail D-55a and D-55b. A more aggressive energy dissipator may be required for severe outlet conditions. Culverts that operate with velocities greater than 10 feet-per-second can cause degradation and may require a hydraulic analysis to establish an effective design. The slope, type of material, tailwater, and other factors can also affect the velocity. Outlet protection designs should be established based on hydraulic analysis results and field observation.

If the velocity discharged from a culvert is greater than the velocity in a downstream natural channel for the design flow, channel stabilization and energy dissipation should be considered.

Natural occurring scour holes at culvert outlets provide efficient energy dissipators. As such, outlet protection for the design storm frequency should be provided where the outlet scour hole depth computations indicate the scour hole:

- Will undermine the culvert outlet
- May cause costly property damage
- Causes a nuisance effect (most common in urban areas)
- Impacts stream continuity and aquatic organism passage
- Will restrict land-use requirements

An energy dissipator should be used at culvert outlets when outlet velocities become excessive for site conditions and downstream scour becomes problematic. (See FHWA HEC 14 Hydraulic Design of Energy Dissipators for Culverts and Channels for scour computations and design of energy dissipators such as riprap basins and aprons in Chapter 10.) [(6-8)]

6.6.6 Bank Protection Design

Bank protection practices are implemented to stabilize and protect the banks of streams or constructed channels and lake shorelines, reservoirs, and estuaries. The primary intent of bank protection is:

- Prevent and/or mitigate the loss of land creating an impediment to an intended land use due to bank failure
- Encourage the maintenance and management of flow capacity of streams and channels
- Mitigate offsite or downstream effects of sediment loading caused by bank failure

According to the USDA, NRCS – Conservation Practice Standard for Streambank and Shoreline Protection | Code 580[(6-9)], causes of stream instability include livestock access, watershed alterations, in-channel modifications, head cutting, water level fluctuations, and boat-generated waves. When bank protection is required, the design should not impede the existing bank line, planned improvements installed as a result of adjacent projects, water chemistry, channel or lake hydraulics, and slope characteristics.

Where bank erosion or degradation has occurred, the following conditions should be considered in the design process:

- If feasible and/or economical, locate any proposed facilities or move existing structures and alignments away from where potential additional erosion may occur.
- Deploy rock slope protection or redirect flow via river training elements such as riprap revetment, earth or rock embankments, rock toe-dikes, crib dikes, bulkheads, flexible revetments, rigid revetments, spurs, guide banks, jack or tetrahedron fields, bendway weirs, vegetative or bioengineered streambank protection and/or channel alterations.
- Minimize the force of the flowing water on the structure through appropriate and applicable vegetative practices. Plantings can reduce stream velocity and root systems add stability to the bank.

6.7 Channel Safety Design Considerations

See GDOT's Design Policy Manual and *AASHTO's Roadside Design Guide* for additional information for applications within the clear zone.

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Chapter 7. Culverts

7.1 Introduction

This chapter provides design requirements for the hydraulic design of highway culverts that are based on FHWA Hydraulic Design Series No. 5 (HDS 5), Hydraulic Design of Highway Culverts.

A culvert or cross drain is a drainage conveyance that allows surface water to flow through embankments. Culverts are often a variety of shapes, sizes, and materials. Culverts are defined according to their shape, size, material, and location. For example, a culvert can be a double 10-ft X 10-ft concrete box culvert, an 18-inch corrugated metal pipe (CMP) side-drain culvert, or a 36-inch reinforced concrete pipe (RCP) cross drain culvert.

Culverts are distinguished from bridges in that they are usually covered with embankment material and are composed of structural material around the entire perimeter of the opening, although some are bottomless. Box, pipe, or arched culverts that have a span width of 20 ft or more, as measured parallel to the roadway centerline between the outermost inside walls, are bridge culverts. See Figure 7.1 below.

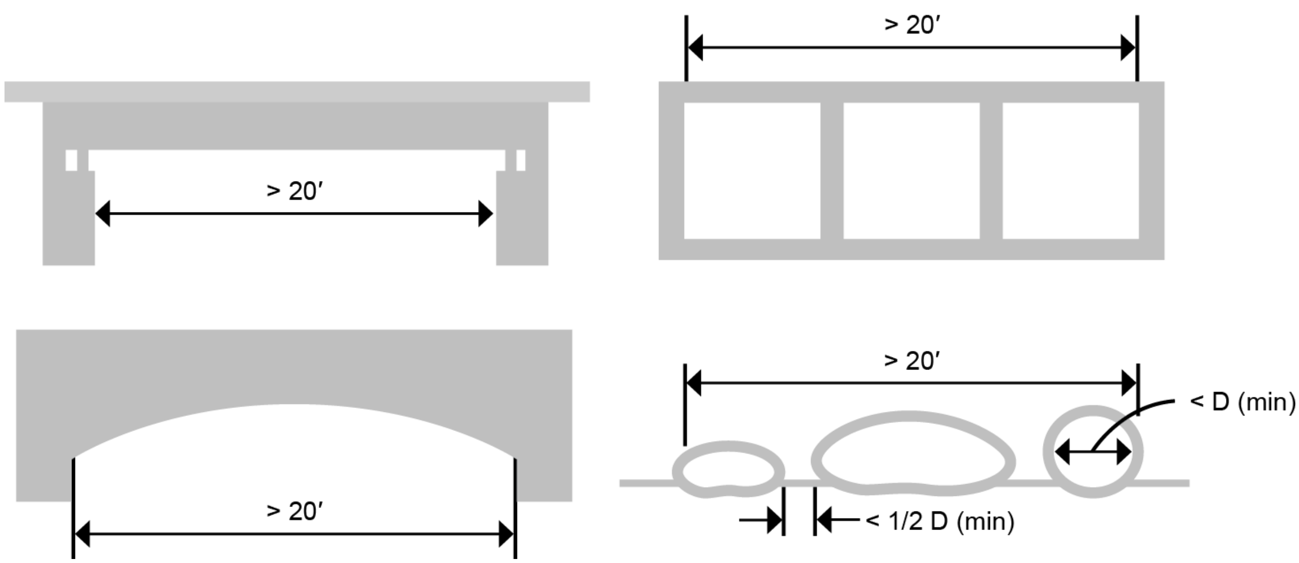

Figure 7.1 Examples of span width for bridge culverts

One exception to the 20-ft span width limit is a multi-barrel pipe culvert. Multi-barrel pipe culverts may exceed the 20-ft clear span width and still may be considered only a culvert as opposed to a bridge culvert if the spacing between the culverts is greater than half a barrel diameter.

Sizing of a culvert and/or bridge culvert should be based on hydrologic analysis. Culverts should be considered for contributing drainage areas of less than 20 square miles. Culverts can be used for drainage areas greater than 20 square miles, but bridges should also be considered.

7.2 Design Storm Frequency

Culverts are sized to accommodate the design storms listed in Table 7.1 without exceeding the design storm freeboard.

Although Table 7.1 lists minimum design frequencies less intense than the 25-year storm event for roads with an ADT less than 400, the 25-year storm event is still recommended as a minimum design guideline.

Table 7.1 Culvert Design Criteria

Facility	Headwater to Roadway Base		Headwater to Shoulder Breakpoint	
	Required Freeboard (ft)	Design Storm Frequency	Required Freeboard (ft)	Check Storm Frequency
Interstates & State Routes	1.0	50-yr	1.0 ft below breakpoint	100-yr
Hurricane Evacuation Routes	1.0	50-yr	1.0 ft below breakpoint	100-yr
Non-state Routes: ADT=0-99 ADT = 100-399 ADT = 400-1499 ADT > 1499	 1.0 1.0 1.0 1.0	 5-yr 10-yr 25-yr 50-yr	 1.0 ft below breakpoint 1.0 ft below breakpoint 1.0 ft below breakpoint 1.0 ft below breakpoint	 10-yr 25-yr 50-yr 100-yr
Driveways	1.0	25-yr	breakpoint not overtopped	50-yr
Temporary Detours	1.0	10-yr	breakpoint not overtopped	25-yr
Temporary Culverts: Local roads, ADT < 400 All other roads	 1.0 1.0	 2-yr 10-yr	 crown not overtopped crown not overtopped	 5-yr 25-yr

Note 1: Both the primary criteria and secondary criteria must be met with the more conservative of the two criteria controlling the maximum headwater elevation
Note 2: Culvert headwater is checked for road overtopping using the 100-year design storm event.
Note 3: When analyzing driveway culverts, criteria for adjacent state or local routes must also be met.
Note 4: Roadway Base refers to pavement aggregate base or the bottom of the pavement structure. See Figure 10.4 of Design Policy Manual for reference.

7.3 Design Criteria

Culverts shall be designed based on the criteria presented in Table 7.1. See GDOT Standard Drainage Design Criteria for reference.

For federally funded projects, section 650.117 of 23 Code of Federal Regulations (CFR) 650A applies and requires that project plans for floodplain encroachment locations contain the following:

- The magnitude, approximate probability of exceedance, and at appropriate locations, the water surface elevations associated with the overtopping storm event or the storm of Sec. 650.115(a)(1) (7-2) (the largest storm event that may be reasonably estimated such as the 500-year storm event).
- The magnitude and water surface elevation of the base storm event, if larger than the overtopping storm. (7-2) (The base storm event is the 100-year storm event).

Note: The overtopping storm event does not need to correspond with the design storm frequency for which the culvert is designed. The culvert should be designed for the event given in section 7.2.2 Design Storm Event.

Culverts less than or equal to 30 inches in diameter are subject to one of the following:

- Extend to the appropriate "clear zone" distance per AASHTO Roadside Design Guide(7-1). An end section appropriate to the culvert material is used; e.g., a flared end section (Ga. Std. 1120).
- Or safety treated with a grate such as GDOT Construction Detail D-5 at any end within the "clear zone."

Culverts greater than 30 inches in diameter are subject to one of the following:

- Extend to the appropriate "clear zone" distance per AASHTO Roadside Design Guide(7-1). An end section appropriate to the culvert material is used.
- Or safety treated with a grate. (GDOT Construction Detail D-6.)
- Protect with a traffic barrier if the culvert cannot be extended, has a channel that cannot be safely traversed by a vehicle, or significant debris may clog a gate.

New culverts will have a beveled edge as shown in the standard drawings.

All calculations should be performed assuming concrete will be used. If the contractor elects to use an alternate material, the culvert design must be checked with this new material and redesigned as necessary. The proper Mannings “n” for the culvert material (concrete, metal plastic, etc.) must be used.

Pipe culvert material alternates are determined by The Office of Materials and Research and shown in the project soil survey summary. These alternates are included in the plans. See the GDOT Geotechnical Manual section 4.5.26 for the Pipe Culvert Material Alternates table. When alternate materials are used that are different from what is assumed in the design calculations, the contractor must perform a hydraulic analysis using the proper Manning’s n for the culvert material (concrete, metal, plastic, etc.). Different materials may require different size structures. Refer to GA Specification 550 for more information.

Culvert should be extended using the same size (or greater in the downstream direction), shape, and material as the existing culvert. Exceptions to this are for existing metal culverts, which should be replaced with concrete pipe instead of extending and 15” culverts, which should be extended or replaced with minimum 18” pipe.

Cutting holes in existing box culverts for pipe outfall is not acceptable.

If both the longitudinal system and the box culvert are proposed structures, then the box culvert may be modified to accommodate either a hole in the top or side of the box culvert barrel to accept flow from a drop inlet structure (Std 9031L2). The elevations of intersection and pipe diameter when compared to the box culvert dimensions may limit this option. In all cases, structural details are required.

7.3.1 Allowable Headwater

The allowable headwater depth (HWd), sometimes called the available head, is the depth of water that can be ponded at the upstream end of the culvert during the design-year storm. This depth is constrained by several factors.

The HW elevation must not exceed the level at which flow would bypass the culvert. For streams located within a FEMA-designated floodway or in communities participating in the NFIP, guidance for establishing the HW elevation can be found in Chapter 2 of this manual. For drainage basins of 200 acres or less, the HWd should generally not exceed 1.5 times the culvert diameter or depth (D). However, under certain conditions, the HWd may be allowed to reach up to 2D, as detailed in Section 8.3.2 of this manual, which also covers additional backwater limitations.

For all new and existing crossings and any size/shape culvert on streams with FEMA-regulated flood zones, the ponding limits shall be controlled through the H&H analysis to meet FEMA requirements (see Guidance for Hydrologic & Hydraulic Studies).

The allowable headwater and other design criteria for culverts are summarized in Table 7.1.

100-year storm ponding outside of a stream buffer and GDOT right-of-way should not be allowed.

7.3.2 Tailwater Relationship

Tailwater relationships vary depending on the particular scenario. The two most common are channels and larger water bodies, including confluences. The following sections discuss each of these scenarios and provide additional information.

Tailwater Relations for Channels

- Evaluate the hydraulic conditions of the downstream channel to determine the tailwater depths for all design flows and the average annual perennial stream flows (see Chapter 5 of this manual).
- For a subcritical hydraulic analysis, use backwater curves or a single, cross-section analysis.
- Use the headwater elevation of any nearby, downstream culvert if it is greater than the depth of flow in the channel.

Confluence or Large-Water Body

- For tributary tailwater conditions, use the backwater elevation of the main stream.
- For two separate rainfall events that occur within the same watershed and both events are independent of one another, use the higher tailwater condition that estimates a reasonable backwater elevation.
- If tidal conditions are present, use the mean high tide. For design methods and requirements for culverts located within a tidal area, see chapter 8 of this manual.

7.3.3 Pipe Culverts and End Treatments

New pipe culverts that cross under a roadway or driveway will have a minimum diameter of 18 inches. This also applies to other shapes, such elliptical or arch pipes, the openings of which are equivalent to that of the 18-inch circular pipe.

Where permitted by the drainage design criteria, existing drainage facilities may be retained within the project's limits. The cross-sectional area is to not be decreased in the direction of flow with the exception of extending existing 15-inch diameter pipes. 15-inch diameter pipes are to be extended with a minimum 18-inch inside diameter. If existing pipes less than 18 inches in diameter require modification which would reduce the cross-sectional area, the existing pipe is to be replaced.

The following end treatments should be used based on the pipe design and flow conditions:

Flared end and safety end sections:

- Inlet ends of all culvert pipes smaller than 48-inch diameter.
- Outlet ends of all storm drain pipes smaller than 48-inch diameter on a 10% or less grade.
- Inlet and outlet ends of all side drain pipes.
- Velocities are less than 12 ft/s.

NOTE: Use riprap protection where appropriate. The use of sand-cement bag riprap on smaller than 48-inch pipes is an acceptable outlet protection for higher priority facilities.

Inlet and outlet concrete headwalls:

- Inlet and outlet ends of all 48-inch or greater storm drain pipes.
- Outlet ends of all storm drain pipes over 10% grade.
- Outlets with velocities greater than or equal to 12 ft/s.

NOTE: projecting ends or ends mitered to the fill slope (no headwall) may be used for pipes less than 48 inches in diameter in certain cases if all other criteria are met.

7.3.4 Box Culverts

The following is applicable to box culverts:

- Standard box culverts have a minimum size of 4 ft. x 4 ft.
- Standard box culverts have a maximum size of 10 ft. x 12 ft. Larger box culverts will require special design details.
- Box culverts are to be used only at sites that have favorable floodplain conditions, which include a well-defined stream channel and a site that is not likely to accumulate silt in the culvert barrels. For example, box culverts located in a backwater condition created by a

pond/lake or other impoundment are prone to excessive siltation and this placement should be avoided.

- Multiple-barrel box culverts should fit within the natural primary channel with only minor widening of the channel permissible in order to avoid conveyance loss through sediment deposit in some of the barrels.
- Bridges should not be replaced with box culverts.
- Proposed box culverts are to be designed without incremental bends.

7.3.5 Culvert Extensions

All culvert extensions should be evaluated using the hydraulic principles discussed in this chapter. See section 7.3.1 for additional information about shape, size, and material selection.

Where culverts have bends and transitions, they may be analyzed as if they are storm drains. Energy losses within the culvert barrel will need to be considered for all bends and transitions when the culvert is operating in outlet control. See FHWA's Hydraulic Engineering Circular 22 for transition and bend losses.

Culverts will be extended with barrel sizes that are equal to or greater than the existing culvert barrel size in the downstream direction.

If additional fill is being placed on an existing culvert, the designer must confirm that the culvert can handle the weight of the additional fill.

Culvert extensions should be made along the same alignment as the existing culvert barrel. When it is not possible or feasible to extend a culvert along the same alignment as the existing culvert, then the maximum allowable deflection angle from the existing culvert alignment is 30 degrees, limited to 15-degree bends at each instance.

Box culvert extensions requiring multiple bends are to be designed according to the GDOT Construction Standards 2312, 2317, and 2318.

7.3.6 Minimum and Maximum Velocity

The minimum velocity in the culvert barrel should result in a tractive force ($t = gdS$) greater than the critical shear stress of the transported streambed material at low-flow rates.

- Use 3 ft/s when the streambed material size is not known.
- If clogging is probable, consider the size of culvert to facilitate cleaning or increase the slope of the pipe.

Culvert slopes that cause uniform design storm flow velocities greater than 12 ft/s could result in erosion and scour at the outlet and should be avoided.

7.3.7 Minimum Required Cover and Clearances

The following criteria should be used for designing culverts in proximity to other structures:

All pipe and box culverts should have a minimum cover of 1 ft or as shown in Standard 1030D for materials other than concrete. The minimum roadway clearance over a culvert is measured from the bottom of the pavement structure to the exterior crown of the culvert. If 1 ft. of cover cannot be achieved, an appropriate mitigation would be to select a Class 5 concrete for pipe material. Class 5 concrete pipe should have a minimum cover of 3 in.

Underground utilities will have a minimum clearance of 0.5 ft from the culvert.

7.3.8 Aquatic Organism Passage (AOP)

At perennial streams, accommodating aquatic organisms and migrating fish is an important design consideration. A primary concern is ecological connectivity between upstream and downstream channels. Design criteria often include minimum flow depths, maximum velocity, natural channel inverts, resting areas in long barrels, and no perched outlets. Minimizing the amount of channel contraction at the culvert inlet is also an important design component for AOP. [7-6] Very wide channels may even require the construction of a bridge spanning the natural stream. However, culvert design elements can often be included to meet the design criteria established by the fish and wildlife agencies, such as the requirements set forth in the USACE's regional permit. [7-9]

The smooth sides and bottom of a standard box culvert, circular pipe, or elliptical pipe tend to increase the velocity of water passing through the culvert. Higher velocities increase erosion at the entrance and exit of the culvert and can cause the invert of the culvert to become perched. The perched invert and increased velocities both create barriers for aquatic life to pass through the culvert. Designing the culvert for AOP reduces the likelihood that the culvert will become perched and generates flow velocities that more closely match the flow characteristics of the surrounding stream. There are several methods by which to design culverts for AOP. The most common design methods are: [7-6]

- Stream Simulation Methods
  - US Forest Service (USFS) Stream Simulation Method [7-7]
  - FHWA HEC 26 Stream Simulation Method [7-5]
- Simplified Method

The premise of either stream simulation method is to mimic the slope, structure, and dimensions of the natural streambed. Since it has similar characteristics to the natural channel, aquatic species should not experience difficulty passing through the stream simulation. [7-7] For more information on the USFS Stream Simulation Method, see the latest revision of Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossings.

The HEC 26 simulation method uses sediment behavior within the streambed as its primary parameter. The idea of using sediment behavior as a model criterion is that aquatic organisms in the stream are exposed to similar forces experienced by the streambed material. The end goal is to design a stream crossing with an equivalent effect. If all conditions are determined to be the same, then the newly designed stream crossing should not present an obstacle to aquatic organisms. [7-5] Refer to FHWA HEC 26 manual for the AOP culvert design guidance.

The requirements set forth in the USACE permit most closely resemble the simplified method, which is also the most commonly used. [7-9] The USACE method requires an embedment depth of 20% of the culvert rise height where the Etowah HCP culvert design policy requires an embedment depth of 30-50% of the rise height.

The following are design guidelines for complying with the USACE AOP requirements and the Savannah District's current Regional Conditions. [7-9] The designer must coordinate with the project ecologist in determining whether any given stream crossing is to be designed for AOP. As with most guidelines, there may be site-specific circumstances where it is not appropriate to design for AOP. This guidance does not relieve the designer from the responsibility of using engineering judgment to determine the appropriateness of designing for AOP for any given circumstance.

There are two cases of stream crossing types where AOP is required. The first is for new installations or replacements and the second is for retrofitting. New installations and replacements offer the flexibility to vary the type, size, shape, alignment, and bed material of the new culvert.

General AOP Guidelines for Culverts on Perennial Streams

1. Culverts should not permanently widen or constrict the channel and should not reduce or increase stream depth. The width of the base flow culvert(s) should be equal to the average channel width. Multiple pipe culverts should not be used to receive base flows.
2. Bankfull flows should be accommodated through maintenance of the existing bankfull, cross-sectional area.

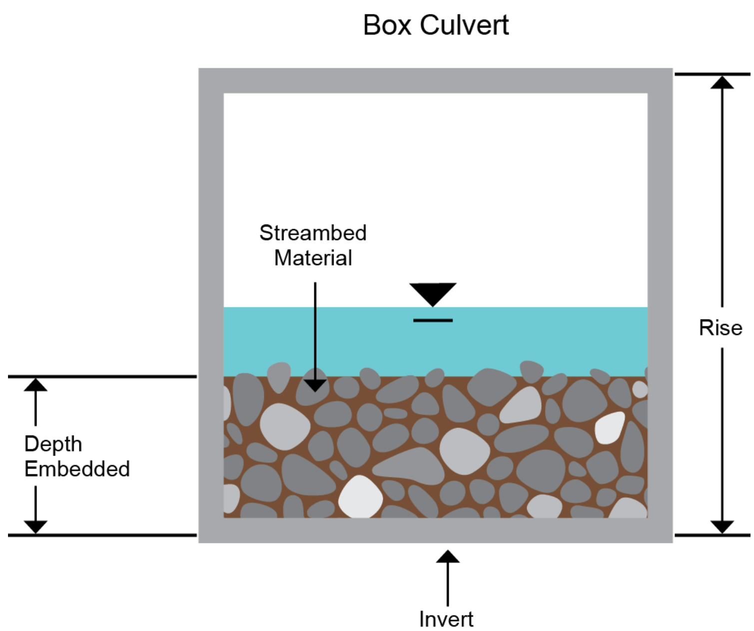

Figure 7.2 - Culvert embedment

3. Both inverts of culverts, except bottomless culverts, need to be buried or embedded to a depth of 20% of the culvert height to allow the natural substrate to colonize the structure's bottom and encourage fish movement. An example of embedment for a box culvert is illustrated in Figure 7.2. When baffles are used to facilitate deposition in box culverts, the quantity of baffles are included in the summary of quantities sheet. In Section 22 of the plans, these culverts need a note referencing the detail.
4. Culvert slope should be consistent with average stream segment slope but should not exceed 4%.
5. Culverts should be an adequate size to accommodate flow in such a way that does not cause flooding of associated uplands or disruption of hydrologic characteristics that support aquatic habitat at either end of the culvert. Consult the project ecologist to determine headwater and tailwater conditions that support the aquatic life.
6. Where adjacent floodplain is available, flows that exceed bank-full conditions should be accommodated by equalizer culvert barrels at the floodplain elevation. This may be additional barrels adjacent to the primary with flow restriction weirs set to the floodplain elevation.
7. Unless specifically described in the USACE's Pre-Construction Notification (PCN), the use of an undersized culvert to detain stormwater or for pollutant treatment is not authorized. [(7-9)]

Bridges and Bottomless Culverts for AOP

Bridges and bottomless culverts cause minimal or temporary impacts to AOP. However, these structures should be assessed with a stream simulation method to evaluate the geomorphic and hydraulic parameters that affect AOP for proper stream placement. Single span bridges and bottomless culverts should be evaluated against other applicable designs with regards to cost, constructability, scour, maintenance and environmental concerns.

Figure 7.3 is an image of a bottomless culvert. It is important to note that there is room for the stream to meander somewhat and there is no change in stream bed material. The impacts to the stream as a result of this culvert installation are minimal.

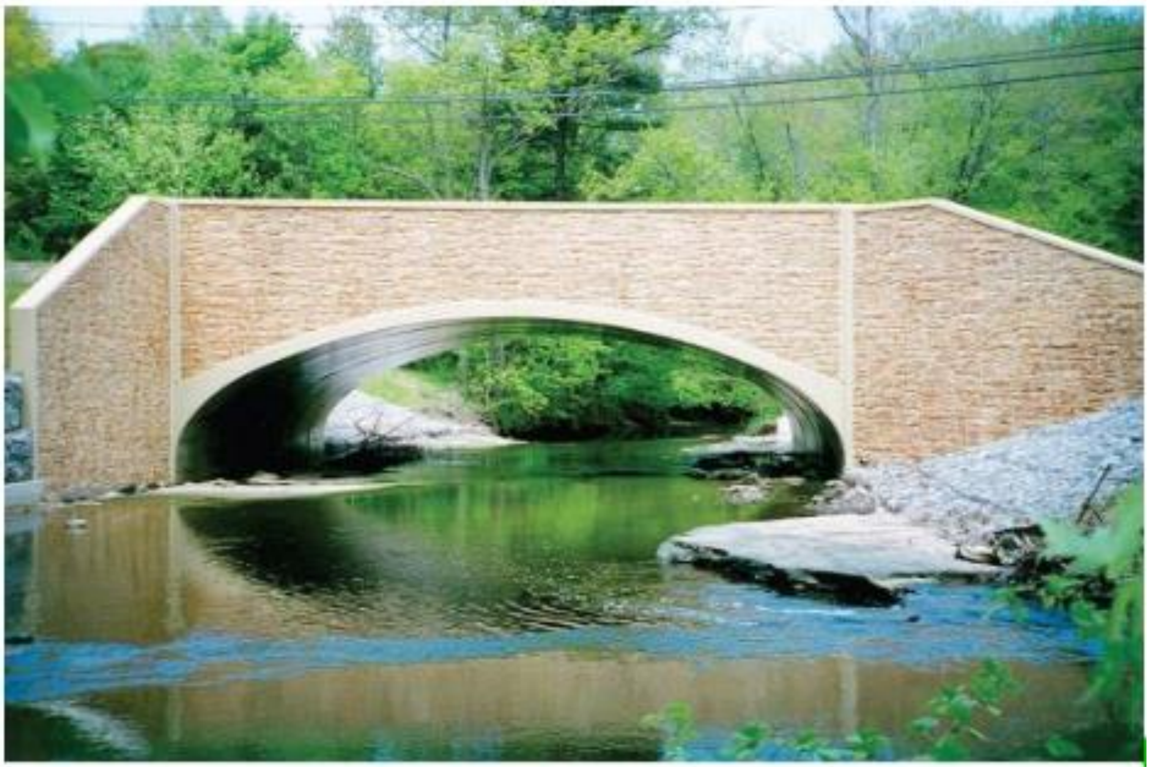

Figure 7.3 - Bottomless culvert (7-6)

Bottomless culverts should be used in locations where it is necessary to maintain the natural streambed through the culvert to meet environmental regulatory requirements. They can also be an alternative to a multi-barrel box culvert in areas that are prone to collecting debris.

The footings for a bottomless culvert should be placed below the streambed elevation on scour resistant material. The culvert foundations should be placed deep enough to withstand the possible channel migration and scour.

Due to the potential for scour problems, a scour analysis must be performed for all bottomless culverts similar to the bridge scour analysis.

Box Culverts for AOP

Box culverts both inherently have an artificial bottom surface that is not the same material or nature as that of the surrounding streambed. For this reason, the embedment requirements of the USACE's

Regional Conditions will apply. Photograph 7.2 shows an embedded box culvert designed to comply with the USACE embedment requirements.

Figure 7.4 - Embedded box culvert

Designing Box Culverts for AOP

1. Select a standard culvert width that most closely matches the average top of bank to the top of bank width of the stream for the portion of stream that is to flow through the culvert.
2. Calculate the height of the culvert needed to pass the design and check storm events. Manning's n values will need to be calculated to account for differing n values between the natural stream material on the bottom and smooth concrete sides of the culvert. Increase this height by 20% (rounded up to the nearest foot) to include the required buried depth to determine the final height of the proposed box culvert.
3. If a single- or multi-barrel box culvert that matches the stream bank to stream bank width is unable to pass the required storm event, additional flanking barrels may be added with weirs to restrict flow to overbank or floodplain flows. These additional barrels do not need to be embedded. The height of the weirs within these barrels should match the adjacent overbank elevations to minimize the risk of scouring.
4. In areas with high bedrock, it may be economically more feasible to construct a bottomless box culvert directly onto bedrock than to install a box culvert since the bedrock needs to be approximately 4 ft below the bottom of the streambed to avoid blasting. [(7-6)] Consult with the GDOT Bridge Design Office for special culvert design on bedrock.
5. Embedded box culverts should include the fish baffle detail (D-48) found on the GDOT details webpage.

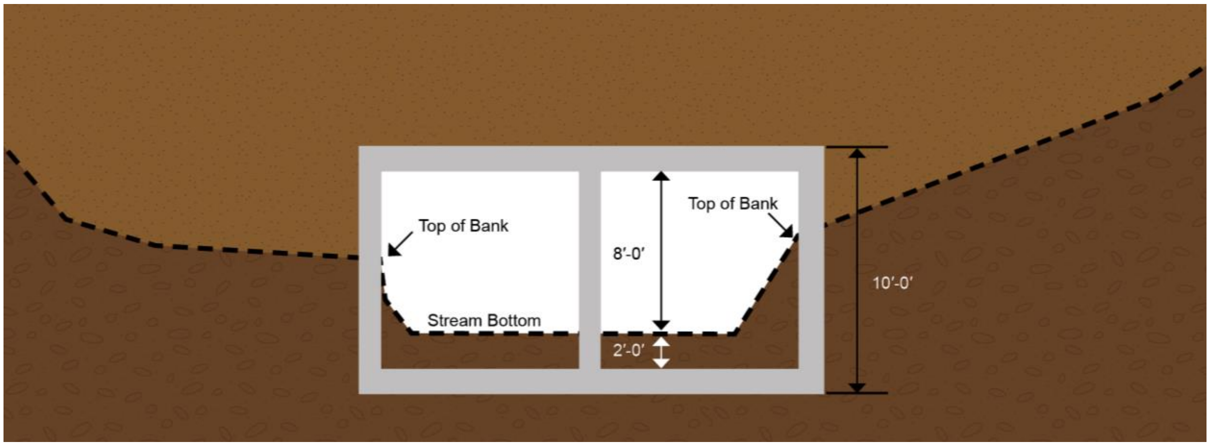

Figure 7.5 - Culvert barrel partially buried to preserve natural stream bed

When designing flanking structures, the non-embedded flanking structure is generally more hydraulically efficient and will tend to carry more flow than the embedded structure. To design parallel, dissimilar culverts, it is necessary to construct separate performance curves (elevation versus discharge) for each culvert. The two performance curves are added together at equal elevations to obtain the combined performance curve. This technique is described in FHWA's HDS 5 [(7-6)] for multiple-barrel culverts with unequal invert elevations.

7.3.9 Hydrologic and Hydraulic Study

Culvert that meet any of the conditions given below will require that a hydrologic and hydraulic (H & H) study be completed and submitted with the PFPR request for review.

- For existing or proposed culverts that have a total span length along the roadway of 20 feet or more
- For all sites located on streams where the 100-year floodplain has been delineated on FEMA maps
- For all sites located on streams that are named on county and/or USGS maps
- For all sites that have a significant risk associated with the project such as existing or potential flooding problems
- For all sites that are affected by downstream constrictions, obstructions, or abnormal flood stages from another stream

Culvert extensions that meet the following criteria will require a hydraulic analysis but not a detailed hydraulic study:

1. Existing culvert barrels are extended by less than 50% of the original length.
2. The profile grade of the roadway is not being raised.
3. No existing scour or flooding problems and the potential for any significant problem is low.

H&H reports completed by GDOT Office of Roadway Design, District Design offices, or GDOT Menu of Services consultants, are submitted to Roadway Hydraulics Group in the Office of Design Policy and Support for a quality assurance review. H&H reports completed by consultant teams are reviewed by a qualified person within their teams.

In addition to the requirements above, the cover sheet of the completed hydraulic and hydrologic study must state “Hydraulic & Hydrologic study prepared by” and must include the signature and Georgia P.E. stamp for the engineer who prepared the study. The cover sheet of the study must also state “QC/QA performed by” and must include the PE stamp and signature of the engineer performing the QC/QA for the study. The engineer who prepared the study may not be the same engineer performing the QC/QA.

GDOT Project Manager would ensure that an H&H report includes the necessary stamps and signatures on its cover.

7.3.10 Culverts Located Within a FEMA Regulatory Floodway

If the culvert is located within a FEMA regulatory floodway, FEMA guidelines must also be satisfied. See chapters 2 and 8 of this manual for more information on FEMA regulations and hydraulic modeling.

7.3.11 Special Culvert Configurations

- Sag culverts – Sag culverts or inverted siphons, are typically used for conveying irrigation water and should be avoided for typical roadway cross drainage. Special cases where a vertical obstruction must be avoided should be coordinated with the GDOT Roadway Hydraulics group.
- Equalizer culverts – Equalizer culverts are typically only used in flat areas where there is no natural outflow and stormwater tends to stand until the water percolates into the soil. The purpose of the equalizer culvert is to allow water on either side of the road to reach equilibrium. These culverts can be prone to siltation and should be sized based on ease of maintenance rather than for conveyance.
- Culvert Junctions – Flow from two or more separate culverts or storm sewers may be combined at a junction into a single culvert barrel. For example, a tributary and a main stream intersecting at a roadway crossing can be accommodated by a culvert junction. A drainage pipe collecting runoff from the overlying roadway surface and discharging into a culvert barrel is an example of a storm sewer/culvert junction. Loss of head may be important in the hydraulic design of a culvert containing a junction. Attention should be given to streamlining the junction to minimize turbulence and head loss in supercritical flow. Also, timing of peak flows from the two branches should be considered in analyzing flow conditions and control. Loss of head due to a junction is not of concern if the culvert operates in inlet control, but the junction must be streamlined to avoid causing a hydraulic jump to occur.
- Skewed Culverts – Culverts that are skewed to the roadway (not perpendicular) are often required to maintain the existing stream channel alignment and avoid channel changes,
- Skewed inlets/outlets – Where culverts are skewed to the roadway, pipe culvert inlets/outlets should align with a culvert; box culvers inlets/outlets should be parallel to the roadway alignment. In general, skewed inlets slightly reduce the hydraulic performance of the culvert under inlet control but the reduction is minor.
- Broken back culverts – A broken back culvert is a culvert with more than one slope and are typically only used in areas of high relief. They often consist of a steep upper section and a flatter runout section on the downstream end or low slope sections both upstream and downstream with a steep section in the middle. The purpose of this type of culvert configuration is to reduce the high outlet velocities associated with culverts on steep slopes.

7.4 Culvert Design Analysis

7.4.1 General Design Guidance

Inlet and outlet control are the two basic types of flow control defined in the research conducted by the National Bureau of Standards (NBS) and the FHWA (formerly Bureau of Public Roads - PR). The basis for the classification system was the location of the control section. The characterization of pressure, subcritical, and supercritical flow regimes played an important role in determining the location of the control section and thus the type of control. The hydraulic capacity of a culvert depends on a different combination of factors for each type of control.

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Figure 7.6 shows one typical inlet control flow condition. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. The upstream water surface elevation and the inlet geometry represent the major flow controls. The inlet geometry includes the inlet shape, inlet cross-sectional area, and the inlet configuration.

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. Figure 7.7 shows two typical outlet control flow conditions. All of the geometric and hydraulic characteristics of the culvert play a role in determining its capacity. These characteristics include all of the factors governing inlet control, the water surface elevation at the outlet, and the barrel characteristics (Table 7.4).

The factors in Table 7.4 distinguish between the geometric properties of the inlet versus the barrel to account for the effect of tapered inlets used on some culverts. For a culvert without a taper the inlet area and shape would be equal to the barrel area and shape. The slope of the culvert is called barrel slope to distinguish it from other slope parameters that may exist at the entrance, such as when a depressed inlet is used. The barrel slope is the primary factor influencing whether or not a culvert will be in inlet or outlet control. In the case of a mitered culvert, the length of the barrel is based on where the crown intersects the fill slope.

Table 7.4 Factors Influencing Culvert Design

Factor	Inlet Control	Outlet Control
Headwater	X	X
Area	X	X
Shape	X	X
Inlet Configuration	X	X
Barrel Roughness	-	X
Barrel Length	-	X
Barrel Slope	X	X
Tailwater	-	X

Note: For inlet control the area and shape factors relate to the inlet area and shape. For outlet control they relate to the barrel area and shape.

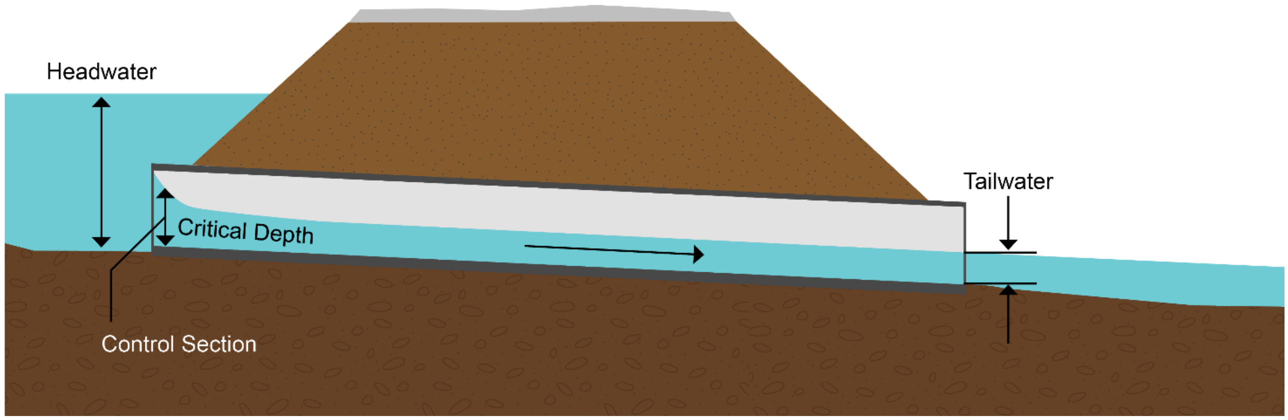

Figure 7.6 Typical inlet control flow section (7-6)

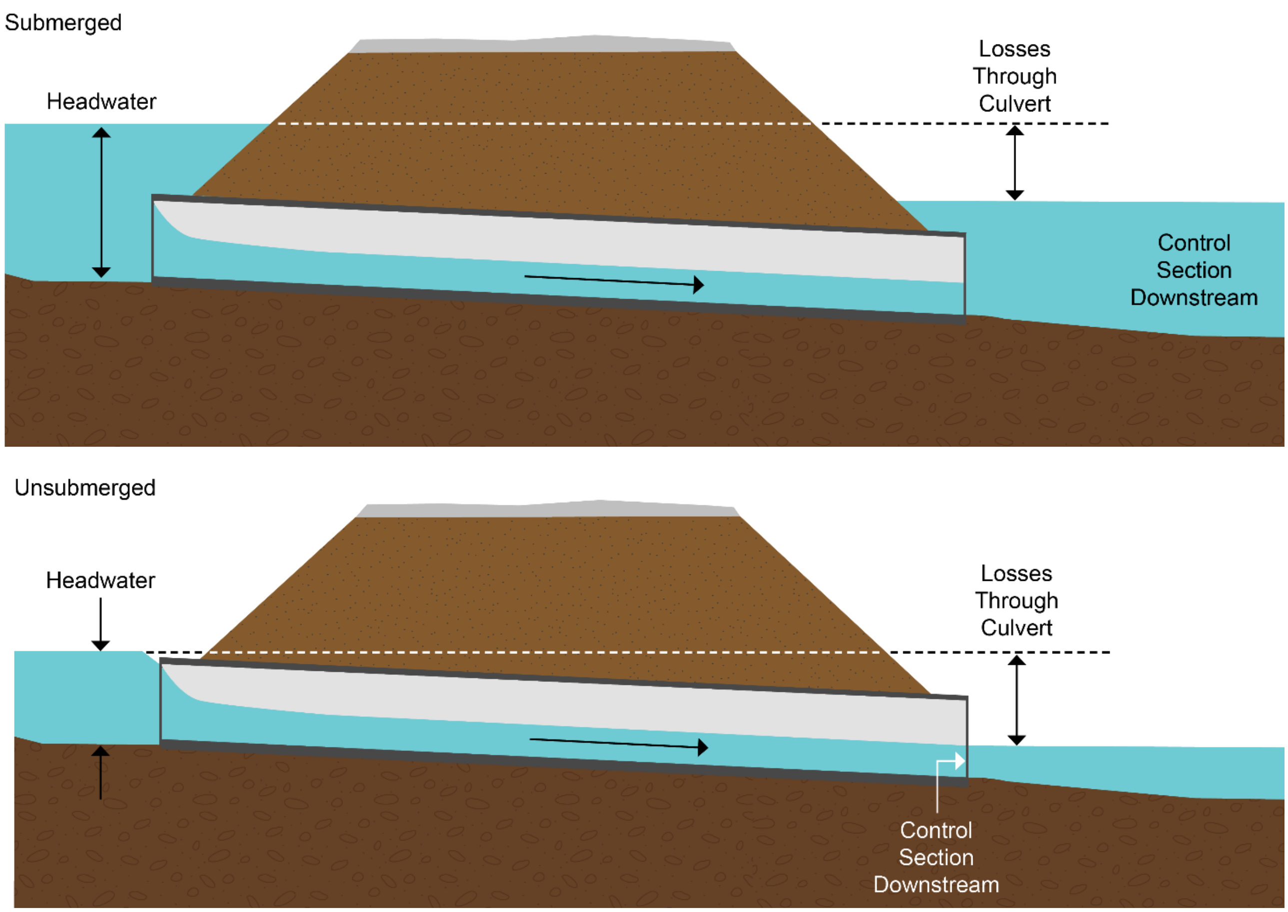

Figure 7.7 Typical outlet control flow conditions (7-6)

Box culvert Bends – Straight culverts are preferred for simplicity of construction and ease of inspection & maintenance. However, box culvert extensions may have incremental bends to maintain stream alignment to minimize channel realignment or to avoid a fixed structure such as a utility or a bridge column. Culvert bends for extensions should be limited to 15 degrees for every 20 ft of culvert length with a maximum of 30 degrees over the entire length of culvert. **Bends in new box culverts are considered special design requiring special design details to be reviewed by GDOT Design Policy & Support.**

Survey information for culverts should include topographic features, channel characteristics, aquatic life, highwater information, utilities, and existing structures. [(7-4)]

All culvert designs that fall within an ESA should be coordinated with environmental personnel to ensure that all permitting requirements are met.

7.4.2 Shape and Material Selection

7.4.2.1 Shapes

Numerous cross-sectional shapes are available for both closed conduit and open-bottom culverts. The most common closed conduit shapes are circular, box (rectangular), elliptical, and pipe-arch (Figure 7.8). These typical manufactured culvert shapes have the same material on the entire perimeter. Shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance. These shapes can be constructed with embedment which is a depression below the streambed of both the inlet and outlet inverts. Typical open bottom culvert shapes are various box and arch configurations shown in Figure 7.8. The cross-section shapes shown in Figures 7.8 and 7.9 along with a user-defined shape comprise the standard shapes available in the FHWA culvert design computer program HY-8.[(7-3)]

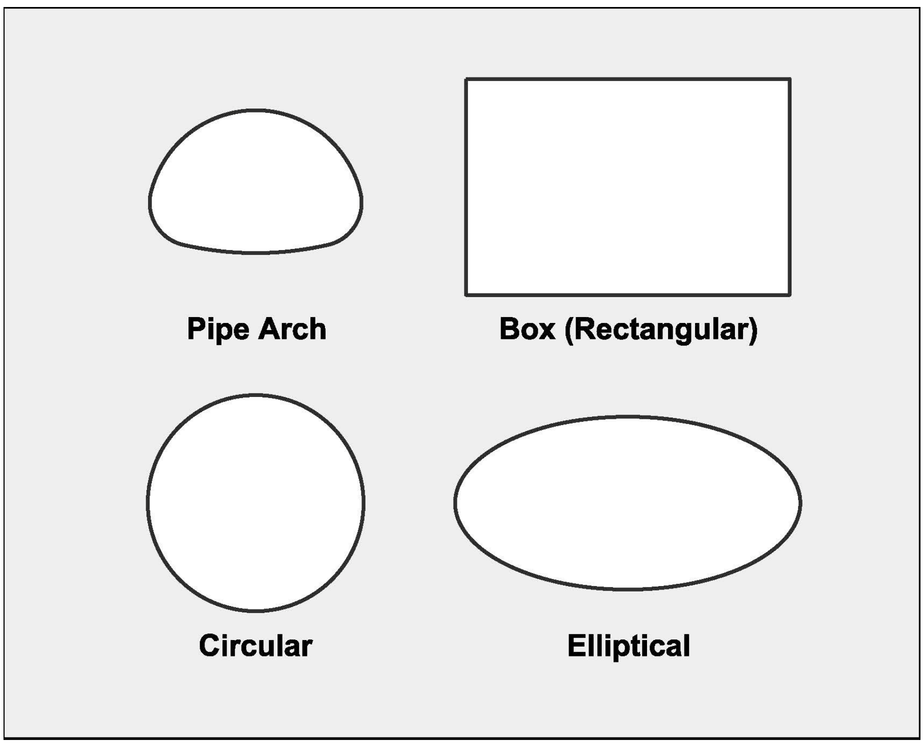

Figure 7.8 Commonly used culvert shapes [(7-6)]

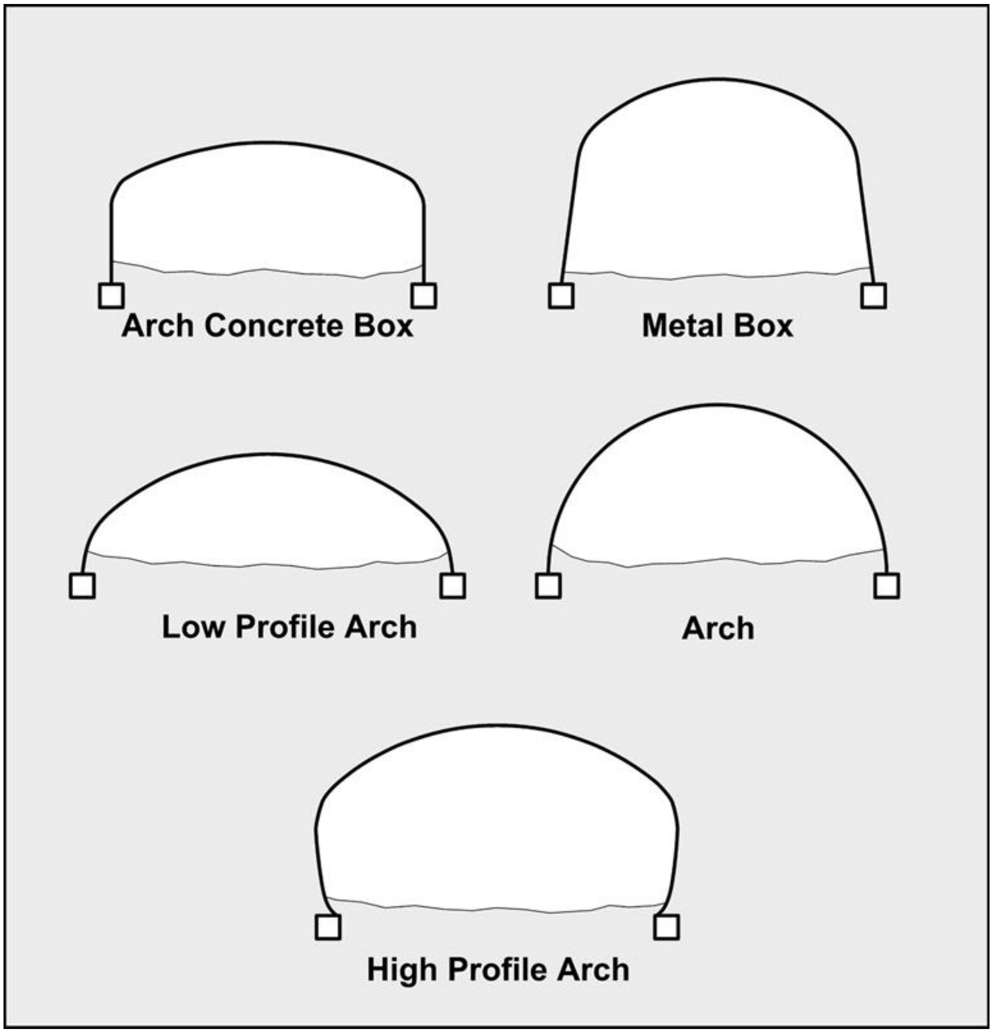

Figure 7.9 Commonly used open-bottom culvert shapes (7-6)

Elliptical pipes should be designed to be equivalent in capacity to circular pipes. See Table 5.2 for reference.

Table 7.5 — Elliptical Pipe Equivalent Round Size		
Equivalent Round Size, in	Reinforced Concrete Elliptical Pipe, in	
	Span	Rise
15 – 18	23	14
21 – 24	30	19
27	34	22
30	38	24
36	45	29
42	53	34
48	60	38
54	68	43
60	76	48
66	83	53
72	91	58

7.4.2.2 Materials

GDOT's preference is to use concrete pipe for all new construction. The OMAT Pipe Culvert Material Alternates chart and the GDOT Construction Standards 1030D1, 2, and 3 should be used to select a suitable alternative material. The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability (corrosion and abrasion resistance), and constructability. The most commonly used culvert materials are concrete (both reinforced and non-reinforced), corrugated metal (aluminum or steel) and plastic (high-density polyethylene (HDPE) or polyvinyl chloride (PVC)). A concrete box culvert and a corrugated metal pipe are depicted in Figures 7.10 and 7.11, respectively. Less commonly used materials include clay, stone, and wood, as might be found in historic culvert structures. Materials for culverts continue to be developed and, in the future, could include various types of plastics, fiberglass, and composite materials. Culverts may also be lined with other materials to inhibit corrosion and abrasion, or to reduce hydraulic resistance. For example, corrugated metal culverts may be lined with asphaltic concrete or a polymer material.

Figure 7.10 Concrete box culvert [(7-6)]

Figure 7.11 Corrugated metal pipe [(7-6)]

7.4.3 Design Software

Specific design software is not mandated by the Department; however, culvert analysis programs should use HY-8 as the basis of their analysis and design of culverts and energy dissipators.

A performance curve is necessary for any energy dissipator design and analysis.

If the tailwater at the culvert site is affected by downstream controls such as natural stream constrictions, irregular downstream cross sections, obstructions, impoundments, or backwater from another stream or body of water, the tailwater elevation to be used in HY-8 is first determined by performing a backwater analysis using a HEC-RAS water surface profile computer model. See section 7.3.3 for additional information on tailwater.

Additionally, HEC-RAS program should be used to perform analysis and design for bridge culverts (>20 ft). This will provide more detailed results located further upstream of the crossing and will be able to document headwater elevations at any structures that are subject to flooding.

HEC-RAS should be used for culvert analysis in the following situations:

- Wide floodplains
- When residential and/or commercial structures are located within or near the floodplain limits
- When more than one distinct culvert or other hydraulic opening is present (multiple openings)
- When FEMA has published flood elevations

Note that HEC-RAS modeling is typically performed by hydraulic engineers as opposed to roadway engineers.

7.5 Culvert Design Procedures

7.5.1 Acceptable Culvert Design Methods

For economic considerations, the designer should strive to select the smallest size culvert that can handle the required design flow and meet the allowable headwater depth. There may be other environmental constraints, such as AOP, that require the structure to be much larger than the normal culvert hydraulics require.

Multiple barrel culverts may be necessary due to site conditions, stream characteristics, or economic considerations. Roadway profiles with low fills often dictate the use of a series of small culverts. Multiple barrel culverts are also used in wide, shallow channels to limit the flow constriction. To accommodate overbank flood flows, relief culverts with inverts at the flood plain elevation are occasionally used. Multiple barrel box culverts are more economical than a single wide span because the structural requirements for the roof of the long span are costly.

Figure 7.12 Multiple barrel box culvert (7-6)

The most significant problems associated with the use of multiple barrel culverts are sedimentation and debris. In alluvial channels, normal flows tend to pass through one of the barrels, while sediment and debris collect in the others. To reduce this problem, weir walls may be used in the outer barrels.

(Figure 7.13). This will encourage the flow and sediment to follow the lower barrel. Sediment and debris accumulation in the other barrels will be reduced since the barrels will only be used to convey higher than normal flows.

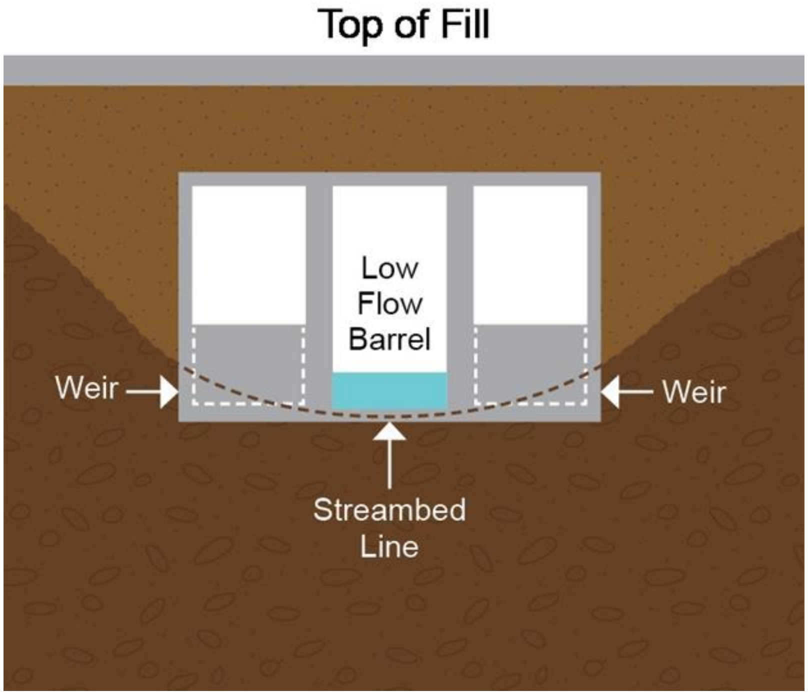

Figure 7.13 Multiple barrel culverts with one low flow barrel.

7.5.2 Improved Inlets

Economic considerations are important factors in determining the use of inlet improvement beyond the standard beveled edge. Such improvements should be evaluated by comparing costs and benefits.

7.5.3 Scour at Inlets and Outlets

A culvert barrel normally constricts the natural channel, thereby forcing the flow through a reduced opening, increasing flow velocities and erosive potential of the stream. Therefore, wingwalls are to be proposed for all culverts, and concrete aprons should be considered at the mouth of box culverts to limit scour and erosion around the culvert. Alternative protective measures may be appropriate depending on the characteristics of the stream and culvert design.

Scour at Inlets

As the flow contracts at the inlet of a culvert, vortices and areas of high velocity flow impinge against the upstream slopes of the fill and may tend to scour away the embankment adjacent to the culvert. In many cases, a scour hole also forms upstream of the culvert as a result of the acceleration of the flow as it leaves the natural channel and enters the culvert. Upstream slope paving, channel paving, headwalls, wingwalls, and cutoff walls help to protect the slopes and channel bed at the upstream end of the culvert.

Scour at Outlets

Scour and erosion at culvert outlets is a common occurrence. The natural channel flow is usually confined to a lesser width and greater depth as it passes through a culvert barrel. An increased velocity results with potentially erosive capabilities as it exits the barrel. Turbulence and erosive eddies form as the flow expands to conform to the natural channel. However, the velocity and depth of flow at the culvert outlet and the velocity distribution upon reentering the natural channel are not the only factors which need consideration. The characteristics of the channel bed and bank material, velocity, and depth of flow in the channel at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Due to the variation in expected flows and the difficulty in evaluating some of these factors, scour prediction is subjective. Erosion in the vicinity of a culvert outlet can be the result of local scour or stream degradation, or both occurring simultaneously. Local scour produces a scour hole at the culvert outlet that is the result of high exit velocities. The effects of local scour only extend a limited distance downstream. Coarse material scoured from the circular or elongated hole is typically deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole can change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow and can partially refill as flows diminish. Methods for estimating local scour hole dimensions are found in HEC-14[(7-8)]

In contrast, stream degradation extends over long distances and time frames as a result of natural and manmade changes in the watershed[(7-10)]. Degradation can be caused by a culvert or be completely independent of the culvert. For example, a linear embankment running transverse to overland flow usually concentrates flow at culvert locations, increasing the discharge over natural conditions. This concentration of flow can initiate deep and large-scale degradation downstream of the culvert that can extend great distances, particularly in arid regions. Degradation can also occur from changes in the watershed that are unrelated to culverts. Examples include reduced sediment supply, channel straightening and bed lowering in channels downstream of the culvert location. The identification of a degrading stream is an essential part of the original site investigation. Methods for evaluating degradation are provided in HEC-20[(7-10)].

7.5.4 Energy Dissipators

The maximum velocity at the culvert outlet should be examined, which may include a sediment transport calculation examining the streambed shearing stress of the sediment. For culverts that operate with velocities greater than 10 ft/s, downstream scour and erosion can become problematic. The culvert design methodology (discussed in section 7.4) does not control the outlet velocity in total. The slope, type of material, tailwater, and other factors can also affect the velocity. See the design methodology section for more details.

If the velocity discharged from the culvert is greater than the velocity in a downstream natural channel for the design flow, the following should be considered:

- Channel stabilization
- Energy dissipation

Scour holes often occur at culvert outlets. As such, outlet protection for the design storm event should be provided where the outlet scour hole depth computations indicate that the scour hole:

- Will undermine the culvert outlet

- May cause costly property damage
- Causes a nuisance effect (most common in urban areas)
- Blocks aquatic life stream sustainability
- Will restrict land-use requirements

An energy dissipator should be used at culvert outlets when outlet velocities become excessive for site conditions and downstream scour becomes problematic. (See FHWA HEC 14 Hydraulic Design of Energy Dissipators for Culverts and Channels [(7-8)] for scour computations and design of energy dissipators.)

General Guidelines for Energy Dissipators

Energy dissipators should be considered for the following conditions:

- The potential erosion at the culvert outlet will become a risk to the roadway itself or a downstream property or structure.
- Culvert outlet velocities are greater than 15 ft/s.

When considering energy dissipators for culvert outlets, determine if the native bed material is erodible. It should be noted that energy dissipators may not always be necessary. Conditions such as bedrock-lined stream channels or steep stream slope may not require energy dissipation design.

Refer to Hydraulic Design of Energy Dissipators for Culverts and Channels [(7-8)] and policies within this chapter for energy dissipation design guidance.

The depth of the riprap at the endrolls is normally 2 feet. The Department of Transportation uses two sizes of riprap: Type 1 riprap has a D50 of 1.14 feet and Type 3 riprap has a D50 of 0.64 feet. Woven plastic filter fabric is placed under the riprap (refer to GDOT Specification 881.2.05).

Note: Type 1 riprap should be used at all locations. Type 3 riprap may be substituted where placement problems prevent the practical use of Type 1 riprap **AND** it is shown that Type 3 riprap is sufficient protection against scour damage during the 100-year flood.

7.5.5 Internal Energy Dissipators

In situations where there is limited right-of-way for an energy dissipator beyond the discharge point of a culvert or chute, there are several options for internal or integrated dissipators, including adding internal roughness elements throughout the culvert or chute or just prior to the outlet. These approaches may be applicable within closed culvert barrels (conventional or broken-back) as well as in open, usually rectangular, chutes. Roughness elements are sometimes a convenient way of controlling outlet velocities for culvert installations where the culvert barrel is not used to capacity because it is operating in inlet control. These roughness elements may be designed to slow the velocity in the culvert including, at the limit, creation of a condition of tumbling flow, where the outlet velocity is reduced to critical velocity. Such internal roughness elements may be placed throughout the entire length of the culvert or chute, or simply near the end prior to the outlet, depending on the hydraulic conditions and desired outlet conditions. [(7-8)]

7.5.6 Channel Changes

To reduce potential environmental mitigation requirements and to minimize costs associated with structural excavation and/or channel work; channel changes should be avoided if at all possible. In cases when a channel change is necessary and cannot be avoided, the abrupt stream transitions at either end of the culvert should be avoided. Environmental coordination and permitting will be required for any necessary channel modifications.

7.5.7 Culvert Constructability and Staging

Reinforced concrete box culverts (RCBC) and precast culvert boxes and arch culverts can be constructed in stages. This typically involves constructing enough of the structure in the first stage such that traffic can be shifted to the new structure section while the remainder of the structure is complete. The limits of the excavation need to be carefully considered when evaluating the constructability, as shoring may be required to between stages to maintain the excavation area.

Maintenance of the stream flow during construction should also be considered. The engineer should recognize that the base flow rate (at a minimum) will need to be accommodated by pumping around the construction or by staging the barrels such that base flow is conveyed through the existing structure until at least one of the new barrels is constructed. The selected diversion method must not have an adverse impact on the surrounding area. The contractor should be required to submit a plan for diverting or controlling the culvert flow.

In some cases, environmental considerations such as an endangered species may dictate that the removal of an existing culvert and/or the construction of a new culvert be completed in a single stage to minimize the in-water disturbance. The hydraulic engineer should coordinate with the environmental staff to ensure that these environmental obligations are met.

7.5.8 Culvert Rehabilitation and Maintenance

In cases where an existing culvert satisfies the hydraulic criteria but has some structural issues, a rehabilitation plan should be considered. Many existing culvert condition factors and indicators must be assessed and considered in selection of appropriate repair and rehabilitation methods and technologies. Techniques intended to address culvert deterioration will also influence hydraulic performance of the culvert. Given an accurate assessment of culvert condition, the decision between repair and replacement can be made. The typical outcomes of this decision-making process are to repair with a lining, repair based on man-entry, or to simply replace. The Culvert Pipe Liner Guide and Specifications[(7-10)] provides information on common lining techniques and methods. The ***Culvert Assessment and Decision-making Procedures Manual*** [(7-12)] provides information on assessment procedures and repair options.

The key factors surrounding culvert maintenance include keeping the opening clear of silt and debris. Multi-barrel structures tend to collect silt in one or more barrels, which leads to a reduction in culvert capacity. In addition, culverts of all sizes are prone to collecting debris and become completely blocked if the debris is not cleared in a timely manner. The majority of debris comes from along the stream banks so the removal of visible woody debris immediately upstream of culvert structures will improve the effectiveness of this maintenance activity.

7.6 Chapter 7 References

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Chapter 8. Bridge Hydraulics

8.1 Introduction

This chapter provides hydraulic design criteria for all existing and/or proposed riverine and tidal bridge sites and for bridge culverts. Bridges over water should be designed to minimize flood hazards, to minimize impacts to environmental resources, and to pass design storm flows across the right-of-way. Due consideration should be given regarding risk to the facility, property and structures in the floodplain affected by the facility, and the traveling public while optimizing costs. Floodplains are the low areas bordering a stream that are subject to inundation by floods and the term is used in this chapter both generally and to refer to specific flood boundaries such as the 100-year floodplain. From a hydraulic perspective, a bridge is defined as a part of a stream crossing system that includes the roadway approaches and all hydraulic openings. Hydraulic engineers should be involved from the outset of a bridge project to ensure that the bridge span arrangement and superstructure type/depth along with the required roadway profile is optimized with regard to the design criteria and overall project cost. Bridge hydraulic design includes regulatory requirements, specific approaches for bridge hydraulic modeling, bridge design impacts on scour and stream stability, and sediment transport.

The hydraulic analysis and design of bridges is as vital as the structural design. Bridges should be designed for the following:

- minimum cost subject to design criteria;
- desired level of hydraulic performance;
- mitigation of impacts on the stream environment and surrounding floodplain;
- safe movement of desired traffic volume under an acceptable level of service; and
- accomplishment of social, economic, and environmental goals. [(8-1)]

8.2 Design Storm

Design flood (or storm) frequencies are established based on hydraulic performance criteria set per risk-based assessments of local site conditions including, but not limited to, backwater, clearance, and overtopping. The criteria established accounts for consideration of traffic service, environmental impact, property damage, hazard to human life, and floodplain management criteria. See Chapter 3 “Hydrology” of this manual for methods in determining discharges for the applicable storm frequencies.

All bridges over water are to be designed to convey the design flood through the bridge opening.

8.2.1 Riverine Bridge Replacements / New Locations

Riverine bridge replacements as well as new riverine and new parallel bridge locations shall be designed per the design storm and freeboard criteria outlined in Table 8.1 below. Not meeting freeboard requirements for a bridge shall require the prior approval of a Design Variance from the GDOT Chief Engineer or designee such as the GDOT Bridge Hydraulics Engineer. See GDOT Standard Drainage Design Criteria for reference.

Table 8.1 – Riverine Bridge Hydraulics Design Storms and Freeboard Criteria

Facility	Design Traffic (ADT)	Design Storm (Yr)	Freeboard Criteria above Highwater Elevation (Whichever is Greater)	
Interstate/ State Routes	N/A	50	2 ft above Design Flood	1 ft above the 100 Year Flood
Non-State Route	0 – 99	5	2 ft above Design Flood *	0.5 ft above the 100 Year Flood
Non-State Route	100 – 399	10	2 ft above Design Flood *	0.5 ft above the 100 Year Flood
Non-State Route	400 – 1,500	25	2 ft above Design Flood	0.5 ft above the 100 Year Flood
Non-State Route	Over 1,500	50	2 ft above Design Flood	0.5 ft above the 100 Year Flood
Temporary Detour	Interstate/State Routes/ Non-State Routes > 400	10	1 ft above Design Flood	N/A
Temporary Detour	Non-State Routes < 400	2	1 ft above Design Flood	N/A

<u>* For riverine bridge replacement, A minimum of 1 ft of freeboard above the roadway overtopping flood is required for overtopping storms less than the 100-year flood</u>.

8.2.2 Tidal Bridge Replacements/New Locations

Tidal bridge replacements as well as new tidal and parallel bridge locations shall be designed per the design storm frequency outlined in Table 8.2 below.

Table 8.2 – Tidal Bridge Hydraulics Design Storm	
Facility	**Design Storm Tide (Yr)** (8-4)
Interstates	50
State Routes	25
Non-State Route	Mean High Spring Tide

8.2.2.1 Additional Design Storm Frequency Considerations in Tidal Areas

Bridges on tidal streams should be designed to protect the bridge structure itself. Most of the surrounding land and the approach roadways may be inundated by relatively frequent (10- to 25-year) tidal storm surges.

At sites that have a combination of riverine and tidal flows, the various combinations of flows should be analyzed to determine the controlling flow. Note that controlling flow for maximum velocities may differ from flow used to determined maximum water surface elevation.

8.3 Design Criteria

Parameters required for the design of safe bridges include determining the waterway opening, roadway grade, low beam, or low chord elevation, scour potential, and countermeasure design as it relates to stream stability. The term countermeasure is defined as a measure intended to prevent, delay, or reduce the severity of hydraulic problems. The following sections list standard GDOT design criteria used in the hydraulic design of bridges.

8.3.1 Freeboard Policy

Freeboard refers to the vertical clearance between the lowest structural member of the bridge superstructure to the water surface elevation of a flood. GDOT criteria for freeboard requirements at bridges are set based on the design flood criteria and the corresponding design highwater elevation (Table 8.1) The design highwater elevation for evaluating freeboard and determining the minimum low chord elevation should represent the highest water-surface at the upstream face of the bridge through the bridge (Figure 8.1). Based on the highwater elevation along the upstream embankment, and consultation with the structural design engineer, the minimum finished grade for the bridge should be determined and supplied to the road designer for use in determining an appropriate finished grade elevation.

Freeboard ensures that the bridge structure is protected from damage from debris, damage to bearings and beam seats from the corrosive effects of water, and to reduce the possibility of pressure flow which tends to produce more severe scour. Freeboard requirements for bridge in the Low Impact Bridge Program (LIBP) and similar programs such as State Funded Local Bridge (SFLB) and Local Bridge (LOCBR) should follow practical design where applicable. On low laying bridge often inundated by frequent floods, raising the profile to meet freeboard are often not practical. Please see the LIBP manual for more guidance.

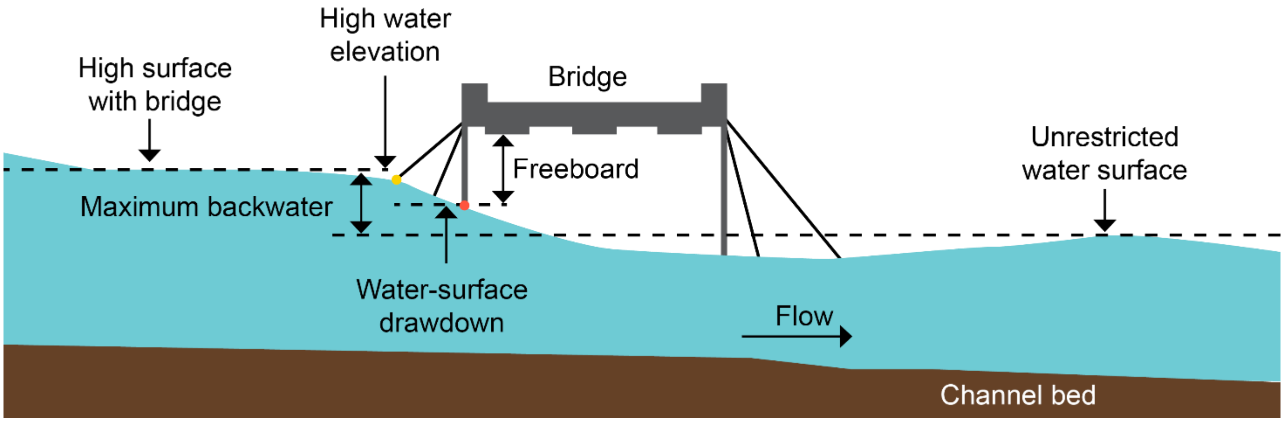

Figure 8.1 Illustration of Bridge Backwater, Highwater elevation, and Freeboard

8.3.1.1 Riverine Bridge Replacements / New Locations

Road subgrade elevation in vicinity of bridges shall be at least 1 foot above the design year highwater level (State Route and Interstate). A decision to design new roads that do not meet the subgrade and roadway freeboard requirements during a design flood event in the vicinity of bridges shall require a comprehensive study by an engineer and the prior approval of a Design Variance from the GDOT Chief Engineer or designee such as the GDOT Bridge Hydraulics Engineer. Additional coordination with the roadway designer is required prior to submitting the Design Variance. See GDOT Standard Drainage Design Criteria for reference.

For existing roads, subgrade elevation near bridges should be at least 1 foot above the design year highwater level and should be analyzed in the hydrologic and hydraulic report. If this cannot be achieved, consult GDOT Bridge Hydraulics Group.

For low volume roads not designated as State Routes, the proposed roadway profile can be set so that the design year highwater is a minimum of one-half foot below the shoulder breakpoint, if the conditions in items (a) and either (b) or (c) exist:

a. Design Year Traffic (ADT) is less than 400 vehicles per day (VPD)
b. Houses and/or buildings are located upstream of a riverine crossing with a high risk of being flooded by the 100-year storm; and raising the profile grade of the roadway would have an adverse effect and/or potential flooding of upstream properties.
c. Raising the profile grade elevation of the roadway would increase the 100-year flood backwater above one foot or would require additional coordination with FEMA at locations where Base Flood Elevations and/or Floodway are established.

<u>Additional design storm frequency and freeboard considerations are as follows:</u>

- If the bridge site is affected by abnormal water surface elevations, the bridge will provide freeboard above the abnormal water surface elevation and be designed for the velocity that occurs without the effects of the abnormal water surface elevation. See section 8.4.2.2 for additional guidance on abnormal floodstages.

- When hunched girders are used, freeboard will be measured at the haunch location on the bridge.
- If the bridge is over a major lake or reservoir where there is boat traffic, the desirable grade must be set so that there is at least 8 feet of freeboard above the maximum operating pool. The required freeboard must be determined through coordination with the lake or reservoir owner and any other applicable stakeholders. This coordination should also determine whether the entire bridge meets the required minimum freeboard or just the main span over the deepest water. The minimum grade should not reduce the freeboard from the existing conditions and can be used if this freeboard meets the above-required minimum clearances and satisfies any requirements concerning boat traffic in the area.
- If debris is a problem at the site, the above-required minimum clearances may be increased with the concurrence of the GDOT Bridge Hydraulics Engineer.
- If the bridge is located over a U.S. Coast Guard navigation channel, the proposed bridge is to be designed to meet the vertical and horizontal clearances as required by the U.S. Coast Guard.
- Since Type I Mod beams are often substituted for reinforced concrete deck girders in the final structural design phase, the initial minimum profile grade for these bridges should be set so that the minimum vertical clearances are obtained using the Type I Mod superstructure.
- In the case of a perched bridge on a low volume roadway, flood events greater than the design year may be allowed to overtop the roadway as long as the bridge freeboard requirements are met.

8.3.1.2 Tidal Bridge Replacements/New Locations

Tidal bridges are to be designed for unsteady flow conditions during the complete rise and fall cycle of a tidal surge. Bridge structures on tidal streams or waterbodies affected by tidal surge/cycles are to be designed to protect the bridge structure itself. Most of the surrounding land and the approach roadways may be inundated by relatively frequent (10- to 25-year) tidal surges. The finished grade of the bridge will be set by considering navigational clearances, the approach roadways, topography, and practical engineering judgment.

Tidal bridge replacements as well as new tidal bridge locations shall be designed per the freeboard criteria outlined in Table 8.3 below. A decision to not meet freeboard requirements during a design flood event for a bridge or bridge culvert shall require a comprehensive study by an engineer and the prior approval of a Design Variance from the GDOT Chief Engineer or designee such as the GDOT Bridge Hydraulics Engineer.

Table 8.3 – Tidal Bridge Hydraulics Freeboard Criteria		
Facility	**Design Tide Event (Yr)**	**Freeboard**
Interstates	50	1 ft above the 100-year tide event and 2 ft above Design Storm Tide
State Routes	25*	2 ft above Design Storm Tide
Non-State Route	N/A	2 ft above the Mean High Spring Tide**
* For tidal bridge replacement, minimum design flood is the overtopping tide event, if less than the 25-year tide event. ** For tidal bridge replacement, minimum of 2 ft of freeboard above the roadway overtopping flood is required for overtopping event less than or equal to the 10-year tide event.		

Freeboard for Road Subgrades in Tidal Areas

- Interstate subgrades should be a minimum of 1 foot above the 50-year tide event.
- One foot above the 25-year tide event elevation is desirable for state route subgrades. Adjoining roadway elevations can be considered when setting minimum roadway grades.
- For roads not designated as state routes, road subgrades should be a minimum of 1 foot above the mean high spring tide elevation.

8.3.1.3 New Parallel Bridges

New bridges built parallel to existing structures should follow the design criteria for bridge replacements.

At a minimum, the bottom elevation of the new parallel bridge’s superstructure should match the existing bridge’s bottom of superstructure elevation, thereby not reducing the existing area of bridge opening, nor existing clearances. This minimum design is only considered if no scour or flooding problems exist and the potential for any significant problems is low.

8.3.1.4 Bridge Widenings

The low chord of the widened bridge should be no lower than the existing bridge, thereby not reducing the existing area of bridge opening, nor existing clearances. Intermediate bent placement should consider existing scour or debris issues.

8.3.2 Bridge Hydraulic Capacity and Backwater

Bridges should be designed to provide sufficient hydraulic capacity to pass the design storm while maintaining acceptable velocity and backwater values. As bridges will tend to generate backwater, a vertical profile that provides the prescribed clearances (freeboard) in Section 8.3.1 over the backwater created by the bridge or other nearby influencing structures should be established. Backwater is defined as the increase in water surface elevation induced upstream of a structure that obstructs or constricts a floodplain relative to the elevation occurring under natural channel and floodplain conditions. Backwater caused by a structure is measured relative to the natural water surface elevation without the effect of the bridge at the approach cross section. The approach section should be located at the point of maximum backwater.

The 100-year backwater should be limited to 1 foot above the unrestricted or natural 100-year water surface profile. This backwater value should include effects from a roadway in the case of a longitudinal encroachment on the floodplain.

The engineer may determine that limiting backwater to 1 foot is not practical for bridge replacement projects. In this case, the 100-year backwater elevation may exceed 1 foot above the unrestricted or natural 100-year water surface profile, but it may not be higher than the existing condition backwater value. This limitation will only be accepted for new drainage structures in rare instances where it can clearly be shown that it is impractical to size the drainage structure to meet backwater requirements.

A decision to not meet backwater requirements for a bridge or bridge culvert shall require a comprehensive study and completion of the GDOT Backwater Waiver by an engineer and the prior approval from the GDOT Bridge Hydraulics Engineer. See GDOT Standard Drainage Design Criteria for reference.

A template of the Backwater Waiver may be obtained from the GDOT Bridge Hydraulics Group.

Justification for waiving the backwater requirement must be clearly shown in the hydraulic and hydrological study including construction cost comparisons to backwater. In all cases, the drainage structure must be sized so that the drainage structure and roadway are protected against failure during major flood events.

For bridge widening and paralleling projects, the existing backwater may already be in excess of 1 foot over the unrestricted or natural 100-year water surface profile. If there are no existing scour or flooding issues, the existing backwater would be considered acceptable.

For new location crossings, flood delineation maps for the natural and proposed conditions are to be developed to ensure that no structures would be flooded by the proposed construction. Finished floor elevation should be surveyed as required. In addition to the above limitations, the hydraulic design needs to conform to NFIP requirements for stream crossings with flood elevations provided by the NFIP's studies.

Negative backwater will not be acceptable unless in an abnormal flood condition. When channel modifications result in negative backwater, the modified channel should be incorporated in a second natural conditions to evaluate proposed backwater. Future development, current conditions, and past historical flooding conditions in the upstream and downstream floodplains should be considered for all cases.

8.3.3 Flow Velocities

Flow velocities through the bridge structure(s) should be analyzed and the structures should be designed to avoid damage to the highway facility or adjacent property. For example, when an adjacent property is located in the immediate vicinity of the bridge, flow direction and velocity should be considered. This may require the use of a 2D model. Acceptable stream channel and overbank velocities should be determined by comparison with the natural velocities and existing bridge velocities, along with any scour problems, or lack thereof, at the existing structure. The type of soil at the site (highly erodible or not) should be considered. Bridge culverts should be sized with acceptable flow velocities to minimize potential scour.

To minimize scour and impacts to the highway facility and adjacent properties, the maximum desirable stream channel velocity values for new bridges should be in the range of 1.5 to 1.75 times the natural/unrestricted channel velocity for the design year and 100-year storms. Values should be obtained within the bridge opening.

8.3.4 Bridge Scour and Stream Stability Criteria

Based on the Technical Advisory (TA 5140.23) issued by FHWA in October 1991, GDOT requires a scour evaluation for existing and proposed bridges over waterways. Bridges should be designed to withstand scour from large floods and from stream instabilities expected over the life of a bridge including, but not limited to, designing the bridge to be stable from scour at the piers, abutments, and across the contracted opening.

8.3.4.1 Bridge Scour Design Flood and Check Flood Criteria

For riverine bridges, the scour analysis is to be completed using all standard storm frequencies between 2-year storm and 100-year storm (scour design flood), and including design storm (according to section 8.2), or the overtopping flood if it is less than or equal to the 100-year storm. Scour is also to be computed for the 500-year flood (scour check flood) or the overtopping flood if it is more intense than the 100-year flood and less intense than the 500-year flood. This event should be used to evaluate the proposed bridge opening to ensure the resulting potential scour will produce no unexpected scour hazards. General contraction scour calculations for the stream channel and overbanks should be completed as well as local pier scour calculations for each intermediate bent. Bridges crossing waterways with abnormal floods are to follow the same criteria but also be analyzed for abnormal floods.

For tidal bridge structures, the scour analysis is to be completed using the 100 and 500-year or overtopping upland riverine floods along with the appropriate tidal influences, as well as the 100 and 500-year or overtopping storm tidal surges combined with the appropriate upland riverine flows. General contraction scour calculations for the stream channel and overbanks should be completed as well as local pier scour calculations for each intermediate bent.

For bottomless culverts, the scour analysis should follow the same general guidance as riverine bridges.

8.3.4.2 Bridge Scour and Stream Stability Countermeasure Design Criteria

Bridges should be designed to withstand scour from large floods and from stream instabilities expected over the life of a bridge. Countermeasure design should be considered and may include abutment protection, guide banks, or other less common methods.

8.3.4.2.1 Bridge Abutment Protection

Stub abutments, built on top of an embankment with a 2:1 slope normal to the end bent are normally used for new bridges. Riprap protection for these endrolls should be sized using the method shown in the latest version of the FHWA HEC-23, [8-5] *Bridge Scour and Stream Instability Countermeasures*. The 100-year flood should be used for this design. This riprap protection (apron) should be entrenched 2 feet below and embedded flush with the natural ground line and extend 2 feet above the 100-year water surface elevation. In rare cases where the 100-year storm is above the berm elevation, the maximum size of the riprap is to be 18" (Type I) instead of the default 24" Type I riprap. The riprap protection should be embedded flush to the original groundline and extended a minimum distance of 20 feet beyond the end of the wingwalls. A riprap apron with a width equal to twice the 100-year storm flow depth in the overbank area from a minimum width of 8 feet to a maximum of 25 feet should be used to protect the endroll toes. The riprap apron should not extend beyond the top of the stream channel bank. The depth of the riprap at the endrolls is normally 2 feet. The Department of Transportation uses two sizes of riprap: Type 1 riprap has a D50 of 1.14 feet and Type 3 riprap has a D50 of 0.64 feet. Woven plastic filter fabric is placed under the riprap (refer to GDOT Specification 881.2.05).

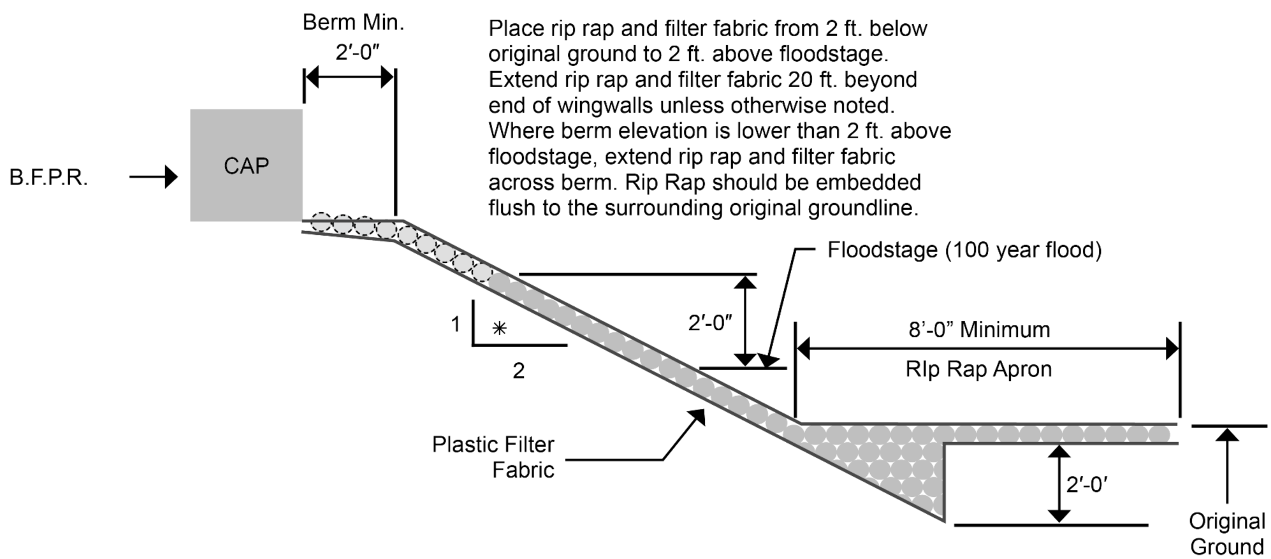

Figure 8.2 Illustration of Riprap at Bridge Abutment

Note: Type 1 riprap should be used at all locations. Type 3 riprap may be substituted where placement problems prevent the practical use of Type 1 riprap **AND** it is shown that Type 3 riprap is sufficient protection against scour damage during the 100-year flood.

8.3.4.2.2 Guide Banks

Guide bank calculations should be performed as shown in the latest version of the FHWA HEC-23, (8-5) *Bridge Scour and Stream Instability Countermeasures* and are based on the 100-year flood. Guide banks are not required to be built where the calculated length is less than 150 feet. Based on FHWA practice, GDOT recommends a maximum length guide bank of 150 feet be built where the calculated length is 150 feet or more. The design engineer, with concurrence from the GDOT bridge hydraulics engineer, may specify a longer or shorter guide bank be built or that no guide bank be built. The reasons supporting these options should be included in the hydraulic and hydrological report.

As a general rule, it is desirable to size new bridges so that guide banks will not be required. This can be accomplished by extending the new bridge to the wide side of the floodplain and/or the addition of overflow structures.

8.3.5 Detour Structures

The detour structure is sized to convey the 10-year storm and is recommended to be placed downstream of the proposed bridge site. **The detour bridge superstructure shall be designed according to Table 8.1**.

Detour structures in tidal areas are sized based on the high tide flow conditions.

8.3.6 Non-vehicular (Pedestrian) Bridge Structures

For non-vehicular (pedestrian) bridges, the designer should provide a copy of the hydraulic study along with the preliminary bridge plans to the Department addressing each of the following items. This study must be signed and stamped by the registered professional engineer who prepared the study.

- The hydraulic and hydrological study should meet the applicable guidelines and recommendations in this chapter.
- The study should include a theoretical scour analysis for the 100-year and 500-year flood frequency.
- For sites located within a regulatory floodway, the proposed bridge should be sized to obtain a no-rise certification. Coordination with the community is required and a copy of the community's letter of concurrence must be included in the study. If a no-rise certification cannot be obtained, coordination with the community and with FEMA is required. Include a copy of the community's letter of concurrence and a copy of the approved Conditional Letter of Map Revision (CLOMR) in the study.
- The backwater for the 100-year storm is not to exceed 1 foot above the natural conditions.
    1) The minimum design year storm for pedestrian bridges is the 25-year storm for drainage areas less than 100 square miles. For drainage areas greater than 100 square miles, the design storm is the 50-year storm.
    2) It is preferred to have 1 foot or more of clearance above the design storm. However, a minimum of one-half foot is required.

For a pedestrian bridge located adjacent to a highway bridge, the pedestrian bridge will be designed so that the hydraulic opening is equivalent to or larger than the adjacent highway bridge. In addition, any intermediate pedestrian bridge bents will be placed to line up with the existing intermediate highway bridge bents. If the pedestrian bridge meets this criterion and is not located within a regulatory floodway, a hydraulic and hydrologic study is not required. In all other cases, a study must be performed and submitted with the plans.

Pedestrian trails located entirely on boardwalks and having no fill within the limits of the floodplain are not considered bridges. A hydraulic study will not be required for these situations.

8.3.7 Longitudinal Roadway Encroachments

Since longitudinal encroachments into the base floodplain (100-year floodplain) and floodway by new and widened roadways have a major effect on the flood elevations of the affected stream, these encroachments should be avoided if possible. The project manager and location engineer should use the following basic rules for roadway widenings, parallelings, and new locations:

1. For roadway widening projects, the typical section should be set to avoid or minimize the placing of additional roadway fill within the adjacent base floodplain.
2. For roadway paralleling projects, the new parallel roadway should be placed to avoid or minimize longitudinal encroachments on the base floodplains.
3. New location roadway projects should be aligned to avoid or minimize longitudinal encroachments on base floodplains.
4. For all cases, longitudinal encroachment on a delineated FEMA regulatory floodway should be avoided and is often prohibited per local NFIP guidelines.

8.3.8 Walls Adjacent to Water

8.3.8.1 Walls Adjacent to Impounded Waters (Lakes and Ponds)

Locate retaining walls near impounded water such that the front edge of the foundation is offset a minimum of 12 feet from the normal pool shoreline or the ordinary highwater mark (OHWM). Detail the bottom of wall elevation so that it is a minimum of 2 feet above the normal pool elevation and embedded a minimum of 3 feet below the proposed ground line. The bottom of wall elevation is equal to the top of footing for cast-in-place walls or the maximum leveling pad elevation for Mechanically Stabilized Embankment (MSE) walls. Detail a 10 feet wide berm in front of the wall to give access for future maintenance of the wall. Coordinate with the Office of Bridge Design and Maintenance to develop an alternative if any of the minimums stated above are not possible.

For all retaining wall structures within 50 feet of impounded water that is at least 10 acres in size, the protection thereof is to be based on the consideration of the wave impacts of the impounded water and/or the water surface elevation of the 100-year flood. For impounded water that is less than 10 acres with no motorized boat traffic, wave impacts may be possible in the future during the life cycle of the wall if the impoundment size is increased. Coordination with the impounded water owner is recommended for any possible changes to future conditions and to ensure proper wall protection.

For adjacent impounded water where the upper wave limit or the 100-year flood is more than 25 ft away from the face of the wall, follow the guidance in the USDA NRCS's Technical Release 56, "A Guide for Design and Layout of Vegetative Wave Protection for Earthen Embankments and Shorelines." Where the upper wave limit or the 100-year flood is within 25 ft of the face of the wall, Detail Type-I rip rap and filter fabric in accordance with the figure 8.3 below. Where obtaining right of way is not feasible or additional encroachment into the impounded water is not practical, detail the rip rap and filter fabric to embed the minimum 12-foot offset. Coordinate with the impounded water owner to ensure proper slope protection is in place to prevent the undermining of wall rip rap beyond the limits of the right of way.

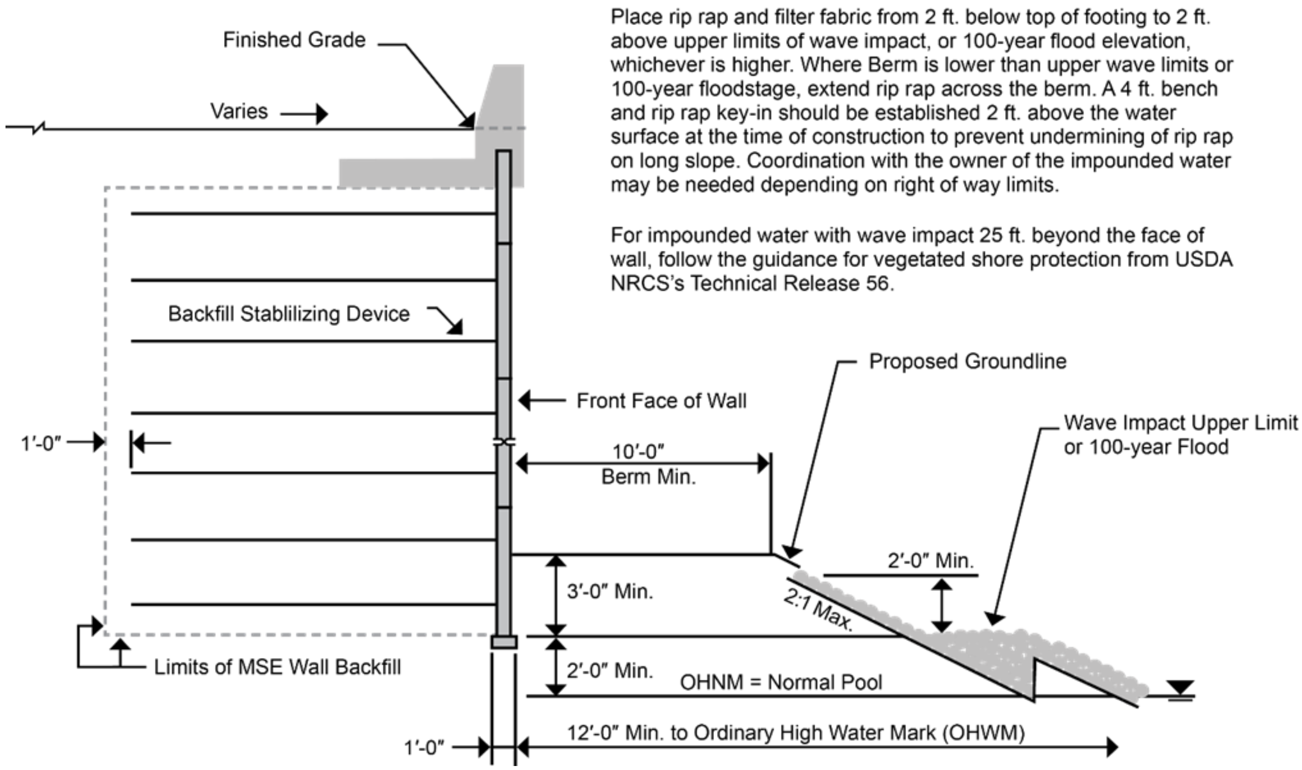

Figure 8.3 Section Thru Wall Adjacent to Water

8.3.8.2 Walls Adjacent to Streams

Locate retaining walls such that the front edge of the foundation is a minimum of 10 feet horizontally from the bankfull location for all perennial and intermittent streams. Coordinate with the Office of Bridge Design and Maintenance to develop an alternative if the minimum is not possible. For all retaining walls with a foundation located within 50 feet of the bankfull location of a perennial or intermittent streams, determine the theoretical scour depths caused by the 100-year flood condition.

Use the 500-year storm as check flood. Calculated scour effect from adjacent structures should also be taken into account when determining wall scour potential. Where the angle of attack between the approaching flow and the wall is greater than 25 degrees and the flow is not entering a culvert that passes through the wall, a 2D hydraulic model is required. See section 6.4.2.2 and 8.4.1.2 for more information on 2D flow analysis.

Calculate wall scour using a method similar to that presented in FHWA HRT-07-026 “Bottomless Culvert Study: Phase II Laboratory Report”. Detail an apron width equal to twice the predicted scour depth at the face of the wall or 5 feet, whichever is greater. Design rip rap in accordance with Hydraulic Engineering Circular (HEC) 23 “Bridge Scour and Stream Instability Countermeasures’. When placement of rip rap is not practicable, calculate scour in accordance with HEC-18 “Evaluating Scour at Bridges” (NCHRP abutment scour method).

Detail the bottom of cast in place footings below the 500-year scour depth and at least 2 feet below the 100-year scour depth, at the toe of the footing.

Detail the maximum leveling pad elevation for MSE walls below the 500-year scour depth and at least 3 feet below the 100-year scour depth at the wall face.

Detail the bottom of the cast in place concrete facing for soldier pile or tie back walls below the 500-year scour depth and at least 3 feet below the 100-year scour depth at the wall face.

When a culvert penetrates the wall, scour at the entrance, and exit of the culvert is calculated in accordance with Hydraulic Engineering Circular (HEC) 14 “Hydraulic Design of Energy Dissipators for Culverts and Channels”. Detail both ends of the culvert to allow for the installation of wingwalls to channel water into the culvert and away from retaining wall.

For ephemeral streams, locate retaining walls such that the front edge of the foundation is a minimum of 10 feet horizontally from the 2-year flow elevation. For drainage ditches, locate retaining walls such that the front edge of the foundation is a minimum of 5 feet horizontally from the 2-year flow elevation. Submit the channel lining and flow calculations to the Office of Design Policy and Support and the Office of Bridge Design and Maintenance for review if the minimum offset is not possible. Channel lining is calculated according to other GDOT policies.

8.4 Bridge Hydraulic Design Computations

The design for a stream crossing system requires a comprehensive engineering approach that includes the consideration of alternatives, data collection, and selection of the most cost-effective alternative according to established criteria and documentation of the final design. The hydraulic analysis of the stream crossing system requires developing an appropriate water surface profile model and using data from the hydraulic model to compute the required computations for the design of a bridge(s), e.g., scour computations, countermeasure computations. Flow through bridges may be computed using a one-dimensional or a two-dimensional model for hydraulic analysis. It is important for the hydraulic engineer to be aware of and understand the assumptions of different hydraulic models.

8.4.1 Bridge Hydraulic Modeling

The hydraulic analysis should be performed using a nationally recognized and readily available computer program to determine the impact of the proposed crossing on flood elevations, velocities, and flow distribution.

All numerical hydraulic models (1D and 2D) incorporate simplifying assumptions, require certain types of input data, and operate under specific implementation limitations. It is the goal of any hydraulic model study to simulate anticipated flow conditions as accurately as possible within project constraints without violating the assumptions and ignoring the limitations of the model. Therefore, a modeling approach should be selected based primarily on its advantages and limitations, though also considering the importance of the structure, potential interactions with the waterway, cost, and schedule. For this reason, 2D hydraulic modeling is recommended for the following hydraulic modeling situations:

- Wide floodplains
- Highly variable floodplain roughness
- Highly sinuous channels
- Multiple embankment openings

- Unmatched multiple openings in series
- Highly skewed roadway alignment (>30°)
- Bends and confluences with significant momentum shifts
- Angle of attack analyses
- Multiple channels
- Large tidal waterways and wind-influenced conditions
- Detailed flow distribution and complex multiple openings at bridge crossings
- Significant roadway overtopping
- Upstream controls
- Significant upstream storage
- Countermeasure design

The use of 2D hydraulic modeling in complex situations does not preclude use of these tools for simpler situations as project resources and context allow. See Section 8.4.1.3 for a list of models approved by GDOT.

8.4.1.1 One-Dimensional Flow Analysis

The hydraulic analysis for bridge(s) at natural channels or streams with steady, gradually varied flow that can be represented by a series of cross sections that are taken perpendicular to the assumed flow direction (one-dimensional flow) can be analyzed using HEC-RAS to complete the step-backwater analysis. One-dimensional models are best suited for in-channel flows, narrow to moderate floodplain widths, and when the degree of bridge constriction is small along the floodplain and the vegetation is not highly variable. All assumptions made in one-dimensional models are crucial such as cross section placement and orientation to ensure reasonable results.[8-1] See Chapter 5 – One-Dimensional Bridge Hydraulic Analysis in HDS-7[8-8] or the HEC-RAS Users Manual[8-7] for the proper procedures for setting up a HEC-RAS model for a bridge hydraulic crossing.

8.4.1.2 Two-Dimensional Flow Analysis

Two-dimensional models avoid many assumptions required by one-dimensional models and include a significant improvement in calculating hydraulic variables at bridges. Preference for the use of two-dimensional models over one-dimensional models for complex waterway and/or complex bridge hydraulic analyses should be considered. GDOT encourages the use of 2D models in appropriate situations consistent with staff expertise and project resources. See HDS-7[8-8] or the HEC-RAS Users Manual[8-7] for more information on criteria for selecting hydraulic models.

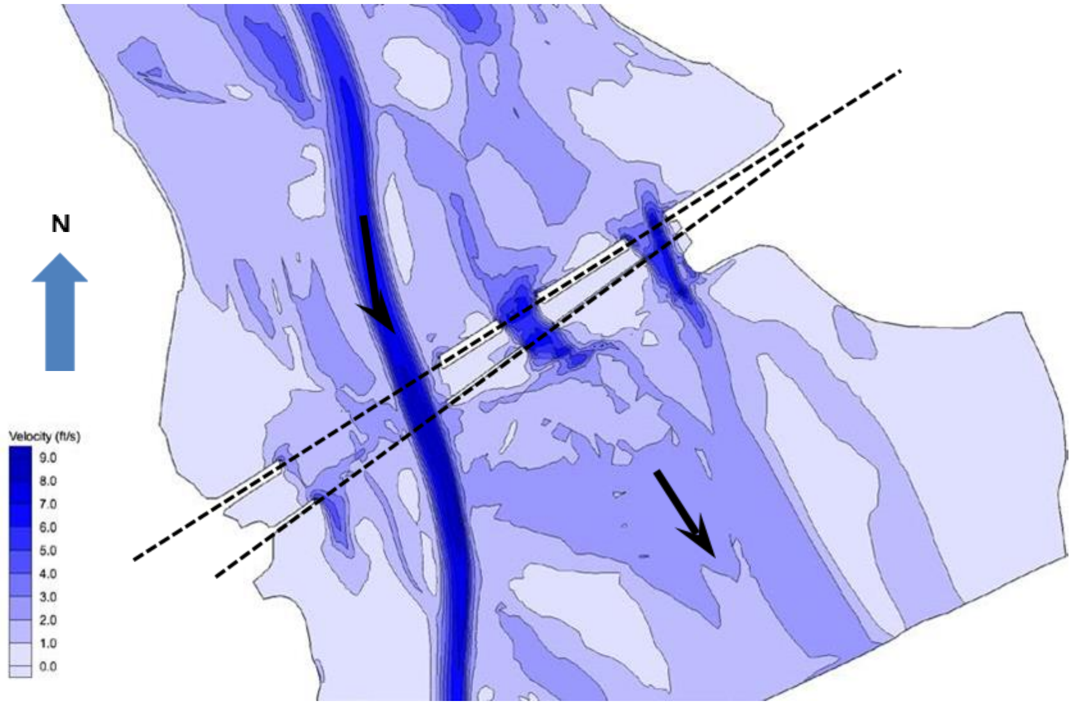

Figure 8.4 Two-dimensional Model Velocities for Wide Floodplain, Variable Vegetation, Multiple Openings (8-1)

Two-dimensional models generally provide more accurate representations of:

- Flow distribution
- Velocity distribution
- Water Surface Elevation
- Backwater
- Velocity magnitude
- Velocity direction
- Flow depth
- Shear stress (8-1)

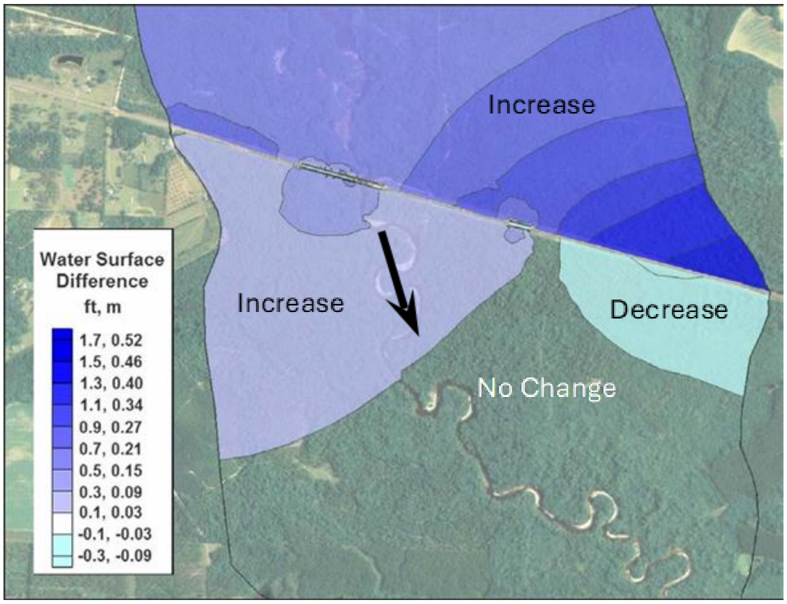

Figure 8.5 Two-dimensional Model Backwater at a Skewed Highway Crossing (8-1)

Figure 8.6 is an example of the complex channel and hydraulic conditions that occur more frequently in tidal waterways than in upland rivers. The figure depicts channels in blue, flood-prone areas in green, and roadway alignments in red. The area is subject to riverine and tidal flooding. Not only are there nine crossings (five on I-95 and four on SR 17), but there are more than 20 individual channel segments or reaches. Tidal waterways include inlets, estuaries, bays, and passages. Many bays and estuaries are crossed by causeways with multiple bridge openings and the potential for overtopping and wave attack. Many two-dimensional models include wind stress acting on the water surface as a boundary condition. Therefore, two-dimensional models need to be used for many coastal bridge hydraulic analyses.

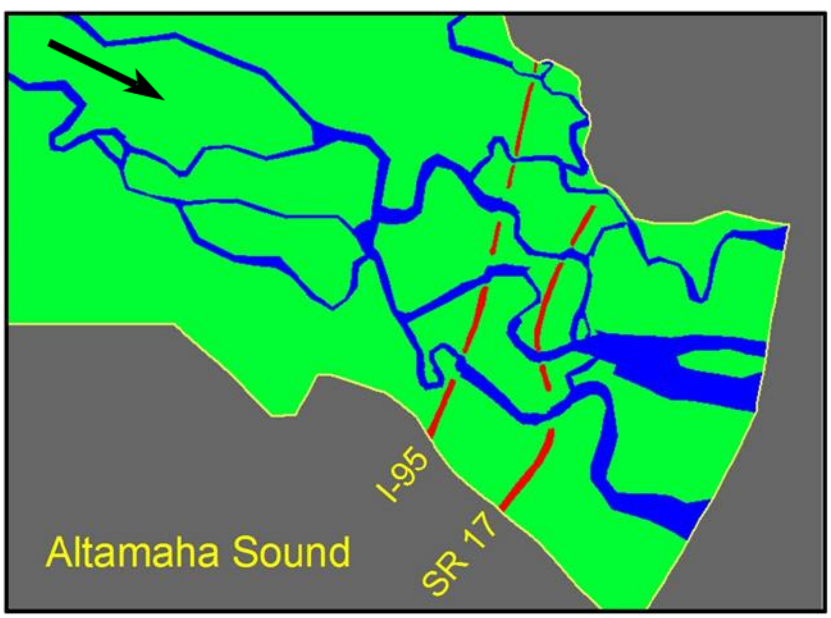

Figure 8.6 Channel Network at Altamaha Sound, Georgia (8-1)

8.4.1.3 GDOT Acceptable Computer Models

Two of the most common hydraulic models used for hydraulic bridge crossings include the following and are acceptable by GDOT:

1. Hydrologic Engineering Center's River Analysis System (HEC-RAS)[(8-7)], USACE
2. SRH-2D, Sedimentation and River Hydraulics – Two-Dimension, USBR, in conjunction with Surface-Water Modeling System (SMS), Aquaveo

For culvert crossings, the following computer models are acceptable by GDOT:

1. FHWA HY-8 Culvert Hydraulic Analysis Program
2. HEC-RAS utilizing culvert routine

For projects that involve in-channel flows, narrow to moderate floodplain widths, and when the degree of bridge constriction is small along the floodplain and the vegetation is not highly variable, HEC-RAS computer models can be used to size and analyze these drainage structures at crossings.

If the project is within a FEMA regulatory floodway, FEMA regulations must also be satisfied. For regulatory FEMA hydraulic models produced from the USACE software HEC-2, HEC-RAS will be used to duplicate the current regulatory FEMA hydraulic model from HEC-2 to produce the floodway and profile runs. Other models for bridge hydraulics may be acceptable with approval of GDOT Hydraulic Engineer including, but not limited to, HEC-RAS 2-D, TUFLOW, RMA2, FESWMS and ADCIRC.

HEC-25 should be referenced to determine if a bridge is located in a tidal area and requires hydrodynamic modeling. See Figure 8.7 from HEC-25 showing when and where to apply riverine hydraulics and coastal hydrodynamic models as a function of distance from the coast and importance of infrastructure. [(8-3)]

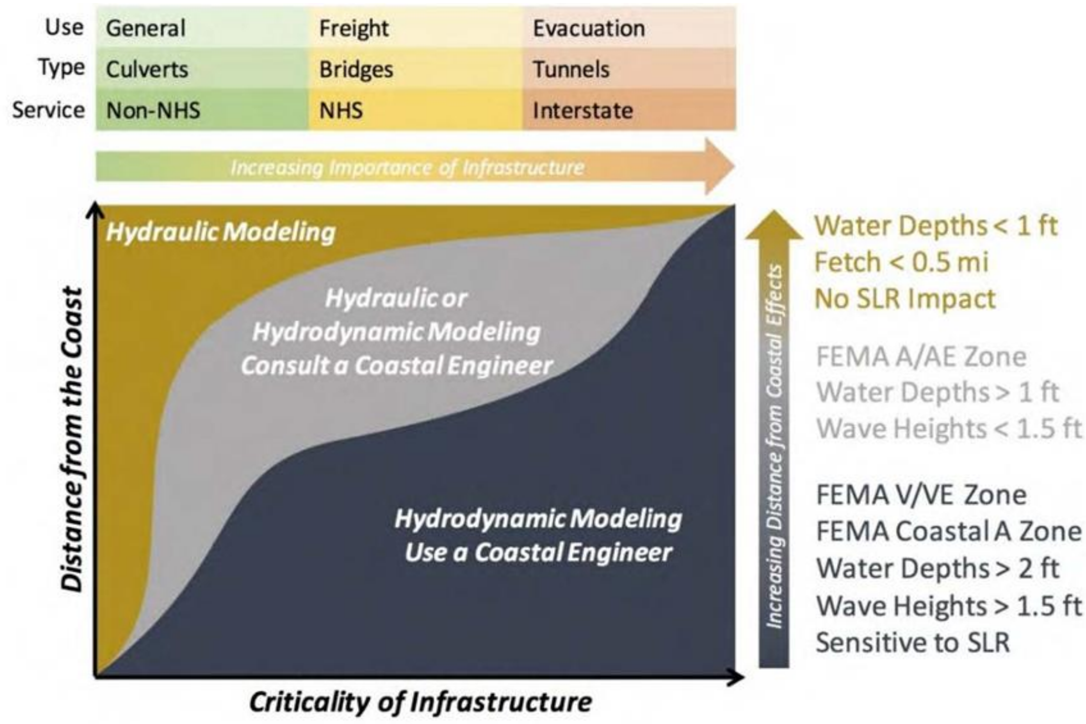

Figure 8.7 Function of a Site's Coastal Proximity and Criticality (8-3)

The closer a site is to the coast (and by extension, coastal hazards such as storm surge and wave attack) and the more important the infrastructure or roadway is, the more likely coastal hydrodynamic modeling will be needed in analysis and design.

The following hydraulic computer models are approved by GDOT to be used when tidal flow is present:

1. The HEC-RAS computer model. This USACE computer model incorporates the UNET program for unsteady flow analysis
2. The SRH-2D computer model. These two-dimensional models are presently recommended by the FHWA for tidal sites where complicated hydraulics exist. These models should be used in cases where there is a large amount of two-dimensional flow
3. ADCIRC
4. SLOSH
5. MIKE-21

8.4.2 Flow Habit Assumptions

Hydraulic Modeling Floodplain Constrictions/Obstructions and Abnormal Flood Conditions

Effects from natural or man-made conditions upstream or downstream may affect the water surface elevations at the crossing site. These effects must be considered when modeling and analyzing the hydraulic conditions at the project site. The hydraulic engineer must identify and include any of these conditions that will affect the project site in the hydraulic model. Following are some examples:

- Roadway and railroad stream crossings

- Longitudinal roadway encroachments,
- Natural narrowing of the floodplain
- Fill that has been placed within the floodplain
- Reservoirs, dams, and levee structures
- Confluence with another stream (all above items must be considered when modeling this stream)

8.4.2.1 Normal Water Surface Profile Run

The normal water surface profile run includes any floodplain constriction or obstruction that controls or affects the water surface elevations at the project site with the normal storm flows along the stream reach. This computer run is the basic run in all hydraulic studies.

8.4.2.2 Abnormal Water Surface Profile Run

The abnormal water surface profile run includes any backwater effects from a natural or man-made condition that causes a difference in water surface elevations at the project crossing compared to the normal storm flows along the stream reach. For example, an abnormal water surface elevation may result when the studied stream is a tributary to another river or stream, and the flood flows along this river or stream cause water surface elevations to rise at the project site.

A reservoir that affects the water surface elevations at the project site can be considered a normal or abnormal water surface elevation condition depending on the situation. If the water surface elevations at the project site are controlled at all times by the reservoir, this is considered a normal flood condition. If the water surface elevations at the project site are only controlled a portion of the time by the reservoir, then this would be considered an abnormal water surface elevation condition.

If the proposed bridge site is affected by abnormal water surface elevations that result in higher water surface elevations and lower flood flow velocities than a normal flood condition, the bridge is to be designed to provide the required freeboard above the abnormal water surface elevations. In addition, the bridge is also to be designed for the higher storm flow velocities that occur without the effects of the abnormal water surface elevations.

8.4.2.3 Coincidental Occurrence

If the drainage area of the stream contributing to the abnormal water surface elevation differs significantly from the drainage area at the project crossing, consider the use of coincidental occurrence flood frequency. Coincidental occurrence incorporates the varying amount of time required for different size drainage areas to reach peak flow by pairing the project flood flows with higher recurrence flood flows at the stream contributing to the abnormal water surface elevations.

Table 8.4 – Coincidental Occurrences Frequency Combinations			
Area Ratio	**Design Storm Frequency**		
Receiving Storm Area to Storm Drain Area	**25-Year**	**50-Year**	**100-Year**
1,000 : 1	5	5	10
100 : 1	10	10	25
10 : 1	10	25	50
1 :1	25	50	100

8.4.3 Selecting Upstream and Downstream Model Extents

Hydraulic models for bridge hydraulics should extend beyond where the flow is fully expanded both upstream and downstream of the flow constriction (example: proposed project). Flow constriction is often the major contributor to backwater, so complete flow expansion and contraction must be included. For one-dimensional models, the use of the minimum downstream extent does not detract from the results as long as the downstream water surface is known with a high degree of certainty. However, if the water surface is not known with confidence, then extending the model further downstream will decrease uncertainty at the structure. This is illustrated in Figure 8.8, which shows water surface profiles for a simple bridge model. The three profiles are all for the same discharge with the only difference being the downstream boundary condition. Each one of the profiles represents a valid solution to the equations of fluid motion. The downstream boundary is located far enough downstream, so the profiles converge, and the initial differences are eliminated before reaching the bridge.

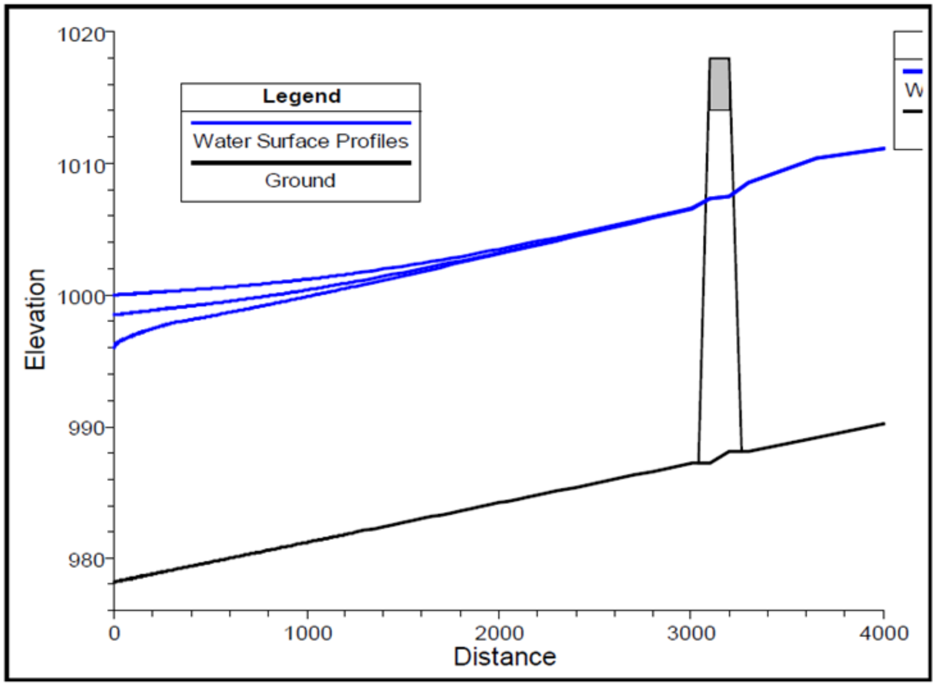

Figure 8.8 Flow profiles with downstream boundary uncertainty (FHWA HDS-7[8-8])

Bridge hydraulic models should extend far enough downstream to minimize the boundary condition influence will have on the location of interest. The further the boundary is from the bridge, the less uncertainty exists at the bridge because channel and floodplain geometry and roughness will dictate the results. A sensitivity analysis may be used to confirm the location of the downstream boundary condition. A common approach to this is to analyze starting water elevations 3 ft above and below the chosen target to ensure that elevations at the crossing are not impacted.

The minimum downstream extent for two-dimensional models is similar to one-dimensional models with flow fully expanded upstream and downstream. It is also desirable to select a location where flow is reasonably one-dimensional, especially at the downstream boundary. This is because the downstream boundary is usually specified as a constant water surface elevation along the boundary. One useful approach is to place the upstream and downstream boundaries at least one floodplain width up- and downstream of the crossing. As with one-dimensional models, the further the boundary is located away from the crossing the less influence the boundary condition will exert of the results.8.8

8.4.4 Hydraulic Modeling Calibration

When a USGS gage is located at or near the bridge site, water surface elevations to calibrate the computer model are to be obtained from the USGS regional office in Atlanta. If reliable highwater information at or near the site is available and the flood frequency of the applicable storm can be determined, the computer model should be calibrated using this information. For more information on calibrating hydraulic models please see applicable FHWA guidance on calibrating one-dimensional and two-dimensional hydraulic models.[8-8]

8.4.5 Scour Analysis

After the bridge waterway opening has been established, the hydraulic designer must evaluate scour and stream instability at the new bridge. A scour analysis will be performed for all bridges using the methods provided in the HEC-18(8-2) and HEC-20(8-6) manuals to evaluate scour and stream instability. Safe bridge design must account for scour conditions that may occur over the life of the bridge. Scour is greatest during flood events when flow velocity and depth is highest, but the event-related scour is in addition to the long-term stream instability components of channel shifting, aggradation, and degradation.

According to AASHTO Drainage Manual (ADM) (8-9), the scour evaluation for proposed new or replacement bridges must include the following:

- Estimate degradation or aggradation of the river, contraction scour, and local scour. Position the foundation to an appropriate depth below the total scour depth, if practicable. Include these estimates as part of the final design.
- Design pier spacing, shape, and orientation. Design abutment alignment and shape. Designs should minimize flow disruption and potential scour.
- Design pier foundations to avoid failure due to scour without the aid of scour countermeasures.
- Design abutment foundations to avoid failure due to scour with proper design of abutment countermeasures.

Furthermore, ADM states that for most new bridges, pier scour will be accommodated by adjusting the pier design in cooperation with the geotechnical and structural engineers, and abutment scour will be mitigated with countermeasures. However, the most cost-effective design may be to modify the opening to reduce the amount of scour or the cost of the scour countermeasures. Considerable judgment will be necessary to make this determination.

Note: The theoretical scour depths for the proposed bridge(s) are normally performed without the benefit of a bridge foundation investigation for the proposed structure. The median grain diameters (D50) of sand that are normally used by GDOT in the FHWA scour equations to estimate the theoretical scour depths are as follows:

Very Coarse Sand:	D_{50} = 0.00492 ft
Coarse Sand:	D_{50} = 0.00246 ft
Medium Sand:	D_{50} = 0.00123 ft
Fine Sand:	D_{50} = 0.00062 ft
Very Fine Sand:	D_{50} = 0.00031 ft

The hydraulic engineer should search the existing bridge files for old bridge foundation investigations or other information that would assist in deciding which D50 would be appropriate for the site. Soil information at nearby crossings along the same stream can also be helpful in this regard.

The predicted scour depths at each intermediate bent of the proposed bridge should be provided to the Office of Materials Soils Lab. The soils engineer should adjust, if necessary, the predicted scour depths depending on the soil conditions at the site. As part of the bridge foundation investigation, the soils engineer should provide the final predicted scour depths to the bridge structural engineer for inclusion in the analysis and design of the bridge foundations.

8.4.6 Stream Instability Countermeasures Design and Analysis

A hydraulic analysis is essential for selecting, sizing, and siting countermeasures that will achieve the desired level of stability. Once the appropriate levels of hydraulic analysis and assessments are completed using the methods provided in the HEC-18(8-2) and HEC-20 manuals(8-6) to evaluate scour and stream instability, countermeasures should be designed as appropriate using the methods provided in the HEC-23 manual (8-5). To provide an acceptable level of service, use established design frequency standards that are based on the classification of the highway facility, the allowable risk for that facility, and the appropriate drainage structure under design. (8-5) For design procedures of countermeasures please see section 8.3.4 of this manual.

8.5 General Design Considerations and Selection of Bridge Types

The hydraulic analysis should consider various stream crossing system designs to determine the most cost-effective proposal consistent with hydraulic design constraints established in this manual and in accordance with the GDOT Bridge and Structures Design Manual. Bridge hydraulic design can affect the Conceptual Design Phase of bridges and is integral in the Preliminary Design Phase because the template for the Final Bridge Plans is set. Therefore, it is critical for the designer to consider all the known aspects that may affect the bridge design in these stages, see Section 2.9 *Preliminary Design in the GDOT Bridge and Structures Design Manual*. Roadway geometry, skew angles, bridge widths, bridge lengths, and bridge types (superstructure and substructure) are all factors of the Preliminary Design. Also, see AASHTO LRFD Bridge Design Specifications (see the GDOT Bridge Manual for the current edition) for LRFD design reference.

The following should be considered in bridge hydraulic design(8.10):

- Evaluation of bridge design alternatives should consider stream instability, backwater, flow distribution, stream velocities, scour potential, flood hazards, tidal dynamics (where appropriate) and consistency with established criteria for the National Flood Insurance Program.
- Studies should be carried out to evaluate the stability of the waterway and to assess the impact of construction on the waterway.
- Consider whether the stream reach is degrading, aggrading, or in equilibrium.
- Consider the effect of natural geomorphic stream pattern changes on the proposed structure.
- For unstable streams or flow conditions, special studies should be carried out to assess the probable future changes to the plan form and profile of the stream and to determine countermeasures to be incorporated in the design, or at a future time, for the safety of the bridge and approach roadways.
- For the scour design flood, the streambed material in the scour prism above the total scour line should be assumed to have been removed for design conditions.
- Locate abutments back from the channel banks where significant problems with ice/debris buildup, scour, or channel stability are anticipated.
- Design piers on floodplains as river piers. Locate their foundations at the appropriate depth if there is a likelihood that the stream channel will shift during the life of the structure or that channel cutoffs are likely to occur.
- The added cost of making a bridge less vulnerable to damage from scour is small in comparison to the total cost of a bridge failure.

8.5.1 Bridge Opening and Road Grade Design Considerations

Hydraulic capacity and freeboard should be a primary consideration in setting the bridge length and height. The bridge must provide enough capacity to:

- Avoid excessive backwater in order to prevent adverse floodplain impacts
- Prevent excessive velocity and shear stress within the bridge waterway

The flow (or conveyance) distribution should be utilized along with the channel bank limits and other significant physical features to establish optimal pier and abutment locations. Optimal locations can generally be modified somewhat in a give-and-take manner with structural and geotechnical engineers to address cost and non-hydraulic design constraints. Prime examples of the latter are span length limitations and scour/foundation treatments. [8.15]

There are several potential bridge opening and road grade considerations that impact hydraulic capacity and upstream flood risk, especially when a road is improved, and the bridge is replaced. These include bridge length, deck width, abutment configuration (spill through or vertical wall), number and size of piers, low chord elevation, freeboard, and road grade. If a crossing with a 25-year level of service is improved to a 50-year level of service, the road elevation may need to be increased. To avoid increased flood risk, the replacement bridge may need to be considerably longer and higher than the existing bridge. If there is inadequate freeboard, debris may collect along the deck and reduce flow conveyance.[8.14]

8.5.2 Guidelines for Selecting Bridge Types

Selection of bridge beams and bridge bents as applied to span lengths should be based on sound engineering design while meeting all applicable design requirements, while maximizing the efficiency and minimizing the costs for the bridge. Cost estimates should be performed for design alternatives and coordination with the Bridge Office or other knowledgeable parties is recommended to obtain current typical bridge costs since the cost of steel can fluctuate.

For ease of structural design and repetition in fabrication, the use of equal span lengths is recommended while following sound hydraulic design practices. For designing typical bridge bents, see Section 2.9 of the Bridge Design Manual.

8.6 Chapter 8 References

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Chapter 9. Bridge Deck Drainage

9.1 Introduction

The information provided in this chapter applies to all bridges. The bridge deck drainage system includes the bridge deck, sidewalks, railings/parapets, gutters, and closed conveyance system. Bridge deck drainage is similar to that for a curbed roadway section, but it can be less efficient because bridge deck cross slopes can be flatter, parapets collect large amounts of debris, and relatively small deck drains have a greater potential for clogging due to debris.

Roadway features considered in bridge deck drainage, gutter flow, and capacity calculations include:

- Longitudinal grade and cross slope
- Pavement width
- Shoulder sections
- Sidewalks
- Buffer Zones
- Barriers leading up to the bridge
- Pavement texture and/or surface roughness
- Drainage on bridge approach slabs

9.2 Gutter Spread Standard Criteria

Gutter spread on bridges is determined using 10-year design storm frequency and shall be limited based on the design speed of the road. For design speeds greater than 45 mph, gutter spread shall be limited to the shoulder area. For design speeds equal to 45 mph or lower, gutter spread shall be limited to the shoulder area and a portion of the outermost travel lane so long as at least 10 feet of the outermost lane is outside of the gutter spread. A decision not to meet the criteria above shall require Design Variance approval from the GDOT Chief Engineer. See GDOT Standard Drainage Design Criteria for reference.

There are no restrictions for gutter spread encroaching into turn lanes on bridges.

9.3 Bridge Gutter Spread Geometry Considerations

Two of the main components that influence gutter flow are the longitudinal and transverse (cross) slopes of the pavement.

9.3.1 Longitudinal Slopes/Grades

0% grades and sag vertical curves requires coordination with Hydraulics Group of the Office of Bridge Design. Closed conveyance systems are unsightly and can interfere with bridge beams. For these reasons, ideally, the longitudinal slope of the bridge should be steep enough to satisfy the gutter-spread requirements without the need for deck drains or a closed conveyance system on the structure. The minimum longitudinal grade for bridge deck drainage should be 0.5%. For long bridges, setting the proposed profile in a crest vertical curve with the high point occurring in the center of the bridge may improve drainage efficiency by reducing the contributing bridge deck drainage area and design flows to each end of the bridge. This vertical geometry reduces the need for deck drains or a closed conveyance system (see Figure 9.1). Any design constraints that lead to the gutter spread requirements not being met will require a Design Variance.

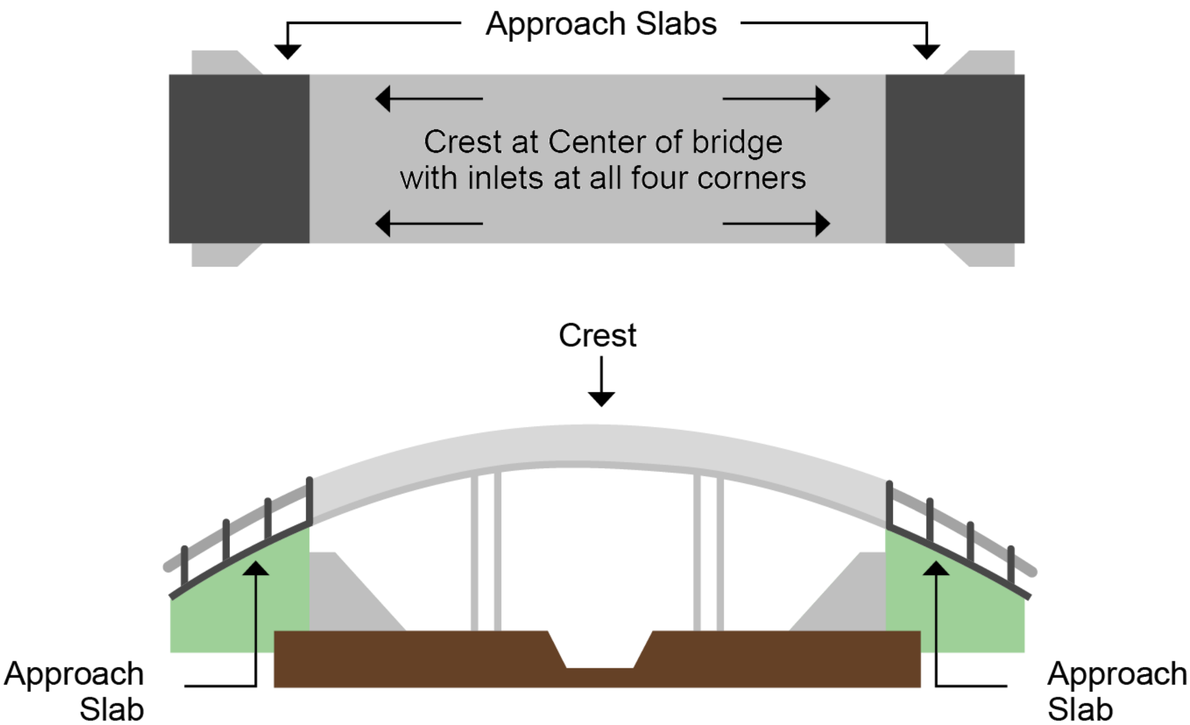

Figure 9.1 – Optimal vertical geometry for deck drainage.

9.3.2 Cross Slopes

Superelevation transitions should be avoided on bridges. Adequate cross slope should be provided so that water flows quickly toward the gutter. The minimum cross slope for bridges with a normal crown is 2%. In the coastal region of Georgia, where intense rainfall is more frequent, a steeper minimum cross slope of 2.5% is recommended. Where three or more travel lanes are provided in each direction, the maximum cross slope is 3%.

9.4 Gutter Flow Computations

Refer to FHWA Hydraulic Engineering Circular No. 21 (HEC-21), Design of Bridge Deck Drainage, May 1993, (9-2) for gutter flow and spread computations.

The Rational Method is typically used for inlet spacing design. Refer to Chapter 3 of this manual. A minimum time of concentration of 5 minutes should be used to estimate the rainfall intensity rate.

Rainfall intensity is calculated using intensity-duration-frequency (IDF) curves and methods consistent with the GDOT drainage manual. Point precipitation frequency estimate are taken from NOAA Atlas 14 Volume (for partial duration time series.

9.5 Bridge Deck Drainage Systems Criteria

When designing deck drainage systems, guidance from Chapter 3.15 of the GDOT Bridge and Structures Design Manual (BDM) should be followed. The following three figures shown below are the typical GDOT deck drain details used for most general design applications. Please coordinate with the GDOT bridge hydraulics engineer for guidance on other types of specific deck drain design applications.

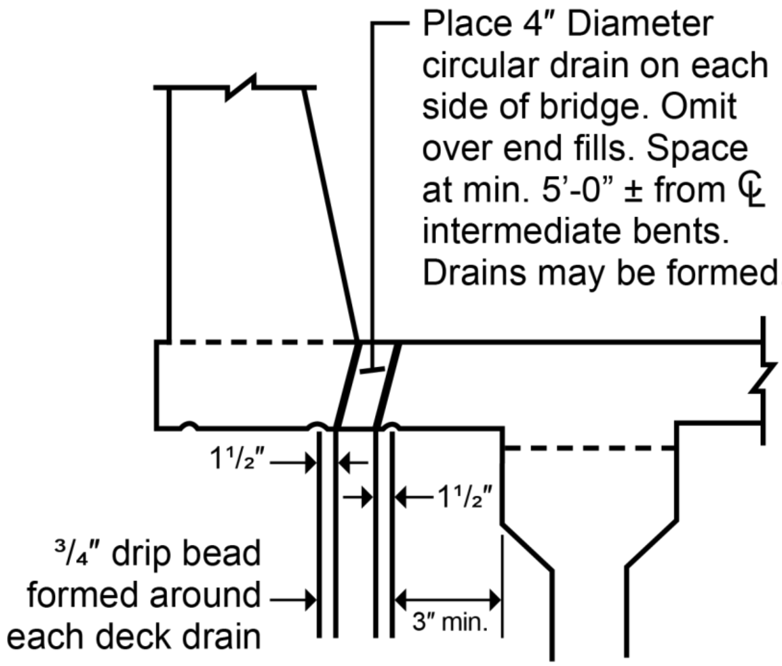

Figure 9.2 - Circular drain detail at barrier

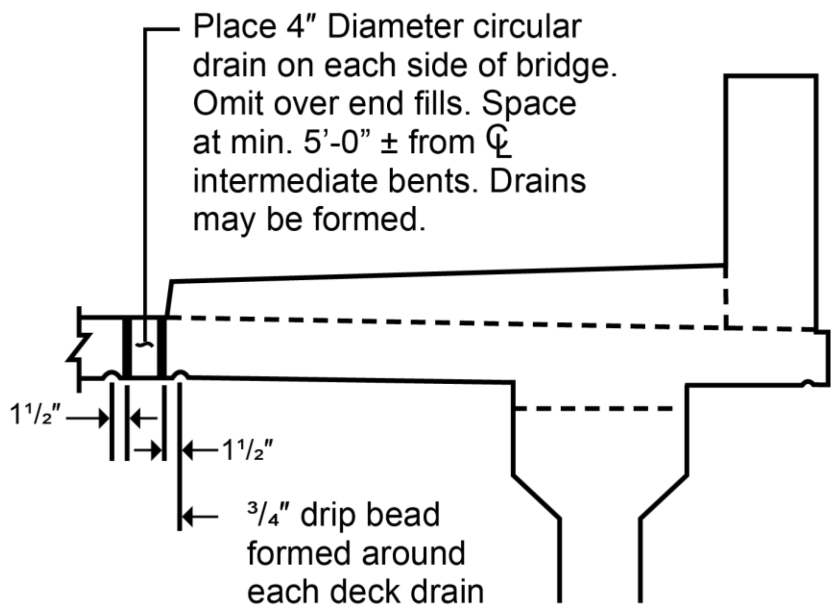

Figure 9.3 - Circular drain detail at sidewalk

If factors such as the location of the bridge beam prevent the standard circular drains from being used, a barrier slot can typically be used in cases without a sidewalk.

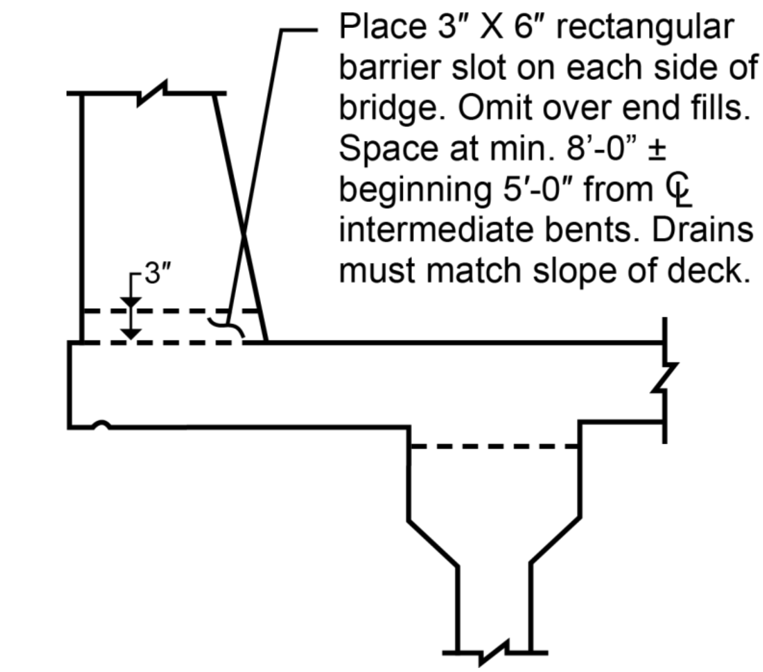

Figure 9.4 - Barrier Slot detail

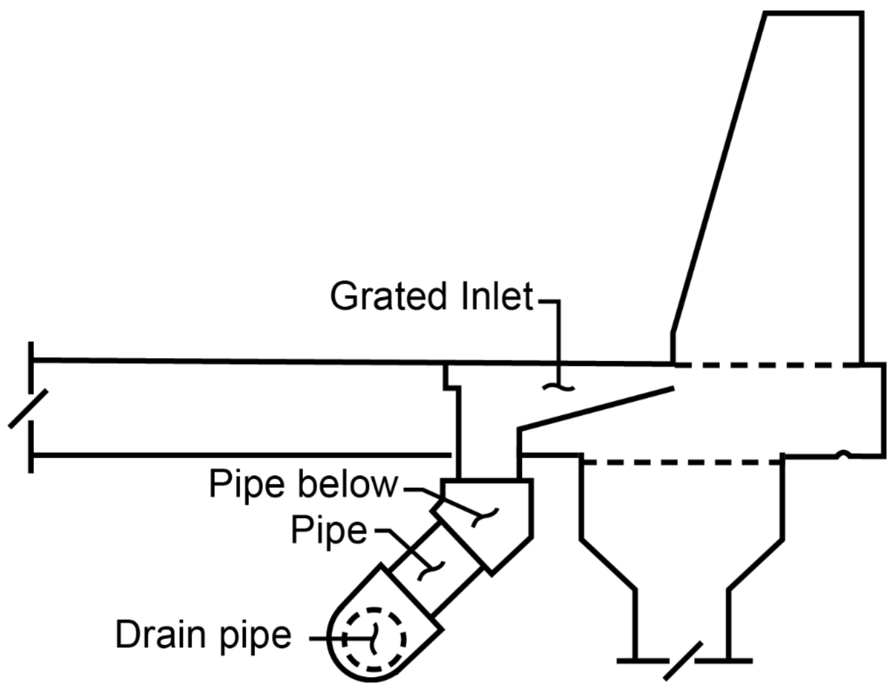

Figure 9.5 – Closed conveyance deck drain system

The following criteria is used in the placement of the above standard and alternate deck drainage systems:

- The circular drains and barrier slots are to be omitted over bridge endrolls and are not for use on grade separation bridges over roadway travel lanes and railroad tracks.
- The circular drain and barrier slot spacing are to begin 5 feet from the centerline of the intermediate bents. No circular drains or barrier slots are to be less than 5 feet from the centerline of intermediate bents. Refer to the GDOT Bridge and Structures Design Manual for additional information.
- If the need for deck drains cannot be eliminated from the bridge due to excessive structure length or width, superelevation, or narrow shoulders, and if there are environmental restraints that would prevent a direct discharge into the stream, a closed drainage system will be required.
- A shoulder is to be designed according to Section 2.9.2 of the GDOT Bridge & Structures Design Manual.
- A closed drainage system should consist of vertical drains with steel grated inlets or openings on the bridge deck with polyvinyl chloride (PVC) pipe to transport the water to a collector via a PVC pipe.
- The slope of the collector pipe should be dictated by the minimum flushing velocity of 3 ft/s.
- The bridge hydraulics engineer will determine the size and spacing of the grated inlets and associated closed drainage system and will work with the structural engineer to maintain structural integrity for the design. A typical closed conveyance system is shown in Figure 9.5.
- The design of the closed system pipe is to follow the same criteria outlined in Table 5.1 of Chapter 5.

9.5.1 Bridge Deck Drainage Design

GDOT bridge on deck drainage design follows the guidance provided in FHWA Hydraulic Engineering Circular No. 21 (HEC-21), Design of Bridge Deck Drainage, May 1993. GDOT provides the Deck Drainage Spreadsheet following principles outlined in HEC-21 as a guidance engineers may use for calculations. The spreadsheet can be found within the Bridge Software > Bridge Resources Tab under Bridge Hydraulics Resources.

9.5.2 Underdeck Collection and Discharge

Refer to FHWA Hydraulic Engineering Circular No. 21 (HEC-21), Design of Bridge Deck Drainage, May 1993, (9-2) for collection pipe sizing computations.

- Cleanouts should be provided near the bent cap, at ground level, and anywhere there is a change in direction for inspection and maintenance.
- The collection pipe is typically connected to the catch basin located just off of the end of the bridge. This section of pipe will be located under the approach slab and is shown on the roadway plans. This interface between bridge deck drainage and the roadway drainage system should be coordinated by the bridge and roadway drainage engineers.
- Closed system outlet pipes should be located on the opposite side of the bridge columns from the oncoming traffic for aesthetics and to reduce the risk of damage.
- The outlet pipe discharge should discharge freely into the receiving channel or storm drain. See storm drain outlet protection criteria provided in Chapter 5.

9.5.3 Bridge End Drainage Inlets

Because of the difficulties in providing and maintaining an adequate deck drainage system, gutter flow from the roadway should be intercepted before it reaches a bridge.

The roadway engineer is to provide drainage inlets to collect the gutter flow from the upslope roadway before it reaches the bridge deck. The roadway designer is to place inlets to collect runoff from the bridge deck immediately after it flows onto the subsequent roadway section (see chapter 6 of this manual).

9.5.4 Temporary Bridge Deck Drainage

Temporary conditions such as a bridge under construction over the Interstate or other high ADT facilities should be evaluated for deck drainage. Temporary conditions can be analyzed for a 2-year 10-minute duration with an intensity of no less than 3 in/hr. Temporary drainage directly over roadways, railroads or highly erodible areas should be redirected through the use of sandbags or other means of flow redirection.

9.6 Information Needed for Design

Below is a list of the basic items needed for bridge deck drainage design:

- Preliminary proposed roadway plans
- Preliminary proposed bridge layout
- FHWA HEC-21
- Catalog of suppliers on List 11 of the GDOT Qualified Products List, “Foundries Supplying Gray Iron Drainage Castings." (9-1)

The design of bridge deck drainage systems requires early coordination with multiple disciplines. In particular, the bridge hydraulic engineer should consider the environmental and final bridge design requirements and limitations, especially bridge width. Failure to do so may result in extensive rework and schedule delays.

9.6.1 Environmental Considerations

- Deck drainage runoff is routed to closed drainage system if the bridge is located within environmentally sensitive areas. Refer to GDOT’s Stormwater Controls Design Manual regarding the use of deck drains and discharge guidelines for bridges located within an MS4 designated areas and for locating MS4 boundaries in Georgia.
- The bridge hydraulics engineer should coordinate with the environmental staff to confirm where direct discharge is acceptable.

9.6.2 Bridge Design Considerations

- Some deck drains may require thickening the deck and/or lowering the low chord, which may impact the hydraulic freeboard.
- The bridge diaphragms, endwalls and edge beams must accommodate the slope of the collection pipes. See Bridge and Structures Manual Table 3.14.1.6-1 for maximum pipe sizes based on beam size.
- It is not acceptable for the collector pipes for a drainage system to hang below the beam low chord.

- In the case of skewed/curved bridges, deck drainage calculations should be computed along the gutter centerline to reflect an accurate drain position along the longitudinal slope.

9.7 Best Practices

Design Methods and procedures follow:

- The bridge hydraulics engineer should coordinate with the bridge structural engineer to ensure the selected deck drain is compatible with the beam spacing.
- The bridge hydraulics engineer should be aware that spread calculations for the flat areas immediately surrounding the crest of a vertical curve on a bridge may require additional analysis to determine an accurate gutter spread.
- The deck drains should be spaced at 10-foot intervals along both sides of the bridge for normal crowns, and along the low side of the bridge for decks with constant cross slopes and superelevation.
- For bridges where standard deck drains are not allowed but supporting calculations indicate that inlets are required on the bridge, coordinate with the roadway engineer to adjust the grade or cross slope to eliminate the need for inlets on the bridge.
- The bridge hydraulics engineer should confirm the validity of drain spacing near any vertical curve crest. This can be done by applying flat grade calculations where longitudinal slopes are less than 0.3% approaching the crest.
- Special consideration should be given to drain spacing on structures with reverse horizontal curves occurring on the bridge. Sufficient drain openings are provided to minimize “cross flow” into travel lanes at superelevation transition areas as much as practical.

9.8 Chapter 9 References

1. Georgia Department of Transportation (GDOT). Qualified Products List.
2. Georgia Department of Transportation (GDOT), Bridge and Structures Design Manual.

Appendix A. Manning's Roughness Coefficient Tables

Manning's Roughness Coefficient (n) for Overland Sheet Flow

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
Residue cover > 20%	0.17
Range (natural)	0.13
Grass	
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Woods*	
Light underbrush	0.40
Dense underbrush	0.80
*When selecting n, consider cover to a height of about 1 inch. This is only part of the plant cover that will obstruct sheet flow.	

Manning's Roughness Coefficient (n) for Various Boundaries

Rigid Boundary Channels	Manning's n
Very smooth concrete and planed timber	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Wood	0.014
Vitrified clay	0.015
Shot concrete, untroweled, and earth channels in best condition	0.017
Straight unlined earth canals in good condition	0.020
Mountain streams with rocky beds	0.040-0.050
MINOR STREAMS (top width at flood stage < 100 ft)	
Streams on Plain	
1. Clean, straight, full stage, no rifts, or deep pools	0.025-0.033
2. Same as above, but more stones and weeds	0.030-0.040
3. Clean, winding, some pools, and shoals	0.033-0.045
4. Same as above, but some weeds and stones	0.035-0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040-0.055
6. Same as 4, but more stones	0.045-0.060
7. Sluggish reaches, weedy, deep pools	0.050-0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075-0.150
Mountain Streams, no Vegetation in Channel, Banks Usually Steep, Trees and Brush Along Banks Submerged at High Stages	
1. Bottom: gavels, cobbles, and few boulders	0.030-0.050
2. Bottom: cobbles with large boulders	0.040-0.070
Floodplains	
Pasture, No Brush	
1. Short Grass	0.025-0.035
2. High Grass	0.030-0.050
Cultivated Areas	
1. No Crop	0.020-0.040
2. Mature Row Crops	0.025-0.045
3. Mature Field Crops	0.030-0.050
Brush	
1. Scattered brush, heavy weeds	0.035-0.070
2. Light brush and trees in winter	0.035-0.060
3. Light brush and trees in summer	0.040-0.080
4. Medium to dense brush in winter	0.045-0.110
5. Medium to dense brush in summer	0.070-0.160
Trees	
1. Dense willows, summer, straight	0.110-0.200
2. Cleared land with tree stumps, no sprouts	0.030-0.050
3. Same as above, but with heavy growth of sprouts	0.050-0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080-0.120
5. Same as above, but with flood stage reaching branches	0.100-0.160

Manning's Roughness Coefficients for Various Boundaries (continued)	
Rigid Boundary Channels	**Manning's n**
MAJOR STREAMS (Top width at flood stage > 100 ft)	
Regular section with no boulders or brush Irregular and rough section	0.025-0.060 0.035-0.100
The n value is less than that for minor streams of similar description, because banks offer less effective resistance.	
Alluvial Sand-bed Channels (no vegetation)	
Tranquil flow, Fr < 1 Plane bed Ripples Dunes Washed out dunes or transition Plane bed	 0.014-0.020 0.018-0.030 0.020-0.040 0.014-0.025 0.010-0.013
Rapid Flow, Fr > 1 Standing waves Antidunes	 0.010-0.015 0.012-0.020
Overland Flow and Sheet Flow	
Smooth asphalt Smooth concrete Cement rubble surface Natural range Dense grass Bermuda grass Light underbrush Heavy underbrush	0.011 0.012 0.024 0.13 0.24 0.41 0.40 0.80

Values of Manning's Roughness Coefficient (n) (Uniform Flow)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
Earth, straight and uniform			
Clean, recently completed	0.016	0.018	0.020
Clean, after weathering	0.018	0.022	0.025
Gravel, uniform section, clean	0.022	0.025	0.030
With short grass, few weeds	0.022	0.027	0.033
Earth, winding and sluggish			
No vegetation	0.023	0.025	0.030
Grass, some weeds	0.025	0.030	0.033
Dense weeds or aquatic plans in deep channels	0.030	0.035	0.040
Earth bottom and rubble sides	0.025	0.030	0.035
Stony bottom and weedy sides	0.025	0.035	0.045
Cobble bottom and clean sides	0.030	0.040	0.050
Dragline-excavated or dredged			
No vegetation	0.025	0.028	0.033
Light brush on banks	0.035	0.050	0.060
Rock cuts			
Smooth and uniform	0.025	0.035	0.040
Jagged and irregular	0.035	0.040	0.050
Channels not maintained, weeds and brush uncut			
Dense weeds, high as flow depth	0.050	0.080	0.120
Clean bottom, brush on sides	0.040	0.050	0.080
Same, highest stage of flow	0.045	0.070	0.110
Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
Minor streams (top width at flood stage < 100 ft)			
Streams on Plain			
Clean, straight, full stage, no rifts, or deep pools	0.025	0.030	0.033
Same as above, but more stones/weeds	0.030	0.035	0.040
Clean, winding, some pools/shoals	0.033	0.040	0.045
Same as above, but some weeds/stones	0.035	0.045	0.050
Same as above, lower stages, more ineffective slopes, and sections	0.040	0.048	0.055
Same as 4, but more stones	0.045	0.050	0.060
Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	0.030	0.040	0.050
Bottom: gravels, cobbles, and few boulders	0.040	0.050	0.070
Bottom: cobbles with large boulders			

Values of Manning's Roughness Coefficient n (Uniform Flow). (continued)			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
FLOODPLAINS			
Pasture, no brush			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Cultivated area			
No crop	0.020	0.030	0.040
Mature row crops	0.025	0.035	0.045
Mature field crops	0.030	0.040	0.050
Brush			
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush and trees, in winter	0.035	0.050	0.060
Light brush and trees, in summer	0.040	0.050	0.080
Medium to dense brush, in winter	0.045	0.070	0.110
Medium to dense brush, in summer	0.070	0.100	0.160
Trees			
Dense willows, summer, straight	0.110	0.150	0.200
Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Major Streams (top width at flood stage > 100 ft)			
Regular section with no boulders or brush	0.025	¾	0.060
Irregular and rough section	0.035	¾	0.100

Manning's Roughness Coefficient (n) for Culverts

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

*NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

Appendix B. SCS (TR-55) Runoff Curve Numbers

SCS (TR-55) Runoff Curve Numbers

Cover Type and Hydrologic Condition		Avg Percent Imperviousness[2]	A	B	C	D
Cultivated Land:	Without Conservation Treatment		72	81	88	91
	With Conservation Treatment		62	71	78	81
Pasture or range land:	Poor Condition		68	79	86	89
	Good Condition		39	61	74	80
Meadow:	Good Condition		30	58	71	78
Wood or forest land:	Thin Stand, Poor Cover		45	66	77	83
	Good Condition		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:	Poor Condition (Grass Cover <50%)		68	79	86	89
	Fair Condition (Grass Cover 50% to 75%)		49	69	79	84
	Good Condition (Grass Cover >75%)		39	61	74	80
Impervious areas:	Paved Parking Lots, Roofs, Driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:	Paved; Curbs and Storm Drains (excluding right-of-way)		98	98	98	98
	Paved; Open Ditches (including right-of-way)		83	89	92	93
	Gravel (including right-of-way)		76	85	89	91
	Dirt (including right-of-way)		72	82	87	89
Urban Districts:	Commercial and Business	85%	89	92	94	95
	Industrial	72%	81	88	91	93
Residential Districts:	1/8 acre or less (townhouses)	65%	77	85	90	92
	1/4 acre	38%	61	75	83	87
	1/3 acre	30%	57	72	81	86
	1/2 acre	25%	54	70	80	85
	1 acre	20%	51	68	79	84
	2 acres	12%	46	65	77	82
Developing Urban Areas and Newly Graded areas	Pervious areas only, no vegetation		77	86	91	94

SCS (TR-55) Runoff Curve Numbers Notes

[1]Average runoff condition, and $I_a = 0.2S$
[2]The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system; impervious areas have a CN of 98 and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS TR-55 method has an adjustment to reduce the effect.
[3]CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Table adapted from Table 3.1.5-1 of Volume 2 of the Georgia Stormwater Management Manual (2016 Edition)

Appendix C. Rational Method Runoff Coefficients

Rational Method Runoff Coefficients (2-, 5-, and 10-Year Return Frequency)[1]			
Type of Cover	Flat (0%-2%)	Rolling (2%-10%)	Hilly (Over 10%)
Pavement and Roofs	0.95	0.95	0.95
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Suburban Residential	0.25	0.35	0.40
Apartment Homes	0.50	0.60	0.70
Single Family Residential	0.30	0.40	0.50
Lawns, Very Sandy Soil	0.05	0.07	0.10
Lawns, Sandy Soil	0.10	0.15	0.20
Lawns, Heavy (clay) Soil	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay, and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodlands and Forests	0.10	0.15	0.20
Meadows and Pasture Land	0.25	0.30	0.35
Pasture with Frozen Ground	0.40	0.45	0.50
Unimproved Areas	0.10	0.20	0.30
Water Surfaces	1.00	1.00	1.00

[1] Rational Method Frequency Adjustment Factors (f_a)	
25-year	1.1
50-year	1.2
100-year	1.25

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