Hydraulic Manual

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1.1 Introduction

This manual has been prepared to outline the general guidelines, procedures, and practices used by the State of Alabama Department of Transportation (hereafter referred to as ALDOT or the "Department") for hydrology and for hydraulic design. ALDOT has revised the manual with the intention of making it policy based and "ALDOT specific", rather than a how-to manual.

This chapter will provide a discussion on the intended use of this manual, general project work flow regarding drainage design for Department projects, and an overview of the manual contents, manual maintenance procedures, and manual acknowledgements.

1.2 How to Use This Manual

The chapter discussions within this manual follow Department design guidelines and state-of-the-practice design procedures. The purpose of this manual is to provide sufficient information on the subjects of hydrologic and hydraulic analyses as related to highway stormwater infrastructure design. During the development of this manual, numerous drainage manuals and guides from the FHWA, other states, and organizations were obtained and referenced. When necessary throughout the chapters, these outside manuals and guides are cited for the designer's reference.

The designer is assumed to be knowledgeable in the use of the referenced items. It is beyond the scope of this manual to incorporate computer program user manuals or keep current with these programs and/or the latest drainage-related federal regulations. Designers of Department drainage structures should follow the guidelines presented in this manual and reference the appropriate user manual or technical support group for computer program use. The FHWA hydraulic-related publications are found at https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm. When the designer encounters a situation that is not described in this manual or in the cited references, the Department Design/Bridge Hydraulic Section or the Department 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.

1.3 General Project Workflow

An adequate drainage structure is defined as one that meets or exceeds the goals of standard engineering practice and is consistent with what a reasonably competent and prudent designer would specify under similar circumstances.

The first step in any drainage design project is a hydrologic analysis to estimate the design discharge. Hydraulic analysis is then completed on those preliminary or trial selections of alternative designs that are judged to meet the site conditions and to accommodate the design discharge. The final step in the design process is the engineering evaluation of the trial designs and the approval of the selected final design. This approval may involve consideration of a wide variety of factors such as legal issues, flood hazards, cost, environmental, and other site-specific concerns.

1.4 Overview of Manual Contents

This manual primarily contains design 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 general contents of each chapter are summarized below.

Chapter 1 - Introduction

Chapter 1 introduces the manual, includes the intended use of this manual and general project work flow regarding drainage design for Department projects, and provides an overview of the manual contents, manual maintenance procedures, and manual acknowledgements.

Chapter 2 - Agency Coordination and Regulations

Chapter 2 provides an overview of the relationship between the roadway drainage design and the regulatory framework under which roadway projects are permitted and constructed. Also included is some background information on the federal laws and regulations regarding NPDES, Federal Emergency Management Agency (FEMA), U.S. Coast Guard (USCG) navigation permits, United States Army Corps of Engineers (USACE) 404 permits, as well as FHWA requirements.

Chapter 3 - Stormwater Planning

Chapter 3 contains an overview of the stormwater planning and design process, in accordance with the Department policies, necessary for both construction and post-construction stormwater measures.

Chapter 4 - Hydrology & Hydraulics

Chapter 4 discusses the methods used to determine peak runoff flow rates and volumes, such as the Rational Method, regression equations, and Technical Release-55 (TR-55). The chapter also introduces the basic concepts and general equations for open-channel and closed-conduit flow.

Chapter 5 - Channels

Chapter 5 discusses roadside and median channel analysis and design and provides an introduction to natural stream channel analysis and design.

Chapter 6 - Pavement Drainage

Chapter 6 discusses pavement drainage and includes curb and gutter flow, spread of water on pavement, hydroplaning, types of inlets, inlet capacity on grades and in sumps, inlet spacing, and flanking inlets.

Chapter 7 - Storm Drain Design

Chapter 7 provides guidance on storm sewer design and discussion, factors related to, and evaluation of the hydraulic grade line and energy grade line.

Chapter 8 - Culverts

Chapter 8 provides design procedures for the hydraulic design of highway culverts, including results of culvert analysis using HY-8 culvert analysis software and a summary of the design philosophy contained in the American Association of State Highway and Transportation Officials (AASHTO) Highway Drainage Guidelines, Chapter 4.

Chapter 9 - Post-Development Stormwater Management

Chapter 9 introduces post-development stormwater management concepts and defines post-construction requirements of the Department's projects.

Chapter 10 - Stream & Wetland Restoration Concepts

Chapter 10 presents an overview of typical stream restoration concepts followed by an overview of wetland restoration designs.

Chapter 11 - Bridge Hydraulic Design Criteria

Chapter 11 provides hydraulic design criteria for all existing and/or proposed river/tidal bridge sites and for culverts that meet any of the several conditions listed in the chapter.

Chapter 12 - Bridge Deck Drainage Systems

Chapter 12 provides the fundamentals of bridge deck drainage design, including pavement design, inlet design, and interception requirements.

1.5 Manual Maintenance

The manual is available through the Department website at: https://www.dot.state.al.us/publications.html It is the designer's responsibility to determine if there are any manual updates by periodically checking the webpage above and/or by contacting the Department.

If errors are discovered in this manual, please report them to the Design Bureau's Hydraulic Section at the address or e-mail address shown below, so that corrections can be made.

Alabama Department of Transportation

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1.6 Acknowledgements

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Dr. Scott Rogers, PE	Mr. John Ammons, PE
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Chapter 2: Agency Coordination and Regulations

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2.1 Introduction

This chapter provides an overview of the relationship between roadway drainage design and the regulatory framework under which roadway projects are permitted and constructed.

The Department's mission is to provide a safe, connected, and environmentally sensitive transportation system that enhances Alabama's economic competitiveness by working efficiently and communicating effectively to create strong partnerships. In keeping with the Department's mission statement, multiple levels of coordination must take place between the Department 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.

Environmental laws require that a reasonable effort be made to avoid or minimize harm to environmental resources such as the following:

- Waters of the United States
- Vegetative buffers on streams
- Threatened and endangered (T&E) species and their habitat (e.g., protection of fish and wildlife)
- Floodplains
- Navigable waters
- Coastal zones
- Historic resources
- Non-historic Section 4(f) resources (publicly owned parks, recreation areas, wildlife and waterfowl refuges)
- Cemeteries
- Archeological sites

2.1.1 Related Publications

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

- AASHTO Highway Drainage Guidelines, 4th Ed., Chapter 5 (2-1)
- AASHTO Drainage Manual, 1st Ed., Chapter 2 (2-2)
- 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)" (2-3)

2.2 Significant Laws Affecting Drainage

Designers should remain informed on proposed and enacted legislation and understand how such legislation relates to roadway drainage and bridge hydraulic decisions when designing a project.

The descending order of law supremacy is federal, State of Alabama, and then local law. Except as provided for in the statutes or constitution of the higher level of government, the higher level is not bound by the laws, rules, or regulations of a lower level. Many laws of one level of government are passed to enable that level to comply with or implement provisions of laws of the next higher level. For example, ADEM can have more stringent regulations than EPA, but said regulations have to be at least as stringent as EPA's.

The impacts of roadway runoff to downstream floodplains and to the downstream built environment must also be considered during the design process. The roadway drainage designer will design in accordance with the minimum FEMA standards. If a community adopts a more stringent floodplain ordinance than the minimum NFIP requirements, the Department may design and construct in accordance with the local ordinance provided the community agrees to pay for the additional design and additional construction cost of the project plus any additional incidental cost that may be associated with this ordinance. The design should be consistent with FEMA regulations, where practicable, to confirm floodplain impacts are within allowable limits and the proper documentation and permits are in place prior to the commencement of construction. Coordination with FEMA, the National Flood Insurance Program (NFIP), and local communities, with respect to the impacts of roadway construction on floodplains, is covered in greater detail later in this chapter.

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)*. Compilations of Federal Statutory Law, revised annually, are available in the *United States Code* (*USC*).

The CFR is available for viewing at:
http://www.gpo.gov/fdsys/browse/collectionCfr.action?collectionCode=CFR A searchable database of the USC is available at: http://uscode.house.gov/

Alabama laws are published in the *Official Code of Alabama,* available at http://alisondb.legislature.state.al.us/alison/codeofalabama/1975/coatoc.htm

2.2.1 Overview of Federal Laws

Federal law has implications that affect drainage design, although it may not directly address drainage. For example, environmental impacts resulting from drainage design will be a significant consideration as regulatory agencies review projects for permit approvals. Some of the more significant federal acts with elements that relate to drainage from roadways are listed below with a brief description of the provisions of each act.

- **THE RIVERS AND HARBORS ACTS (RHA) (33 USC 401, 403, 407).** The original RHA was passed in 1899. Several amendments to the Act have since been passed. These amendments address projects and activities in navigable waters and harbor and river improvements. Several of these amendments provide for a number of regulatory authorities, the implementation of which has evolved over time. Section 9 of the Act gives regulatory authority to U.S. Army Corps of Engineers (USACE) and the U. S. Coast Guard (USCG) regarding construction of structures across navigable waters of the United States. The USCG has regulatory authority over bridges and causeways while the USACE has regulatory authority over any dam, dike, or other similar structure not including a bridge or causeway. Section 13 of the Act grants regulatory authority to the USACE over the discharge of refuse into navigable waters. In the absence of a permit, such discharge of refuse is prohibited. Section 11 of the RHA authorizes the USACE to establish harbor lines, or arbitrary lines beyond which wharves and other structures may not be built.

- **THE TENNESSEE VALLEY AUTHORITY ACT OF 1933 (16 USC 831).** This Act formed the Tennessee Valley Authority (TVA). TVA's mission is to provide flood control within the Tennessee Valley, improve navigation on the Tennessee River, provide electric power, and promote "agricultural and industrial development" in the region.

- **FEDERAL-AID HIGHWAY ACT OF 1956 (23 USC 101 et seq.).** This Act provides for the administration of the Federal-Aid Highway Program. Proposed federal-aid projects must be adequate to meet the existing and probable future traffic needs and conditions in a manner conducive to safety, durability, and economy of maintenance. The projects must also be designed and constructed according to standards best suited to accomplish these objectives and to conform to the needs of each locality. Various amendments to the original Federal-Aid Highway Act have been enacted. Some of the more significant amendments added regulations for the following:

- Protection of Section 4(f) resources
- Addressing environmental justice, or the fair treatment and meaningful involvement of all people regardless of race, color, national origin, or income with respect to the development, implementation, and enforcement of environmental laws, regulations, and policies
- Control of soil erosion from roadway construction

The FHWA administers the Federal-Aid Highway Program in cooperation with the states. Projects classified as exempt are not subject to full FHWA oversight. However, the FHWA retains approval authority for the environmental documentation on exempt projects.

- **DEPARTMENT OF TRANSPORTATION ACT (DOTA) OF 1966 (49 USC 101, 80 Stat. 941).** This Act established the Department of Transportation and set forth its powers, duties, and responsibilities to establish, coordinate, and maintain an effective administration of the transportation programs of the Federal Government.

- **NATIONAL ENVIRONMENTAL POLICY ACT (NEPA) OF 1969 (42 USC 4321).** This Act is the overarching environmental law for federal-aid projects. The NEPA requires entities receiving federal aid to evaluate the impacts of their actions on the environment and prepare a public disclosure of environmental impacts in an environmental document, also known as a NEPA document, before project decisions are made. The NEPA document should not be written to defend a project decision that has already been made. The Council on Environmental Quality (CEQ) was established within the Executive Office of the President to administer NEPA. Each federal agency must assume responsibility for meeting NEPA guidelines with guidance from CEQ and oversight from the U.S. Environmental Protection Agency (EPA). In some circumstances, federal environmental laws may trigger a NEPA review regardless of whether or not a project receives federal funds. The roadway engineer or NEPA analyst and local government sponsors should coordinate with the Department's Environmental Technical Section to determine which federal requirements apply to state-funded projects.

Following are the three classes of environmental documentation under the NEPA:

1. Class I, Environmental Impact Statement (EIS)/Record of Decision (ROD) – An EIS is prepared for projects whose actions will have a significant impact on the environment.
2. Class II, Categorical Exclusion (CE) – A CE is prepared for projects that do not individually or cumulatively have a significant environmental impact.
3. Class III, Environmental Assessment (EA)/Finding of No Significant Impact (FONSI) – An EA is prepared for projects in which the environmental impact is not clearly defined. All actions that are not Class I or II are Class III. All actions in this class require the preparation of an EA to determine the appropriate document required.

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 final decision on the level of documentation is not made until the environmental studies are complete. All environmental documents are subject to Early Coordination.

The environmental document is prepared during the Preliminary Engineering phase as project decisions are being made.

- **THE CLEAN WATER ACT (CWA) OF 1972 (33 USC 1251).** The EPA is responsible for oversight and overall administration of the CWA. The CWA amended the Federal Pollution Control Act of 1948 to provide the statutory basis for the NPDES Permit Program and the basic infrastructure for regulating the discharge of pollutants from point sources to waters of the United States. Section 402 of the CWA specifically requires the EPA to develop and implement the NPDES program. The CWA allows the EPA to authorize the NPDES Program to state governments, which enables states to perform the permitting, administrative, and enforcement functions of the NPDES Program. In Alabama, the NPDES Program is implemented by the Alabama Department of Environmental Management (ADEM).

  Section 404 of the CWA establishes a program to regulate the discharge of dredged and fill material into waters of the United States, including wetlands. Responsibility for administering and enforcing Section 404 is shared by the USACE and EPA. Under Section 404, the USACE is responsible for regulating and issuing permits for proposed discharges into waters of the United States, including wetlands. As the overall Administrator of the CWA, the EPA retains oversight and veto authority over the USACE.

- **THE COASTAL ZONE MANAGEMENT ACT (CZMA) OF 1972 (Public Law 92-583, 86 Stat.1280, 16 USC 1451-1466).** The Act, administered by the National Oceanic Atmospheric Administration's (NOAA) Office of Ocean and Coastal Resource Management (OCRM), provides for management of the nation's coastal resources, including the Great Lakes, and balances economic development with environmental conservation. This Act encourages states to be responsible stewards of coastal land by implementing state-administered management programs.

- **SOIL AND WATER RESOURCES CONSERVATION ACT (RCA) OF 1977 (16 USC 2001-2009).** The RCA provides the United States Department of Agriculture (USDA) broad strategic assessment and planning authority for the conservation, protection, and enhancement of soil, water, and related natural resources. This Act directs the USDA to develop a National Soil and Water Conservation Program (SWCP), and to conduct an appraisal of the nation's soil, water, and related resources at five-year intervals. The SWCP and the appraisals are conducted under the jurisdiction of the Natural Resources Conservation Service (NRCS). Analyses conducted by the NRCS in carrying out the provisions of this Act are to be in conjunction with the Alabama Soil and Water Conservation Committee (SWCC), conservation districts, and appropriate citizen groups. The

SWCC works closely with the NRCS and models many of its recommended soil conservation and water quality practices after NRCS conservation practice standards.

- **FARMLAND PROTECTION POLICY ACT (FPPA) OF 1981 (7 USC 4201).** This Act is contained within the Agriculture and Food Act of 1981. Projects are subject to FPPA requirements if they may irreversibly convert farmland (directly or indirectly) to nonagricultural use and are completed by a federal agency or with assistance from a federal agency. For the purpose of FPPA, farmland includes prime farmland, unique farmland, and land of statewide or local importance.

  The Department's projects receiving federal aid must be coordinated with the NRCS to determine if there is farmland involvement in accordance with the FPPA. If there is involvement, the project engineer or ecologist must further coordinate with the NRCS to calculate a Farmland Conversion Impact Rating. Depending on this rating, additional roadway alignment alternatives may need to be considered to reduce impacts to the farmland. Projects planned and completed without the assistance of a federal agency are not subject to FPPA.

2.2.2 Overview of Alabama State Laws

The more significant state acts with elements that relate to drainage from roadways are listed below with a brief description of the provisions of each act. A timeline presenting the inception dates of the federal and state acts is provided in Figure 2.1.

- **THE ALABAMA ENVIRONMENTAL MANAGEMENT ACT OF 1982, (Section 22-22A-1).** This act consolidated various state commissions, agencies, and programs responsible for implementing environmental law. ADEM is responsible for the enforcement of environmental policy in the State of Alabama. It is authorized to adopt and enforce rules and regulations consistent with the statutory authority granted to the Alabama Environmental Management Commission and ADEM by the United States Environmental Protection Agency (EPA).

- **THE ALABAMA WATER RESOURCES ACT OF 1993 (Section 9-10B-1).** This Act authorized the creation of the Alabama Office of Water Resources (OWR) and a division of the Alabama Department of Economic and Community Affairs (ADECA) with oversight by the Alabama Water Resources Commission. OWR administers programs for river basin management, river assessment, water supply assistance, water conservation, flood mapping, the National Flood Insurance Program and water resources development. Further, OWR serves as the state liaison with federal agencies on major water resources related projects and conducts any special studies on instream flow needs as well as administering environmental education and outreach programs to increase awareness of Alabama's water resources.

2.3 Coordination with Regulatory Agencies

It is the responsibility of the Department or its consulting roadway drainage and bridge hydraulic engineer to coordinate as early as possible in the project schedule and follow-up diligently with regulatory agencies in order to move a project forward. 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 shall apply. In situations where these agencies or regulators contradict one another, it is the designer's responsibility to resolve the matter, most likely through a joint coordination meeting or similar means.

2.3.1 Federal Agencies

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

USCG

https://www.dco.uscg.mil/Our-Organization/Assistant-Commandant-for-Prevention-Policy-CG-5P/Marine-Transportation-Systems-CG-5PW/Office-of-Bridge-Programs/Bridge-Permit-Application-Process/

The USCG has regulatory authority under Section 9 of the RHA of 1899 to approve plans and issue permits for bridges and causeways across navigable rivers. As outlined in 23 CFR 650, the area of jurisdiction of USCG and FHWA is established as follows.

FHWA has the responsibility under the Federal-Aid Highway Act to determine whether or not a USCG bridge permit is required. This determination should be made at an early stage of project development so that any necessary coordination can be accomplished during environmental permitting.

USCG has the responsibility to do the following: 1) to determine whether or not a USCG permit is required for the improvement or construction of a bridge over navigable waters, except for the exemption exercised by FHWA as stated above, and 2) to approve the bridge location, alignment, and appropriate navigational clearances for all applications made to construct a bridge over a navigable waterway.

If a project involves a navigable waterway, the NEPA analyst must complete a bridge permit questionnaire and submit it to the FHWA for a determination of the need for a USCG permit. If the FHWA indicates that the project will require a USCG permit, the Department's Bridge Bureau will prepare and submit the permit application.

According to the USCG Bridge Administration Manual, Chapter 2, Section I (COMDTINST M16590.5C), bridge permit applicants should be encouraged to conduct waterway surveys as part of the application process to help determine bridge vertical clearance requirements. These surveys will help identify existing and prospective vessels using the waterways that exceed established vertical guide clearances, and possibly require an increased clearance for a planned bridge.

USACE

http://www.usace.army.mil/

The USACE has regulatory authority over the construction of dams, dikes, or obstructions other than bridges under Section 9 of the RHA of 1899. USACE also has authority to regulate the provisions of Section 10 of this Act, which prohibits the alteration or obstruction of any navigable waterway with the excavation or deposition of fill material in such waterway.

Section 404 of the CWA prohibits the unauthorized discharge of dredged or fill material into waters of the United States, including navigable waterways. Such discharges require a Section 404 permit from the USACE.

The USACE grants Nationwide General Permits (NWP) under Section 404 for certain minor activities involving discharge of fill material. NWPs were developed to allow projects that cause minimal adverse impacts to waters of the United States. The NWPs most applicable to roadway drainage are as follows:

- NWP 3 – Maintenance
- NWP 7 – Outfall Structures and Associated Intake Structures
- NWP 13 – Bank Stabilization
- NWP 14 – Linear Transportation Projects
- NWP 15 – USCG Approved Bridges
- NWP 23 – Approved Categorical Exclusions
- NWP 33 – Temporary Construction, Access, and Dewatering
- NWP 41 – Reshaping Existing Drainage Ditches

Additional information regarding NWPs can be found at: http://www.usace.army.mil/Missions/CivilWorks/RegulatoryProgramandPermits/NationwidePermits

Regional General Permits (RGP) are similar to NWPs in that they are programmatic permits. Instead of applying on a national scale, RGPs apply only within specific USACE regions.

Projects that do not meet the criteria for a NWP must apply for an Individual Permit (IP) from the USACE under Section 404 of the CWA. Processing IPs involves evaluation of individual and project-specific applications in what can be considered three steps:

1. Pre-application consultation (for larger projects)
2. Formal permit application review
3. Decision-making

Pre-application consultation usually involves one or several meetings between the applicant, USACE staff, interested resource agencies (federal, state, or local), and sometimes the interested public. Once a complete application is received, the formal review process begins. The USACE prepares a public notice (if required), evaluates the impacts of the project and considers all comments received, addresses potential modifications to the project if appropriate, and drafts or oversees drafting of appropriate documentation to support a recommended permit decision. The permit decision document includes a discussion of the environmental impacts of the project, the findings of the public interest review process, and any special evaluation required by the type of activity such as determinations of compliance with Section 404(b)(1) of the CWA.

The USACE's "Obtain a Permit" web page is located here:
www.usace.army.mil/Missions/CivilWorks/RegulatoryProgramandPermits/ObtainaPermit

This web page provides links to the USACE application form ENG FORM 4345, instructions for filling out the form, and applicable regulations and guidance, which are the legal foundation of the USACE permitting program.

When the USACE determines that an IP is required for a project, the Department must prepare a Practical Alternatives Report (PAR). The purpose of the PAR is to conduct an analysis of multiple project alternatives and to demonstrate that the preferred or selected project alternative is the least environmentally damaging practicable alternative (LEDPA).

The Fish and Wildlife Coordination Act (FWCA) [16 U.S.C. 661-667e; 48 Stat. 401], as amended, provides authority for the U.S. Fish and Wildlife Service (USFWS) to review and comment on the effects on fish and wildlife of activities proposed to be undertaken or permitted by the USACE.

FHWA

https://www.fhwa.dot.gov/

The FHWA is an agency within the U.S. Department of Transportation that administers the Federal-Aid Highway Program in concert with state and local governments. The FHWA supports state and local governments in the financing, design, construction, and maintenance of the nation's highway system and various federally and tribal-owned lands (Federal Lands Highway Program). The FHWA is responsible for ensuring that America's roads and highways continue to be among the safest and most technologically sound in the world.

FHWA has the authority to implement the Section 404 Permit Program (CWA of 1977) for federal-aid highway projects processed under 23 CFR 771.115 (b) as categorical exclusions. This authority was delegated to FHWA by USACE to reduce unnecessary federal regulatory controls over activities adequately regulated by another agency. This permit is granted for projects where the activity, work, or discharge is categorically excluded from environmental documentation because such activity does not have an individual or cumulative significant effect on the human environment.

FEMA

https://www.fema.gov/

(See Section 2.4)

U.S. EPA

http://www.epa.gov/

The Department's projects are coordinated through the EPA, Region 4 office. The EPA is responsible for administration of the CWA and for oversight of the NEPA process. Certain sections of the CWA are regulated by other federal or state agencies while the EPA provides oversight and retains veto authority over the other agencies. Examples include Section 402 and Section 404 of the CWA.

The EPA is authorized to prohibit the use of any area as a disposal site when it is determined that the discharge of materials at the site will have an unacceptable adverse effect on municipal water supplies, shellfish beds and fishery areas, wildlife, or recreational areas (Section 404 (c), CWA, 33 USC 1344). Also, the EPA is authorized under Section 402 of the CWA (33 USC 1344) to administer and issue an NPDES permit for point source and non-point source discharges.

Section 404 of the CWA (33 USC 1344) requires any applicant for a federal permit for any activity that may affect the quality of waters of the United States to obtain a water quality certification from ADEM.

USFWS

http://www.fws.gov/

The Fish and Wildlife Act (FWA) of 1956 (16 USC 742 et seq.), the Migratory Game-Fish Act (MGFA) (16 USC 760c-760g) and the FWCA (16 USC 611-666c) provide protection of the quality of the aquatic environment as it affects the conservation, improvement, and enjoyment of fish and wildlife resources. The FWCA requires that the USFWS and Alabama Department of Conservation of Natural Resources be consulted for review and comment whenever a private or public entity's action proposes to modify or impact a stream or body of water in Alabama. This includes drainage impacts. The intent of the above Acts is to conserve wildlife resources by preventing loss of and damage to such resources as well as provide for the development and improvement of such resources.

It is the function of the USFWS to consider and balance all factors, including anticipated benefits and costs in accordance with NEPA, in deciding whether to issue the permit. The Department should initiate contact with the USFWS regarding relevant actions on proposed roadway projects.

USDA

http://www.usda.gov/

Provides leadership on food, agriculture, natural resources, rural development, nutrition, and related issues based on public policy, the best available science, and effective management.

NRCS

http://www.nrcs.usda.gov/

The Department's projects receiving federal aid must be coordinated with the NRCS to determine if there is farmland involvement in accordance with the FPPA. If it is determined that impacted farmland meets the FPPA criteria, the Department should further coordinate with the NRCS to calculate a Farmland Conversion Impact Rating. Depending on this rating, additional roadway alignment alternatives may need to be considered to reduce impacts to the farmland. Projects planned and completed without the assistance of a federal agency are not subject to FPPA.

Early coordination will be completed with the NRCS regarding impacts to farmland as discussed in the paragraph above.

TVA

http://www.tva.gov/

The TVA was established by the TVA Act of 1933. Section 26a of that Act requires that TVA approval be obtained before any construction activities can be carried out that affect navigation, flood control, or public lands along the shoreline of the TVA reservoirs or in the Tennessee River or its tributaries. Permit approvals for construction under Section 26a are considered federal actions and are therefore subject to the requirements of the NEPA and other federal laws.

Among the typical Department structures and projects that require TVA approval under a Shoreline Construction Permit are bridges, culverts, and fill or construction within the floodplain. Section 26a regulations apply to both the location of construction projects and the types of activities carried out.

Note that TVA approval is not required for replacement of culverts or bridges of the same or greater hydraulic capacity, which create no new or additional obstruction and are within the same roadway alignment. This type of construction is considered maintenance activity.

Shoreline Construction Permits are needed for both on-reservoir and off-reservoir activities:

- On-reservoir activities are those that occur in, across, or along TVA reservoirs and regulated rivers and streams in the Tennessee Valley. Regulated rivers and streams are located downstream of TVA dams and are directly impacted by the operation of the dams.

- Off-reservoir activities are those that occur on all other perennial rivers and streams in the Tennessee Valley watershed. The construction standards outlined on this site do not apply to off-reservoir activities, which are considered on a case-by-case basis.

Detailed information regarding Shoreline Construction Permits under Section 26a is available at the following website: http://www.tva.gov/river/26apermits

USGS

https://www.usgs.gov/centers/lmg-water/

The Department's projects that may involve the removal and possible relocation of USGS stream gages and associated benchmarks should be coordinated with the local USGS office either in Montgomery or Tuscaloosa.

2.3.2 State and Local Agencies

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

ADEM

https://adem.alabama.gov/

ADEM is responsible for the enforcement of environmental policy in the State of Alabama. It is authorized to adopt and enforce rules and regulations consistent with the statutory authority granted to the Alabama Environmental Management Commission and ADEM by the United States Environmental Protection Agency (EPA).

ADECA

http://www.adeca.alabama.gov/

ADECA (through OWR) administers programs for river basin management, river assessment, water supply assistance, water conservation, flood mapping, the National Flood Insurance Program and water resources development. Further, OWR serves as the state liaison with federal agencies on major water resources related projects and conducts any special studies on instream flow needs as well as administering environmental education and outreach programs to increase awareness of Alabama's water resources.

Other state agencies that may require coordination efforts with regard to the Department's projects:

Alabama Department of Conservation and Natural Resources

http://www.outdooralabama.com/

Alabama Surface Mining Commission

http://www.surface-mining.state.al.us/

Alabama Soil and Water Conservation Committee

https://alabamasoilandwater.gov/

2.4 National Flood Insurance Program (NFIP)

(http://www.fema.gov/national-flood-insurance-program)

Given the significance and number of river crossings and floodplain-related issues encountered during roadway construction, specific information and guidance related to the FEMA regulations and requirements is provided. The information below is based on the Department's policy and practice.

2.4.1 Flood Insurance

The National Flood Insurance Act of 1968, as amended, (42 USC 4001-4127) requires that communities adopt adequate land-use and control measures to qualify for flood insurance in flood-prone areas. Federal criteria promulgated to implement this provision contain the following requirements that can affect certain roadways:

For riverine situations, when the Administrator of the Federal Insurance Administration has identified a flood-prone area, the community should require that no fill or other proposed use be permitted within the floodplain where base (100-year) flood elevations have been determined, 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 one foot at any point in the floodplain. In FEMA designated flood hazard areas (no base-flood elevations or floodway determined), these same regulations apply. In areas where a regulatory floodway has already been established, the allowable increase in the base flood elevation may be restricted to less than one foot.

After the floodplain special flood hazards, the base flood water surface elevations, and floodway data have been provided, the community should designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood to more than the local ordinance requirement at any point in the floodplain 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 within the community during the occurrence of the 100-year flood discharge.

The participating communities agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the NFIP.

2.4.2 Flood Disaster Protection

The Flood Disaster Protection Act of 1973 (PL 93-234, 87 Stat. 975) denies federal financial assistance to local communities that fail to qualify for flood insurance. Formula grants to states are excluded from the definition of financial assistance, and the definition of construction in the Act does not include highway construction; therefore, Federal aid for highways is not affected by the Act. The Act does require communities to adopt certain land-use controls to qualify for flood insurance. These land-use requirements could impose restrictions on the construction of highways in floodplains and floodways in communities which have qualified for flood insurance.

2.4.3 Local Community

The local community with land-use jurisdiction, whether it is a city, county, or state, has the responsibility of enforcing 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 submit proposals to FEMA for amendments to NFIP ordinances and maps in that community. The Department and its consultants shall coordinate directly with the community and, through them, coordinate with FEMA. Determination of the status of a community's participation in the NFIP and the review of applicable NFIP maps and ordinances are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

2.4.4 NFIP Maps

Where NFIP maps are available, their use is mandatory in determining whether a roadway location alternative will include an encroachment on the base floodplain. The following four types of NFIP maps are published in Alabama:

- Flood Hazard Boundary Maps (FHBM)
- Flood Boundary and Floodway Maps (FBFM)
- Flood Insurance Rate Maps (FIRM)
- Digital Flood Insurance Rate Maps (DFIRM)

A FHBM indicates where the boundaries of the floodplain, mudslide, and related erosion areas having special hazards have been designated. A FHBM is generally not based on a detailed hydraulic study and, therefore, floodplain boundaries shown are approximate. A FBFM, in contrast, is generally derived from a detailed hydraulic study and should provide reasonably accurate information concerning the base floodplain and regulatory floodway. A FIRM is generally produced at the same time using the same hydraulic model and has appropriate rate zones and base flood elevations added. A DFIRM is an electronic product linked to a geographical information system (GIS) database. It includes the same information as a FIRM but can include additional information as well, such as hydraulic structure data. In October 2009, FEMA began converting all maps to DFIRM , which are viewable online. Hydraulic data used in the derivation of these map products, including the effective hydraulic model may be available through FEMA study contactors, OWR (ADECA), and the local communities.

Communities may or may not have published one or more of the above maps depending on their level of participation in the NFIP. Information on community participation in the NFIP is provided in the *National Flood Insurance Program Community Status Book,* which is published semiannually for each state and can be viewed online from the FEMA website.

2.5 NFIP Requirements

All floodplain crossings must comply with FEMA regulations. The Department adheres to the guidelines set forth in the FHWA's Federal-Aid Policy Guide, 23 CFR 650A, September 30, 1992, Transmittal 5, "Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency." A copy of this policy guide is included in Appendix B of this manual.

2.5.1 FEMA Coordination

The Department's coordination with FEMA may arise in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. This is accomplished initially through consultation with the local floodplain administrator of the participating community to determine if the proposed highway project is consistent with existing watershed and floodplain management policy and programs and to obtain information on current and proposed development in the affected watershed(s). The Department will design proposed projects in accordance with the minimum FEMA standards. If a community adopts a more stringent floodplain ordinance than the minimum requirements of the NFIP, the Department may design and construct in accordance with the local ordinance provided the community agrees to pay for the additional costs of the project plus any additional incidental costs that may be associated with this ordinance.

The following circumstances, based on the FHWA/FEMA coordination agreement, would ordinarily require coordination with FEMA:

- When a proposed crossing encroaches on a regulatory floodway and would require a revision to the floodway map,
- When a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated, and the maximum one foot increase in the base flood elevation would be exceeded.
- When a local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are under way,

- When a local community is participating in the emergency program and base FEMA flood elevation in the vicinity of insurable buildings is increased by more than one foot, or where insurable buildings are not affected, it is sufficient to notify FEMA of changes to base flood elevations as a result of highway construction.

During the corridor study, the draft EIS/CE/EA should indicate the NFIP status of affected communities, the encroachments anticipated, and the need for floodway or floodplain ordinance amendments. If coordination with FEMA is required, and a determination by them would influence the selection of and alternative, a commitment from FEMA indicating acceptance of the revision should be obtained prior to the final environmental impact statement (FEIS) or a FONSI through a conditional map revision request. Otherwise, this later coordination may be postponed until the design phase.

2.5.2 Longitudinal Roadway Encroachments

Since longitudinal floodplain and floodway encroachments by new and widened roadways generally have a major effect on the flood elevations of the affected stream, these encroachments shall be avoided if at all possible. The project manager and location engineer shall abide by the following basic rules for roadway widening, dualizations, 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 floodplain.
2. For roadway paralleling projects, the new parallel roadway should be placed to avoid or minimize longitudinal encroachments on floodplains.
3. New location roadway projects should be aligned to avoid or minimize longitudinal encroachments on floodplains.
4. For all cases, longitudinal encroachment on a delineated FEMA regulatory floodway should be avoided if at all possible.

2.5.3 Categories and Recommendations for Bridges and Bridge Culverts

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

1. If the stream has an established regulatory floodway, the structure should be designed, if practical, so that the bridge approaches will not encroach on the regulatory floodway. The bridge superstructure should also clear the floodway elevation. The bridge substructure (i.e. piers/piles) 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 shall be sent a copy of the proposed roadway plans and the preliminary bridge layout along with a letter stating that the proposed construction will 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 to be constructed 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, if practical, so that there will be no change in the base flood elevations, floodway elevations, or floodway widths at any cross section. The Department defines a "No-Rise" project as one that causes 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 the Department's right-of-way. Increases greater than 0.1 foot in the base flood profile or the floodway profile and/or 1 foot in the floodway width inside the right-of-way are considered integral to the bridge structure and do not affect any other property. Designers should be aware of any measurable impacts (0.01 ft) upstream of the project right-of-way where structures (i.e. residential, commercial, industrial, and etc.) exist and could be affected by the project. Liability relating to increased flood stages in these situations should be avoided. Furthermore, designers should also be aware that Alabama OWR along with many local communities throughout the state define a “No-Rise” as a 0.00 foot increase in base flood or floodway elevations.

   For consultant projects, a signed and sealed "No-Rise" certification by a registered professional engineer is required (see Appendix B at the end of this manual). If this criterion is met, two original sets of supporting documentation shall be prepared. One set is for submission to the affected community, requesting a "letter of concurrence" approving the project to be sent to the Department. One set will be retained in the project file for the Department’s records.

   An example of a "No-Rise" condition can be shown in either of the two following cases for the base flood and floodway elevation:

   a. When the total difference in the calculated base flood and 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 the water surface elevation difference exceed 0.05 foot, then a no-rise condition can no longer be claimed according to ALDOT. The designer should note that many local communities have more stringent regulations, such as not increasing base flood and floodway elevations at all (0.00 feet).

   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 ALDOT. The designer should again note that many local communities have more stringent regulations, such as not increasing base flood and floodway elevations at all (0.00 feet).

3. If the stream has an established regulatory floodway, and an encroachment on the regulatory floodway is necessary, and the criteria of category 2 are not met, then the affected community will need to make arrangements and obtain approval from any affected property owners. The community will also be responsible for coordinating with FEMA to revise the effective base flood elevations, floodway widths, and floodway elevations. FEMA can approve an alternative floodway where increases in water surface elevations exceed the one foot maximum only when the following conditions have been met:

    a. A location hydraulic study has been performed in accordance with 23 CFR 650 Subpart A, and the Department finds the encroachment is the only practical alternative.

    b. The constructing agency has made appropriate arrangements with affected property owners and the community to obtain flooding easements or otherwise compensate them for future flood losses due to the effects of the backwater exceeding the one foot limit.

    c. The constructing agency has made appropriate arrangements to ensure that the NFIP and Flood Insurance Fund will not incur any liability for additional future flood losses to existing structures that are insured under the Program and grandfathered in under risk status existing prior to the construction of the project.

    d. Prior to initializing construction, the constructing agency provides FEMA with revised flood profiles, floodplain and floodway mapping, and background technical data necessary for FEMA to issue revised FIRMs and FBFMs for the affected area, upon completion of the structure.

    Revisions such as these often require local funding that may not be available, further coordination will be required by the Department and the local community on a project-specific basis to prepare the necessary map revisions. See Section 2.8 of this chapter for additional information.

    For consultant projects, the Professional Certification Form required by FEMA shall be completed, stamped, and signed by a registered professional engineer (see Appendix B at the end of this manual). For cases such as these, the Department 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 the Department. One set will be retained in the project file for the Department's records. The community's "letter of concurrence" approves the project as designed 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 the Department's design policy.

4. For a bridge crossing a floodplain that is shown on a FIRM map, but does not have a regulatory floodway, the bridge will be sized to limit the backwater to the Department's minimum of no more than a 1-foot increase in the existing base flood elevation. If the local community's ordinance is more stringent than the

minimum requirement, the Department may construct in accordance with the local ordinance provided the community agrees to pay for the additional design and additional construction cost of the project plus any additional incidental cost that may be associated with this ordinance.

5. For bridges that are outside of NFIP communities or NFIP identified flood hazard areas, the bridge should be sized using the Department's design criteria and requirements (see Chapter 11).

2.5.4 Floodway Revisions

Where it is not cost-effective to design a highway crossing to avoid encroachment on an established floodway, a second alternative would be modification of the floodway itself by increasing the conveyance within the floodplain and or channel areas by increasing cross sectional area, reducing/removing vegetative cover, paving channels, or other similar activities. Only when the above options are determined to be inappropriate, should a design which raises the base flood elevation greater than one foot be considered.

The responsibility for demonstrating that an alternative floodway configuration meets NFIP requirements rests with the community. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Floodway revisions should be based on the hydraulic model that was used to develop the current effective floodway but updated to reflect existing encroachment conditions. This will allow the determination of the increase in the base flood elevation caused by the proposed encroachment(s) since the original floodway was established. It is permissible however, to input older model inputs (i.e. E-431, HEC-2) into HEC-RAS to duplicate and then revise the effective FIS model. Duplicated water surface elevations should match within 0.5 feet of the elevations determined in the original model. All current conditions and revised models should extend far enough upstream and downstream to tie back into the original base flood and floodway profiles. These models should tie back to within 0.5 feet of the original profile elevations.

If the input data for the original effective model are unavailable, an approximation should be developed. A new model should be established using original cross section topographic information, original drainage structure geometries, where possible, and the discharges and Manning's roughness coefficients contained in the published FIS report for the respective community. The profile elevations produced in the new model should match the profile elevations from the effective model within 0.5 feet.

2.5.5 Data Submittal for Floodway Revisions

Data submitted to FEMA, through the community, in support of a base flood and/or floodway revision request should include the following:

- A copy of current regulatory Flood Boundary Floodway Map, showing existing conditions, proposed highway crossing, and revised floodway limits.

- A copy of the hydraulic models (input, computation, and output) for the duplicated effective run, revised existing conditions run, and proposed conditions run for the 100-year base flood and 100-year floodway models.

- A copy of the engineering certification required for work performed by private subcontractors.

- Completed MT-2 forms

2.6 Design Data Required for Project Involving FEMA Floodplains

2.6.1 Publications

FEMA regulatory information can be obtained by visiting their Map Service Center Web site at https://msc.fema.gov.

2.6.2 Maps

1. FHBM
2. FBFM
3. FIRM
4. DFIRM
5. Detailed Study Workmaps. These are large-scale topographic maps annotated with cross-section locations, floodplain limits, and floodway boundaries from detailed hydraulic studies.

2.6.3 Survey Data, Plans, Reports

1. All data specified in Chapter 11.2.1 of this manual is required. This section contains a detailed listing of the minimum survey data that is required. The hydraulic engineer should determine the extent of survey data required to accurately model the project site.
2. In-roads and Microstation (digital/electronic) survey files.
3. Existing bridge and roadway plans.
4. Three sets of preliminary proposed roadway plans.

2.6.4 Regulations and Policy Guides

1. Federal-Aid Policy Guide, NS 23 CFR 650A, "Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency (FEMA)." See Appendix B in this manual.

2. The NFIP laws and regulations are available for download at https://www.fema.gov/flood-insurance/rules-legislation/laws.

2.6.5 Computer Models and Manuals

For current hydrologic and hydraulic computer models that meet the minimum requirements of the NFIP, please visit the FEMA web site at https://www.fema.gov/flood-maps/products-tools/numerical-models

2.7 Design Methods/Procedures for all Encroachments

For current design methods and procedures for encroachments, please visit the FEMA web site at https://www.fema.gov/media-library-data/1387560925526-6726c1db628a848154f1c77c3b503fb2/Guidelines_and_Specifications_for_Flood_Hazard_Mapping_Partners_-_Reference_Sections_(Feb_2002).pdf for the FEMA Guidelines and Specifications Volume 2: Map Revisions and Amendments.

2.8 NFIP Map Revisions

FEMA has established administrative procedures for changing or correcting effective FIRMs and Flood Insurance Study (FIS) reports based on new or revised technical data. A physical change to the affected FIRM panels and portions of the FIS report is referred to as a Physical Map Revision (PMR).

A PMR is an official republication of a community's NFIP map to reflect changes to base flood elevations, floodplain boundary delineations, regulatory floodways and planimetric features. These changes typically occur as a result of structural works or improvements, annexations resulting in additional flood hazard areas, or corrections to base flood elevations or Special Flood Hazard Areas (SFHAs).

Changes to NFIP maps may also be made by a Letter of Map Change (LOMC). The three LOMC categories are described below:

- **LETTER OF MAP AMENDMENT (LOMA).** A LOMA is an official revision by letter to an effective NFIP map. A LOMA results from an administrative procedure that involves the review of scientific or technical data submitted by the owner or lessee of property who believes that the property has incorrectly been included in a designated SFHA. A LOMA amends the currently effective FEMA map and establishes that a specific property is not located in an SFHA.

- **LETTER OF MAP REVISION BASED ON FILL (LOMR-F).** A LOMR-F is an official revision by letter to an effective NFIP map. A LOMR-F states FEMA's determination concerning whether a structure or parcel has been elevated on fill above the base flood elevation and is, therefore, excluded from the SFHA.

- **CONDITIONAL LETTER OF MAP REVISION (CLOMR).** NFIP maps must be based on existing, rather than proposed, conditions. Because flood insurance is a financial protection mechanism for real-property owners and lending institutions against existing hazards, flood insurance ratings must be made accordingly. However, communities, developers, and property owners often undertake projects that may alter or mitigate flood hazards and would like FEMA's comment before constructing them. A CLOMR is FEMA's formal review and comment on whether a proposed project complies with the minimum NFIP floodplain management criteria. If it is determined that it does, the CLOMR also describes any eventual revisions that will be made to the NFIP maps upon completion of the project.

Obtaining conditional approval is not automatically required by NFIP regulations for all projects in the floodplain. A CLOMR is required only for those projects that will result in an increase in the water surface elevation greater than 1 foot for the 100-year flood for streams with base flood elevations specified but no floodway designated. A CLOMR is also required for any proposed construction within a regulatory floodway that will result in an increase in the water surface elevation for the base flood. Note that a CLOMR may also be necessary if there is a decrease in flood elevations, which would allow the community to build in areas previously not allowed. The technical data needed to support a CLOMR request generally involve detailed hydrologic and hydraulic analyses and are similar to the data needed for a LOMR request. When the proposed construction is completed, a LOMR request should be made.

A request for a CLOMR by a private individual, including homeowners and land developers, or by any public agency, must be made through the local community participating in the NFIP. The following are reasons why the CLOMR request is made through the community:

1. Community must be aware of changes by the proposed project and determine if they are consistent with local ordinances.
2. Community will collect fees for FEMA that apply to requests for map revisions.
3. Community must determine that the existing FIRM is not accurate and that the hydrologic and/or hydraulic information should be updated as proposed in the CLOMR request.

- **LETTER OF MAP REVISION (LOMR).** A LOMR is an official revision to the currently effective FEMA map. It is used to change flood zones, floodplain and floodway delineations, flood elevations, and planimetric features. All requests for LOMRs should be made to FEMA through the chief executive officer of the community, because it is the community that must adopt any changes and revisions to the map. If the request for a LOMR is not submitted through the chief executive officer of the community, evidence must be submitted that the community has been notified of the request.

R2 Chapter 2 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2007. Highway Drainage Guidelines, 4th Ed.

2. American Association of State Highway and Transportation Officials (AASHTO). 2014. Drainage Manual, 1st Ed.

3. Federal Highway Administration (FHWA), Federal-Aid Policy Guide. “Highways.” Title 23 Code of Federal Regulations (CFR).

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Chapter 3: Stormwater Planning

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3.1 Introduction

For many reasons, stormwater planning is an essential component of the overall project design. Planning minimizes safety hazards on roadways, degradation of receiving waters, and adverse impacts to the environment. To effectively plan for the stormwater component of a linear Department roadway project, it is important to consider stormwater in the earliest stages of the design process. This chapter contains an overview of the stormwater planning and design process, in accordance with Department practices, necessary for both construction and post-construction stormwater measures.

3.2 Objectives and Conceptualization

Traditionally, the main objective of hydraulic planning has been to provide a safe driving environment by preventing ponding on roadway surfaces and designing corridors to effectively pass the design-year event. Stormwater planning is important for the following reasons:

- Maintenance of public safety
- Protection of property upstream and downstream of the Department's project or facility
- Protection of the Department linear facility itself and the function it serves by reducing erosive damage from stormwater effects
- Protection of the surrounding environment from potentially adverse impacts

An important part of the Department's project conceptualization phase is to consider stormwater and how to incorporate it into the planning process. Stormwater planning often requires advanced gathering of data to create alternatives, and present viable cost estimates. The following are some key concepts to consider:

- **Avoidance**: avoid disturbing environmentally sensitive areas (e.g. changing the roadway alignment to avoid these areas)
- **Minimization:** minimize the disturbance required for the project (e.g. selecting a bridge design that will minimize floodplain impacts)
- **Footprint Reduction**: reduce the roadway footprint by considering different alternatives (e.g. reducing the number of lanes, reducing lane width, etc.)

A concurrent process to the stormwater planning and conceptualization phase is defining the project scope. During the scope development, awareness of potential stormwater impacts leads to a better project concept and overall improved design. The stormwater design workflow process is discussed in the next section, in its entirety.

3.3 Project Workflow and Design Considerations

An outline of the project development process, from inception through construction award, can be found in the ALDOT Guide for Developing Construction Plans (GDCP). Stormwater planning should be incorporated into the plan process near the beginning.

In addition to the GDCP, stormwater planning should consider other requirements set forth by the Department. Designers must provide features and practices that cause the post-development hydrology to mimic the pre-development hydrology of the site to the maximum extent practicable, while working within the constraints of the project, at all locations of discharge. The basis for design to meet this requirement shall be small, frequent rain events up to, and including, the 95th percentile of rain event for the site. While working toward this design goal, initial consideration should be the use of decentralized practices and features near the source of the runoff. Design elements that utilize natural materials and processes will be considered whenever possible.

- Small, frequent rain events are those storm events with rainfall depths up to, and including, the 95th percentile event for a specific county.

- Pre-development and post-development hydrology include both peak discharge and runoff volume.

- Pre-development hydrology is the existing hydrological condition of the site, just prior to construction of the planned development or re-development.

- New Development describes the creation of a new transportation facility or a new support facility that causes a ground disturbance of greater than one acre.

- Re-Development, with respect to transportation facilities, describes non-maintenance work performed to, or on, an existing transportation facility that provides for an increased number of thru lanes of travel, and causes a ground disturbance of greater than one acre. Work on an existing road that does not result in an additional thru lane does not constitute re-development.

To address these requirements, the designer should approach stormwater planning as shown in Figure 3.1. The lighter shaded top row indicates steps taken during the concept stage, where the darker shaded rows below represent the preliminary design stage. New design alternatives and iterations of layouts may be necessary to address all requirements.

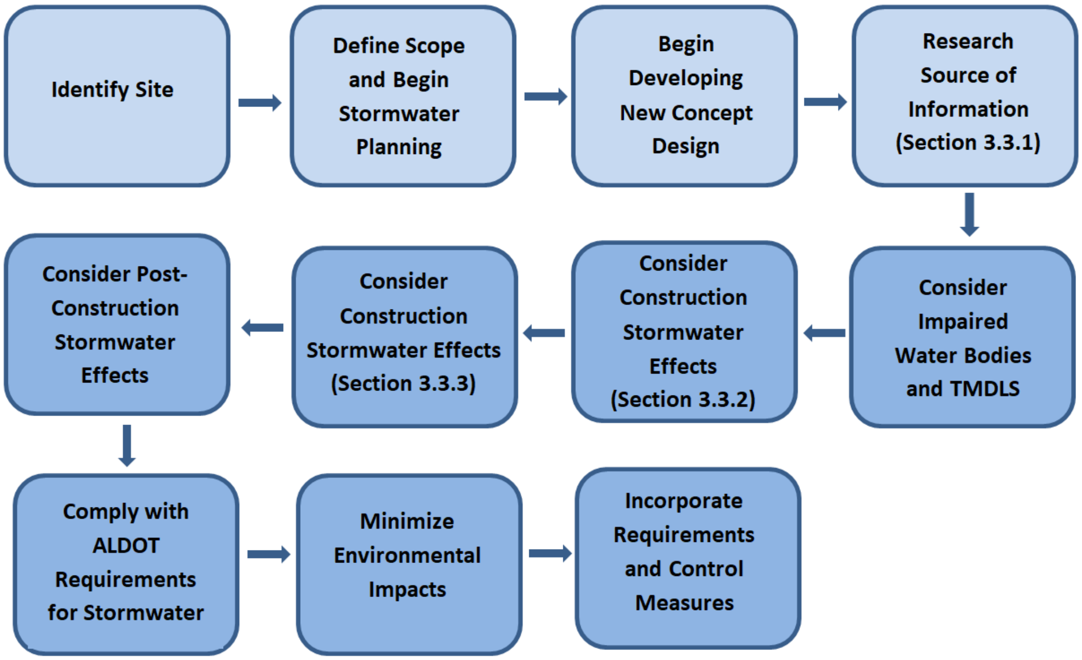

Figure 3.1 – Stormwater Planning Workflow

Figure 3.1 is intended as a guidance tool.

3.3.1 Sources of Information

Stormwater planning will often incorporate local, state, and/or federal regulatory requirements.

Information will be needed to fulfill certain regulatory planning aspects and may include the following sources:

- Floodplain data
- United States Geological Survey (USGS) topographic maps
- State, county, or city maps
- Land surveys
- Geotechnical evaluations and soil surveys
- Wetland maps
- Utility companies
- Aerial photography
- Past project plans
- Other nearby projects
- Current approved 303d list and updated TMDL[1] list with priority construction sites (ADEM)

[1] – Total Maximum Daily Load – identifies maximum amount of pollutant that a body of water can receive and still meet water quality standards.

Refer to Chapter 2 of this manual for agency coordination requirements.

3.3.2 Hydrology and Hydraulic Design

After gathering the information required for the design process to begin, the project area's hydrology must be determined. The Department gives guidance on the hydrologic design method used for each stormwater component. For example, the rational method is used for gutter spread calculations and Technical Release (TR)-55, and small storm hydrology methodologies are used for post-construction stormwater BMPs. Tables 4.1 and 4.2 in Chapter 4 of this manual include a comprehensive list of acceptable design methods and their limitations.

The Department design guidance also gives direction on determining design flow rates based on specific storm events. Table 4.3 of Chapter 4 provides a summary of design storm events used throughout the manual.

Once the hydrologic components of the project are determined, hydraulics of storm sewer systems, culverts, and channels can be evaluated. Additional guidelines and hydraulic design parameters can be found in Chapters 5, 6, 7, and 8 of this manual.

Several stormwater design alternatives will develop as the project design progresses. Within the alternative analysis process, it is important to consider both construction stormwater and post-construction stormwater effects. Making stormwater alterations in the project design usually entails numerous modifications to the overall project (e.g., grading, utility coordination, roadway alignment, etc.) For this reason, the Department urges the designer to consider these planning aspects early in the concept phase.

3.3.3 Construction Stormwater

Construction stormwater refers to runoff that occurs during construction from storm events. This associated runoff can be problematic and contribute to the impairment of Alabama's streams, rivers, and lakes. Currently, the NPDES permit program, operating under the Clean Water Act, regulates the discharge of pollutants to waters of the United States. As previously noted, low impact development (LID) and minimization strategies help to alleviate the effects of construction stormwater runoff. The Department must meet ADEM requirements outlined in the current effective NPDES Construction General Permit (CGP) in these areas. Also, the Department must meet Municipal Separate Storm Sewer System (MS4) permit requirements. For additional information on MS4 Post-Construction permit requirements, see Chapter 9 of this manual.

Interception and concentration of overland flow and constriction of natural waterways from linear highway construction typically results in increased erosion potential. To protect the highway and adjacent areas from erosion, it is sometimes necessary to employ an energy dissipating device.

Energy dissipation should be considered part of the larger design system, which may include the culvert and channel protection requirements (upstream and downstream). The interrelationship of these various components must be considered in designing any one part of the system.

Throughout the design process, the designer should keep in mind that the primary objective is to protect the highway structure and adjacent area from excessive damage due to erosion. One way to accomplish this objective is to return flow to the downstream channel in a condition that approximates the natural flow regime. Several factors involved in designing an energy dissipator can be found in Chapters 5 and 8 of this manual. For a more comprehensive discussion of energy dissipator design, refer to the FHWA publication HEC-14. [3-2]

3.3.4 Post-Construction Stormwater

Post-construction stormwater consists of the permanent controls and practices established to mimic pre-development hydrology at a site. Both poor runoff quality and quantity can have adverse effects on receiving waters, making it important to continually treat and minimize stormwater after construction has been completed. Chapter 9 of this manual provides a detailed explanation on the potential permanent controls and design criteria for post-construction practices related to the Department's requirements. Chapter 9 also discusses the post-construction stormwater detention guidance. BMP design information, and other Department requirements, can be found in Chapter 9 of this manual and the Department's website: https://www.dot.state.al.us/programs/StormwaterPermittingDesign.html. A growing national trend has become the incorporation of LID and Green Infrastructure (GI) into the design of construction and post-construction stormwater practices. The three key concepts listed in section 3.2 are all LID concepts that attempt to minimize construction impacts. Additional information on specific LID/GI practices is detailed in the subsequent BMP sections of Chapter 9.

3.4 Project Requirements

At the beginning of any Department roadway design, understanding project requirements is a fundamental step in the success of the design effort. From a drainage design perspective, knowing the following criteria will make the process much more efficient:

- Applicable Department requirements
- Required documentation (calculation summaries, reports, etc.)
- Permitting and applicable agency coordination

3.4.1 Department Guidance

The majority of the Department's guidance regarding stormwater design is included in this drainage manual. For example, acceptable hydrologic and hydraulic methods are found in Chapter 4, *Hydrology & Hydraulics*, and requirements for gutter spread are found in Chapter 6, *Pavement Drainage*.

3.4.2 Project Documentation

Project documentation varies based on what aspect of stormwater design is being performed, and at what review stage the project resides. A project-specific hydraulics notebook is required for documenting criteria outlined in the GDCP. This notebook will include all the necessary calculations used for stormwater design purposes and, at a minimum, will include the documentation shown in Figure 3.2:

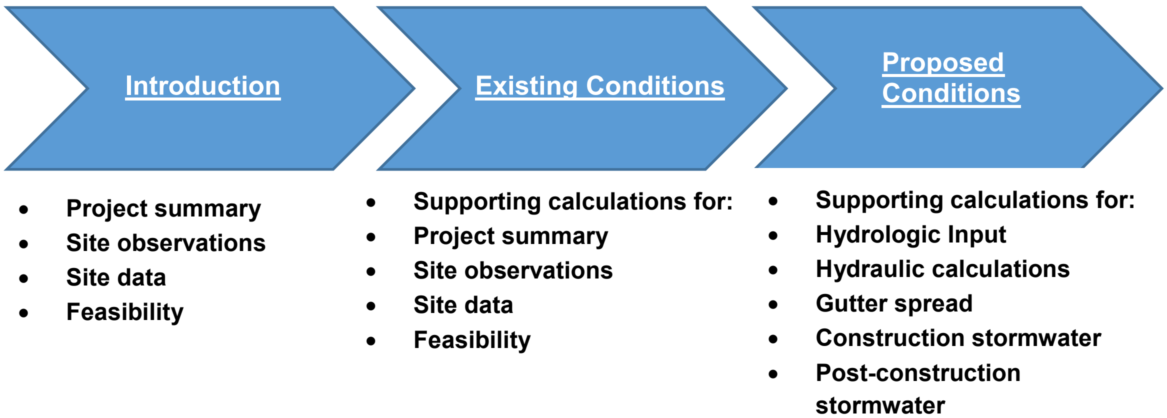

Figure 3.2 – Stormwater Project Documentation Requirements

Project documentation also serves as a method for the designer to demonstrate permit compliance to the maximum extent practicable (MEP) according to the stormwater management plans. The MEP concept acknowledges that not all designs and projects are capable of meeting the same standards but striving to meet those standards should be to the maximum extent practicable. This can be addressed by either documentation of meeting permit requirements or providing a rationale as to why a potential stormwater component was excluded or determined infeasible. This information is provided in the Department's Post-Development Stormwater Management referenced in Chapter 9.

3.4.3 Permitting and Other Agencies

Any design considerations that may have an effect on the environment should be cross-referenced with the National Environmental Policy Act (NEPA). Other rules and requirements that apply may be due to special design considerations, or project location. Location specific considerations may be warranted when a project site is near a historical preservation area or recreation area. See Table 3.1 for a list of these location specific considerations and their associated permits.

Table 3.1 Agencies & Permits

Agency	Permit
USACE (Wetlands)	NWP, IP, Section 404
FEMA (Floodplains)	CLOMR, LOMR
ADEM (Impaired Stream)	NPDES CGP
FWS (Endangered Species)	Regional Endangered Species Permit

Maintenance Challenges

In addition to outside agencies, it is important to take into account intra-agency coordination. One Bureau to consider is the Maintenance Bureau. Planning and location studies should take into consideration potential erosion and sedimentation problems upon completion of highway construction. If a particular location will require frequent and expensive maintenance due to drainage, alternative locations should be considered unless the potentially high-maintenance costs can be reduced by special design features.

Experience in the Region area is the best indicator of maintenance problems, and interviews with maintenance personnel could be extremely beneficial in identifying potential drainage problems. Reference to highway maintenance and flood reports, damage surveys, newspaper reports, and interviews with local residents could be helpful in evaluating potential maintenance problems, as well.[3-1]

During highway construction, channel changes, minor drainage modifications, and revisions in irrigation systems usually carry the assumption of certain maintenance responsibilities by the Department. Potential damage from the erosion and degradation of stream channels, and problems caused by debris, can be of considerable significance from a maintenance standpoint.[3-1]

Legal Consideration

A goal in highway drainage design should be to perpetuate natural drainage, insofar as practicable.

In general, designers should not address a question of law without the aid of legal counsel. Whenever drainage problems are known to exist or can be identified, drainage and flood easements, or other means of avoiding future litigation, should be considered, especially in locations where a problem could be caused or aggravated by the construction of a highway.[3-1]

If a question pertaining to law arises the designer should consult with their manager. Additionally, if a citizen reaches out directly to the designer, the designer should direct them to the proper channels within the Department.

It is often helpful in the planning and location phase of a project to document the history and present the status of existing conditions or problems and to supplement the record with photographs and descriptions of field conditions.[3-1]

R3 Chapter 3 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2014. Drainage Manual, 1st Ed.

2. 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.

Chapter 4: Hydrology and Hydraulics

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4.1 Hydrology

4.1.1 Guidelines

Drainage design requires knowledge of the hydrologic characteristics of the area. The Department uses several methods to determine peak runoff flow rates and volumes that have proven to be reliable for use in design, operation, and maintenance of the Department's highway systems. This chapter provides the Department's practices and an explanation of these methods. Designers should see the *References* at the end of this chapter for other publications that offer a more thorough explanation of the background and theory of these methods.

4.1.1.1 Acceptable Design Methods

The designer should use the hydrologic method that is consistent with the characteristics of the basin under consideration. See Table 4.1 and Table 4.2 below for more information on hydrologic methods.

Table 4.1 Typical Applications of Acceptable Hydrologic Methods

Application	Hydrologic Methods			
	Rational Method	NRCS TR-55 Method*	USGS Equations	Small Storm Hydrology
Post Construction				X
Channel Protection	X	X		
Overbank Flood Protection		X	X	
Extreme Flood Protection		X	X	
Storage Facilities		X		
Outlet Control Structures	X	X	X	
Gutter Spread	X			
Storm Drain Pipes	X			
Culverts	X	X	X	
Bridges		X	X	
Small Channels	X	X	X	
Natural Channels		X	X	
Energy Dissipation	X	X	X	

*NRCS TR-55 method shall be calibrated to USGS regression equations when used.

Table 4.2 Limitations for Hydrologic Methods

Method	Watershed Area Limitation
Rational	0 - 200 acres
NRCS TR-55 Method	Usually < 2,000 acres and has hydrologic homogeneity
USGS Urban Regression Equations	See most current USGS publication
USGS Rural Regression Equations	See most current USGS publication

*USGS regression equations should not be used to calculate peak flow in basins less than 200 acres.

Existing Information: Hydrologic studies resulting in established flow rates for given design flood events have been completed for many locations in Alabama. These studies have many forms and provide valuable information. Some sources of these studies include the following:

- Flood Insurance Studies (FIS) – FEMA link: (http://www.fema.gov)
- Floodplain Information Reports – USACE
- Local community drainage master plans
- Nearby local projects completed by other entities

Published Flow Records: The designer should use published flow records when available. Flow records are typically collected on larger watersheds, and therefore, this approach in defining design peak discharge is more commonly used for bridge and large culvert projects. A minimum record of 10 consecutive years should be used to provide a reasonable statistical model.[4-9] This flow data can be gathered from a variety of agencies, such as:

- USGS – USGS data for Alabama can be found at the following website: http://waterdata.usgs.gov/al/nwis/nwis
- FEMA Effective FIS data can be found at the following website: https://msc.fema.gov/portal

Statistical analysis is used for estimating the design peak discharge for the gauged site and for nearby sites on the same stream.

Peak annual stream flows are measured primarily for streams with significantly large drainage areas or for streams that are located in hydrologically sensitive areas. Where peak stream flow is measured, the data can be statistically fit to a frequency distribution to estimate peak flow rates for flood events with specific recurrence intervals. "Guidelines for Determining Flood Flow Frequency Bulletin 17C" [4-9] establishes the Log-Pearson Type III frequency distribution as the base statistical method to analyze an annual series of peak flow rates. Manual computation using computer programs such as the U.S. Geological Survey's Statistical Software Package (PeakFQ), or websites such as https://pubs.er.usgs.gov/publication/tm4B5 can be used to complete these calculations of peak flow rates.

Regional Evaluation: Peak stream flow records have also been used together with known basin characteristics to produce generalized peak flow rate equations applicable to all streams within similar physiographic regions. Four such regions are delineated for Alabama as shown in Figure 4.1. The USGS developed regression equations 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. Separate equations are used for large and small basins that are primarily rural and for those that are primarily urban. A watershed is considered urban if it has 20 percent or greater developed area within the basin.[4-4] Refer to Section 4.1.3.1 for further information regarding regional evaluation using the USGS equations for Alabama.

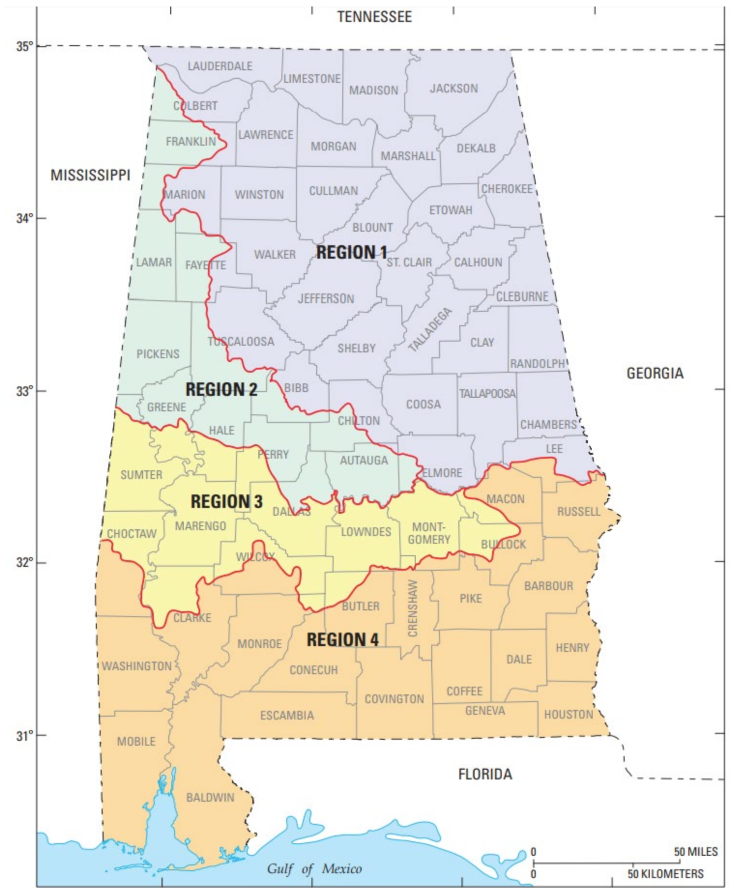

Figure 4.1 - Alabama flood frequency region map [4-4]

Regional regression equations are used to estimate the peak flow rates. USGS reports [4-3,4-4] describe these regression equations, which vary in applicability by region and can be used for various drainage area ranges (see Section 4.1.3.1 for area limitations by region).

Rational Method: The rational method was developed for estimating the peak flow rates and can be used for 2-year to 200-year rainfall events in small urban drainage basins. This method is recommended for use in basins with drainage areas up to 200 acres in size. This method estimates a peak discharge.

NRCS TR-55 Method: The TR-55 method provides simplified procedures to calculate hydrographs, particularly in urbanizing areas based on NRCS (formerly Soil Conservation Service (SCS)) procedures. The peak discharges should be calibrated to regression equations when used. If the drainage area is less than the regression equations lower limit then the model can be extended and calibrated downstream to where it is within the equations limits. Sub-basins may be used to determine discharges further into the upper part of the watershed when extending the model. This method is fully described in the National Engineering Handbook, Part 630, Hydrology (NEH630).[4-2] TR-55 can be used on basins up to 2,000 acres in size as long as the drainage basin is hydrologically homogeneous. Because larger basins are less likely to be hydrologically homogeneous, basins over 2,000 acres should be carefully examined before using this method. The latest version of TR-55 should be used and is available at https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf.

4.1.1.2 Design Discharge Criteria

Design frequency for the Department's roadway drainage facilities is based on achieving a balance between construction cost, maintenance needs, amount of traffic, potential flood hazard to adjacent property, and expected level of service. The design frequencies presented in Table 4.3 are the minimum that will achieve this balance for the various road classifications and types of drainage facilities.

Drainage structures should be designed on the basis of the design frequencies in Table 4.3 such that they shall not increase the flood hazard for upstream or downstream properties.

The design frequency for a given flood event is the reciprocal of the probability that a flood event will be equaled or exceeded in a given year. For example, if a flood event has a 10 percent chance of being equaled or exceeded in a year, the flood event will probably be equaled or exceeded on average every 10 years. The designer should note that the 10-year flood event will not be equaled or exceeded once every 10 years, but has a 10 percent chance of being equaled or exceeded in any given year. Therefore, the 10-year flood event could conceivably occur in consecutive years, or possibly even more frequently. See all design frequencies for a given flood event below:

2-Year – 50 percent chance of being equaled or exceeded in any given year

5-Year – 20 percent chance of being equaled or exceeded in any given year

10-Year – 10 percent chance of being equaled or exceeded in any given year

25-Year – 4 percent chance of being equaled or exceeded in any given year

50-Year – 2 percent chance of being equaled or exceeded in any given year

100-Year – 1 percent chance of being equaled or exceeded in any given year

200-Year – 0.5 percent chance of being equaled or exceeded in any given year

500-Year – 0.2 percent chance of being equaled or exceeded in any given year

Table 4.3 Design Flood Criteria for Culverts

Type	Item	ADT	Return Frequency Years	Check Frequency
Interstate & state highways	Bridge and roadway culverts	NA	50	200
Interstate & state highways	Cross drain pipes and dissipators	NA	50	200
Interstate & state highways	Median ditches, inlets & storm drains	NA	50	200
Interstate & state highways	Lateral ditches[1], inlets[2] & storm drains[2]	NA	10	25
County/municipal collector or local road[3]	Bridge or roadway culverts or cross drain	1-99[3]	1.5 to 25	5 to 50
County/municipal collector or local road[3]	Bridge or roadway culverts or cross drain	100- 399[3]	10 to 25	25 to 50
County/municipal collector or local road	Bridge or roadway culverts or cross drain pipes	400+	25	50

[1] Slope paved ditches should be designed for at least a 50 year return frequency because the liner can be lost if the ditch is overtopped.

[2] Use check storm for design at underpasses and depressed sections where water can only be removed through the storm drain system. In areas where water can spill over the back of the curb or gain relief by another means, analyzing the check storm may not be necessary.

[3] Design flood should be commensurate with the type of road and risk the County/Municipality desires.

4.1.1.3 Design Flood Characteristics

Stream-flow measurements for determining a design flood frequency relationship at a site are generally unavailable. Therefore, peak runoff rates and hydrographs can be estimated using statistical or empirical methods. The design discharge should be reviewed for other structures over the stream, historical data, and previous studies including FIS. The design discharge that best reflects local project conditions should be used, with the reasons documented.

Peak-runoff rate for the design condition is adequate for conveyance structures such as storm drains, open channels, or culverts. However, if the design must include flood event routing for detention, retention, post-construction stormwater ponds, or any other attenuating structure or system, then a hydrograph for the storm event will be required.

Drainage structure design is based on peak flow rate. Methods described in Section 4.1.3 include procedures for estimating the peak flow rate.

Volumetric runoff rate is depicted as a hydrograph with discharge in cubic feet per second plotted against time. The area under the curve is the volume of flow. Published flow records include data for the actual hydrograph experienced which can be of value in identifying volume. However, this information would likely require adjustment to provide the specific temporal, spatial, and frequency characteristics that are needed. The TR-55 method provides a simplified tabular method to compute the runoff volume. The USGS regional regression equations can also be used to produce flood hydrographs.

Certain data are required prior to using many of the hydrologic methods presented in this manual. The following is a description of the typical data required to begin a hydrologic study and how to obtain it.

4.1.2 General Design Data

Size of the Drainage Basin: The drainage area can be determined from field surveys, USGS topographic maps, or geospatial information.

Slope: The slope of the drainage area can be determined from field surveys, USGS topographic maps, or geospatial information.

Land Use: Land use conditions can be determined by field surveys, aerial photography, or geospatial information.

Soil and Geological Data: 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.

Rainfall: The amount, spatial distribution, and duration for various frequency rainfall events for Alabama are described in NOAA Atlas 14 Volume 9 and are published through Precipitation Frequency Data Server (https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html).[4-5]

The Rational Method Runoff Coefficient: The runoff coefficient C value reflects land use, soil type, and slope. The C value can be thought of as a factor used to compute the effective drainage basin area. It is directly related to the percent imperviousness. The higher the C value, the higher the runoff rate.

The NRCS Curve Number (CN): The NRCS CN value also reflects land use, soil type, and slope. In addition, the CN value also includes the hydrologic soil group and hydrologic condition. Like the Rational C value, a CN value is directly related to percent imperviousness.

4.1.3 Peak Flow Determination Procedures

4.1.3.1 Regional Evaluation

The designer should first check to see if the drainage basin or any portion of it is gauged. Where there are published flow records within the drainage basin, the recorded hydrologic data should be used.

Rural Regression

For rural ungauged drainage basins, regression equations are used to determine peak flow rates. The equations are based on watershed and climate characteristics within each of the four hydrologic regions in Alabama. The drainage area limitations on each region are shown below:

Region 1	0.94 to 1027 square miles
Region 2	0.13 to 1766 square miles
Region 3	0.34 to 1097 square miles
Region 4	0.69 to 1650 square miles

Note: The regression equations are updated periodically; be sure to use the most current equations.

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 in Alabama, 2015[4-3]

The referenced USGS equations are applicable for rural ungauged sites with drainage basin areas meeting the guidelines of the most recent publication for any given hydrologic region. 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. (4-3)

Small Streams

Small stream regression equations are available for determining peak flow rates and are suggested to be limited on streams up to fourteen square miles in drainage area. They are especially recommended in drainage areas less than five square miles and should be used where appropriate. The equations outlined in the latest version of the USGS publication *Magnitude and frequency of floods in Alabama, 2015*(4-3)

Urban Regression

Regression equations are also available for determining peak flow rates in urban areas from 1 square mile to 43 square miles with greater than 20 percent developed areas and should be used where appropriate. The equations outlined in the latest version of the USGS publication *Magnitude and frequency of floods for urban streams in Alabama, 2007*(4-4) should be used for urban calculations.

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 three sets of regression equations are updated periodically; be sure to use the most current equations.

4.1.3.2 The NRCS TR-55 Method

The TR-55 method is also used to estimate peak discharge. This method is primarily used for the design of post-construction stormwater BMPs, although it can be used for other calculations as well. Documentation and computer programs (WINTR55) for completing calculations using this method can be located at www.wcc.nrcs.usda.gov or from the two following links:

- https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf
- https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1042901

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), USACE Gridded Surface Subsurface Hydrologic Analysis (GSSHA) model with AQUAVEO's Watershed Modeling System (WMS).

4.1.3.3 The 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. This produces a peak rate of runoff, which remains constant as long as the rain continues at the same rate. The rational method may be used for areas up to 200 acres. As the drainage area gets larger, the assumptions related to time of concentration and a uniformly distributed rainfall occurring at a constant rate begin to break down.

The formula for the rational method is depicted below in Equation 4.1:

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

Where:

Q = Peak rate of flow (ft^3/s)
C = Runoff coefficient, the ratio of runoff to total rainfall (dimensionless)
I = Average rainfall intensity for a duration equal to the time of concentration (in/hr)
A = Drainage area (acres)

Runoff Coefficient: The runoff coefficient, C, in the rational formula is a ratio expressing the total precipitation that becomes stormwater runoff. Selecting the runoff coefficient for a drainage area requires careful engineering judgment by the designer. The runoff coefficient is a function of the land use, ground slope, topography, rainfall infiltration rate into the soil, and other factors. Table 4.4 gives applicable values for runoff coefficients.

The runoff coefficient should never be greater than 0.95 except for water-covered surfaces.

Where the drainage area is a composite of several land use types, a weighted runoff coefficient is calculated by using the following equation:

$$C_{weighted} = \frac{C_1 A_1 + C_2 A_2 + \ldots + C_n A_n}{A_{total}} \tag{4.2}$$

Table 4.4 Rational Method Runoff Coefficients

	Soil Texture		
Topography and Vegetation	Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland			
Flat 0-5% Slope	0.10	0.30	0.40
Rolling 5-10% Slope	0.25	0.35	0.50
Hilly 10-30% Slope	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82
	30% of Area Impervious	50% of Area Impervious	70% of Area Impervious
Urban Areas			
Flat	0.40	0.55	0.65
Rolling	0.50	0.65	0.80
Soil characteristics should be determined through field investigation or by consulting the Soil Conservation Service soil surveys.			
All water tight roof surfaces			0.75 – 0.95
Asphalt pavements			0.80 – 0.95
Concrete pavements			0.70 – 0.90
Gravel of Macadam pavements			0.35 – 0.70
Ponds and Lakes			0.95 – 0.98
Densely built up area where streets, walks and yards are paved And the remaining area is practically all roof area as in downtown districts			0.75
Areas adjacent to downtown district where streets and alleys are paved and yards are small			0.70
Densely built up residential district where streets are paved and houses are close together			0.65
Ordinary residential areas			0.55 – 0.65
Areas having small yards and medium density population			0.45 – 0.55
Sparsely built up areas or those having large yards			0.35 – 0.45
Suburbs having gardens and large lawns and with paved streets			0.30
Parks, golf course, etc., covered with sod and having no pavement			0.20
During selection of the above coefficients, consideration is to be given to future development.			

Time of Concentration: The time of concentration, T_c, is the time required for stormwater runoff to travel 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 land, land use, rainfall intensity, and how the runoff is conveyed. There are many equations used for computing Tc. The Department uses the Kirpich Formula for calculating T_c for the Rational Method. In this method, the time of concentration is directly dependent upon "L", the distance in feet from the most hydraulicly distant point in the drainage area along a flow path and the discharge point and "H" the difference in elevation in feet between the two points. Note that L is measured along the path of flow, even if this is a meandering ditch. The relationship is expressed by the empirical formula:

$$Tc = 0.0078\ (L^{1.5}/H^{0.5})^{0.770} \tag{4.3}$$

Often there are two or more flow paths in a drainage area which may seem reasonable to give a time of concentration. When that occurs, both are checked, and the one that gives the longest time is used.

Many factors can affect Tc. Concrete gutters, multiple types of surfaces, highly varied slopes, and surface storage of water (ponds, swamps, etc.) affect time of concentration and thus affect maximum runoff. Flow modifications have been developed for the Tc to account for these affects and can be found in Table 4.5.

Table 4.5 Flow Modification Factors for Tc

Condition	Factor
1. Natural basins with well-defined channels, overland flow on bare earth, mowed grass roadside	1.0
2. Overland flow on grassed surface	2.0
3. Overland flow on concrete or asphalt surfaces	0.4
4. Flow in concrete channels	0.2

Rainfall Intensity: In the Rational Method, rainfall intensity, I, depends on storm duration. The designer can determine rainfall intensity, I, for a computed duration and desired frequency by using the NOAA Atlas 14 website: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html. At the top left of the page next to "Data type" there is an option to select "Precipitation intensity." Also, next to "Time series type", "Partial Duration" should be selected. The user selects the location in the map and the intensities are generated in a table with frequency and duration. These can be plotted manually and plotted on the webpage by clicking the "PF Graphical" tab. It is recommended to use the plotted curve on the website for a check only, due to its scale and resolution. Using the computed Tc, an intensity can be picked off of the graph. Alternatively, a computer program can be used to compute the intensity to be used in the equation.

The Department’s Design Bureau Hydraulic Section has developed an acceptable alternate for computing discharges using a spreadsheet for the Rational Method and can be found on the Department’s website. An example using this method can be found in Appendix G.

4.1.4 Hydrograph Types and Development

A partial list of different types of hydrographs includes the following:

1. Natural hydrographs obtained directly from stream gauge data.
2. Synthetic hydrographs obtained from watershed parameters and storm characteristics to simulate natural hydrographs.
3. A natural or synthetic unit hydrograph for 1 inch of direct runoff occurring uniformly over the entire watershed from a storm of a specified duration. The direct-runoff volume is determined, and the ordinates of the direct-runoff hydrograph are divided by the observed runoff in inches.
4. Dimensionless unit hydrograph (See USGS Alabama Hydrograph in Appendix G), which eliminates the effect of basin size and much of the effect of basin shape. The hydrograph is made dimensionless by expressing the ordinate (vertical axis) values as the ratio of discharge to peak discharge and the abscissa (horizontal axis) values as the ratio of the time to time-to-peak.

4.1.5 Other Relevant Hydrologic Information

One of the most common methods to develop a hydrograph is based on the NRCS curve number method. Many standard hydrology textbooks and references detail the application of this method. A simplified tabular hydrograph method is provided in TR-55. Other complex watersheds require the use of computer programs such as the NRCS WinTR20, USACE’s HEC-HMS or GSSHA. A list of approved hydrologic programs can be found on the Design Bureau Hydraulic Section’s website.

For sites affected by regulation from dams 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).

Helpful tidal site: http://tbone.biol.sc.edu/tide/

4.2 Hydraulics

4.2.1 Introduction

Basic concepts and general equations for gravity flow (open-channel) and pressure flow (closed-conduit) will be briefly discussed in this section. Further discussions on gravity and pressure flow follow in Chapters 5 and 7.

Since these concepts are elementary in nature and their derivations are not shown here, refer to applied hydraulic textbooks or to FHWA publications for additional information.

4.2.2 General

The design of drainage structures requires the use of the continuity, energy, momentum, and other equations. These equations were derived on the basis of fundamental equations by a combination of mathematics, laboratory experiments, and field studies.

4.2.3 General Flow Classification

Flow can be classified as either gravity (non-pressure) or closed-conduit (pressure) flow. Gravity flow can then be further defined as: (1) uniform or non-uniform flow; (2) steady or unsteady flow; and (3) subcritical (tranquil) or supercritical (rapid) flow. Likewise, closed-conduit flow can be further defined as either steady or unsteady flow; and either laminar or turbulent flow.

Whether fluid flow is laminar or turbulent depends on surface roughness of the conveyance and a dimensionless number called the Reynolds number, Re, which is the ratio of inertial forces to viscous forces. This number is defined mathematically as:

$$Re = \frac{\rho \times D \times V}{\mu} \tag{4.4}$$

Where:

V = velocity, ft/s
D = diameter of conveyance, ft
ρ = fluid density, lbm/ft^3
μ = fluid viscosity, lbf s/ft^2

Depending on surface roughness, laminar flow generally occurs when the Reynolds number is less than 2,100. Turbulent flow generally occurs when the Reynolds number is above 4,000, except for extreme smooth materials. A transitional zone exists between 2,100 and 4,000.

4.2.4 Basic Principles

The basic equations of flow are continuity, energy, and momentum. They are derived from the laws of (1) the conservation of mass; (2) the conservation of energy; and (3) the conservation of linear momentum. Conservation of mass is another way of stating that (except for mass-energy interchange) matter can neither be created nor destroyed. The principle of conservation of energy is expressed in the Bernoulli Equation which states that energy must at all times be conserved in flowing fluids. The principle of conservation of linear momentum is based on both Newton's second law of motion and third law which states that a mass (of fluid) accelerates in the direction of and in proportion to the applied forces on the mass.

Analysis of flow problems are much simplified if there is no acceleration of the flow or if the acceleration is primarily assumed to be in one direction, which is considered one-dimensional flow. Equations given in the manual are written specifically as they apply to the analysis of one-dimensional flow and not two-dimensional or more complex fluid flow.

4.2.4.1 Continuity Equation

The continuity equation is based on conservation of mass. For steady flow of incompressible fluids, it is:

$$V_1 A_1 = V_2 A_2 = Q = VA \text{ or alternatively } Q_{in} = Q_{out} \tag{4.5}$$

Where:

V = Average velocity in the cross-section perpendicular to the area, ft/s
A = Area perpendicular to the velocity, ft^2
Q = Volume flow rate or discharge, ft^3/s

This form of the continuity equation is applicable when the fluid density is constant, the flow is steady, there is no significant lateral inflow or seepage (or they are accounted for), and the velocity is perpendicular to the area (Figure 4.2).

For unsteady flow, conservation of mass requires that the net rate of fluid mass flow into any elemental control volume be equal to the time rate of change of fluid mass storage within the element, and the continuity equation takes the following form:

$$Q_{in} - Q_{out} = dS/dt \tag{4.6}$$

Where:

Q_{in} = Volumetric fluid flow into the control volume, ft^3/s
Q_{out} = Volumetric fluid flow out of the control volume, ft^3/s
dS = Volumetric change in fluid mass storage, ft^3
dt = Change in time across control volume, s

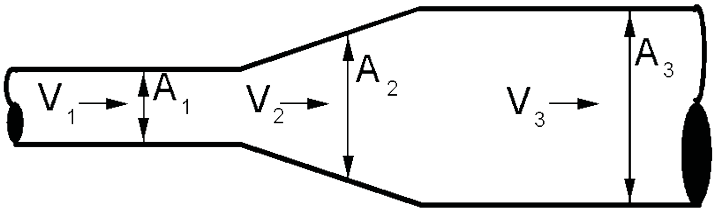

Figure 4.2 - Sketch of continuity concept through a control volume element

4.2.4.2 Energy Equation

The energy equation, in terms of the three components of total head, is derived from the first law of thermodynamics, which states that energy is a conserved physical quantity. The three head components in Equation 4.7 are the velocity head (h_v), the pressure head (h_p), and the elevation head (h_z). The head loss (h_L) equals the amount of energy lost and converted into thermal energy. Equation 4.7 represents a fluid state for steady incompressible flow and is shown as:

$$h_{v1} + h_{p1} + h_{z1} = h_{v2} + h_{p2} + h_{z2} + h_L$$

or (4.7)

$$\frac{V_1^2}{2g} + \frac{p_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + Z_2 + h_L$$

Where:

V = Average velocity in the cross section, ft/s
g = Acceleration of gravity, 32.2 ft/s^2
p = Pressure, lbs/ft^2
γ = Specific weight of water, 62.4 lbs/ft^3
Z = Elevation above a horizontal datum, ft
h_L = Head loss due to friction and form losses, ft

The energy grade line (EGL) is a representation of the total specific energy, shown as the elevation that equals the sum of the h_v, h_p, and h_z, the total head. The hydraulic grade line (HGL) is below the EGL by the amount of the velocity head, or is the sum of just the pressure and elevation heads. The application of the energy equation in gravity and pressure flow is illustrated in Figures 4.3 and 4.4.

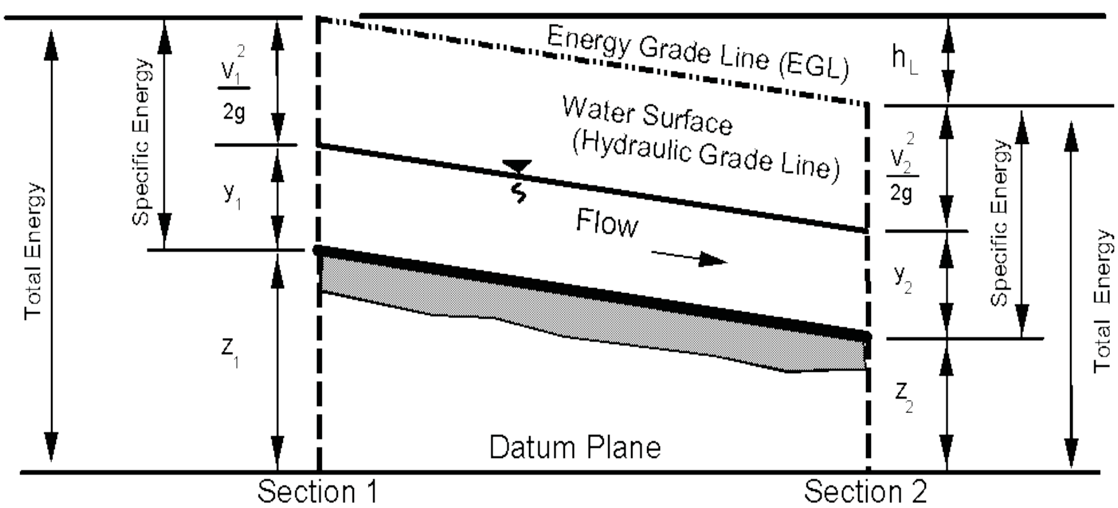

Figure 4.3 - Gravity flow (open-channel)

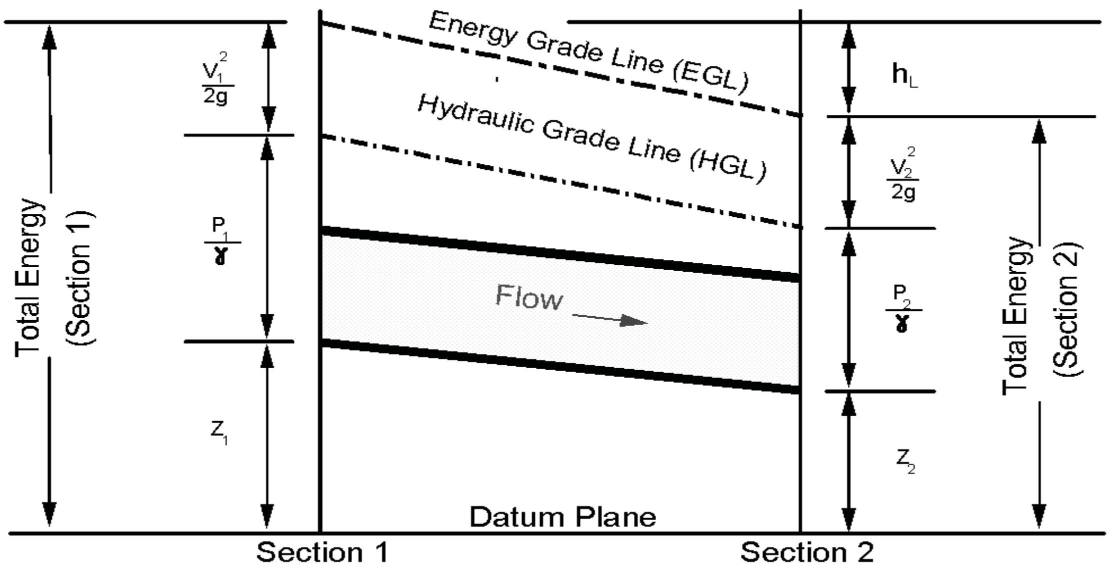

Figure 4.4 - Pressure flow (closed-conduit flow)

Since highway stormwater pipe joints are not designed to be watertight under pressure, the HGL should not exceed the pipe crown if practicable. When the HGL rises above the crown of the pipe at an upstream structure, the storm system becomes hydraulically surcharged. Similarly, if an open-channel flow condition in a storm drain is supercritical, care must be taken to ensure that a hydraulic jump does not occur which might also create a hydraulically surcharged scenario with the HGL above the roadway elevation.

4.2.4.3 Momentum Equation

The momentum equation is derived from Newton's second law which states that the summation of all external forces on a system is equal to the change in momentum (the impulse). In the x-direction for steady flow with constant density, it is:

$$\sum F_x = \rho Q (V_{x2} - V_{x1}) \quad (4.8)$$

Where:

F_x = Forces in the x direction, lbs
ρ = Density, 1.94 slugs/ft^3
Q = Volume flow rate or discharge, ft^3/s
V = Velocity in the x direction, ft/s

The momentum equation is used to estimate forces on pipe bends and to analyze hydraulic jumps.

4.2.5 Weirs and Orifices

4.2.5.1 Weirs

A weir is typically a notch of regular shape (rectangular, square, or triangular), with a free surface. The edge or surface over which the water flows is called the crest. A weir with a crest where the water springs free of the crest at the upstream side is called a sharp-crested weir. If the water flowing over the weir does not spring free and the crest length is short, the weir is called a not sharp-crested weir, round-edge weir, or suppressed weir. If the weir has a horizontal or sloping crest sufficiently long in the direction of flow that the flow pressure distribution is hydrostatic it is called a broad-crested weir (Figure 4.5). As with orifices, weirs can be used to measure water flow. Strictly speaking, a sharp-crested weir used for measurement purposes, must be aerated on the downstream side and the pressure on the nappe downstream must be atmospheric. Examples of weir flow that are of interest to the highway engineer are flow into grates, flow spilling through curb inlets, flow into culverts, outlet structures for detention basins, and flow-over approach embankment.

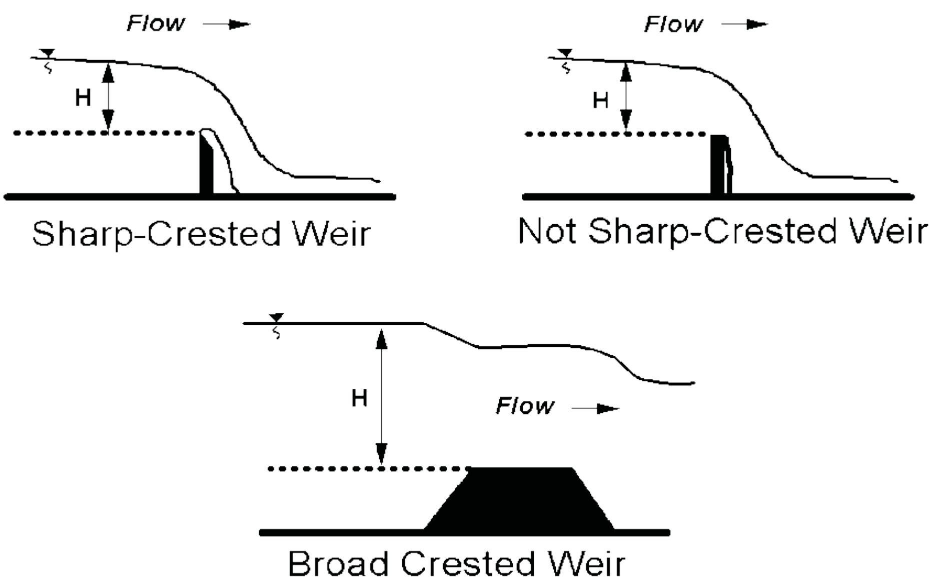

Figure 4.5 - Weir types

The discharge across a weir (sharp-crested or broad-crested) is calculated using Equation 4.9 below:

$$Q = C_D \times L \times H^{3/2} \tag{4.9}$$

Where:

Q = Discharge, ft^3/s
C_D = Coefficient of discharge for weirs, sharp-edge or broad-crested
L = Weir length (equal to the width of the bottom of the crest), ft
H = Head on the weir, ft (depth of flow above the weir crest measured upstream at the normal depth)

Roadway overtopping is modeled as broad-crested flow because the weir length will be greater than one-half of the head. The equation of flow is the same as Equation 4.9, but the coefficient of discharge is a function of weir length and head height. The coefficient normally ranges from 2.63 to 3.33.

Coefficients of discharge are given in most handbooks (e.g., HEC-22, HDS-5) for the different types of weirs and flow conditions. Note that correction factors are also available if the weir is submerged.[4-1,4-7] As long as the tailwater is less than critical depth, submergence is not a factor.

4.2.5.2 Orifices

An orifice is an opening with a regular shape (e.g., circular or rectangular) through which water flows in contact with the total perimeter. If the opening is flowing only partially full, the orifice operates as a weir. An orifice with a sharp upstream edge is called a sharp-edged orifice. If the jet of water from the orifice discharges into the air, it is called a free discharge. If it discharges under water, it is called a submerged orifice. Orifices are common fluid discharge measuring devices (Figure 4.6), but orifice type flow occurs under other circumstances where head loss, backwater, etc. needs to be determined. Examples of orifice flows of interest to highway engineers are flow through bridges when they are overtopped, flow through culvert inlets, curb inlets flowing full, etc. When a bridge is overtopped the flow through the bridge is orifice flow, but the flow over the bridge is weir flow.

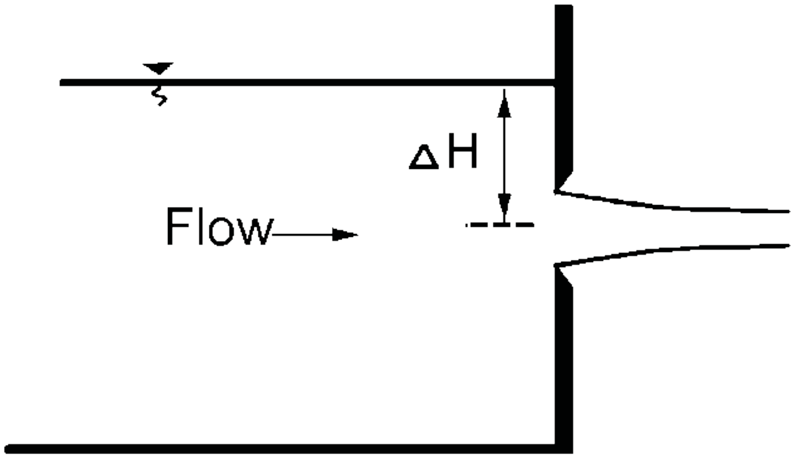

Figure 4.6 - Orifice

The discharge through an orifice is calculated using Equation 4.10 below:

$$Q = C_D A \sqrt{2g\Delta H} \qquad (4.10)$$

Where:

Q = Discharge, ft^3/s
C_D = Coefficient of discharge, 0.62 for a sharp-edged orifice
A = Area of the orifice, ft^2
g = Acceleration of gravity = 32.2 ft/s^2
ΔH = Difference in head across the orifice, ft

Coefficients of discharge are given in most handbooks.[4-1] For an unsubmerged orifice, the difference in head across the orifice is measured from the centerline of the orifice to the upstream water surface. For a submerged orifice, the difference in head is measured from the upstream water surface to the downstream water surface.

4.2.6 Open-Channel Flow

4.2.6.1 Introduction

Open-channel flow, or gravity flow, occurs when the water surface is at atmospheric pressure, which creates a free surface. It occurs in open channels such as curb and gutters, roadside channels, streams, and rivers. Open-channel flow also occurs in closed conduits that are not flowing full such as storm drains and culverts. All of the basic equations apply to open-channel flow: continuity, energy, and momentum equations. Open-channel flow, however, is more complex than closed-conduit flow since the cross-sectional flow area is not constant. The water surface may vary from steady uniform flow conditions to rapidly varied flow situations, from one-dimensional flow to two- and three-dimensional flow, and from steady to unsteady flow. Each of these flow variations adds complexity to the analysis of open-channel flow.

4.2.6.2 Detailed Flow Classification

The classification of gravity flow is summarized as follows:

Steady flow occurs when the flow velocity and depth at any given location does not vary with time.

1. Uniform flow occurs when flow velocity and depth do not change along a channel with a constant slope and cross section. This flow type rarely occurs in natural channels.

2. Varied flow occurs when the flow velocity and depth changes along a channel due to a change in channel slope, cross section, or roughness. Varied flow consists of two types:

   a. Gradually varied flow – changes occur slowly in flow for longer channel distances.

   b. Rapidly varied flow – changes occur faster due to short channel distances and transitions.

The steady, uniform flow case and the steady, non-uniform flow case are the most fundamental types of flow treated in highway engineering hydraulics. For the design of most highway drainage structures, steady flow is often assumed and will be the basis of the discussion in the section. However, the engineer must confirm that this assumption is reasonable. For structures in tidally influenced areas, this basic assumption may not be valid, and a more appropriate analysis may be required. For these situations, contact the Department's Hydraulic Group.

4.2.6.3 Manning's Equation

Uniform flow exists when the gravitational energy resulting from the longitudinal channel slope is balanced with the losses due to friction between the wetted perimeter and the boundary of the channel. Therefore, the slope of the water surface, channel bed, and the energy grade line are parallel. Numerous equations have been developed to analyze this flow condition. The one most commonly used by highway engineers was developed by Robert Manning. Equation 4.11 follows:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{4.11}$$

Where:

V = Mean velocity, ft/s
n = Manning's coefficient of roughness, dimensionless
R = Hydraulic radius, ft
S = Slope, ft/ft

The hydraulic radius, R, is a measure of hydraulic efficiency that depends on the shape of the channel and depth of flow. Of all cross-sectional shapes, the circular shape is the most hydraulically efficient. Moreover, the maximum rate of discharge under gravity flow in a circular pipe with a fairly constant n-value occurs when the flow depth is 94% of the pipe's diameter. The hydraulic radius is given by Equation 4.12:

$$R = A / P \tag{4.12}$$

Where:

A = Area perpendicular to flow, ft^2

P = Wetted perimeter, ft

When the Manning's equation is combined with the continuity equation, Equation 4.13 is then used to compute discharge:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{4.13}$$

Note that Manning's equation is valid also for pressure flow, but other equations, such as the Darcy-Weisbach equation, are preferred.

For gravity flow, Manning's equation is strictly applicable only to uniform flow. Even though uniform flow is rarely attained in highway stormwater infrastructure, uniform flow is assumed, and Manning's equation is usually used for steady gradually varied flow where the change in velocity from section to section is very small. The error by assuming uniform flow is small in comparison to the error in determining the design discharge.

Individual structures may be constructed of several materials with varying Manning's n-values. Embedded culverts are a common example when the sides of the culvert are constructed of concrete and the bottom is embedded in natural streambed material. In this case, a weighted Manning's n-value should be calculated.

Several programs, including Hydrologic Engineering Center's River Analysis System (HEC-RAS), and HY-8 will calculate a weighted average n-value directly. In the absence of computer aid, the designer will need to calculate the average n-value by hand.

Several methods are available for calculating the average n. The methods all have one thing in common: they are all some form of a finite series that involves the summing of terms. Hand calculation of the average n-value varies from being extremely tedious to being relatively simple depending on the method used.

As shown in HDS-5, Horton's method, Equation 4.14, uses the length of wetted perimeter as the weight.

$$n = \left(\frac{\sum_{i=1}^{n} (p_i n_i^{1.5})}{p} \right)^{0.67} \quad (4.14)$$

Where:

n = Weighted Manning's n-value
p_i = Wetted perimeter of material i, ft
n_i = Manning's n value for material i
p = Total wetted perimeter, ft

In the case of an embedded culvert, the formula can be reduced to the following as shown in Equation 4.15:

$$n = \left(\frac{(p_b n_b^{1.5})(p_s n_s^{1.5})}{p} \right)^{0.67} \quad (4.15)$$

Where:

p_b = Wetted perimeter of the bottom of the culvert, ft
n_b = Manning's n-value for the bottom of the culvert
p_s = Total wetted perimeter of the sides and the top (if applicable) of the culvert, ft
n_s = Manning's n-value for the sides of the culvert
P = Total wetted perimeter, ft

4.2.6.4 Froude Number

The Froude Number is a very important parameter in open-channel flow. It is an index of flow regime: subcritical, critical, or supercritical and is defined as the ratio of the inertial forces to the gravitational forces, normally expressed as shown in Equation 4.16 below:

$$\mathrm{Fr} = \frac{V}{\sqrt{gy}} \quad (4.16)$$

Where:

Fr = Froude Number, dimensionless
V = Velocity of flow, ft/s
g = Acceleration of gravity, ft/s^2
y = Hydraulic depth of flow, ft

If the channel is rectangular, the hydraulic depth is simply the depth "d." For trapezoidal and circular channels, y = A/T, the flow area "A" divided by the top width "T". In general, the hydraulic depth is the flow area divided by the top width of flow.

V and y can be the mean velocity and depth in a channel or the velocity and depth in the vertical. If the former is used, then the Froude Number is for the average flow conditions in the channel. If the latter are used, then it is the Froude Number for that vertical at a specific location in the cross section. The Froude Number uniquely describes the flow pattern in open-channel flow.[4-6]

$$c = \sqrt{g y_o} \tag{4.17}$$

Note that the denominator of the Froude Number is the same as the celerity of a shallow water wave of small amplitude (the velocity of the wave relative to the velocity of the flow, shown in Figure 4.7).

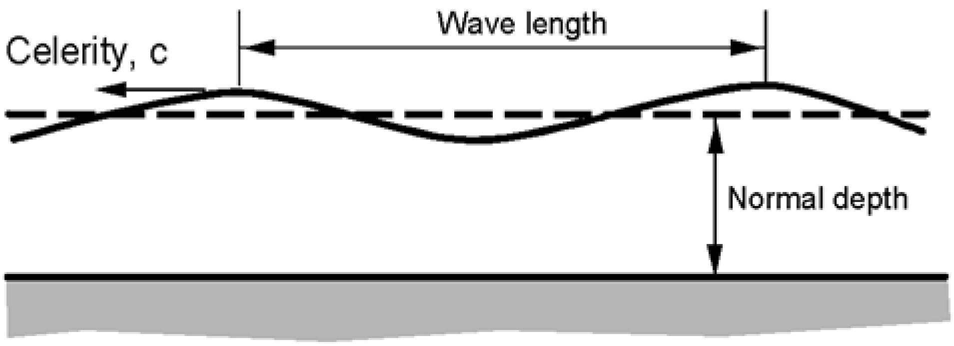

Figure 4.7 - Definition sketch for small amplitude waves

When the velocity of the flow is less than the celerity (speed of the wave with respect to fluid with which it is travelling) of the wave, a small amplitude wave resulting from a disturbance will move upstream, and the Froude number will be less than one (Fr < 1). This type of flow regime is subcritical or tranquil flow. In other words, the effects of a downstream flow disturbance will propagate upstream.

When the velocity of the flow is greater than the celerity of the wave, the effect of a flow disruption will not be carried upstream, and the Froude number will be greater than one (Fr > 1). This type of flow regime is supercritical or rapid flow.

The fact that waves (or surges) cannot move upstream when the Froude Number is greater than 1.0 means the stage discharge relation at a cross section cannot be affected by downstream conditions.

If the velocity of flow is the same as the celerity of the wave, the wave will be stationary, and the Froude number will be one (Fr = 1). This flow regime is called critical flow, and the depth of this flow is the critical depth. Flow going from supercritical to subcritical must pass through the critical depth in what is called a hydraulic jump. In a hydraulic drop the flow goes from subcritical to supercritical and again passes through the critical depth.

4.2.6.5 Specific Energy Diagram and Evaluation of Critical Depth

If the elevation head is removed from the energy equation, the sum of the two remaining terms is called the specific energy, or specific head, H, defined as:

$$H = \frac{V^2}{2g} + y = \frac{q^2}{2gy^2} + y \tag{4.18}$$

Where:

H = Specific energy, ft
q = Unit discharge, defined as the discharge per unit width ($ft^3/s/ft$) in a rectangular channel
V = Velocity, ft/s
g = Acceleration of gravity, 32.2 ft/s^2
y = Depth of flow, ft

The specific energy, H, is the height of the total energy above the channel bed. The relationship between the three terms in the specific energy equation, q, y, and H, are evaluated by holding the discharge constant and by examining the relationship between H and y in the specific energy diagram. For any given discharge, there are two flow depths that have the same specific energy: a deep, low velocity flow called subcritical and a shallow, high velocity flow called supercritical. These diagrams for a given discharge or energy are then used in the design or analysis of transitions or flow through bridges. They are explained in the next two sections.

For a given q, Equation 4.18 can be solved for various values of H and y. When y is plotted as a function of H, Figure 4.8 is obtained. There are two possible depths called alternate depths for any H larger than a specific minimum. Thus, for specific energy larger than the minimum, the flow may have a large depth with small velocity or small depth with large velocity. Flow for a given unit discharge q cannot occur with specific energy less than the minimum. The single depth of flow at the minimum specific energy is called the critical depth, y_c, and the corresponding velocity, the critical velocity, $V_c = q/y_c$. The relation for y_c and V_c for a given q (for a rectangular channel) is shown as Equation 4.19:

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \frac{V_c^2}{g} \tag{4.19}$$

Note that for critical flow, Equations 4.20 and 4.21 are:

$$\frac{V_c}{\sqrt{gy_c}} = 1 = Fr \tag{4.20}$$

and

$$H_{\min} = \frac{V_c^2}{2g} + y_c = \frac{3}{2} y_c \tag{4.21}$$

Thus, flow at minimum specific energy has a Froude Number equal to 1. Flows with velocities larger than critical (Fr > 1) are called rapid or supercritical and flow with velocities smaller than critical (Fr < 1) are called tranquil or subcritical.

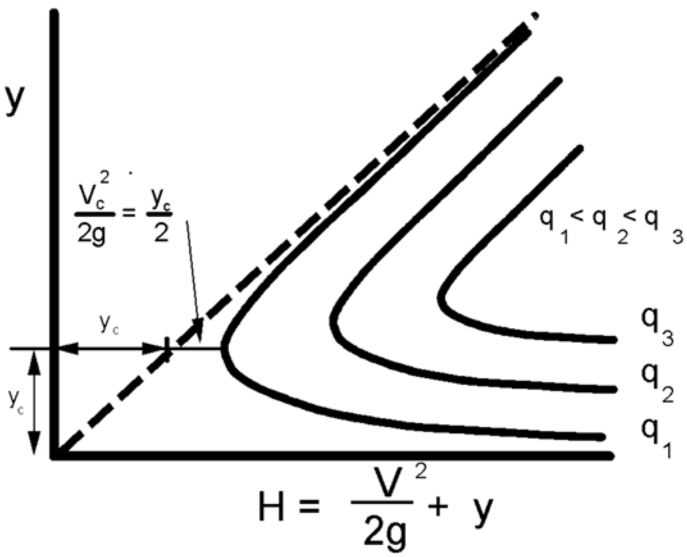

Figure 4.8 - Specific energy diagram

Distinguishing between the types of flow and how the water surface reacts with changes in cross section is important in channel design; thus, the location of critical depth and the determination of critical slope for a cross section of given shape, size, and roughness becomes necessary. Equations for direct solution of the critical depth are available for several prismatic shapes; however, some of these equations were not derived for use in the metric system.

For any channel section, regular or irregular, critical depth may be found by a trial-and-error solution of the following equation:

$$\frac{A_c^3}{T_c} = \frac{Q^2}{g} \tag{4.22}$$

where: A_c and T_c are the area and top width at critical flow. An expression for the critical velocity (V_c) of any cross section at critical flow conditions is:

$$V_c = \sqrt{g\, y_c} \tag{4.23}$$

where: $y_c = A_c / T_c$ (4.24)

Uniform flow within about 10% of the critical depth is unstable and should be avoided in design. As the flow approaches the critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific head curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design.

4.2.7 Closed-Conduit Flow

4.2.7.1 Types of Flow in Closed Conduits

Flow conditions in a closed conduit can occur as open-channel flow, full gravity flow, or pressure flow. The analysis of open-channel flow in a closed conduit is no different than any other type of open-channel flow and all the concepts and principles previously discussed are applicable. Full gravity flow occurs when the conduit is flowing full but not under any pressure greater than atmospheric. Pressure flow occurs when the conduit is flowing full and under a pressure greater than atmospheric.

Due to the additional wetted perimeter and increased friction that occurs in a full gravity pipe, a partially full pipe with a 94% depth will actually carry greater flow. The average velocity for a closed conduit flowing one-half full is the same as full gravity flow (Figure 4.9). Full gravity flow condition is usually assumed for purposes of storm drain design.

The Manning's equation combined with the continuity equation for a circular section flowing full can be rewritten as the following:

$$Q = \frac{0.46}{n} D^{\frac{8}{3}} S^{\frac{1}{2}} \tag{4.25}$$

Where:

Q = Discharge, ft^3/s
n = Manning's coefficient, dimensionless
D = Pipe diameter, ft
S = Slope, ft/ft

This equation allows for a direct computation of the required pipe diameter. Note that the computed diameter must be increased in size to a larger nominal dimension in order to carry the design discharge without creating pressure flow.

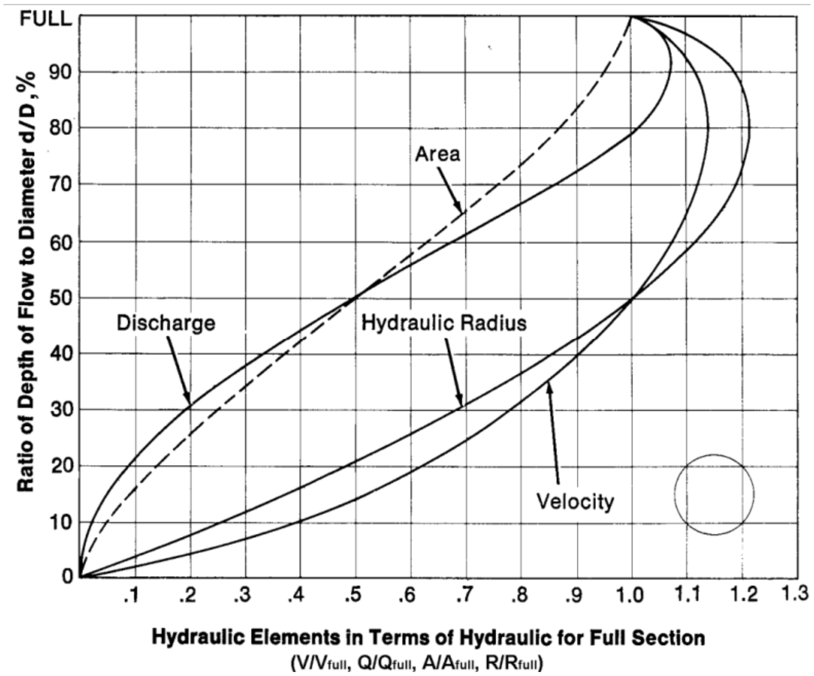

Figure 4.9 - Partially full flow relationships for circular pipes

4.2.7.2 Energy Losses

When using the energy equation, all energy losses should be identified. Energy losses can be classified as friction losses or form losses. Friction losses are due to forces between the fluid and boundary material, whereas form losses are the result of various hydraulic structures along the closed conduit. These structures, such as access holes, bends, contractions, enlargements, and transitions, will each cause velocity head losses and potentially major changes in the energy grade line and hydraulic grade line across the structure. The form losses are often called "minor losses," which is misleading since these losses can be large relative to friction losses.

R4 Chapter 4 References

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Chapter 5: Channels

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5.1 Introduction

An open channel is a natural or constructed conveyance for water in which the water surface is exposed to the atmosphere (free-surface flow), and gravity alone is the driving force.

Open channels associated with transportation facilities can be described by two main categories: natural stream channels and constructed channels, such as ditches or swales.

A natural stream channel is described as:

- A channel with its size and shape determined by means of natural forces
- A compound cross section with a main channel for conveying low flow and a floodplain to transport flood flow
- Geomorphologically shaped due to the long-term history of sediment load and water discharge which it experiences

A constructed channel can be a roadside channel, interceptor ditch, or drainage ditch which can have a regular geometric cross section and is unlined or lined with constructed or natural material to protect against erosion. Culverts or storm drains are also constructed conveyances where the principles of open-channel hydraulics are applicable during free-surface flow.

The purpose of this chapter is to:

- Establish design practices
- Specify design criteria
- Outline channel design procedures

This chapter is to be used as a tool that will aid the designer when approached with roadside or median channel design. In addition to roadside and median channel design topics, Section 5.3 provides guidance on stream channel analysis and design. Some of the stream channel topics introduced include stream morphology, cross sections, Manning's n values, calibration, one-dimensional gradually varied flow profile analysis, and a few special analysis techniques. However, for more information regarding stream studies, assessments of existing stream channels, or guidance on relocating a stream, the designer should refer to Chapter 10. In general, this chapter begins with a brief discussion on practices which is followed by an extensive discussion on open-channel hydraulics topics and concludes with roadside and median channel guidelines and criteria and design procedures.

The designer should consult other chapters of this manual, as appropriate, for additional information regarding open channels, including the following:

- Chapter 4 – Hydrology & Hydraulics
- Chapter 9 – Post-Construction Stormwater Design Guidelines
- Chapter 10 – Stream & Wetland Restoration Concepts
- Chapter 11 – Bridge Hydraulic Design Criteria

5.2 Design Practice

The purpose of open channel design is to provide a channel configuration that will convey the naturally occurring flow and/or design stormwater runoff through or adjacent to the transportation facility or through a BMP and back to its original course. See Section 5.4 for additional details on roadside and median channels. In general, the following guidance applies to all channel designs:

- Channel designs and/or designs of highway facilities that impact channels shall satisfy the policies of the FHWA applicable to floodplain management if federal funding is involved.
- FEMA floodway regulations and USACE permit conditions/regulations for wetland restrictions and stream impacts shall be satisfied.
- Coordination with other federal, state, and local agencies concerned with water resources planning shall have high priority in the planning of highway facilities.
- Safety of the general public shall be an important consideration in the selection of cross-sectional geometry of constructed drainage channels.
- The design of constructed drainage channels or other facilities shall consider the frequency and type of maintenance expected; and make allowance for access of maintenance equipment.
- A stable channel is the goal for all channels that are located on highway right-of-way or that impact highway facilities.
- Environmental impacts of channel modifications, including disturbance of fish habitat, wetlands, and channel stability shall be assessed. Channels should not be placed within the limits of delineated wetlands.
- For design storm event requirements, see Table 4.3 of Chapter 4 in this manual.

5.3 Open-Channel Hydraulics

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

- Changes in erosive potential resulting from changes in velocity magnitude and direction
- Need for channel lining to prevent erosion
- Post-construction assessment of infiltration potential / soil permeability of subsurface

This section will specifically discuss guidelines and design criteria applying to open-channel hydraulics for roadside and median channels and stream modifications. For more information, the designer should consult Chapter 4 of this manual which provides a general discussion of hydraulics with links to valuable references.

5.3.1 Types of Flow

Open-channel flow is generally classified using the following characteristics:

- Steady or unsteady
- Uniform or non-uniform (varied)
- Subcritical or supercritical

Of these, non-uniform, unsteady, subcritical flow is the most common type of flow in open channels. Due to the complexity and difficulty involved in the analysis of non-uniform, unsteady flow, most hydraulic computations are made with certain simplifying assumptions that allow the application of steady-uniform or gradually-varied flow principles and one-dimensional methods of analysis.

The use of steady flow methods implicitly assumes that the flow rate at a point does not change with time, and the use of uniform flow methods assumes that there is no change in velocity, magnitude, or direction with distance along a streamline. Steady-uniform flow is thus characterized by constant velocity and flow rate from section to section along the channel.

Steady-uniform flow is an idealized concept of open-channel flow that seldom occurs in natural channels and is difficult to obtain even in model channels. However, for most practical highway channel applications, the assumption of steady and uniform flow is often adequate for design purposes since changes in width, depth, or direction (resulting in non-uniform flow) is sufficiently small. The changes in channel characteristics occur over a long distance such that flow is gradually varied. For these reasons, use of uniform flow theory is usually within acceptable degrees of accuracy.

The designer must consider non-uniform and/or unsteady flow conditions in some instances, such as gradually-varied flow in spillways and receiving channels, rapidly-varied flow in energy dissipators (hydraulic jumps), and around bridge piers. Refer to Section 5.3.3.7 for more information on complex hydraulic modeling principles.

5.3.2 Manning's Equation for Mean Velocity and Discharge

Water flows in a sloping drainage channel because of the force of gravity. The flow is resisted by the friction between the water and wetted surface of the channel. As discussed in Chapter 4, the Manning's Equation is used to compute the mean velocity in an open channel with steady-uniform flow as shown in Equation 5.1:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \quad (5.1)$$

Where:

V = Mean velocity, ft/s
n = Manning's coefficient of channel roughness
R = Hydraulic radius (R = A/P), ft
S = Slope, ft/ft
A = Area, ft^2
P = Wetted perimeter, ft

When the Manning's equation is combined with the continuity equation, Equation 5.2 is then used to compute discharge, Q:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \quad (5.2)$$

Typical values of the Manning's n roughness coefficient for various channel types are given in Appendix D.

5.3.3 Stream Channel Analysis and Design

Stream channels are usually natural channels with their size and shapes determined by natural forces. Stream channels are also usually compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows. Rehabilitation of disturbed or relocated natural channels shall incorporate cross section geometry that will effectively convey the design frequency, minimize erosive forces, and provide sufficient floodway as required. See Chapter 10 for additional information regarding natural channel design requirements and analyses.

The analysis of a natural stream channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway bridge in such a manner that it will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining floodplain. The designer should refer to Chapter 10 for more information regarding stream studies, assessments of existing stream channels, or guidance on relocating a stream.

The following sub-sections cover the general guidance for stream channel analysis and design. See Section 5.4 for design information on engineered channels.

5.3.3.1 Stream Morphology

A study of the plan and profile of a stream is very useful in understanding stream morphology, or the form or shape of a stream. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. Conversely, a highway crossing or encroachment can inadvertently change a variable, adversely affecting the stream. Additional information can be obtained through FHWA publications, such as *HEC-20 Stream Stability at Highway Structures* and *HDS-6 River Engineering for Highway Encroachments.*

5.3.3.2 Cross Sections

In order to define how the natural flow of a stream is conveyed, hydraulic modeling is conducted with specific data requirements. One hydraulic data requirement includes cross sections. Cross sections provide the designer with factors such as channel depth, channel width, water surface elevation, bank failure, etc.

Cross sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible. In wide floodplains or bends, it may be necessary to use a section along intersecting straight lines, i.e., a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as for overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection. Selection of cross sections and the vertical subdivision of a cross section are shown in Figure 5.2.

5.3.3.3 Manning's n Value Selection

Hydraulic roughness is the measure of the amount of frictional resistance water experiences when passing over channels and flood plains. Manning’s n represents this resistance. These values have been calculated for various types of channels based on stream flow and are provided in the USGS Water-Supply Paper 2339, *Guide for Selecting Manning’s Roughness Coefficients for Natural Channels and Flood Plains.*[5-2] This publication can be found at https://pubs.usgs.gov/wsp/2339/report.pdf and is a useful tool that aids the designer in the determination of Manning’s n values. Pictures (Figure 5.1) are also provided in the document which offer visual representations of natural channels and floodplains Manning's n values. An example is provided below.

Figure 5.1 – Floodplain photograph with Manning's n value(5-2)

Manning's n values for constructed channels are more easily defined than for natural stream channels. Appendix D lists typical n values of both constructed channels and natural stream channels.

5.3.3.4 Calibration

The equations should be calibrated with historical high-water marks and/or gauged 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. Proper calibration is essential if accurate results are to be obtained.

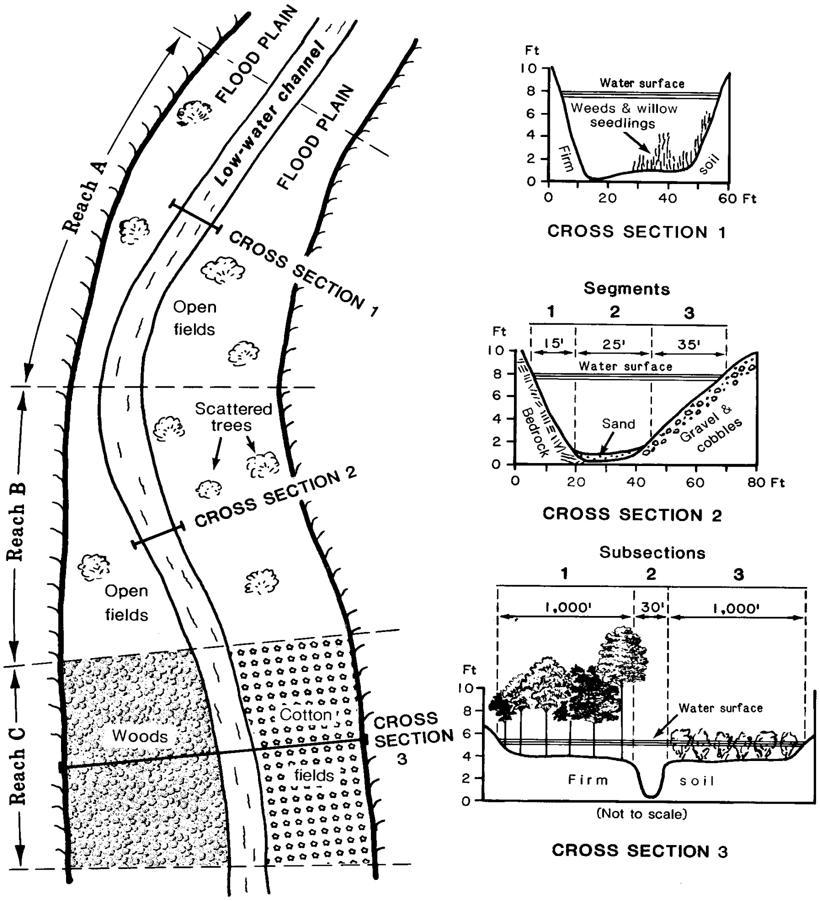

Figure 5.2 – Hypothetical cross section showing reaches, segments, and subsections used in assigning n values (5-2)

5.3.3.5 Slope-Area Method

A common method used for channel design is the slope-area method (also known as single-section method or slope conveyance method). It is simply a solution of Manning's equation for the normal depth of flow given the discharge and cross section properties including geometry, slope, and roughness (Manning's n value). This method implicitly assumes the existence of steady-uniform flow; however, uniform flow rarely exists in either constructed or natural stream channels. Nevertheless, the slope-area method is often used to design constructed channels for uniform flow as a first approximation and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. This relationship should cover a range of discharges up to at least the base (100-year) flood.

The stage-discharge curve can be determined as follows:

1. Select the typical cross section at or near the location where the stage-discharge curve is needed.

2. Subdivide cross section and assign n-values to subsections as previously described.

3. Estimate water-surface slope. Because uniform flow is assumed, the average slope of the streambed can usually be used.

4. Apply a range of incremental water surface elevations to the cross section.

5. Calculate the discharge using Manning's equation for each incremental elevation. Total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross section and not along the vertical water interface between subsections.

6. After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge should be made. This plot is the stage-discharge curve, and it can be used to determine the water surface elevation corresponding to the design discharge or other discharge of interest.

5.3.3.6 One-Dimensional Gradually-Varied Flow Profile Analysis

Another common method used for channel design is the standard step backwater method. This method employs the energy equation to determine the water surface profile along a roadside channel or stream channel during gradually-varied flow. In gradually-varied flow, which is a type of steady non-uniform flow, any changes in depth and velocity take place slowly over large distances. The resistance to flow dominates and acceleration forces are neglected under this type of flow. There are many different flow profile types for gradually-varied flow; the FHWA publication *Introduction to Highway Hydraulics (HDS-4)* (5-4) provides the background on flow profile types and the standard step method. The manual calculation process for the standard step backwater method is cumbersome and tedious for channels of any length or with numerous variations in cross section shape, roughness, slope, or discharge within the area of interest.

Thus, HEC-RAS, WSPRO or another acceptable computer program should be used to calculate water surface profiles when this method is required. (5-6)

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.* (5-6)

Water surface profile computation for the standard step method requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section; but, in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the slope area method or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the profiles do not converge, then the analysis may need to be extended downstream, a shorter cross section interval should be used, or the range of starting water surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 5.3).

Given a long enough stream reach, the water surface profile computed by the standard step method will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define the limits of data collection and subsequent analysis.

Calculations must begin sufficiently far downstream to assure accurate results at the structure site and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 5.4).

The USACE publication *Accuracy of Computed Water Surface Profiles* (5-5) provides equations for determining upstream and downstream reach lengths as follows:

$$L_{dn} = 8{,}000\ (HD^{0.8}/S) \tag{5.3}$$

$$L_u = 10{,}000\ [(HD^{0.6})(H_L^{0.5})]/S \tag{5.4}$$

Where the following occurs:

L_{dn} = Downstream study length (along main channel), ft (for normal depth starting conditions)

L_u = Estimated upstream study length (along main channel), ft (required for convergence of the modified profile to within 0.1 feet of the base profile)

HD = Average hydraulic depth (1% chance event flow area divided by the top width), ft

S = Average reach slope, ft/mi

H_L = Head loss ranging between 0.5 feet and 5 feet at the channel crossing structure for the 1% chance flood, ft

The USACE publication referenced above[5-5] and the USGS publication for navigable waterways, *Computation of Water Surface Profiles in Open Channels* [5-3] are valuable sources that provide additional guidance on the practical application of the standard step method to highway drainage problems involving open channels. These references contain more specific guidance on cross section determination, location and spacing, and stream reach determination. The USACE document [5-5] also investigates the accuracy and reliability of water surface profiles related to n value determination.

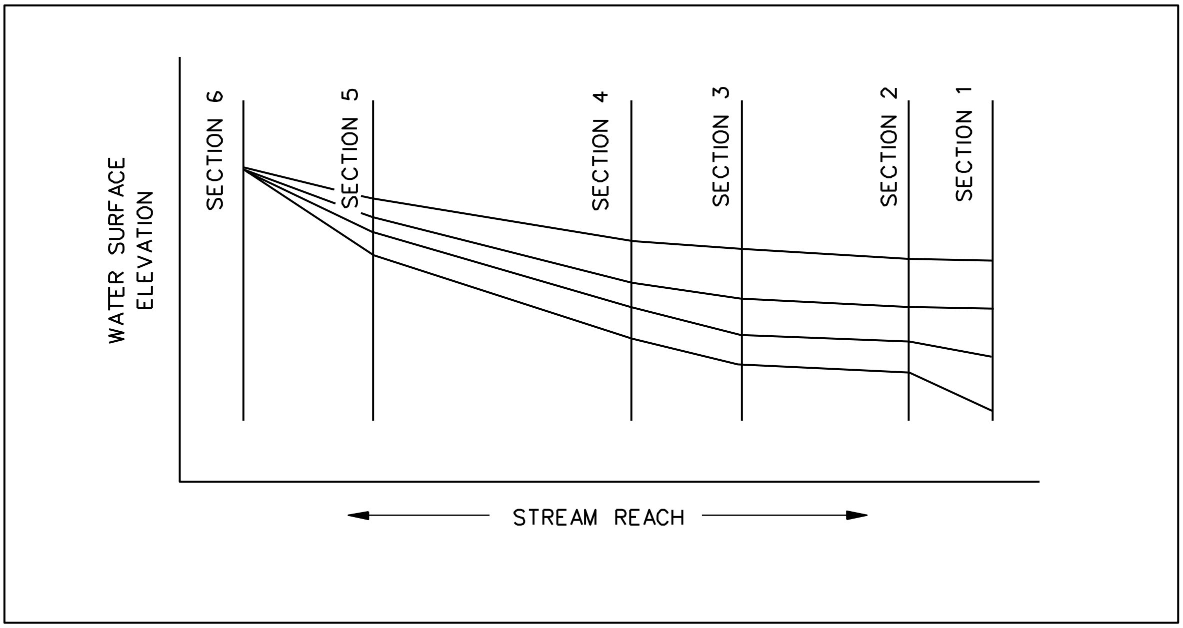

Figure 5.3 - Profile convergence pattern backwater computation

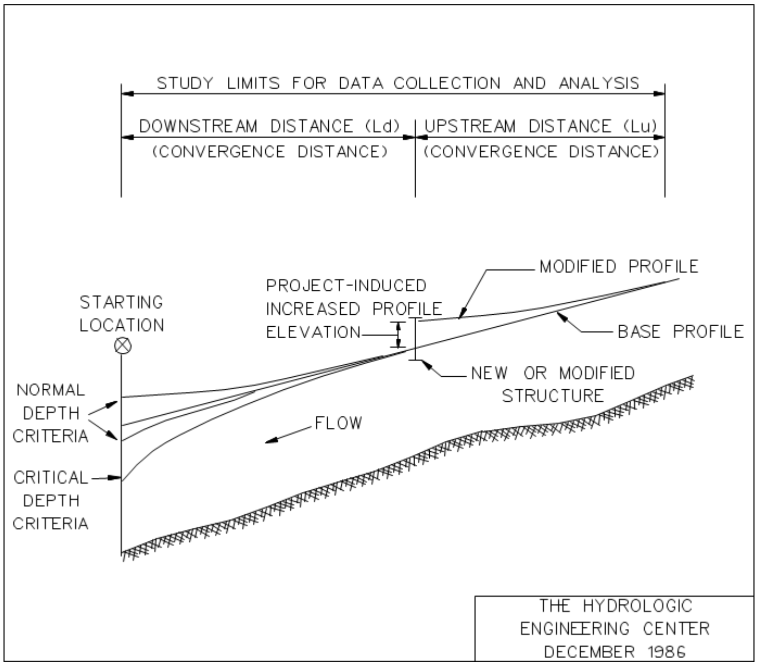

Figure 5.4 - Profile study limits

5.3.3.7 Special Analysis Techniques

Open-channel flow problems sometimes arise that require a more detailed analysis than a slope area method or the computation of a water surface profile using the standard step method. More detailed analysis techniques include two-dimensional analysis, water and sediment routing, and unsteady flow analysis. Computer programs are available for the analysis techniques discussed in this section.

5.3.3.7.1 Two-Dimensional Analysis

Two-dimensional (2-D) models simulate flow in two directions, longitudinal and transverse, at a series of user-defined node points. Flow in the vertical direction is assumed to be negligible. These models can account for transverse flow due to lateral velocities and water surface gradients that cannot be accounted for with one-dimensional models. Examples of such conditions include skewed bridges, floodplain crossings with multiple openings, channel bifurcation, flow around channel bends, and flow around islands.

A 2-D model should be considered for major projects with complex flow patterns that one-dimensional models cannot adequately analyze. Examples of situations where 2-D models should be considered are as follows:

- Wide floodplains with multiple openings, particularly on skewed embankments
- Floodplains with significant variations in roughness or complex geometry such as ineffective flow areas, flow around islands, or multiple channels
- Sites where more accurate flow patterns and velocities are needed to design better and cost-effective countermeasures such as riprap along embankments and/or abutments
- Tidally-affected river crossings and crossings of tidal inlets, bays, and estuaries
- High-risk or sensitive locations where losses and liability costs are high

Following are three commonly used computer programs for 2-D modeling:

FESWMS: *Finite Element Surface Water Modeling System version 3.3.3*

The FESWMS package consists of FST2DH that can model flows in open channels. FST2DH, is a 2-D finite element surface water computer program that computes the direction of flow and water surface elevation in a horizontal plane. FST2DH has the ability to model hydraulic structures commonly used by hydraulic engineers. FESWMS is usually recommended for highway crossings of rivers and floodplains because it supports both super and subcritical flow analysis and can analyze roadway overtopping, culverts, and bridges.

See http://www.xmswiki.com/wiki/SMS:FESWMS for information regarding FESWMS.

SRH-2D: *Sedimentation and River Hydraulics – Two-Dimensional Model Version 3.1*

SRH-2D is a hydraulic model developed by the U.S. Bureau of Reclamation that incorporates very robust and stable numerical schemes with a seamless wetting-drying algorithm. The model uses a flexible mesh that may contain arbitrarily shaped cells, both quadrilateral and triangular elements, which promotes solution accuracy while minimizing computing demand. SRH-2D modeling applications include flows with in-stream structures, through bends, with perched rivers, with side channel and agricultural returns, and with braided channel systems. SRH-2D is well suited for modeling local flow velocities, eddy patterns, flow recirculation, lateral velocity variation, and flow over banks and levees.

See http://www.xmswiki.com/wiki/SMS:SRH-2D or information regarding SRH-2D.

SMS: *Surface-water Modeling System Version 12.2*

Surface-water Modeling System (SMS) is a comprehensive user interface for one- and two- dimensional models dealing with surface water applications. The hydrodynamic models cover a range of applications including river flow analysis, rural and urban flooding, estuary and inlet modeling, and modeling of large coastal domains.

FESWMS and SRH-2D are modules included in SMS. For information regarding SMS see http://www.aquaveo.com/sms.

5.3.3.7.2 Unsteady Flow Analysis

One-dimensional, unsteady flow can be analyzed with the HEC-RAS computer program. Some of the features of HEC-RAS are the network simulation of split flow and combined flow. The effect of storage areas can also be analyzed. This feature is useful when the effects of a stream channel and/or overbank floodwater storage areas are sufficient to allow a significant reduction in peak rates approaching a drainage structure or series of structures.

This program can provide more realistic estimates of headwater produced at a series of closely spaced highway drainage structures. HEC-RAS allows the user to analyze lateral overflow into storage areas over a gated spillway, weir, levee, through a culvert, or a pumped diversion. The user can apply several external and internal boundary conditions, including flow and stage hydrographs, gated and controlled spillways, bridges, culverts, and levee systems. HEC-RAS can be an effective tool to analyze tidally-affected river crossings and crossings of tidal inlets, bays, and estuaries.

Two-dimensional, unsteady flow can be analyzed with either FESWMS or SRH-2D.

5.3.3.8 Switchback Phenomenon

If the cross section is improperly subdivided, the mathematics of the Manning's equation causes a switchback. A switchback results where the calculated discharge decreases with an associated increase in elevation (Figure 5.5). This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter.

Simultaneously, there is a corresponding small increase in cross sectional area that causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross sectional area, a discharge is computed that is lower than the discharge based upon the lower water depth. More subdivisions within such cross sections should be used to avoid the switchback (Figure 5.6).

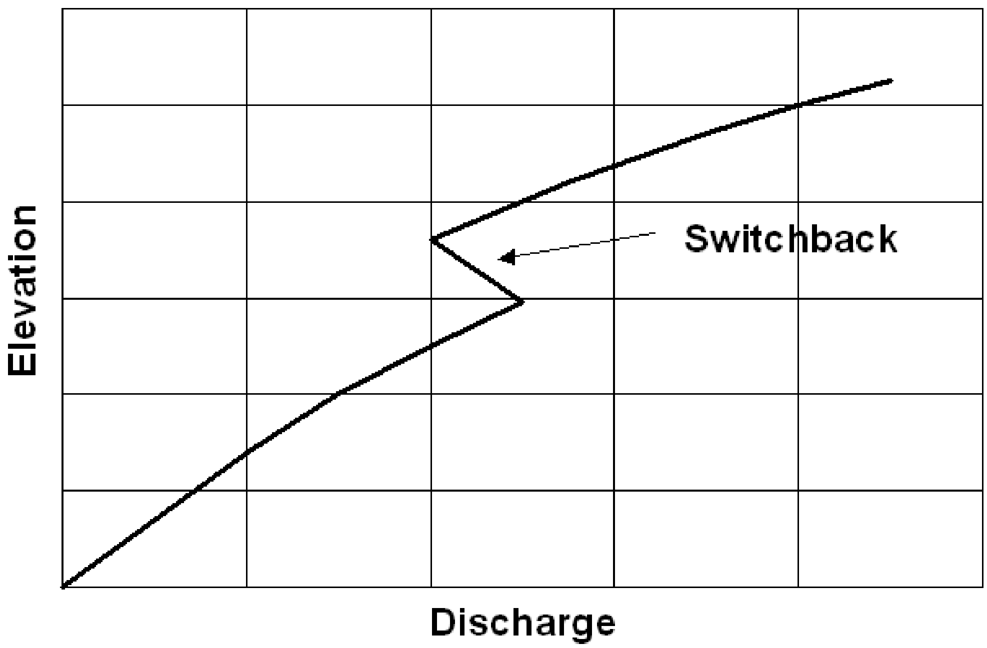

Figure 5.5 - Switchback phenomenon

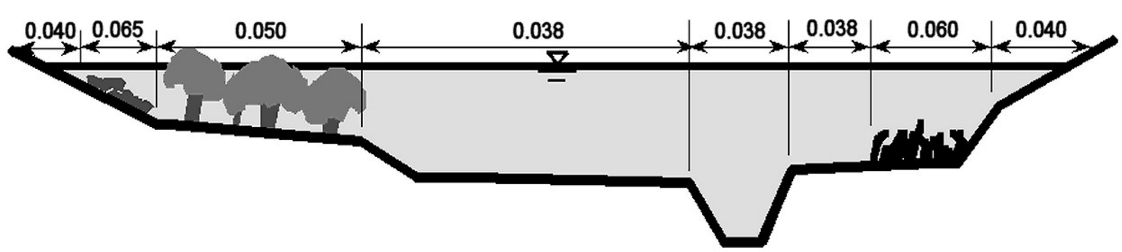

Figure 5.6 - Cross section subdivision

This phenomenon can occur in any type of conveyance computation, including the step backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross section being used in a step-backwater program. For this reason, the cross section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n value itself may be the same in adjacent subsections.

5.4 Roadside and Median Channel Guidelines and Criteria

5.4.1 Design Storms

Table 4.3 in Chapter 4 of this manual provides design storm frequencies for roadside channels. Roadside and median channel design should be based on the 50-year storm for interstate systems and arterials with full access control, while the roadside and median channel design for other facilities should be based on the 10-year storm. The channel should be provided with sufficient capacity that the design high water elevation will be below the bottom of the subgrade. 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 higher design storm frequency to provide additional protection for the subgrade of the roadway.

5.4.2 Channel Shape and Protection

Roadside channels are typically trapezoidal or V-shaped in cross section and lined with grass or other protective linings such as riprap. Refer to the "Elements of Channel Sections (p.9)" figure in the USDA National Engineering Handbook, Section 5 for typical geometries for various channel sections. The shape of a roadside channel is governed largely by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner that assures the safety of the motorist and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects. Section 5.6 addresses safety issues related to open-channel drainage facilities.

In order to minimize future maintenance, the channel should be designed with the appropriate channel properties and lining. Hydraulic Engineering Circular 15, Design of Roadside Channels with Flexible Linings (HEC-15), provides design guidance for straight channels, side slope stability, composite lining design, stability in bends, steep slope design, and maximum discharge approach for the design of channel lining types including bare earth, grass, concrete, erosion control products and others. Once the channel properties are known (shape, slope, and roughness) and discharges have been estimated, the designer can compute velocity and shear stress. These can be compared to known permissible shear stresses (Table 5.1). If the permissible shear stress is not exceeded then there should be no erosion. HEC-15 has coefficients to account for various vegetation types and their influence on soil shear stress. For other erosion control products, see manufacturer's permissible shear stress. The design process is iterative. Working through the design procedure in HEC-15, the properties can be modified to use a desired lining or vice versa. There are multiple programs available for performing these computations and performing lining design. Any lining design program should compare favorably with the results of HEC-15. If riprap is used for the lining, see Section 4.2.2 Design Guidelines for Riprap Revetment in HEC-23 Volume II.[5-9] The Hydraulic Toolbox program uses this method for riprap design.

Protective channel linings are an important aspect of any transportation project. Site conditions, long term maintenance, and cost should also be considered when choosing channel linings. Lining in a channel requires permanent or semi-permanent type erosion control measures to protect the channel from degradation. The most commonly implemented measures include grass channel lining, concrete channel lining, riprap channel lining, or turf reinforced mats (TRMs). Transitions between channels of dissimilar materials will also warrant protection from scour and erosion. For example, a concrete-lined channel transitioning to a vegetated channel would likely warrant a riprap-lined portion at the transition. It is the responsibility of the design engineer to check these areas for proper erosion control measures, both permanent and temporary. See Section 5.6 for safety of riprap lining use.

Table 5.1 Permissible shear stress for bare soil and stone linings (5-8)

		Permissible Shear Stress	
Lining Category	**Lining Type**	**N/m^2**	**lb/ft^2**
Bare Soil[1] Cohesive (PI = 10)	Clayey sands	1.8-4.5	0.037-0.095
	Inorganic silts	1.1-4.0	0.027-0.11
	Silty sands	1.1-3.4	0.024-0.072
Bare Soil[1] Cohesive (PI ≥ 20)	Clayey sands	4.5	0.094
	Inorganic silts	4.0	0.083
	Silty sands	3.5	0.072
	Inorganic clays	6.6	0.14
Bare Soil[2] Non-cohesive (PI < 10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	1.0	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	5.6	0.12
	Gravel D_{75}=15 mm (0.6 in)	11	0.24
Gravel Mulch[3]	Coarse gravel D_{50}=25 mm (1 in)	19	0.4
	Very coarse gravel D_{50}=50 mm (2 in)	38	0.8

[1]Based on Equation 4.6 of HEC-15 assuming a soil void ratio of 0.5
[2]Based on Equation 4.5 of HEC-15
[3]Based on Equation 6.7 of HEC-15 with Shield's parameter equal to 0.047

5.4.3 Channel Alignment

Roadside channels will parallel the roadway alignment and lie within the limits of the right-of-way of the roadway.

Changes in alignment should be as gradual as the right-of-way and terrain permit. Whenever practicable, changes in alignment should be made in sections with flatter grades where flow is subcritical.

5.4.4 Channel Grade

The following guidelines and design criteria should be followed when considering the grade required for a channel:

- The grade on surface channels at the top of cut slopes will be controlled primarily by the contour of the land. Surface channels should be constructed approximately 2-feet deep with low points draining into roadway channels by use of pipes down the back slope.

- The grade on grass-lined channels should be greater than 0.5% with the grade kept as constant as practicable. The grade on concrete lined channels may go as low as 0.3%.

- The grade affects both the size of the channel required to carry a given flow and the velocity at which the flow occurs. The flow should be kept subcritical wherever possible in order to minimize soil erosion.

- Alignment changes should be kept to a minimum for paved channels on steep slopes flowing in a supercritical flow regime.

5.4.5 Stream-Bank Protection from Erosion

Stream-bank stabilization shall be provided, when appropriate, as a result of any stream disturbance and shall include both upstream and downstream banks as well as the local site. The choice of stabilization used should be appropriate from an engineering and environmental aspect.

5.4.6 Typical Design Data Required

The following list includes data required for a typical design:

- Contour maps, quadrangle maps

- LIDAR data (if available)

- Field measured topography or digital terrain model (DTM)

- Stream profile and cross sections

- Soil survey and soil erosion index

- Drainage basin size and characteristics
- Rainfall intensity
- Determination of the design runoff volume or discharge
- C or CN Runoff Factors (C - Rational Method runoff coefficient; CN - SCS curve number)
- Available gauge data
- Regulatory flood data

5.5 Roadside and Median Channel Design Procedures

The primary function of roadside channels 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 of support for the pavement or to provide a positive outlet for subsurface drainage systems such as pipe underdrains.

Median channels perform the same functions as roadside channels and shall be designed using the same criteria. Basic design steps, as adapted from HEC-22, are as follows:

Step 1 Establish a conceptual roadway plan

- Collect available site data
- Obtain or prepare existing and proposed plan and profile layout

Step 2 Obtain or establish cross section data

- Provide channel depth adequate to drain subbase
- Select channel side slopes based on safety clear zone, economics, soil stability, and access
- Establish bottom width or shape of channel
- Identify features which may restrict cross section design, e.g., right-of-way constraints, environmentally sensitive areas, utilities, and existing drainage facilities

Step 3 Determine channel grades

- Plot initial grades on plan and profile layout

- Provide minimum grade of 0.5% to minimize ponding and sediment accumulation (if the channel is concrete lined the slope minimum may be 0.3%)

- Consider influence of grade on lining type.

Step 4 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. The desirable minimum allowable freeboard is 6 inches.

- 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, and/or provide smoother channel lining

- Provide smooth transitions at changes in channel cross section

Step 5 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

5.6 Safety

The Department hydraulic design criteria and practices found in each design chapter meet the primary responsibility for traffic safety which is to provide drainage structures which convey floodwaters and which avoid hazardous flooding and failure of the highway. Another important responsibility is to locate drainage structures so that they will present a minimum hazard to traffic.[(5-1)]

Drainage structures shall be located to present a minimum hazard to traffic and people or protected, if appropriate, using Department Construction Standards and Details.

Roadside channels that are outside of the clear zone can be designed with a trapezoidal cross section that has side slopes as steep as 2H:1V with appropriate erosion slope protection. If riprap is used for the slope protection and channel lining, it should only be used outside of the clear zone.

For applications within the clear zone, see AASHTO's Roadside Design Guide for additional information. Channels that are within the clear zone and are not screened by guard rail should be designed to be traversable using Figures 5.7 and 5.8. These figures show the AASHTO recommended foreslopes and backslopes for traversable channel configurations. Additionally channels should be designed to prevent ponded water exceeding 2 ft in depth during the design and check storm event. If this can't be avoided, it shall be considered as a hazard that warrants some protection.

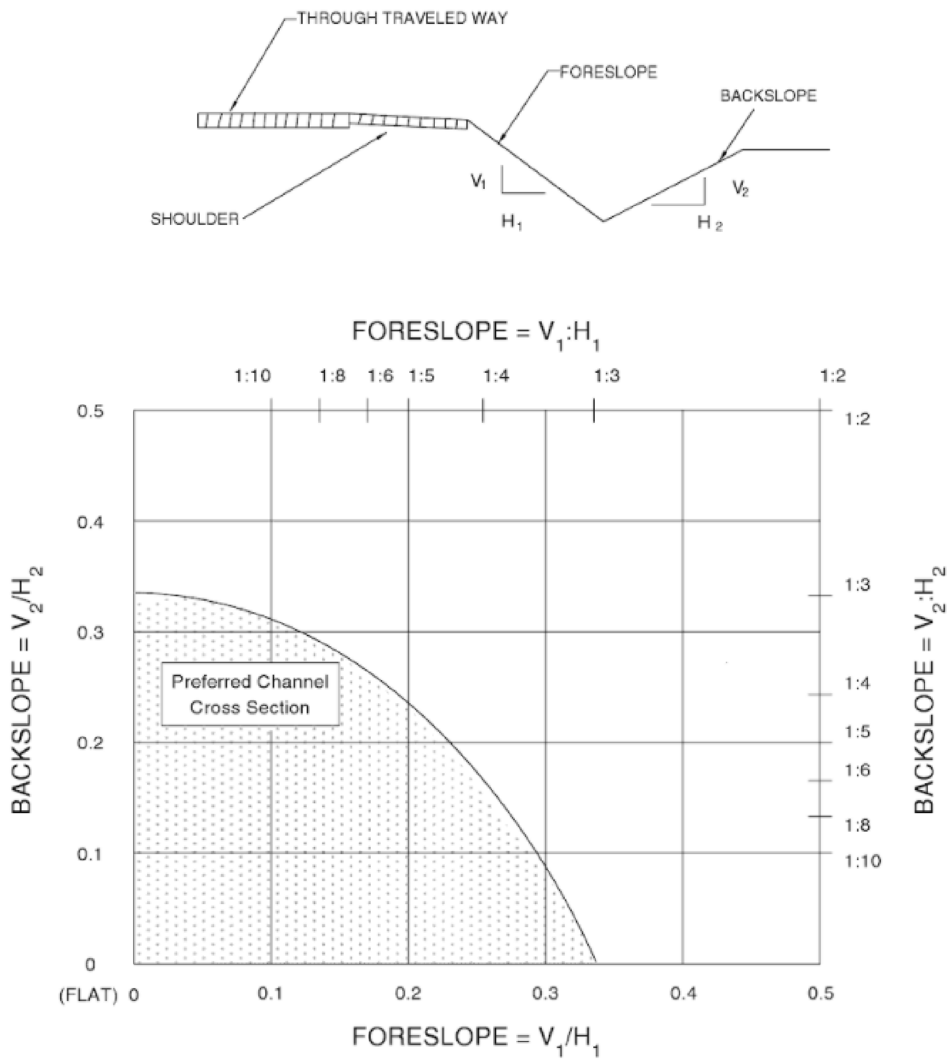

Figure 5.7 - Traversable channel geometry for Vee ditches, rounded channels with a bottom width less than 8 feet and trapezoidal channels with a bottom width less than 4 feet. Source: AASHTO *Roadside Design Guide*

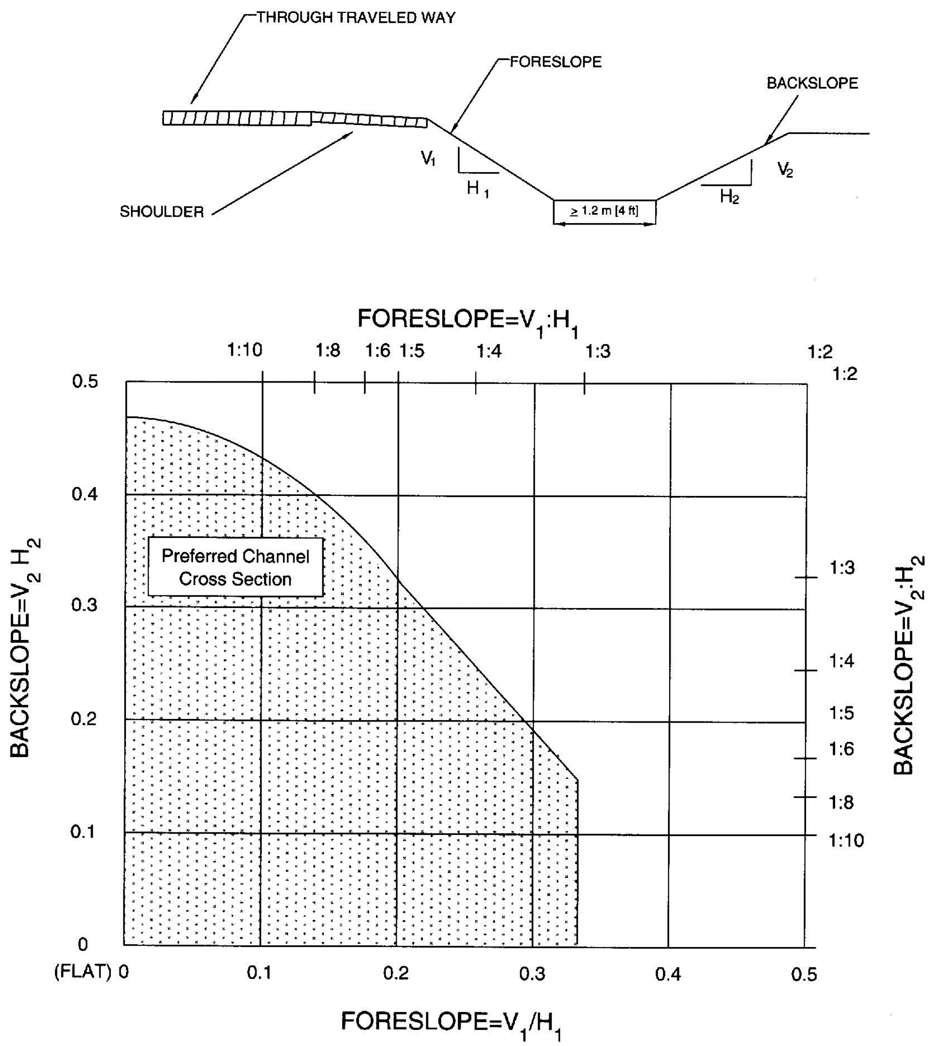

Figure 5.8 - Traversable channel geometry for rounded channels with a bottom width greater than 8 feet or trapezoidal channels with a bottom width equal to or greater than 4 feet. Source: AASHTO *Roadside Design Guide*

Channel sections that fall outside the shaded region of Figures 5.7 and 5.8 are not desirable and their use should be limited where high-angle encroachments might occur, such as the outside of relatively sharp curves. Channel sections outside the shaded region may be acceptable for projects with restrictive right-of-way, resurfacing, restoration, or rehabilitation (3R) construction projects, or on low-volume or low-speed roads, particularly if the bottom and backslopes do not have any fixed objects.

R5 Chapter 5 References

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Chapter 6: Pavement Drainage

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6.1 Guidelines

Good drainage design entails 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, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles.[6-3]

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

The guidelines included herein should be considered minimum standards. The designer should consistently strive to provide optimum and functional drainage facilities.

Plans, drainage profiles, and the quantities for the drainage system design should be developed in accordance with the current Department Roadway Plan Preparation Manual.

6.1.1 Introduction

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

- Longitudinal and cross slope
- Curb and gutter sections
- Pavement texture/surface roughness
- Roadside and median ditches
- Bridge decks

The pavement width, cross slope, profile and pavement 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.

6.1.2 Hydroplaning

Pavement drainage is an integral component in providing a safe roadway for the traveling public. An important part of that is removing water from the pavement to minimize the potential for hydroplaning. For additional details on the hydroplaning phenomenon, see FHWA's HEC-22.[6-4]

Hydroplaning is not evaluated as a standard project design procedure. Depending on the roadway characteristics, gutter spread calculations are sufficient. For areas where crash rates are escalated during wet weather conditions, hydroplaning calculations may be necessary. The designer should also be aware of the potential for hydroplaning in areas with zero super elevation in a crest or sag, turn lanes, median openings, and any other areas susceptible to problems.

Hydroplaning conditions can be evaluated based upon the relationships between the following primary controlling factors:

- Vehicular speed
- Tire conditions (pressure and tire tread)
- Pavement micro and macrotexture
- Roadway geometrics (pavement width, cross slope, grade)
- Pavement conditions (rutting, depressions, roughness)

Vehicular speed appears as a significant factor in the occurrence of hydroplaning; therefore, it is considered to be the driver's responsibility to exercise prudence and caution when driving during wet conditions. [6-2] This is analogous to the prudence and caution that drivers must exercise when ice or snow is on the roadway.

The following guidance is taken from FHWA *Hydraulic Engineering Circular No. 21.*[6-5] The prevention of hydroplaning is based on pavement and geometric design criteria for minimizing hydroplaning. An empirical equation for the vehicle speed that initiates hydroplaning is:

$$V_a = SD^{0.04}\, P_t^{0.3}\, (TD+1)^{0.06}\, A_T \qquad (6.1)$$

where A_T is a Texas Transportation Institute empirical curve fitting relationship. A_T is the greater of A_{T1} and A_{T2}, where

$$A_{T1} = \frac{10.409}{d^{0.06}} + 3.507 \;\; and \;\; A_{T2} = \left[\frac{28.952}{d^{0.06}} - 7.817\right] TXD^{0.14} \qquad (6.2)$$

Where:

V_a	=	Vehicle speed, mph
SD	=	Spindown (percent); hydroplaning is assumed to begin at 10% spindown. This occurs when the tire rolls 1.1 times the circumference to achieve a forward progress distance equal to one circumference.
P_t	=	Tire pressure, psi
TD	=	Tire tread depth (1/32 in)
d	=	Water film depth, in
TXD	=	Pavement texture depth, in

For given values of V_a, SD, P_t, TD, and TXD, Equations (6.1) and (6.2) can be solved simultaneously for film depth, d.

For example, given the following parameters, d is estimated to be 0.0735 in.

V_a = 55 mph

SD = 10% (by definition)

P_t = 27 psi (50 percentile level)

TD = 7/32 in (50 percentile level)

TXD = 0.038 in (mean pavement texture depth)

This value of d (0.0735 in) is suggested as a sound design value, since it represents the combination of the mean or median of all the above parameters. However, a designer could adjust the values of the target design parameters to match the anticipated prevailing road conditions for a given project.

For example, a designer might groove a deck to increase TXD, which would increase the water film depth, d, at which hydroplaning would be expected to occur at the design speed. Or, a designer might adjust the parameters to design for a smaller d at higher vehicle speeds. Multiple combinations of adjustments can be made to the parameters to control the design for hydroplaning.

Once a design d is determined, it is assumed that the thickness of the water film on the pavement should be less than d. Water flows in a sheet across the surface to the edge of the gutter flow. The length of sheet flow is designated as L_f. At the edge of the gutter flow, the design hydroplaning depth is d.

By combining the rational equation, the Manning's equation, and Equations (6.1) and (6.2), Equation 6.3 solves for the rainfall intensity that will cause hydroplaning.

$$i = \left[\frac{64904.4}{C\ n}\right]\left[\frac{S_x}{(S_x{}^2 + S^2)^{0.25}}\right]\left[\frac{d^{1.67}}{L_f}\right] \qquad (6.3)$$

Where:

C = Runoff coefficient from rational equation, (dimensionless)
n = Manning's coefficient for pavement, (.016)
S_x = Pavement cross slope (ft/ft)
S = Longitudinal slope, (ft/ft)
d = Design hydroplaning depth depending on speed, (ft)
L_f = Travel distance across the pavement for water flow, (feet)

Additional accepted methods by the Department for hydroplaning computations include a software program called HP, developed by the Florida Department of Transportation (FDOT) and University of South Florida. The program has two components:

1. A methodology to predict water film thickness (WFT) on the pavement being analyzed; and

2. A methodology to predict potential hydroplaning speed given the WFT determined.

The program offers four different formulas for calculating the WFT and three different formulas for predicting the potential hydroplaning speed. The engineer should use best judgment for applying the correct formulas for the specific site. The analysis tool also takes into account the observed speed reduction of motorists in a rain event based on rainfall intensity. Visibility is reduced when intensity exceeds 2 in/hr and becomes poor when intensity exceeds 3 in/hr.

The Hydroplaning Tools and Design Guidance can be downloaded at the Florida Department of Transportation's website at http://www.fdot.gov/roadway/Drainage/Manualsandhandbooks.shtm.

The rainfall intensity, related to hydroplaning, is independent of the storm event frequency.

Designers do not have control over all factors involved in hydroplaning. However, the following practical remedial measures should be considered by the designer during development of a project to reduce hydroplaning potential:

Pavement Sheet Flow

- Maximize transverse slope
- Maximize pavement roughness
- Use of graded course (porous pavements)
- Use of transverse grooves to reduce water depth
- Additional consideration should be given in superelevation transition areas

Gutter Flow

- Limit gutter flow (by decreasing inlet spacing)
- Maximize interception of gutter flow above superelevation transitions

Sag Areas: Limit ponding duration and depth.

Overtopping: Limit depth and duration of overtopping flow.

If suitable measures cannot be implemented to address an area of high potential for hydroplaning or an identified existing problem area, the installation of advance warning signs, although not common, could be considered as a last course of action.

The above measures are in accordance with Chapter 9 of the AASHTO Highway Drainage Guidelines.(6-2)

6.2 Gutter Spread and Design Storm Frequency

Following are two of the more significant variables that must be considered in the design of highway pavement drainage:

- The allowable gutter spread
- The frequency of the design storm event

Gutter spread and design storm frequency are interrelated variables.

6.2.1 Gutter Spread

Gutter spread is defined as the perpendicular distance from the face of curb or barrier to the furthest extent of the water on the roadway during the design storm (Figures 6.1 and 6.2).

Limiting the gutter spread width is a very important design criterion and will vary depending on the roadway classification and speed of traffic. Gutter spread shall be limited to the widths shown in Table 6.1.

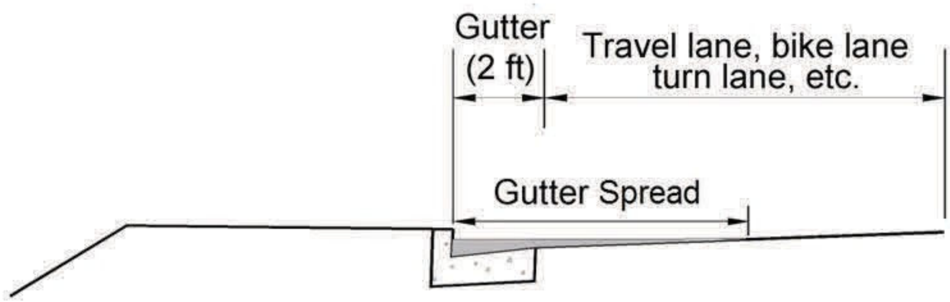


Figure 6.1 - Gutter spread in a typical urban section

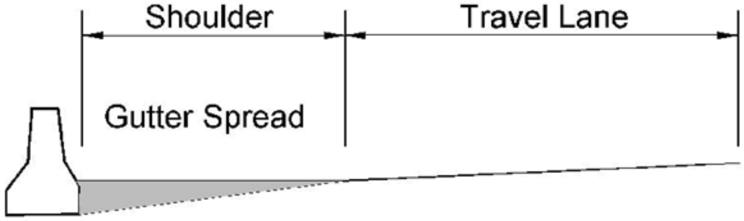


Figure 6.2 - Gutter spread confined to a shoulder on an interstate

6.2.2 Design Storm Frequency

Inlet spacing should be designed to accommodate the gutter spread limits given in Table 6.1.

Table 6.1 Pavement Drainage Design-Minimum Frequencies and Spreads Based Upon the Type of Highway and the Traffic Speeds

Road Classification	Design Frequency (Q-Years) 10	Shoulder or Parking Lane	Design Speed Partial Driving Lane (outside)
1. High Volume Divided Highway (with Median two or more travel lanes each direction)			
1a. Design Speed < or = 45 mph	x		½ lane width, Note 4
1b. Design Speed > 45 mph	x	x	Note 5
1c. Sag Point	x		Notes 2, 3, 4, 5
2. High Volume Bidirectional (with Median-two or more travel lanes each direction)			
2a. Design Speed < or = 45 mph	x		½ lane width, Note 4
2b. Design Speed > 45 mph	x	x	Note 5
2c. Sag Point	x		Notes 2, 3, 4, 5
3. Collector			
3a. Design Speed < or = 45 mph			
3a1. Two or more travel lanes each direction	x		½ lane width, Note 4
3a2. One lane each direction	x		¼ lane width
3b. Design Speed > 45 mph	x	x	
3c. Sag Point including flanking inlets	x		Notes 2, 3, 4, 5
4. Local Streets			
4a. Low ADT	x		Notes 1, 2, 5
4b. High ADT	x		Notes 1, 2, 5
4c. Sag Point	x		Notes 1, 2, 5

Note 1: The design speed for Local Streets can be larger or smaller (¼, ½, ¾ of a lane, etc.) based upon the designer's discretion after a review of the specific factors involved in the design. The sag point design (Q_{10}) should include the sag and flanking inlets only.

Note 2: The design width of spread for sag and flanking inlets (Q_{10}) should be the same width as for continuous grades (Q_{10}); therefore insuring consistency of design.

Note 3: The sag point design (Q_{10}) should include the sag and flanking inlets only and be designed as follows:

Design Speed	< 45 mph:	½ Driving Lane
Design Speed	= 45 mph:	Shoulder and Parking Area Only
Design Speed	> 45 mph	Shoulder and Parking Area Only

Note 4: In crowned or superelevated sections where the passing or high speed lane is near a curb, island, or barrier the design spread should be as follows:

Design Speed	< 45 mph	¼ Driving Lane
Design Speed	= 45 mph:	Shoulder and Parking Area Only
Design Speed	> 45 mph	Shoulder and Parking Area Only

Note 5: Generally, for depressed sections and underpasses on highways where water can be removed only through the storm sewer system, a 50-year frequency is used for a Design Storm.

The use of a less frequent event, such as a 100-year storm, to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.(6-4)

A check storm should be used any time runoff could cause unacceptable flooding. Inlets should always be evaluated for a check storm when a series of inlets terminates at a sag vertical curve where ponding to hazardous depths could occur. (6-4)

6.3 Gutter Flow

The basis for the gutter flow principles discussed in this chapter can be found in Section 4.3 of the HEC 22(6-4) manual.

Gutter flow calculations are necessary to relate the total quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane, or pavement section. For the purposes of this chapter, the term gutter refers not only to the typical 2-foot wide concrete gutter but, to the area covered by the water spread on the pavement. Two of the main components that influence gutter flow are the longitudinal and transverse (cross) slopes of the pavement. Longitudinal slope may also be referred to as gutter grade.

6.3.1 Longitudinal Slope - Gutter Grades

Longitudinal slope (grade) is important for curbed roadways (e.g., roadway shoulders with curb and gutter, v-gutters, concrete barrier walls, etc.) because stormwater runoff can accumulate and spread against the curb. It should be noted that flat slopes on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge. The gutter grade will be the same as the longitudinal grade on tangent sections; however, at superelevation transitions the gutter grade will vary from the roadway longitudinal grade, and the actual grade at the gutter should be used for spread and inlet efficiency calculations.

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 is accurately sloped and supported on firm subgrade.(6-1) Longitudinal grades less than 0.3% should be used only in extreme conditions such as increased road cross slope or decreased inlet spacing.

A minimum longitudinal grade of 0.3% should be reached 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. (6-1)

Where retrofitting and reconstructing existing roadways, in the event it is not possible to prevent short distances of zero grade, additional drainage inlets/trench drains may be considered.

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. (6-1)

K values greater than 167 (ft/%) may be required to provide a safe sight distance on crest vertical curves for design speeds greater than 60 mph. This may be of particular concern for night driving on highways without lighting. (6-1)

6.3.2 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 2% pavement cross slope for travel lanes. Cross slope should be increased to 2.5% in areas where an increase is practicable and justified. On multi-lane roadways, the cross slope may be broken at 0.5% intervals not to exceed 4% on any lane. Steeper cross slopes (4% maximum) should be considered for roadways draining more than three travel lanes in the same direction or in a 4-lane divided section where the gutter grade is less than 0.5%. Be sure to check superelevation transitions in the areas of cross slope less than 0.5%.

6.4 Gutter Flow Computations

In establishing the capacity of the gutter flow for a given width of spread, the type of gutter is important. Some of the more common types of gutters are shown in Figure 6.3 (gutter spread is shown as the variable “T” in Figure 6.3).

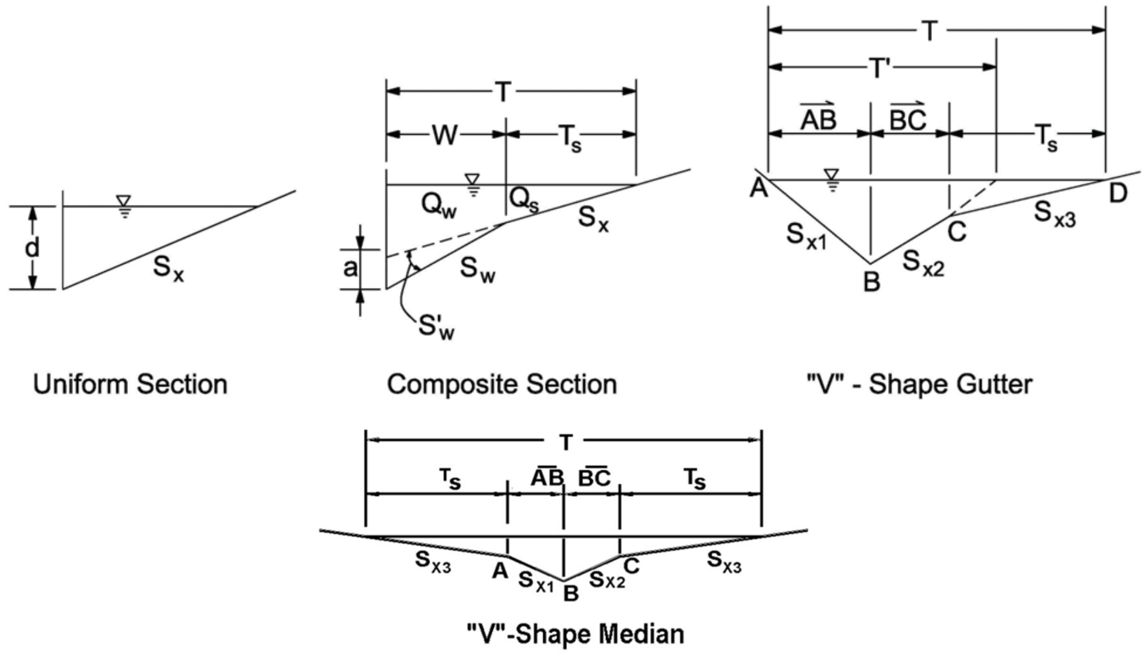


Figure 6.3 - Common gutter types

Figure 6.4 - Photograph of typical V-inlet

The uniform and composite gutter shapes are more conventional. The V-shaped gutter (Figure 6.4) is often used in median areas and along shoulders where surface water runs onto the pavement. The composite gutter will carry more flow for a given width than the uniform gutter.

6.4.1 Uniform Cross Section Procedure

In calculating the flow capacity of gutters with a uniform cross slope, a modified version of Manning's equation is used.

$$Q = \frac{0.56}{n} S_x^{1.67}\, S_L^{0.5}\, T^{2.67} \qquad (6.4)$$

Rearranged to solve for gutter spread (T), this equation is expressed as

$$T = \left(\frac{Q_n}{0.56\, S_x^{1.67} S_L^{0.5}}\right)^{0.375} \qquad (6.5)$$

Where:

Q = Total flow rate, ft^3/s
n = Manning's coefficient
S_x = Pavement cross-slope, ft/ft
S_L = Longitudinal slope, ft/ft
T = Width of flow (gutter spread), ft

The resistance of the curb face is negligible and is therefore not accounted for in Equation 6.4.

Manning's n Coefficient for Pavements

The roughness of the pavement surface affects water spread. The methods for determining spread provided in this chapter use Manning's roughness coefficient (n). Normally a value of 0.016 is used for curb and gutter flow. [(6-4)]

Table 6.2 provides additional Manning's roughness coefficients for specific types of pavement conditions.

Table 6.2 Manning's n for street and pavement gutters

Type of Gutter or Pavement	Manning's n
Concrete Gutter, troweled finish	0.012
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete Gutter, Asphalt Pavement	
Smooth texture	0.013
Rough texture	0.015
Concrete Pavement	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above n values by:	0.02
Source: Reference [(6-5)]	

For depth of flow, in feet, at curb (d):

$$d = T\,S_x \qquad (6.6)$$

There are numerous ways of solving Equation 6.5 to find gutter spread, T. A common and practical method is:

- Model the system in a HEC 22 based computer program approved by the Department

6.4.2 Composite Cross Slopes

Pavements with composite cross slopes are composed of a pavement section with a cross slope that is different from the gutter cross slope. Figure 6.5 depicts a typical composite cross slope section.

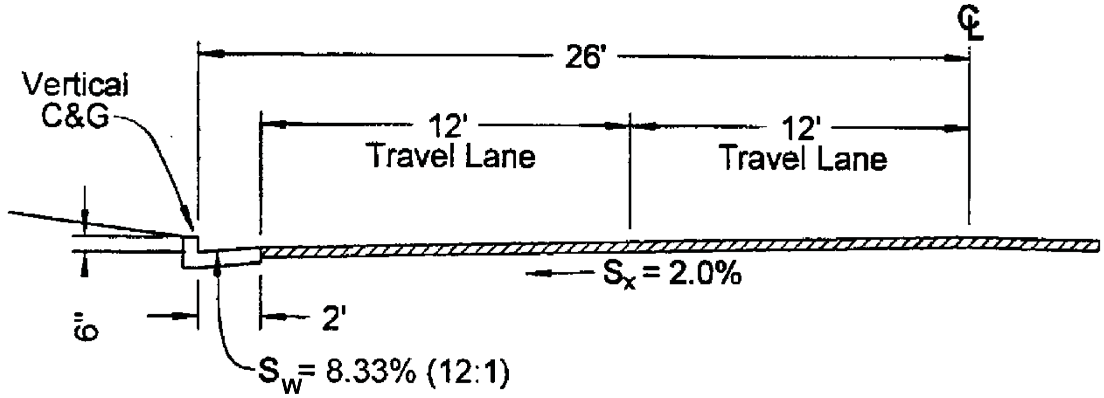


Figure 6.5 - Typical pavement with composite cross slope

For pavement with composite cross slopes, the total rate of flow in the channel may be expressed as the sum of the flow in the gutter section (Q_w) and the flow outside of the gutter section (Q_s):

$$Q = Q_w + Q_s \tag{6.7}$$

The total flow Q may also be expressed as:

$$Q = \frac{Q_s}{(1 - E_o)} \tag{6.8}$$

Where E_o (gutter flow ratio) is defined as the ratio of the flow in the width of the gutter section (Q_w) to the total channelized pavement flow (Q):

$$E_o = \frac{Q_w}{Q} \tag{6.9}$$

Further, E_o can be determined using the below expressed relationship between Sw, Sx, T, and W:

$$Eo = \frac{1}{1 + \left[\frac{\frac{Sw}{Sx}}{\left(1 + \frac{\frac{Sw}{Sx}}{\frac{T}{W} - 1}\right)^{2.67} - 1} \right]}$$

(6.10)

Where:

E_O	=	Gutter flow ratio (Q_W/Q)
S_w	=	Gutter slope (ft/ft)
S_x	=	Pavement cross-slope (ft/ft)
T	=	Width of flow (gutter spread), ft

Note:

S_w is defined as S_x + a/W, where a = gutter depression depth at inlet (ft) and W = width of the gutter or grate (ft). See the composite section in Figure 6.3 or Figure 6.8 for a graphical depiction of a and W.

6.4.3 V-Sections

V-sections are used where curbs are not needed or may present a safety problem. This section is also used at offset turn lanes. When solving for Sx, Equation 6.11 may be used. (6-4)

$$Sx = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \quad (6.11)$$

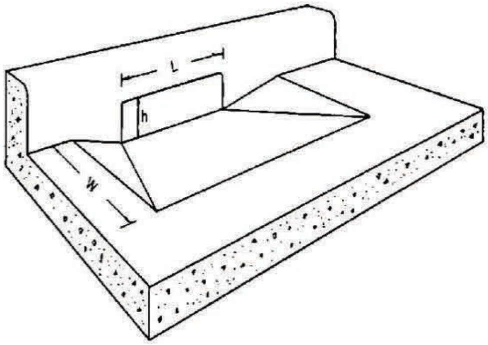
S_{x1} and S_{x2} are defined in the V-shaped gutter graphic shown in Figure 6.3.

6.5 Inlet Types

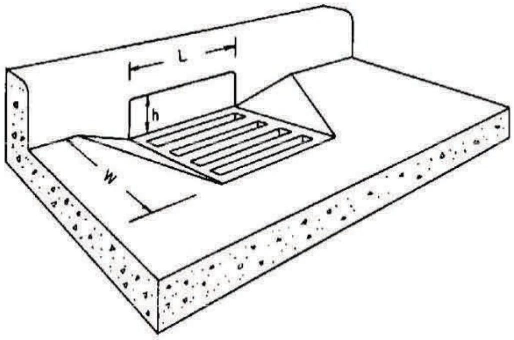
Inlets used for the drainage of pavement surfaces can be divided into four major classes (Figure 6.6). These classes are as follows:

- Curb Opening Inlet (S1, S2, S3 and S4)
- Combination Inlet (E, E1/E2, E3/E4)
- Grate Inlet (B, PR, CV-4.5, CV-6, PB, V-1, V-2, P1, P2 and P3)
- Slotted Drain Inlets

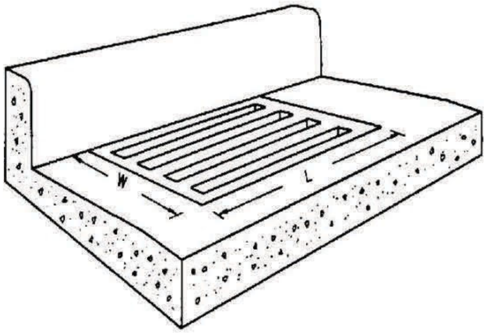
A list of Department type grates can be found in Table 6.3. Standard Drawings for some of the listed inlets above can be found on the Department of Transportation web site: http://alletting.dot.state.al.us/Docs/Standard_Drawings/StdDrawingSelect.htm



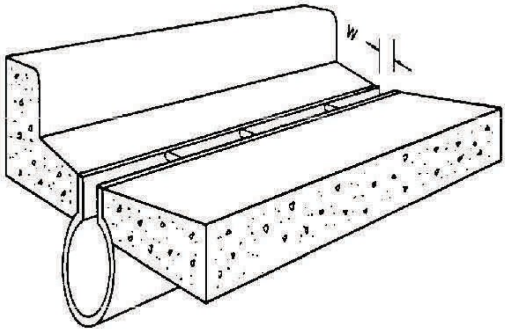
Curb Opening Inlet



Combination Inlet



Grate Inlet



Slotted Drain Inlet

Figure 6.6 - Inlet types used in roadway drainage design

Table 6.3 Alabama Department of Transportation Inlet Dimensions
Standard Curb Inlets On Grade

Inlet Std.	% Clogged	Length (Ft.)	Effective Length (Ft.)
S1 One Wing	0	10.25	10.25
S1 One Wing	10	10.25	9.23
S1 One Wing	25	10.25	7.69
S1 One Wing	50	10.25	5.13
S2 One Wing	0	10.25	10.25
S2 One Wing	10	10.25	9.23
S2 One Wing	25	10.25	7.69
S2 One Wing	50	10.25	5.13
S3 One Wing	0	10.25	10.25
S3 One Wing	10	10.25	9.23
S3 One Wing	25	10.25	7.69
S3 One Wing	50	10.25	5.13
S4 One Wing	0	12.75	12.75
S4 One Wing	10	12.75	11.48
S4 One Wing	25	12.75	9.56
S4 One Wing	50	12.75	6.38

Table 6.3 (Cont'd)
ALDOT Standard Curb Inlets In Sump

Inlet Std.	% Clogged	Length (Ft.)	Effective Length (Ft.)
S1 Two Wings	0	16.50	16.50
S1 Two Wings	10	16.50	14.85
S1 Two Wings	25	16.50	12.38
S1 Two Wings	50	16.50	8.25
S2 Two Wings	0	16.50	16.50
S2 Two Wings	10	16.50	14.85
S2 Two Wings	25	16.50	12.38
S2 Two Wings	50	16.50	8.25
S3 Two Wings	0	17.00	17.00
S3 Two Wings	10	17.00	15.30
S3 Two Wings	25	17.00	12.75
S3 Two Wings	50	17.00	8.50
S4 Two Wings	0	19.50	19.50
S4 Two Wings	10	19.50	17.55
S4 Two Wings	25	19.50	14.63
S4 Two Wings	50	19.50	9.75

The lengths shown on this chart are approximations of a length of a curb opening inlet as shown in Figure 6.6. The S1 through S4 inlets consist either of a section of weir parallel to the curb and a section perpendicular to the curb or a partial arc of a circular weir plus a depressed gutter partially obstructed by one or more pedestals.

The designer should use prudence when curb opening inlet lengths are selected for inlet design or pipe design. For inlet design, it is suggested to use the least length which could possibly be used on the project. For pipe design, it is suggested to use the longest length which could possibly be used on the project.

Table 6.3 (Cont'd) ALDOT Special Grate Inlets On Grade

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Eff. Grate Width (Ft.)	FHWA Tested Grate for Calculations
B(Curb/Barrier)	1.250	1.583	0	1.583	Reticuline
B(Curb/Barrier)	1.250	1.583	10	1.425	Reticuline
B(Curb/Barrier)	1.250	1.583	25	1.187	Reticuline
B(Curb/Barrier)	1.250	1.583	50	0.792	Reticuline
P1/P2/P3(Curb)	3.000	3.000	0	3.000	P 1-$^{1}/_{8}$
P1/P2/P3(Curb)	3.000	3.000	10	2.700	P 1-$^{1}/_{8}$
P1/P2/P3(Curb)	3.000	3.000	25	2.250	P 1-$^{1}/_{8}$
P1/P2/P3(Curb)	3.000	3.000	50	1.500	P 1-$^{1}/_{8}$

Table 6.3 (Cont'd) ALDOT Special Project Detail Grate Inlets On Grade

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Eff. Grate Width (Ft.)	FHWA Tested Grate for Calculations
PB(Curb)	2.021	1.542	0	1.542	P 1$^{7}/_{8}$-4
PB(Curb)	2.021	1.542	10	1.388	P 1$^{7}/_{8}$-4
PB(Curb)	2.021	1.542	25	1.156	P 1$^{7}/_{8}$-4
PB(Curb)	2.021	1.542	50	0.771	P 1$^{7}/_{8}$-4
PR(Curb)	3.438	4.042	0	4.042	P 1$^{7}/_{8}$-4
PR(Curb)	3.438	4.042	10	3.638	P 1$^{7}/_{8}$-4
PR(Curb)	3.438	4.042	25	3.031	P 1$^{7}/_{8}$-4
PR(Curb)	3.438	4.042	50	2.021	P 1$^{7}/_{8}$-4
CV-4.5	2.271	4.500	0	4.500	Curved Vane (CV)
CV-4.5	2.271	4.500	10	4.050	Curved Vane (CV)
CV-4.5	2.271	4.500	25	3.375	Curved Vane (CV)
CV-4.5	2.271	4.500	50	2.250	Curved Vane (CV)
CV-6	2.271	6.000	0	6.000	Curved Vane (CV)
CV-6	2.271	6.000	10	5.400	Curved Vane (CV)
CV-6	2.271	6.000	25	4.500	Curved Vane (CV)
CV-6	2.271	6.000	50	3.000	Curved Vane (CV)

Table 6.3 (Cont'd) ALDOT Special Comb. Curb Grate Inlets On Grade

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Eff. Grate Width (Ft.)	FHWA Tested Grate for Calculations
E(Curb/C&G)	2.833	1.833	0	1.833	Reticuline/CV
E(Curb/C&G)	2.833	1.833	10	1.650	Reticuline/CV
E(Curb/C&G)	2.833	1.833	25	1.375	Reticuline/CV
E(Curb/C&G)	2.833	1.833	50	0.917	Reticuline/CV
E1/E2(2′MB)	2.833	1.833	0	1.833	Reticuline/CV
E1/E2(2′MB)	2.833	1.833	10	1.650	Reticuline/CV
E1/E2(2′MB)	2.833	1.833	25	1.375	Reticuline/CV
E1/E2(2′MB)	2.833	1.833	50	0.917	Reticuline/CV
E3/E4(2.5′MB)	2.833	1.833	0	1.833	Reticuline/CV
E3/E4(2.5′MB)	2.833	1.833	10	1.650	Reticuline/CV
E3/E4(2.5′MB)	2.833	1.833	25	1.375	Reticuline/CV
E3/E4(2.5′MB)	2.833	1.833	50	0.917	Reticuline/CV

Table 6.3 (Cont'd) ALDOT Special Grate Inlets In Sump

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Grate Opening Perimeter (Ft.)	Grate Opening Area (Sq. Ft.)
B(Curb/barrier)	1.250	1.583	0	4.416	1.250
B(Curb/barrier)	1.250	1.583	10	4.099	1.125
B(Curb/barrier)	1.250	1.583	25	3.625	0.938
B(Curb/barrier)	1.250	1.583	50	2.833	0.625
PR(No Curb)	3.438	4.042	0	14.958	9.519
PR(No Curb)	3.438	4.042	10	14.150	8.570
PR(No Curb)	3.438	4.042	25	12.938	7.140
PR(No Curb)	3.438	4.042	50	10.917	4.762

Table 6.3 (Cont'd) ALDOT Special Project Detail Grate Inlets In Sump

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Grate Opening Perimeter (Ft.)	Grate Opening Area (Sq. Ft.)
PB(Curb)	2.021	1.542	0	5.104	1.885
PB(Curb)	2.021	1.542	10	4.796	1.697
PB(Curb)	2.021	1.542	25	4.333	1.415
PB(Curb)	2.021	1.542	50	3.563	0.994
PR(Curb)	3.438	4.042	0	11.521	9.519
PR(Curb)	3.438	4.042	10	10.713	8.570
PR(Curb)	3.438	4.042	25	9.500	7.140
PR(Curb)	3.438	4.042	50	7.479	4.762
PR(No Curb)	3.438	4.042	0	14.958+	9.519
PR(No Curb)	3.438	4.042	10	14.150	8.570
PR(No Curb)	3.438	4.042	25	12.938	7.140
PR(No Curb)	3.438	4.042	50	10.917	4.762

Table 6.3 (Cont'd) ALDOT Special Comb. Grate Inlets In Sump

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Grate Opening Perimeter (Ft.)	Grate Opening Area (Sq. Ft.)
E(Curb/C&G)	2.833	1.833	0	6.499	2.979
E(Curb/C&G)	2.833	1.833	10	6.132	2.681
E(Curb/C&G)	2.833	1.833	25	5.583	2.234
E(Curb/C&G)	2.833	1.833	50	4.666	1.490
E1/E2(2′MB)	2.833	1.833	0	6.499	2.979
E1/E2(2′MB)	2.833	1.833	10	6.132	2.681
E1/E2(2′MB)	2.833	1.833	25	5.583	2.234
E1/E2(2′MB)	2.833	1.833	50	4.666	1.490
E3/E4(2.5′MB)	2.833	1.833	0	6.499	2.979
E3/E4(2.5′MB)	2.833	1.833	10	6.132	2.681
E3/E4(2.5′MB)	2.833	1.833	25	5.583	2.234
E3/E4(2.5′MB)	2.833	1.833	50	4.666	1.490

Table 6.3 (Cont'd) ALDOT Special Project Detail Grate Inlets On Grade

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Eff. Grate Width (Ft.)	FHWA Tested Grate to Use
V1	3.396	3.083	0	3.083	P $1^{7}/_{8}$-4
V1	3.396	3.083	10	2.775	P $1^{7}/_{8}$-4
V1	3.396	3.083	25	2.312	P $1^{7}/_{8}$-4
V1	3.396	3.083	50	1.542	P $1^{7}/_{8}$-4
V2	5.375	3.417	0	3.417	Reticuline
V2	5.375	3.417	10	3.075	Reticuline
V2	5.375	3.417	25	2.562	Reticuline
V2	5.375	3.417	50	1.708	Reticuline

Table 6.3 (Cont'd) ALDOT Special Project Detail Grate Inlets In Sump

Inlet Std.	Length (Ft.)	Width (Ft.)	% Clogged	Grate Opening Perimeter (Ft.)	Grate Opening Area (Sq. Ft.)
V1	3.396	3.083	0	12.958	8.730
V1	3.396	2.775	10	12.342	7.533
V1	3.396	2.312	25	11.417	6.278
V1	3.396	1.542	50	9.875	4.185
V2	5.375	3.417	0	17.583	11.776
V2	5.375	3.075	10	16.900	10.598
V2	5.375	2.562	25	15.875	8.832
V2	5.375	1.708	50	14.167	5.888

6.5.1 Characteristics and Uses 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.

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. A caveat to this general rule is that grate inlets or combination inlets are preferable in heavily urbanized areas.

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. The Department's only combination inlet is the E inlet with the grate placed alongside the curb opening. On grade the interception capacity of the E inlet differs little from that of a grate only, and the grate capacity is used to determine the E inlet's capacity. The curb opening is useful if the grate becomes clogged or if an object is too large for the grate to pass such as a can floats down to the inlet. Combination inlets are more desirable than grate inlets in sags because they can continue to receive stormwater flow when the grate becomes clogged.

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 E combination inlet for capacity calculations.

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. Curved vane inlets are the least likely grate inlets to clog on grade (Table 6.4). Special-design (oversized) grate inlets can be utilized at major sag points if sufficient capacity is provided for clogging. Grates should be bicycle safe, where bicycle or wheel chair traffic is anticipated and structurally designed to handle the appropriate loads when subject to traffic.

Table 6.4 Average Debris Handling Efficiencies of Grates Tested

Rank	Grate	Longitudinal Slope	
		0.005	0.04
1	Curved Vane	46	61
2	30° - 85 Tilt Bar	44	55
3	45° - 85 Tilt Bar	43	48
4	P - 50	32	32
5	P - 50 x 100	18	28
6	45° - 60 Tilt Bar	16	23
7	Reticuline	12	16
8	P - 30	9	20

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. Although slotted drains are more easily clogged in sags than other inlet types, at sharp radiuses where the top slabs of S inlets are subject to destruction by truck trailers, PB inlets with an attached slotted drain or drains are a practical alternative. Slotted drain inlets can be used on curbed or uncurbed sections

6.5.2 Inlet Flow Capacity and Interception

The interception capacity of a slotted drain inlet, curb-opening inlet or grate inlet on grade is equal to the efficiency of the inlet multiplied by the total flow:

$$Q_i = EQ \qquad (6.12)$$

Inlet capacity calculations can be performed using HEC 22 based computer programs approved by the Department, such as InletsoftAL. Regardless of the method used, the Department requires that the results be included on the standard Department inlet computation form for uniformity and ease of review.

Curb-Opening Inlets and Slotted Drains on Grade

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Analysis of data from the FHWA tests of slotted inlets with slot widths ≥1.75 in indicates that the length of slotted inlet required for total interception can be computed by Equation 6.13. Chart 6.3 (page 6-39), is therefore applicable for both curb-opening inlets and slotted inlets. Similarly, Equation 6.14 is also applicable to both curb-opening inlets and slotted inlets.[6-4]

Figure 6.7 - Curb inlet placed on continuous roadway grade

When slotted drains are used to capture overland flow, research has indicated that with water depths ranging from 0.38 in to 0.56 in, the 1, 1.75 and 2.5 in wide slots can accommodate 0.025 ft^3/s /ft with no splash-over for slopes from 0.5% to 9%. At a test system capacity of 0.40 ft^3/s /ft, a small amount of splash over occurred. Within these ranges, slotted inlets are equivalent in efficiency to curb-opening inlets. When these depths and flow rates are greater than the maximum values in the range, curb-opening inlets are more efficient and should be specified rather than slotted drains.[6-4]

Curb-opening inlets are preferable to grate inlets in locations where grates would be in traffic lanes, where greater debris handling capability is required, and where it is desirable to provide a smooth path for bicycle traffic (e.g., a narrow shoulder).

Both curb-opening inlets and slotted drain inlets offer little interference to traffic operations. [6-4]

Non-Depressed Curb-Opening Inlets and Slotted Drains on Grade

The length of a non-depressed curb-opening inlet (i.e., uniform section) required for total interception of flow on a pavement section with a straight cross slope is expressed by the following:

$$L_T = KQ^{0.42} S_L^{0.3} (1/nS_x)^{0.6} \quad (6.13)$$

Where:

L_T	=	Curb-opening length required to intercept 100% of gutter flow, ft
K	=	0.6

For composite cross slopes, substitute S_e for S_x where $S_e = S_x + S'_W E_O$ and $S'_W = a/W$.

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - (1 - L/L_T)^{1.8} \quad (6.14)$$

Where:

L	=	Curb-opening length (shorter than L_T), ft

Depressed Curb-Opening Inlets and Slotted Drains on Grade

The length of inlet necessary for required interception by locally depressed curb-opening inlets or curb openings in continuously depressed gutter sections (i.e., composite cross slopes) can be found by the use of an equivalent cross slope, Se, in Equation 6.15 in place of S_x:

$$Se = S_x + S'_W E_O \quad (6.15)$$

Where:

S'_W	=	Gutter cross slope measured from the pavement cross slope= a/W, ft/ft
E_O	=	Ratio of flow in the gutter (or depressed) section to total gutter flow
a	=	Gutter depression at inlet, ft (as shown on Figure 6.8)
W	=	Gutter width, ft

E_O is determined by the gutter configuration upstream of the inlet as discussed in the section on composite cross slope gutter flow computations.

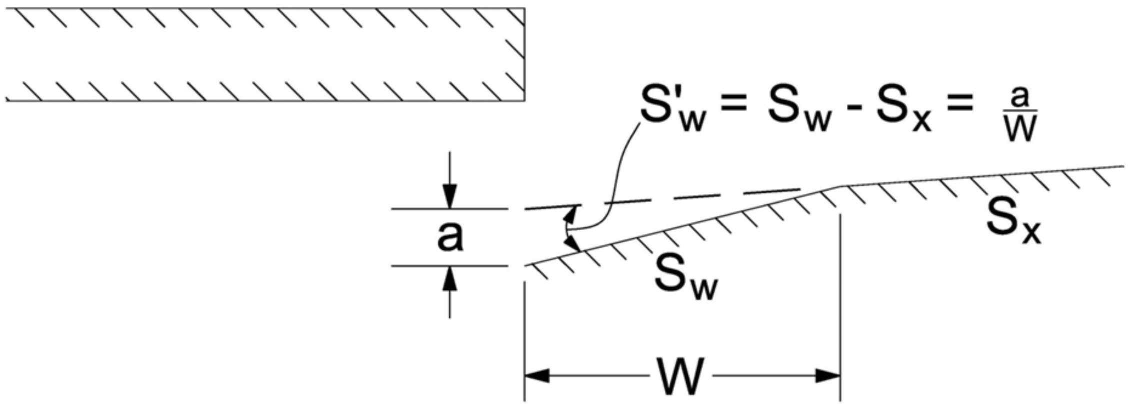


Figure 6.8 - Definition sketch of S'W

Curb Inlets in Sag

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

Weir Flow for Depressed Curb-Opening Inlets in Sag

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir $(d \leq h)$ is:

$$Q_i = C_W (L + 1.8W)d^{1.5} \quad (6.16)$$

Where:

C_W = 2.3
L = Length of curb opening, ft
W = Width of depression, ft
d = Depth of water at curb measured from water surface to the projected normal cross slope gutter flow line, ft

This weir equation uses an effective weir length and coefficient that is representative of the line of gutter transition to the depression. The user is cautioned not to use the depth from the water surface to the depressed inlet throat for d, but to use the un-depressed depth “d” (or more specifically, the projected depth at the curb face as shown in Figure 6.9). Otherwise, the capacity for weir flow will be overestimated.

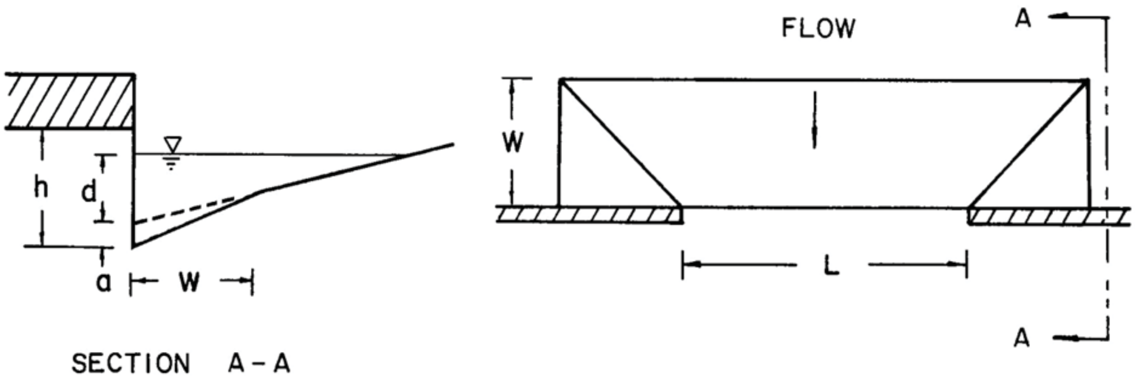


Figure 6.9 - Definition of weir flow parameters used in Equation 6.16

Weir Flow for Curb-Opening Inlets without Depression

The weir equation for curb-opening inlets without depression becomes

$$Q_i = C_W L d^{1.5} \quad (6.17)$$

Where:

C_W = 3.0
L = Length of curb opening, ft
d = Flow depth, ft

Note: At curb-opening lengths greater than 12 ft, Equation 6.17 for a non-depressed inlet produces intercepted flows that exceed the values for depressed inlets computed using Equation 6.16. Since depressed inlets will perform at least as well as non-depressed inlets of the same length, Equation 6.17 should be used for all curb-opening inlets having lengths greater than 12 ft.

Orifice Flow for Curb-Opening Inlets

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity can be computed by

$$Q_i = C_o A \left[2g\left(d_i - h/2\right)\right]^{0.5} \quad (6.18)$$

Where:

C_O = Orifice coefficient (0.67)
A = Clear area of curb-opening = h x L, where L is the horizontal length of curb opening, ft^2
g = Acceleration of gravity, 32.2 ft/s^2
d_i = Depth at lip of curb opening as defined in Figure 6.10, ft
h = Height of curb-opening orifice as defined in Figure 6.10, ft
d_O = Effective head at the centroid of the orifice, ft See Figure 6.10 for a graphical depiction of the parameters used in this equation.

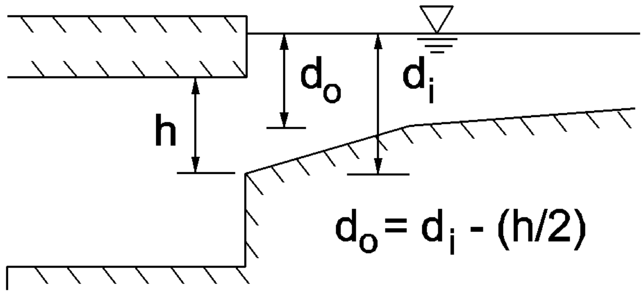


Figure 6.10 - Parameters for calculating orifice flow into an inclined curb inlet

Slotted Drain Inlets in Sag

The use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of these inlets to intercept debris and clog. However, there may be locations where it is desirable to supplement an existing low-point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to approximate depths of 0.2 ft, dependent on slot width and length. At depths greater than 0.4 ft, they perform as orifices. Between these depths, flow is in a transition stage.

The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8LW\,(2gd)^{0.5} \tag{6.19}$$

Where:

L = Length of slot, ft
W = Width of slot, ft
g = Acceleration of gravity, 32.2 ft/s^2
d = Depth of water at slot, ft

For a slot width of 1¾ in, Equation 6.19 becomes

$$Q_i = 0.94Ld^{0.5} \tag{6.20}$$

The interception capacity of slotted inlets at depths between 0.2 ft and 0.4 ft can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of slotted inlet.

For depths that are transitional between weir and orifice flow, refer to HEC 22[(6-4)] for further information.

Flow Over Grates

There are three types of flow to consider when evaluating the interception capacity of a grate inlet. They are frontal flow, side flow, and splash-over.

- Frontal flow is the portion of the flow that passes over the upstream side of the grate.
- Side flow is the portion of flow that passes along the side of the grate.
- Splash-over is the portion of frontal flow that skips or splashes over the grate and is not intercepted.

Capacity and Interception of Grate Inlets on Grade

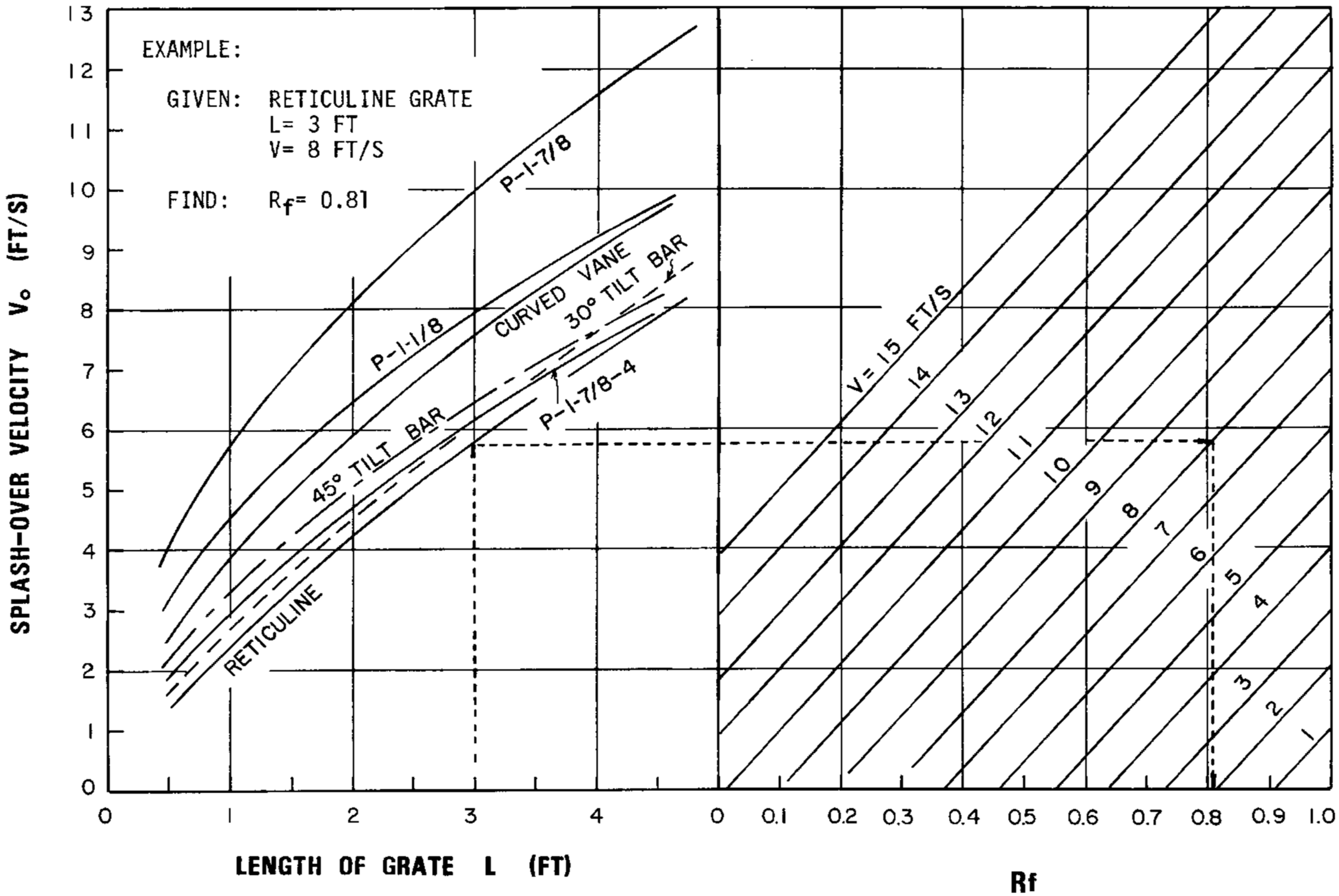
The interception capacity of a grate inlet is dependent upon the following parameters:

- Shape or geometry
- Cross slope
- Longitudinal slope
- Total flow
- Depth of flow
- Pavement roughness

The depth of water next to the curb is the major factor in the interception capacity of grate inlets and curb-opening inlets. At low velocities, all of the frontal flow is intercepted by grate inlets and a small portion of the side flow is intercepted. Splash-over tends to increase on steep longitudinal slopes.

While the parallel bar grates are the most efficient grates on steep slopes, they are not bicycle safe unless they have closely spaced cross bars such as the P1-7/8-4 grate. The bicycle safe roadway grate inlets the Department has are the PB, PR, CV-4.5, CV-6, V-1, and V-2 inlets, and those E inlets which have curved vane grates. The grates tested in a FHWA research study are described in HEC 22. [(6-4)]

Chart 6.1 can be used to determine splash-over velocities for various grate configurations and the portion of frontal flow intercepted by the grate.



. Grate Inlet Frontal Flow Interception Efficiency

Chart 6.1 Source HEC 22

The ratio of frontal flow to total gutter flow, E_O, for a straight cross slope is given by the following equation:

$$E_O = Q_W/Q = 1 - (1 - W/T)^{2.67} \quad (6.21)$$

Where:

Q_W	=	Frontal flow in width W, ft^3/s
Q	=	Total gutter flow, ft^3/s
W	=	Width of depressed gutter or grate, ft
T	=	Total spread of water on pavement, ft

The ratio of side flow, Q_S, to total flow is

$$Q_S/Q = 1 - E_O \quad (6.22)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\ (V - V_O) \quad (6.23)$$

Where:

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

Note that R_f cannot exceed 1.

This ratio is equivalent to frontal-flow interception efficiency. Chart 6.1 provides the splash-over velocity as well as a solution of Equation 6.23 that incorporates grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed is total gutter volumetric flow divided by the cross-sectional area of flow.

The ratio of side flow intercepted to total side flow, R_s, or side-flow interception efficiency, is expressed by the following:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \quad (6.24)$$

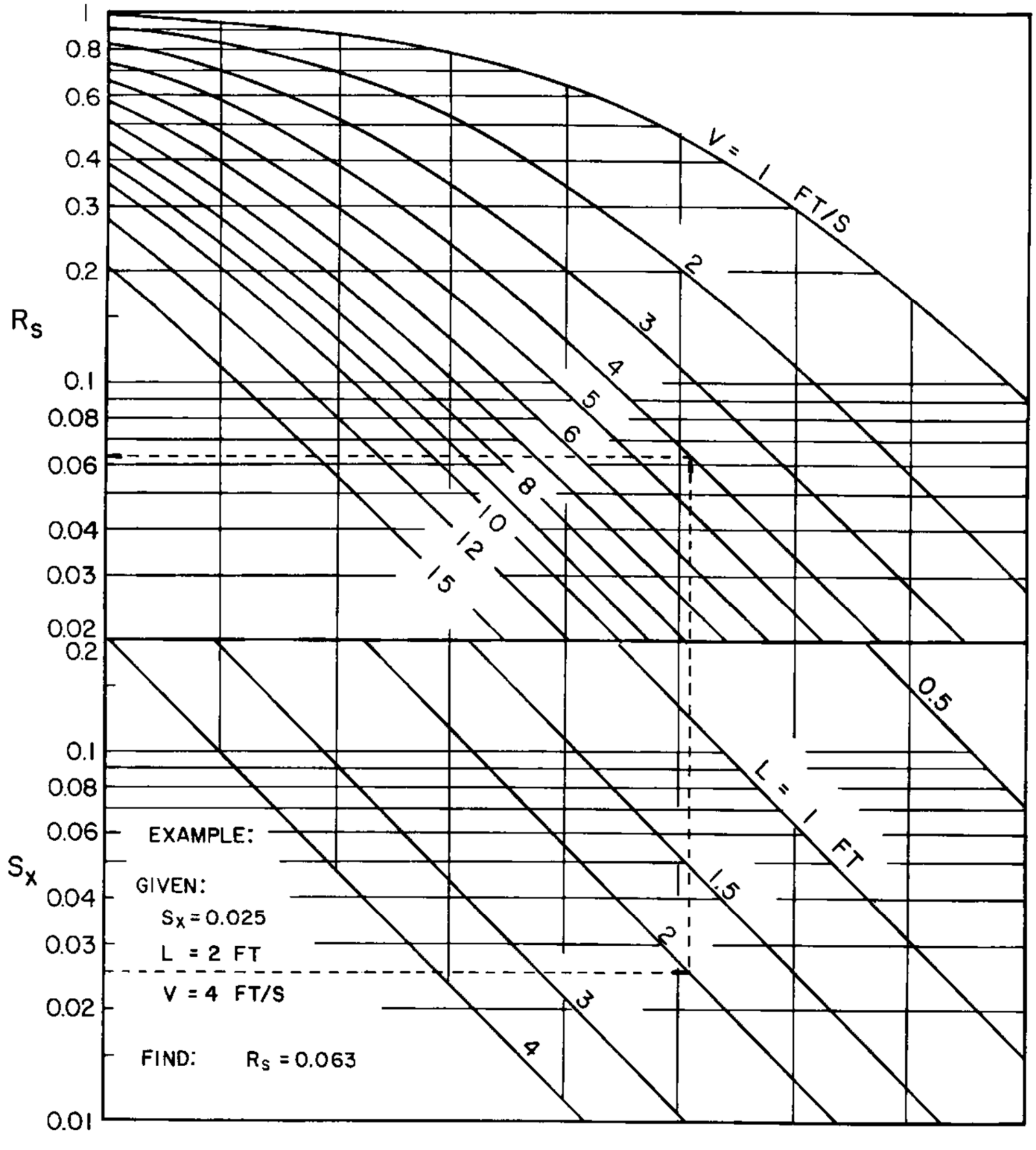
Where:

V	=	Velocity of flow in the gutter, ft/s
S_x	=	Cross slope, ft/ft
L	=	Length of the grate, ft

The efficiency, E, of a grate is expressed as

$$E = R_f E_O + R_s(1 - E_O) \qquad (6.25)$$

Chart 6.2 provides a graphical solution to Equation 6.24.



Grate Inlet Side Flow Intercept Efficiency

Chart 6.2 Source HEC 22

Capacity of Grate Inlets in Sag

Although curb-opening inlets are generally preferred to grate inlets at sag locations, grate inlets without a curb opening can be used successfully at minor sag points where debris potential is limited. An example of a minor sag point might be on a side road as it joins a mainline.

For major sag points, such as on divided highways, a curb-opening inlet or combination inlet is preferable to a grate inlet because of its hydraulic capacity and debris-handling capabilities.

A grate inlet in a sag operates as a weir up to a depth of about 0.4 ft and as an orifice for depths greater than 1.4 ft. Between these depths, a transition from weir to orifice flow occurs. The capacity of a grate inlet operating as a weir is:

$$Q_i = CPd^{1.5} \tag{6.26}$$

Where:

C = 3.0 weir coefficient

P = Perimeter of grate, ft, disregarding the side against the curb (as shown on Chart 6-3).

d = Average flow depth across the grate, ft, see Figure 6.11

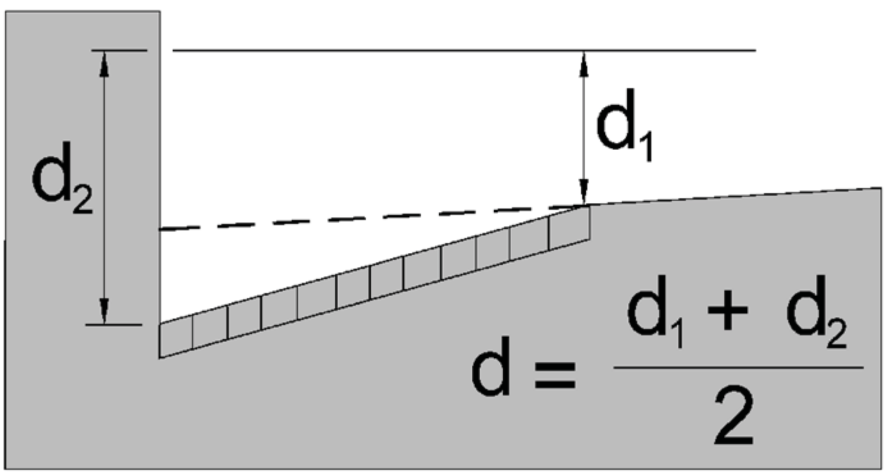


Figure 6.11 - Average flow depth for grate inlet

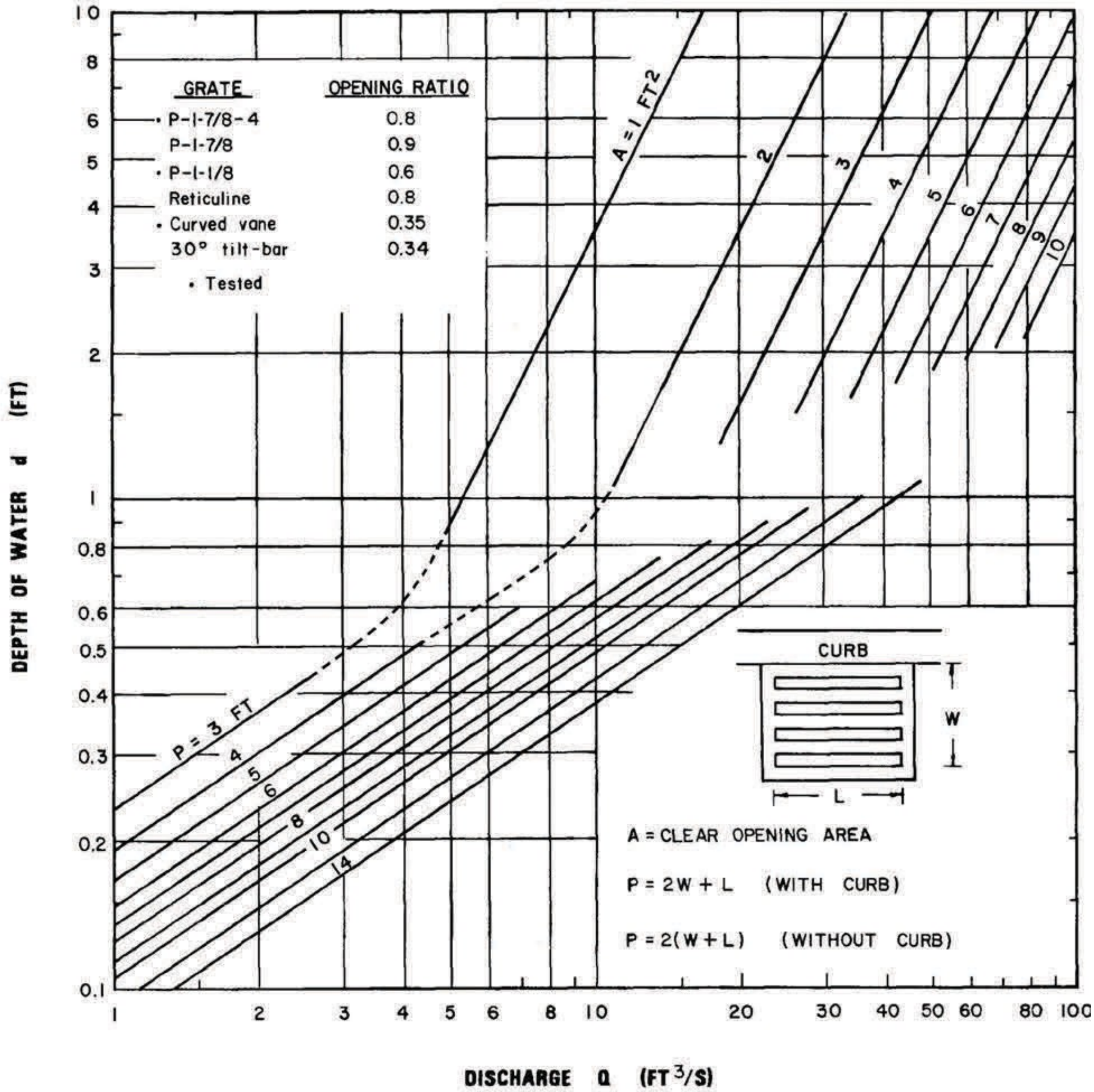
The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA(2gd)^{0.5} \tag{6.27}$$

Where:

C	=	0.67, orifice coefficient
A	=	Clear opening area of the grate, ft^2
g	=	32.2 ft/s^2
d	=	Average flow depth across the grate, ft, see Figure 6.11

Chart 6.3 is a plot of Equations 6.26 and 6.27 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or the orifice equation. Drawing in a curve between the lines representing the perimeter and net area of the grate to be used can approximate this capacity.



Grate Inlet Capacity in Sump Conditions - English Units

Note: Dashed lines are example representations of the curve that can be drawn between the perimeter and net area of a given grate. Drawing this type of curve allows approximation of the capacity of the grate through the transition from weir to orifice flow.

Chart 6.3 Source: HEC 22

6.6 Design Procedures

The following is a summary of the design procedures for pavement drainage design:

6.6.1 Collect and Analyze Existing Data

6.6.2 Preliminary Layout - Placement of Inlets Due to Geometric Controls

6.6.1 Collect and Analyze Existing Data

The following information is required 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 4)
- Existing pipe data

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.
- Assess condition and type of existing pipes and structures to determine if any deficiencies exist.

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

6.6.2 Preliminary Layout - Placement of Inlets Due to Geometric Controls

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 regardless of the contributing drainage area. The following list provides guidance for required placement of drainage structures on roadway projects:

- Inlets are to be placed at all sag locations and low points in the gutter grade.
- Inlets are to be placed on continuous grades to control gutter spread per Table 6.1 in Section 6.2.2.

- Inlets are to be placed in locations to minimize sheet flow across the roadway.
- Curb-opening inlets are not to be used in small radii. PB inlets are recommended in small radii.
- Inlets should not be placed within crosswalk locations.
- Inlets are to be placed immediately upstream of median breaks, entrance/exit ramp gores, crosswalks, and street intersections (i.e., at any location where water could flow onto the travelway).
- Inlets are to be placed immediately upstream of bridges to prevent pavement drainage from flowing onto bridge decks.
- Inlets are to be placed immediately downstream of bridges to intercept bridge deck drainage even where deck drain systems exist.
- Inlets are to be placed within approximately 50 ft upgrade of flat cross slopes in superelevation transition areas. Practically all surface runoff should be intercepted to prevent flow back across travel lanes.
- Inlets are to be placed immediately upgrade of pedestrian crosswalks.
- Inlets are to be placed on side streets immediately upgrade from intersections.
- Inlets are to be placed in low areas behind curbs, shoulders or sidewalks.
- Inlets are to be placed 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. In addition check high side of superelevated transitions for pocketed water.
- Use manholes rather than junction boxes when outside the roadway travel lanes and when site-specific obstacles don't exist in order to provide access.
- Special drainage systems such as trench and slotted drains should be considered and utilized as necessary to control gutter spread within tolerable limits.
- 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.

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. Never locate a low point or a high point on the longitudinal grade near the following locations:

- A flat cross slope in superelevation transition areas
- Intersections
- Sags in cut areas
- Sags on bridges

6.6.3 Determine Drainage Areas and Flows

The Rational Method (see Chapter 4 of this manual) is typically used for inlet design. Selection of design frequency (storm year) should be obtained from the policy in Section 6.2.2.

6.6.4 Determination of Spreads and Placement of Inlets on Continuous Grades

Placement and spacing of inlets on continuous roadway grades is dependent upon the gutter spread. It 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 6.1 in Section 6.2.2.

Selection of inlet locations on continuous grade may be done using a HEC 22 based computer program such as InletsoftAL or by using a hand tabulation method (see the Hydraulic Section website for a spreadsheet to be used in the tabulation method).

Whatever calculation method is chosen, it should be thoroughly documented so the calculations may be easily followed and reproduced by a reviewer. It should be noted when the CA values are calculated, they should be recorded so they do not have to be recalculated for the storm drain design.

6.6.5 Low Point and Adjacent Inlet Structures

At low points where significant ponding can occur such as at underpasses and sag vertical curves in depressed sections, or at the low point of the sag, or in unusually long sag vertical curves where it might be difficult to determine the low point, and where water cannot escape by overtopping the curb, it is desirable to place a flanking inlet (Figure 6.12) on each side of the inlet required at the low point should that inlet become completely clogged. Inlets should be placed on the low-gradient approaches to the low point to limit spread within the tolerances of Table 6.1. Where stormwater has the potential to escape over the curb, the shoulder slope should be flattened or even reversed at the sag to provide an outlet.

If flanking inlets are used to act in relief of the sag inlet, the flanking inlets are to be located so that they will receive all the flow when the primary inlet at the bottom of the sag is clogged. The following procedure demonstrates the distance to locate flanking structures using depth criteria.

If the flanking inlets are the same dimension as the primary inlet, they will each intercept one-half the design flow when they are located so that the depth of ponding at the flanking inlets is 63% of the depth of ponding at the low point. This is depicted in profile in Figure 6.13. If the flanking inlets are not the same size as the primary inlet, it will be necessary to either develop a new factor or do a trial and error solution using assumed depths with the weir equation to determine the capacity of the flanking inlet at the given depths.

The inlet spacing required for various depths at curb criteria and vertical curve lengths is defined as follows:

$$K = L / (G2 - G1) \quad (6.28)$$

Where:

L	=	Length of the vertical curve in feet
$G1$, $G2$	=	Approach grades in percent

The AASHTO Policy on Geometric Design of Highways and Streets recommends a maximum K value of 167 feet per percent change in grade in order to facilitate drainage to inlets located at sag points and away from the level point on crest vertical curves. (6-1)

The distance from the bottom of the sag to the flanking inlet is:

$$X = (74\, d\, K)^{0.5} \quad (6.29)$$

Where:

X	=	Maximum distance from bottom of sag to flanking inlet, ft
d	=	Depth of water over inlet in bottom of sag as shown in Figure 6.14, ft
K	=	Rate of vertical curvature commonly used as the measure for stopping sight distance

6.6.6 Clogging Reduction Factor for Inlet Design

During the design process, a clogging factor should not be used to reduce the widths of grates or lengths of curb inlets on grade, except for flat grades where there is an indication of a need. If a clogging factor is warranted on flat grades or sag vertical curves, this factor should be determined in consultation with maintenance personnel. Without maintenance information, a reasonable factor should be 10%. Clogging factors are for inlet designs only, but storm drain pipes should be designed as if the inlets are unclogged.

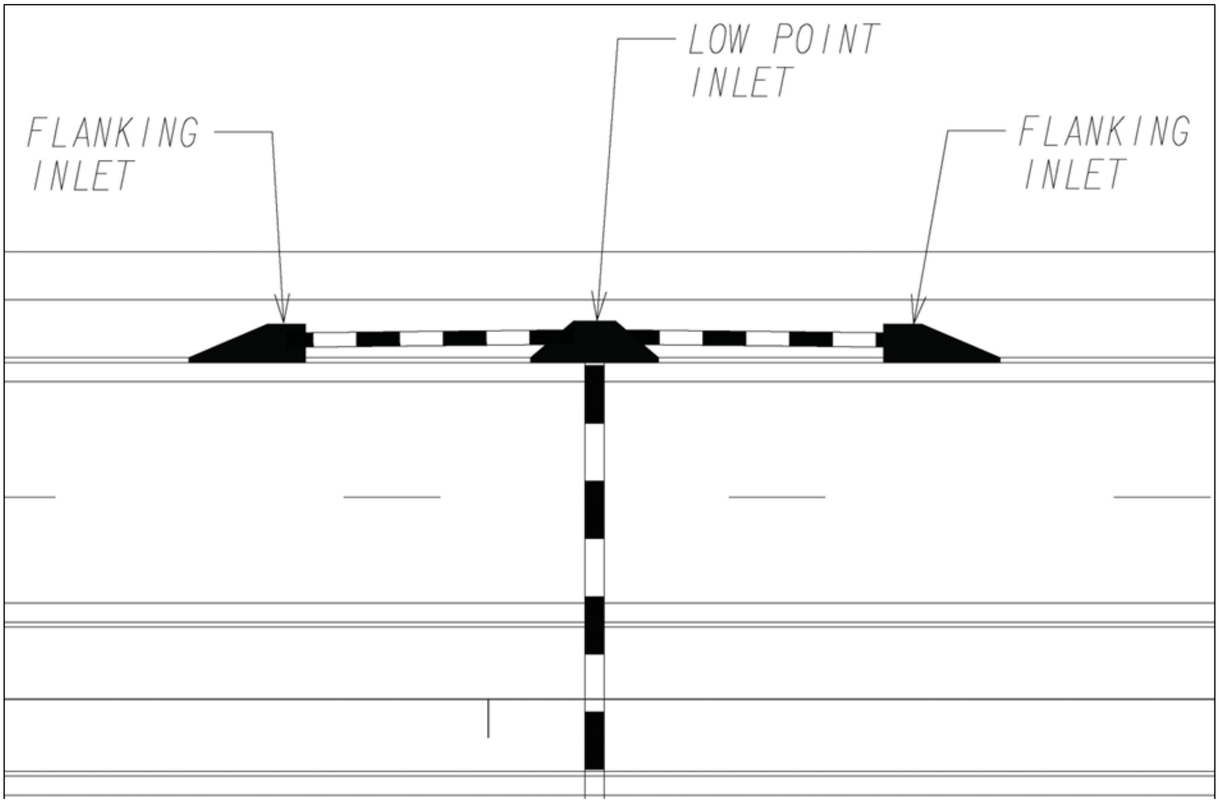


Figure 6.12 - Example of typical placement of flanking inlets (plan view)

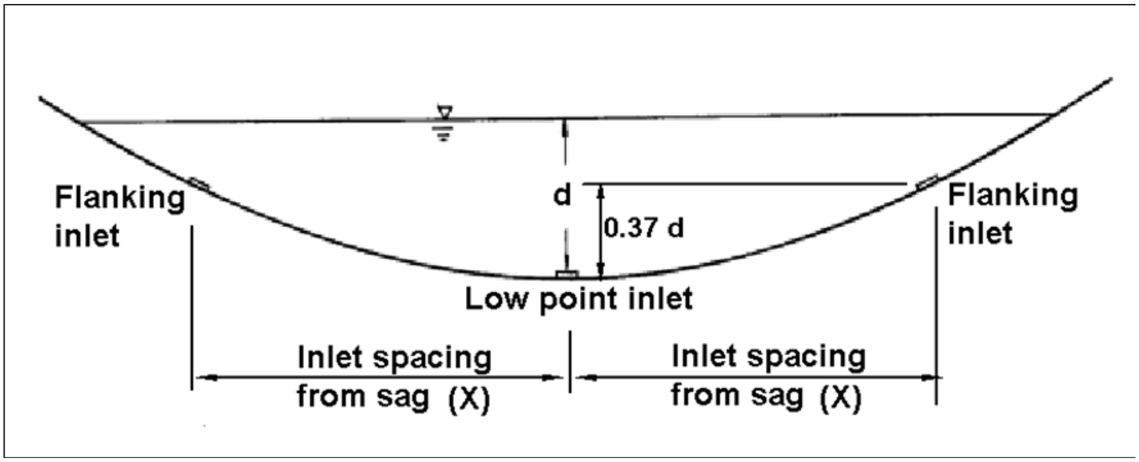


Figure 6.13 - Flanking inlet schematic

Step 1. Determine the K value for the sag curve.

Step 2. Determine the depth at design spread, d = $S_X\,T$ (S_X = cross slope, T = gutter spread) Step

Step 3. Establish X from Equation 6.29. This distance is the maximum distance that can be used.

R6 Chapter 6 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2011. Geometric Design of Highways and Streets

2. American Association of State Highway and Transportation Officials (AASHTO). 2007. Highway Drainage Guidelines, 4th Ed.

3. American Association of State Highway and Transportation Officials (AASHTO). 2014. Drainage Manual, 1st Ed.

4. 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.

5. Federal Highway Administration. 1961, Design Charts for Open-Channel Flow, Hydraulic Design Series No. 3, FHWA-EPD-86-102. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C

6. Young, G.K, Walker, S.E., Chang, F. 1993, Design of Bridge Deck Drainage, Hydraulic Engineering Circular No. 21, FHWA-SA-92-010. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.

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Chapter 7: Storm Drain Design

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7.1 Introduction

The design of a drainage system should address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region can be more complex. This can be attributed to areas with a heavy concentration of development and associated conflicts with existing utilities and the drainage system.

This chapter provides guidance on storm drain design and analysis based on procedures presented in the American Association of State Highway and Transportation Officials (AASHTO publication, *Drainage Manual*[7-1] and the FHWA publication, *Urban Drainage Design Manual* (HEC 22)[7-3]). Prior to starting a storm drain design, the designer should already have a basic understanding of the hydraulic behavior of closed conduits and open channels, and the concepts related to their hydraulic performance. In addition to storm sewer design guidance, this chapter also includes discussions, factors related to, and evaluation of the hydraulic grade line (HGL) and energy grade line (EGL).

The design procedures presented here assume that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

The designer should consult other chapters of this manual as appropriate for additional information relating to storm sewer design principles.

Chapter 4 – Hydrology & Hydraulics

Chapter 5 – Channels

Chapter 6 – Pavement Drainage

Chapter 8 – Culverts

7.1.1 Definition

A storm drain is the portion of the highway drainage system that receives surface water through inlets and conveys the water through conduits to a pipe outlet. It is composed of different lengths and sizes of pipe or conduit connected by structures. A section of conduit connecting one inlet or structure to another is termed a "segment" or "run." The storm drain is usually a circular pipe but can also be a box or other enclosed conduit shape. Structures may include inlets (excluding the actual inlet opening), access holes, junction chambers, and other miscellaneous structures. [7-3] The designer should refer to Chapter 6 for more information on drainage structures used in pavement design.

7.2 Design Guidelines

The guidelines listed below are to be followed unless the Department provides other guidance. In general, the placement and hydraulic capacity of a storm drainage facility should be designed to consider:

- Damage to adjacent property
- Traffic interruption due to flooding
- Traffic service requirements
- Existing utilities
- Minimization of erosion at outlets
- Proposed staging of large construction projects to maintain an outlet throughout the construction project

7.2.1 Design Storm Frequency

The guidelines listed below are to be followed unless the Department provides other guidance. In general, the storm drainage components listed below should consider the following design storm frequencies:

Lateral longitudinal pipes for storm drains shall be designed to accommodate the 10-year frequency storm. Median storm drains shall be designed for a 50-year frequency.

If the storm drain has a cross-drain pipe, then the cross-drain will be designed in accordance with the guidance provided in chapter 8 and any of the Department's system downstream of that junction will be designed based on the same design frequency.

Storm drain systems shall be designed to accommodate the 50-year storm in areas where the flow has no outlet except through the storm drain system. The design should accommodate the 50-year storm when failure of the drainage system could result in flooding or inundation of the roadway in areas such as low points in cuts or depressed roadways. If the flow can overtop the curb and escape overland, the 50-year design criterion is not required.

Where no significant ponding can occur, check storms are normally unnecessary. Where the use of check storms are warranted, the check storm should normally be a 50-year storm. A check storm should be used to check the system (including inlets) if the system terminates at a sag vertical curve where ponding to hazardous depths could occur, or if a sizeable area which drains to the highway could cause unacceptable flooding during events greater than the design event. If a project is located in a FEMA study area, a check storm is required for a 100-year event. Refer to Chapter 6 for additional guidance regarding the 100-year design criteria.

7.2.2 Maximum Structure Spacing

Drainage structures shall be spaced to facilitate regular maintenance. Adequate access is required for inspection and cleanout of storm drain systems. The following distances in Table 7.1 are the maximum allowable spacing intervals between access points within a closed storm drain system:

Table 7.1 Maximum Structure Spacing

Pipe Size	Maximum Spacing Interval
≤ 36 in.	400 ft
> 36 in.	600 ft

7.2.3 Conduit Criteria

7.2.3.1 Minimum Pipe Size and Material

The guidelines listed below are to be followed unless the Department provides other guidance. In general, pipe sizes and materials should adhere to the following guidelines:

New round pipe under a roadway (roadway pipe) shall have a minimum diameter of 24" if the roadway geometry allows. New round pipe under side drains (storm sewer pipe) shall have a minimum size of 18" if the geometry allows, but otherwise 15" is permitted. Consideration should be given to arch pipe if round pipe will not fit the geometry or if the hydraulic grade line (HGL) needs to be lowered. Specific projects may dictate a minimum pipe size larger than 18 and 24 inches to account for sediment accumulation and clogging, such as in flat terrain.

Storm sewer pipe material selection for all classifications of a roadway is based on the site specific geotechnical, environmental, and regional conditions. For pipe material selection, refer to the Department's GFO 3-22.1.

7.2.3.2 Minimum Cover / Clearances

The minimum allowable depth of cover for all conduits under design loads (pipes, boxes, etc.) shall be 1 ft, measured from the bottom of the subgrade to the outside surface of the pipe.

The minimum clearance between underground utilities and the exterior surface of storm sewer conduits shall have a minimum clearance of 18 inches, but they can go to 1 ft from the exterior crown of the culvert if approved by the State Utility Engineer. A 1 ft minimum cover will be desired from the top of pipe to top of ground in areas where pipe is no longer under the roadway.

Minimum cover shall be maintained at all points where a pipe is beneath travel lanes or shoulders. In particular, this may become an issue when designing a pipe to connect to the catch basin at the sag point of a steep grade. The pavement grades between the sag inlet and the next upstream curb inlet will be curved. However, the pipe connecting the two inlets will be straight. Thus, if the pipe is at or near minimum depth of cover at the catch basins, the depth of cover will be less than allowable at some point near the middle of the pipe run. In extreme cases, the top of the pipe might even "daylight." An additional catch basin placed at the point of minimum cover will usually be sufficient to correct this problem. Showing catch basins and manholes at the correct scale on the roadway profile drawings will facilitate checking of the minimum cover criteria (refer to Chapter 6 for more information on structures). [(7-5)]

7.2.3.3 Minimum / Maximum Velocity

The guidelines listed below are to be followed unless the Department provides other guidance. In general, the following minimum and maximum velocities are provided:

- Generally, storm drains should be designed to provide a velocity of at least 3 ft/s to prevent silting [(7-1)] at a 2-year design frequency if possible.

- The designer should strive to keep velocities between 5 and 7 ft/s in the storm sewer system at the design frequency. The maximum velocity should not exceed 10 ft/s to prevent excessive head losses. Where velocities exceed 10 ft/s, consider adding a drop inlet to include some of the elevation change at the inlet.[(7-1)] The 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 must not exceed the manufacturer's recommendations.

Refer to Section 7.3.2 in this chapter for a more in-depth discussion of minimum grades for closed conduits.

7.2.4 Data Collection and Preliminary Sketch

The design of a storm drain system evolves as a project develops. Preliminary sketches or schematics featuring the basic components of the intended design are useful and serve as a starting point for the storm drainage design. The designer should acquire or address the following minimum necessary information:

- Project survey information, including existing utility locations (look for potential conflicts)
- Off-site drainage information, including land-use patterns and soil types
- Existing drainage information, including information on the existing storm drainage system and existing pipe outlets
- Local information, including comprehensive stormwater management plans and floodplain ordinances

- Federal and state regulatory requirements
- Flood elevations and historical high water marks
- Water quality requirements at environmentally sensitive discharge points

The following should be included in the preliminary sketch:

- Watershed areas and land use
- Existing drainage patterns
- Plan and profile of the roadway
- Roadway cross section
- Typical sections
- Street and driveway layout with respect to the project roadway
- Underground utility locations and elevations
- Locations of proposed retaining walls, bridge abutments, and piers
- Logical inlet and access hole locations
- Preliminary lateral and trunk line layouts
- Clear definition of the discharge points and characteristics

Unless the proposed system is very simple and small, the designer should develop a preliminary sketch as described above. The next step in creating a preliminary design of the storm drainage system is discussed in Section 7.4.

7.2.5 Cooperative Storm Drainage Projects

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.

In order to keep down the tendency for expanding costs in the design phase of highway drainage the following guide should be followed:

- Drainage facilities upgrading in the project vicinity, but not vital to the project, shall be the responsibility of the controlling agency for that area. Such areas may include cities, counties, railroads, schools, private concerns, etc. The state may notify the appropriate agency of a deficiency found during the design process.

7.2.6 Outlet Concerns

In the design of a storm drain system, establish the location of the pipe outlets. The outlets become one of the control points that will influence the grade and the subsequent design of the system. Always strive for a gravity flow system. The flow line of the outlet structure should be equal to or higher than the flow line of the outfall conveyance to avoid the need of a pump station. Pumping stations are to be avoided except in extreme circumstances and never proposed without consultation with the Department.

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. Table 7-3, "Frequencies for Coincidental Occurrence," found in FHWA's Urban Drainage Design Manual, HEC 22 3rd Ed., Sept 2009 can be used for estimating the design frequency discharges for computing tailwater elevations.

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. If the pipe outlet structure cannot be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outlet discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipating structure could be used at the storm drain outlet (See Chapter 8). Either method of stream bank stabilization may require coordination with the United States Army Corps of Engineers (USACE) for additional permitting. See Chapter 2 for agency coordination information.

Because highway systems may increase peak discharge and volume due to increases in the impervious area and decreases in the time of concentration or lag time, accumulation or diversion of flow may also result in an increase in runoff at storm drain outlets. The channel stability of the discharge channels/storm drain systems should be assessed, especially when there are significant changes in discharges due to highway projects or developments.

When necessary, backflow protection should be provided in the form of flap gates. These gates offer negligible resistance to the release of water from the system, and their effect upon the hydraulics of the system may be neglected.

7.2.7 Access Holes

Access holes are used to provide entry to continuous underground storm drains for inspection and cleanout. When entry into the system can be provided by a grate inlet, some agencies opt to use these in lieu of access holes. The use of grate inlets provides the benefit of achieving extra stormwater interception with minimal additional cost.

The following are some typical locations where access holes should be specified:

- Where two or more storm drains converge
- At intermediate points along tangent sections
- Where pipe size changes
- Where an abrupt change in alignment occurs
- Where an abrupt change of the grade occurs

Access holes should not be located in traffic lanes. The spacing of access holes should be in accordance with Section 7.2.2.

7.2.8 Curved Alignment

Curved storm drains are permitted where necessary. Long-radius bend sections are available from many suppliers and are a more preferable means of changing direction in pipes 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 conveyance system. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Using large access holes solely for changing direction may not be cost effective on large-size storm drains.

7.3 Hydraulics of Storm Drain Systems

Hydraulic design of storm drainage systems requires an understanding of basic hydrologic and hydraulic concepts. Important hydraulic principles include flow classification, conservation of mass, conservation of momentum, and conservation of energy.

The desired flow regime for the design of a drain system should be open channel flow. Pressure flow, or the surcharging of drain systems, is not as desirable, but can be accommodated if adequate separation is provided within the storm structure(s) to withstand pressurized flow. A check of the HGL should also be included to evaluate the containment of the HGL within the drain system as well as energy losses for the desired design storm and larger storm events. The HGL check is needed to verify that the pressure flow (or surge) within the drain system for larger storm events is controlled and released at outlet points where flooding can be minimized. A factor of safety is often desired where headroom within the drain system is needed for pressure flow as supported from the HGL check.

The designer should consult Chapter 5 for a general discussion of the above mentioned hydraulic principles; the following sections assume a basic understanding of these topics.

7.3.1 Sizing of Storm Drain

7.3.1.1 Full Flow

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's equation. The Manning's equation was introduced in Chapter 4 and further explained in Chapter 5 for computing the flow capacity for roadside and median channels.

For any shape of conduit, Manning's equation, as introduced in the earlier chapters of this manual, should be used. However, for circular storm drains flowing full, where the hydraulic radius equals the diameter divided by 4 (R = D/4), Manning's equation solved for V and Q, becomes:

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{7.1}$$

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \tag{7.2}$$

Where:

Q = Flow rate, ft^3/s

V = Mean velocity of flow, ft/s

n = Manning's coefficient of channel roughness

D = Diameter of pipe, ft

S = Slope of the energy grade line, ft/ft

Equation 7.2 may be rearranged to solve for the diameter directly.

$$D = \left(\frac{Q\,n}{0.463\,S^{0.5}} \right)^{0.375} \qquad (7.3)$$

Nomographs have been developed as an alternate method to solve the Manning's equation for full flow in circular conduits. For guidelines on using nomographs, the designer should reference the FHWA publication, *Hydraulic Design of Highway Culverts (*HDS-5*).* [7-4]

A full table of the Manning's coefficient for various storm drain materials are provided in Appendix D. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

It is the Department's policy to use concrete pipe for the design of all storm drains. For round storm sewer system pipes, Chapter 8 provides the Department's determination of the hydraulic equivalency of alternate pipe materials. If an alternate pipe type is allowed by specification, the size of the alternate pipe type supplied shall be determined based on the hydraulic equivalency in Table 8.4 and sampling shall be performed per GFO 3-22.1.

An investigation will be conducted by Area personnel of existing drainage structures in similar geological areas to determine their age and condition. This should include evaluations of potential abrasion, pollution and other physical factors which might affect the drainage structure. Written information of this investigation should be included in the materials write-up for the project.

7.3.1.2 Part-Full Flow

The hydraulic elements graph in Figure 7.1 is provided to assist in the solution of the Manning's equation for part-full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

- Peak flow occurs at 93% of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
- Velocity in a pipe flowing half-full is the same as the velocity for full flow.
- Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
- As the depth of flow drops below half-full, the flow velocity drops off rapidly.

The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape.(7-3)

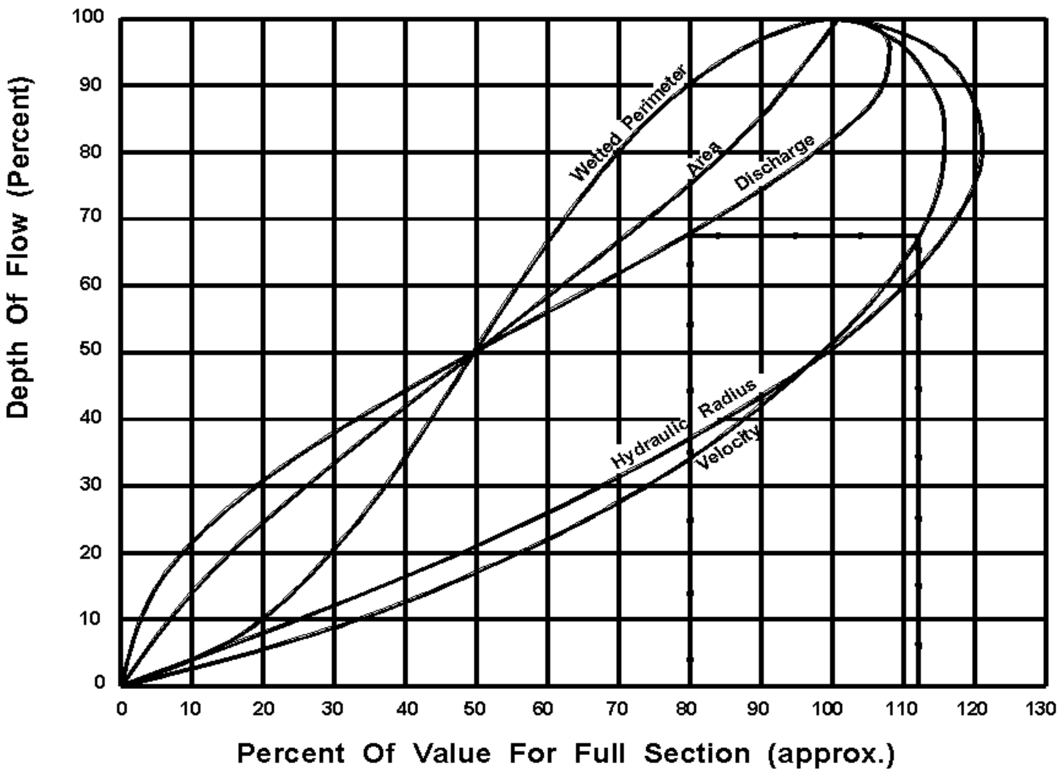

Figure 7.1 - Hydraulic elements chart

7.3.2 Minimum Grades

As stated in 7.2.3, all storm drains should be designed such that velocities of flow will not be less than 3 ft/s. For very flat grades, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to verify that there is sufficient velocity (i.e., 3 ft/s) in all drains to deter settling of particles. Minimum slopes required for a velocity of 3 ft/s can be calculated by the rearranged Manning's equation (7.4), or obtained using Table 7.2.

$$S = 2.87\left[\frac{nV}{D^{0.67}}\right]^2 \qquad (7.4)$$

Where:

S = Slope of the energy grade line, ft/ft

n = Manning's coefficient of channel roughness

V = Mean velocity of flow, ft/s

D = Diameter of pipe, ft

Table 7.2 Minimum slopes necessary for velocity of 3 ft/s in circular pipes flowing full

Pipe Size (in)	Full Pipe (ft^3/s)	Minimum Slopes (ft/ft)		
		n = 0.012	n = 0.013	n = 0.024
15	3.68	0.0028	0.0032	0.0111
18	5.30	0.0022	0.0026	0.0087
21	7.22	0.0018	0.0021	0.0071
24	9.43	0.0015	0.0017	0.0059
27	11.93	0.0013	0.0015	0.0051
30	14.73	0.0011	0.0013	0.0044
33	17.82	0.00097	0.0011	0.0039
36	21.21	0.00086	0.0010	0.0034
42	28.86	0.00070	0.00082	0.0028
48	37.70	0.00059	0.00069	0.0023
54	47.71	0.00050	0.00059	0.0020
60	58.90	0.00044	0.00051	0.0017
66	71.27	0.00038	0.00045	0.0015
72	84.82	0.00024	0.00040	0.0014

7.4 Design Procedures

This section will focus on the design procedures for a system including the calculations necessary for determining pipe sizes and the evaluation of the hydraulic gradeline. The following subheadings under 7.4 Design Procedures will follow the steps required in the design progression.

7.4.1 Energy Loss Estimation for Preliminary Layout

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 7.5. Applicable coefficients (K_{ah}) are tabulated in Table 7.3. This method can be used to estimate the initial pipe flow line (F.L.) drop across an access hole or inlet structure to offset energy losses at the structure. Where pipe sizes change at a box, the crowns of the pipes normally should match rather than the flow lines. The flow line drop is then used to establish the appropriate pipe invert elevations. However, this method is used only in the preliminary design process and should not be used in the EGL calculations. For calculation of the HGL, a more detailed and precise procedure will be used (see Section 7.5).

$$H_{ah} = K_{ah}\left(\frac{V_O^2}{2g}\right) \quad (7.5)$$

Where:

H_{ah} = Estimated energy loss (head loss) across the structure, ft

K_{ah} = Head loss coefficient as illustrated in Figure 7.2

V_O = Velocity of flow leaving structure in outflow pipe, ft/s

g = Acceleration of gravity (32.2 ft/s^2)

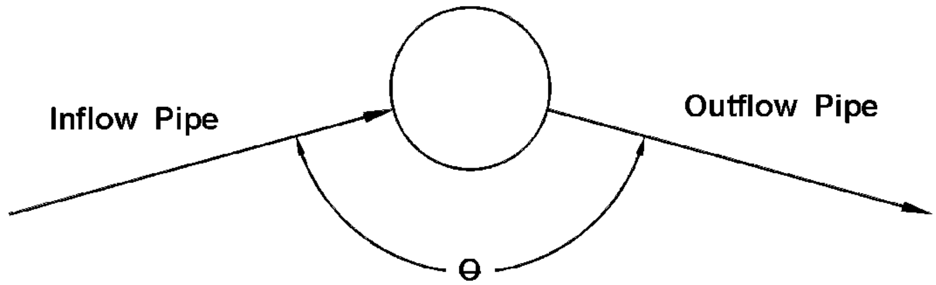

Figure 7.2 - Interior angle

Table 7.3 Head loss coefficients (Adapted from 7-3)

		K_{ah}
Inlet - straight run		0.50
Inlet - angled through	90°	1.50
Manhole - straight run		0.15
Manhole - angled through	90°	1.00
	120°	0.85
	135°	0.75
	157.5	0.45

7.4.2 Preliminary Layout

The subsequent procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. 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 (see Section 7.5).

Discharge Computations for Pipe Sizing

At each point in the system, the drainage area, A, served by the specific inlet is determined, along with the runoff coefficient, C (runoff coefficient values can be found in Chapter 4). These two values are multiplied to determine the parameter "CA" which, is then added to the total "CA" values computed at all of the upstream inlets.

The total flow time from the beginning of the system to the point of interest is then computed. This flow time is used to determine a value of rainfall intensity from the intensity-duration-frequency (IDF) curves for that location. This is multiplied by the total of the “CA” values to determine the design discharge for the site.

The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less than this total. The Rational Method is the most common means of determining design discharges for storm drain design. The time of concentration is very influential in the determination of the design discharge using the Rational Method. The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of interest. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration for inlet spacing is the time required for water to flow from the hydraulically most distant point of the unique drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

The time of concentration for pipe sizing is defined as the time required for water to travel from the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest.

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the high runoff coefficient (C value) and high intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.

- Secondly, calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

- First, calculate the runoff from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.

- Secondly, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The portion of the larger primary area to be considered is determined by the following equation:

$$Ac = A\,(tc_1 / tc_2) \tag{7.6}$$

Where:

Ac	=	Most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area.
A	=	Area of the larger primary area
tc_1	=	Time of concentration of the smaller, less pervious, tributary area
tc_2	=	Time of concentration associated with the larger primary area as is used in the first calculation

The C value to be used in this computation should be the weighted C value that results from combining C values of the smaller less pervious tributary area and the area Ac. The area to be used in the Rational Method would be the area of the less pervious area plus Ac. This second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, the results of these calculations should be compared, and the largest value of discharge should be used for design.

The preliminary design of storm drains can be accomplished by using the preliminary storm drain computation sheet provided in Figure 7.3 and the following steps:

Step 1 Determine inlet location and spacing as outlined earlier in this chapter.

Step 2 Prepare the plan layout of the storm drainage system establishing the following design data:

a. Location of storm drains

b. Direction of flow

c. Location of access holes

d. Check crossing with existing utilities located during the preliminary sketch (e.g., water, gas, underground cables, and existing and proposed foundations)

Step 3 The CA values should have already been computed and shown on the Inlet Design Data Form for unclogged inlets so they should not have to be redone. They can be entered on Column 7 of the Preliminary Storm Drain Computation Sheet. For the most upstream catch basin in the system, determine the following:

- the drainage area, A_r, runoff coefficient, C_r, and time of concentration, Tc_r, for the roadway
- the drainage area, A_o, runoff coefficient, C_o, and time of concentration, Tc_o, for any off-site runoff to that catch basin

Step 4 Compute "Sum CA" for the catch basin as

$$\sum CA = C_r A_r + C_o A_o$$

Step 5 Determine the time of concentration, Tc, for the first catch basin as the longest of Tc_r, Tc_o and 5 minutes. Determine the rainfall intensity, i, corresponding to the time of concentration from the IDF curves which apply to the project site.

Step 6 Determine the design flow rate as:

$$Q = \left(\sum CA\right) i$$

Step 7 For each subsequent catch basin, determine the drainage area, runoff coefficient and time of concentration for the roadway, and any additional off-site areas draining to that catch basin. Compute

$$\sum CA = (Upstream \sum CA) + C_r A_r + C_o A_o$$

Where C_r , A_r , C_o, and A_o are as defined in Step 3.

Step 8 Determine the time of concentration, Tc, for the catch basin as the longest of the following:

- Longest flow time for roadway flows to the inlet, Tcr

- Longest flow time for off-site flows to the inlet, Tco

- [Upstream Tc] + upstream pipe travel time as determined from the pipe capacity computations

Step 9 Determine the rainfall intensity, i, corresponding to the time of concentration from the IDF curve which applies to the project site.

Step 10 Determine the design flow rate

$$Q = \left(\sum CA\right)i$$

Step 11 Repeat Steps 5 through 8 for each catch basin, proceeding in the downstream direction to the system discharge point.

Step 12 Complete the design by calculating the hydraulic grade line as described in Section 7.5. The design procedure should include the following:

- Storm drain design computations can be made on the computation sheet as illustrated in Figure 7.3.

- All computations and design sheets should be clearly identified. The designer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be provided.

- If the designer chooses to use software for assistance in storm drain design computations, the output should be formatted in such a way to agree with the computation sheet shown in Figure 7.3.

PRELIMINARY STORM DRAIN COMPUTATION SHEET

COMPUTED BY ______________________ DATE ________________
CHECKED BY ________________________ DATE ________________
PAGE ________ OF _________

ROUTE ______________________________
SECTION ______________________________
COUNTY ______________________________

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	SLOPE	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP
FROM (1)	TO (2)	( ) (3)	INC ( ) (4)	TOTAL ( ) (4)	(6)	INC ( ) (7)	TOTAL ( ) (8)	INLET (MIN) (9)	SYSTEM (MIN) (10)	(IN/HR) (11)	(FT^3/S) (12)	( ) (13)	(/) (14)	(FT^3/S) (15)	FULL (FT^3/S) (16)	DESIGN (FT^3/S) (17)	(MIN) (18)	U/S ( ) (19)	D/S ( ) (20)	( ) (21)

Figure 7.3 - Preliminary Storm Drain Computation Sheet

7.5 Energy Grade Line / Hydraulic Grade Line

The designer should reference Chapter 4 for an introduction of the energy equation and for a discussion on the EGL and HGL.

Knowing the location of the EGL is critical to understanding and estimating the location of the HGL. The HGL is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. Refer to Figure 7.4, as well as Figures 4.3 and 4.4 in Chapter 4 for the application of the energy equation in open channel flow and pressure flow.

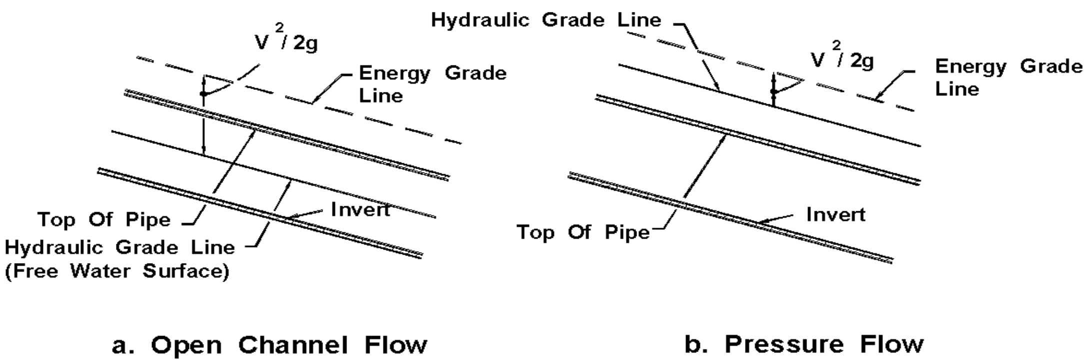

Figure 7.4 - Hydraulic and energy grade lines in pipe flow

In storm drains, the HGL location varies and corresponds to one of the two flow conditions listed below:

- Open channel flow - When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface.
- Pressure flow - When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe and is the level to which water would rise in a vertical tube at any point along the pipe.

Full gravity flow, a specific state of open channel flow, can be classified as the flow in the pipe just before reaching the point where the pipe is flowing full. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL is still at the water surface, which coincides with the crown of the pipe.

Inlet surcharging and possible lid displacement of access holes can occur if the hydraulic grade line rises above the ground surface. Storm drainage systems can often alternate between pressure and open channel flow conditions from one section to another. The designer should check pipe sizes and inverts to prevent this type of hydraulically surcharged condition.

7.5.1 Evaluating Tailwater

For each run of pipe the hydraulic grade line analysis must begin from a "known" tailwater elevation. For the first pipe in the closed drainage system, this elevation should be determined based on a hydraulic analysis of the channel or other conveyance at the system discharge location. Otherwise, the tailwater elevation will have to be determined from an analysis of the downstream pipe.

At the system discharge location, the tailwater conveyance can typically be approximated assuming open channel flow with a normal depth and flow velocity, which can be calculated using Manning's equation as described in Chapters 4 and 5. This will usually result in a measurable amount of velocity head. In these situations, the designer should calculate both the EGL and the HGL downstream of the pipe. Where the tailwater condition is determined by a catch basin or manhole in a surcharged condition (i.e., the water surface is above the crown of the outlet pipe), the EGL and HGL may be assumed to be approximately equal since turbulence within the structure renders the velocity negligible and difficult to determine. However, where the depth in a structure is less than the crown of the outlet pipe, it may be necessary to determine the EGL and HGL separately since the bench in the structure can help to organize the flow. (7-5)

7.5.2 Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. This section presents relationships for estimating typical energy losses in storm drainage systems.

7.5.2.1 Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion at the outlet, the exit loss is as follows:

$$H_o = C_o \left[\frac{V_o^{\,2}}{2g} - \frac{V_d^2}{2g} \right] \tag{7.7}$$

Where:

H_o = Outlet velocity head, ft

C_o = Exit loss coefficient (1.0)

V_o = Average outlet velocity, ft/s

V_d = Channel velocity downstream of outlet, ft/s

Note that, when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For partially full flow where a properly aligned pipe discharges into a channel with moving water, the exit loss may be reduced to virtually zero.

7.5.2.2 Pipe Friction Loss

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

$$H_f = S_f L \tag{7.8}$$

Where:

H_f = Friction loss, ft

S_f = Friction slope, ft/ft

L = Length of pipe, ft

The friction slope in Equation 7.8 is also the slope of the hydraulic gradient for a particular pipe run. Because this design procedure assumes steady uniform and open channel flow, the friction slope will match the pipe slope for part-full flow. Pipe friction losses for full flow can be determined by combining Equation 7.8 with the Manning's equation as follows:

$$S_f = \left(\frac{Q\ n}{1.486\ A R^{2/3}} \right)^2 \tag{7.9}$$

Equation 7.9 is applied for any shape of conduit. For a circular pipe flowing full, the following equation may be developed:

$$S_f = \left(\frac{Q\ n}{0.463\ D^{2.67}} \right)^2 \tag{7.10}$$

Combining Equations 7.8 and 7.10, the following equation may be developed:

$$H_f = 4.665 \left(\frac{Q\ n}{D^{2.67}} \right)^2 L \tag{7.11}$$

7.5.2.3 Bend Loss

The bend loss coefficient for storm drain design is minor, but it can be evaluated using the following formula:

$$h_b = 0.0033\ (\Delta)(V_o^2 / 2g) \qquad (7.12)$$

Where:

Δ = Angle of curvature, degrees

V_o = Average outlet velocity, ft/s

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

7.5.2.4 Junction Loss

$$H_J = \frac{(Q_0 V_0) - (Q_L V_i) - (Q_i V_L\ Cos\,\theta)}{0.5\,g\,(A_0 + A_i)} + h_i - h_o \qquad (7.13)$$

Where:

H_J = Junction loss, ft

Q_o, Q_i, Q_L = Outlet, inlet, and lateral flows, respectively, ft^3/s

V_o, V_i, V_L = Outlet, inlet, and lateral velocities, respectively, ft/s

h_o, h_i = Outlet and inlet velocity heads, respectively, ft

A_o, A_i = Outlet and inlet cross-sectional areas, ft^2

q = Angle between the inflow and outflow pipes (Figure 7.2)

As introduced in Section 7.4.1, the energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Experimental studies have determined that the K value can be approximated by the relationship in Equation 7.14 when the inflow pipe invert is below the water level in the access hole.

7.5.2.5 Access Hole and Inlet Losses

$$K=K_O C_D C_d C_Q C_p C_B \tag{7.14}$$

Where:

K = Adjusted loss coefficient

K_o = Initial head loss coefficient based on relative access hole size

C_D = Correction factor for pipe diameter (pressure flow only)

C_d = Correction factor for flow depth (non-pressure flow only)

C_Q = Correction factor for relative flow

C_p = Correction factor for plunging flow

C_B = Correction factor for benching

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in the FHWA publication, *Hydraulic Design of Highway Culverts* (HDS-5).[7-4] If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K to K_e as reported in Table 7.4. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the FHWA inlet control charts that can be found in HDS-5, *Hydraulic Design of Highway Culverts.*[7-4]

Table 7.4 Entrance loss coefficients.[7-4]

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (grove end)	0.2
Square-edge	0.5
Rounded (radius – D/12)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

7.5.2.6 Relative Access Hole Size

K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes (see Figure 7.2):

$$K_o = 0.1(b/D_o)(1 - \sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta \tag{7.15}$$

Where:

- b = Access hole diameter, ft
- D_o = Outlet pipe diameter, ft
- θ = Angle between inflow and outflow pipes, degrees

7.5.2.7 Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure-flow situations where the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. Therefore, it is only applied in such cases as follows:

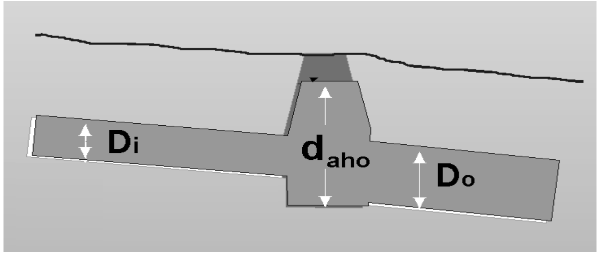

Figure 7.5 - Pipe diameter

$$C_D = (D_O / D_i)^3 \tag{7.16}$$

Where:

- D_o = Outlet pipe diameter, ft
- D_i = Incoming pipe diameter, ft

7.5.2.8 Flow Depth

The correction factor for flow depth is significant only in free surface flow or low pressures, where the d_{aho}/D_o ratio is less than 3.2 and is only applied in such cases. Water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

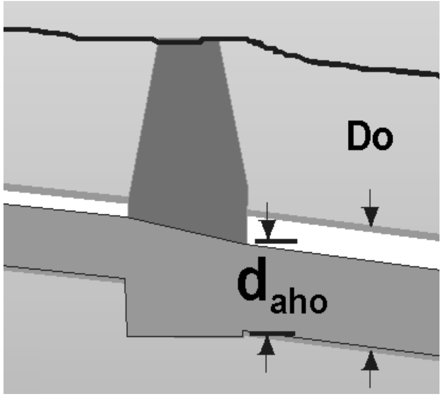

Figure 7.6 - Flow depth

$$C_d = 0.5\left(\frac{d_{aho}}{D_0}\right)^{0.6} \tag{7.17}$$

Where:

d_{aho} = Water depth in access hole above the outlet pipe invert, ft

D_o = Outlet pipe diameter, ft

7.5.2.9 Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \tag{7.18}$$

Where:

C_Q = Correction factor for relative flow

θ = Angle between the inflow and outflow pipes, degrees

Q_i = Flow in the inflow pipe, ft^3/s

Q_o = Flow in the outlet pipe, ft^3/s

As can be seen from Equation 7.18, C_Q is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access hole shown in Figure 7.7 and assume the following two cases to determine the impact of Pipe No. 2 entering the access hole:

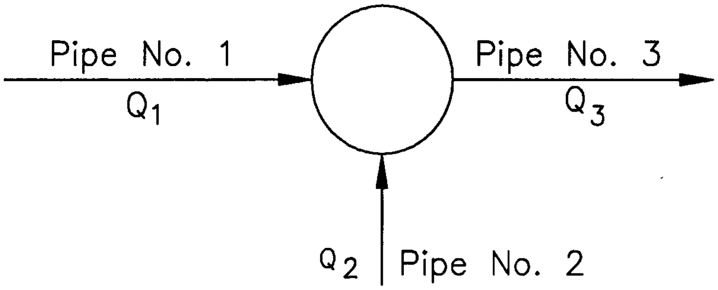

Figure 7.7 - Relative flow

Case 1

Q_1 = 3.2 ft^3/s, Q_2 = 1.0 ft^3/s, Q_3 = 4.2 ft^3/s

$C_{Q1\text{-}3}$ = $(1 - 2\sin 180°)(1 - 3.2/4.2)^{0.75} + 1 = 1.34$

Case 2

Q_1 = 1.0 ft^3/s, Q_2 = 3.2 ft^3/s, Q_3 = 4.2 ft^3/s

$C_{Q2\text{-}1}$ = $(1 - 2\sin 90°)(1 - 3.2/4.2)^{0.75} + 1 = 0.66$

7.5.2.10 Plunging Flow

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. The correction factor is only applied when h > d. The correction factor for plunging flow, Cp, is calculated by the following:

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{aho}}{D_o}\right) \tag{7.19}$$

Where:

C_p = Correction for plunging flow

h = Vertical distance from flow line of incoming pipe to center of outlet pipe, ft

D_o = Outlet pipe diameter, ft

d_{aho} = Water depth in access hole relative to outlet pipe invert as shown in Figure 7.8, ft

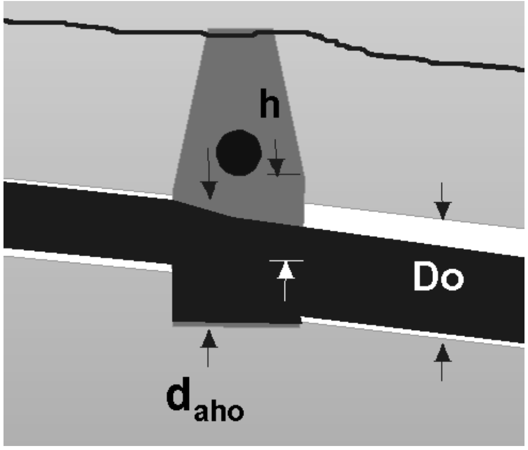

Figure 7.8 - Plunging flow

7.5.2.11 Benching

The correction for benching in the access hole, C_B, is obtained from Table 7.5. Benching tends to direct flows through the access hole, resulting in reductions in head loss (Figures 7.9 and 7.10). For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed. Benching should only be used where energy losses must be kept to a minimum. In areas where energy is not a problem, there is no need to use benching.

Table 7.5 Corrections for benching

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat or depressed floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02

*pressure flow, $d/D_o>3.2$ **free surface flow, $d/D_o<1.0$

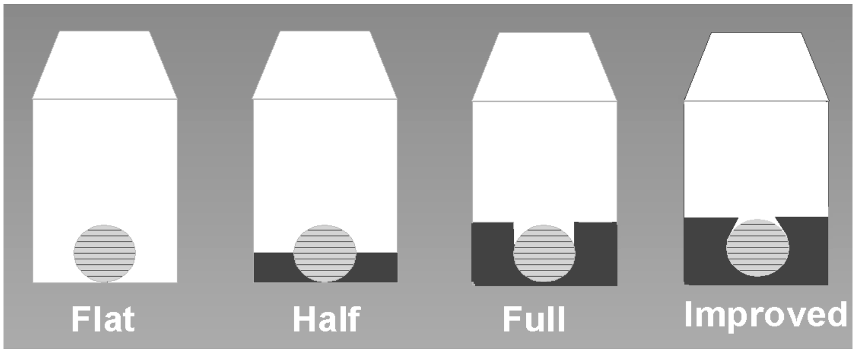

Figure 7.9 - Types of benches

Figure 7.10 - Example of a bench in an access hole

7.5.2.12 Energy Losses

There are other types of energy losses that may be part of the storm drain system, but are not covered here and should be evaluated when present. These losses may be caused by transitions due to expansions and contractions or obstructions. For information on how to handle these losses see HEC 22.

7.6 Energy Grade Line Evaluation

For most storm drainage systems, computer methods are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process in order to better interpret the output from the computer generated storm drain designs.

Figure 7.11 provides a sketch illustrating the use of the two grade lines in developing a storm drainage system. A step-by-step procedure that can be used to manually compute the EGL and HGL can be found in HEC 22 using the two forms shown in Figure 7.12 and 7.13.[(7-3)]

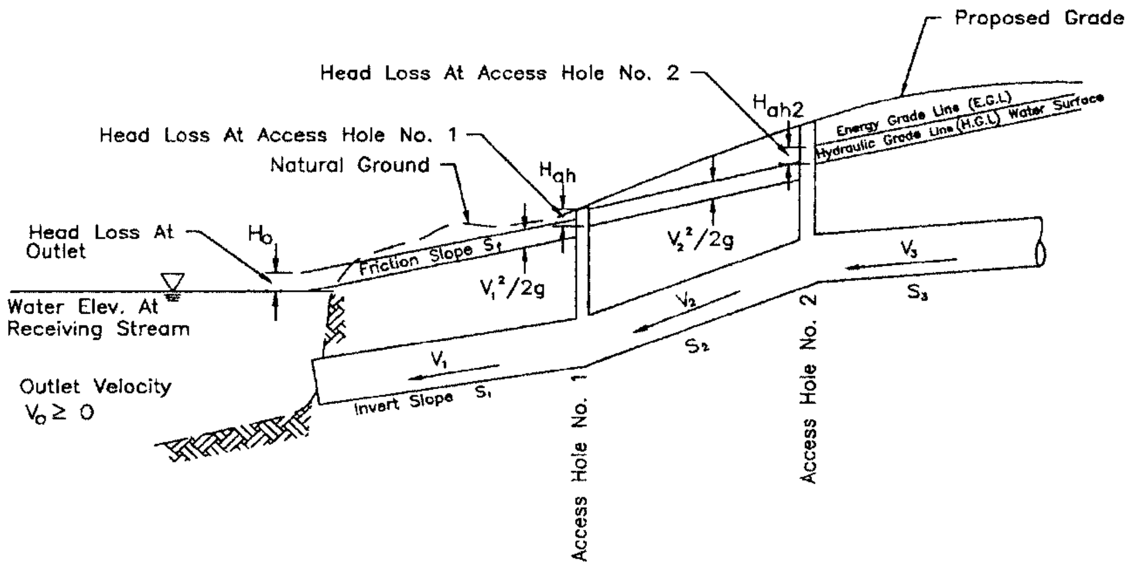

Figure 7.11 - Energy and hydraulic grade line illustration

Before going to the HEC 22 reference that outlines the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines, the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity).

Table A (Figure 7.12) is used to calculate the HGL and EGL elevations while Table B (Figure 7.13) is used to calculate the pipe losses and structure losses. Values obtained in Table B are transferred to Table A for use during the design procedure.

ENERGY GRADE LINE COMPUTATION SHEET – TABLE A

COMPUTED BY ________ DATE ________ ROUTE ____________________

CHECKED BY ________ DATE ________ SECTION ____________________

PAGE ________ OF ________ COUNTY ____________________

INITIAL TAILWATER ELEV. ________________

Struct. ID (1)	D (ft, m) (2)	Q (cfs, cms) (3)	L (ft, m) (4)	V (ft/s, m/s) (5)	d (ft, m) (6)	d_c (ft, m) (7)	$V^2/2g$ (ft, m) (8)	EGL_o (ft, m) (9)	HGL_o (ft, m) (10)	S_f (ft/ft, m/m) (11)	Total Pipe Loss (Table B) (ft, m) (12)	EGL_i (ft, m) (13)	HGL_i (ft, m) (14)	E_a (Table B) (ft, m) (15)	EGL_a (ft, m) (16)	U/S TOC (ft, m) (17)	Surface Elev. (ft, m) (18)

Figure 7.12 - Energy grade line computation sheet – Table A

ENERGY GRADE LINE COMPUTATION SHEET – TABLE B

COMPUTED BY ______ DATE ______ ROUTE ______

CHECKED BY ______ DATE ______ SECTION ______

PAGE ______ OF ______ COUNTY ______

Struct.	Exit	Pipe Losses (ft, m)						Structure Losses (ft, m)								
ID (1)	H_o (2)	H_f (3)	H_b (4)	H_c (5)	H_e (6)	H_j (7)	Total (8)	E_i (9)	$y+(P/\gamma)$ (10)	DI (11)	E_{ai} (12)	C_B (13)	C_θ (14)	C_P (15)	H_a (16)	E_a (17)

Figure 7.13 - Energy grade line computation sheet – Table B

EGL computations begin at the outlet and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow or full barrel flow, pipe and access hole losses are summed to determine the upstream EGL levels. **If supercritical flow occurs, pipe and access losses are not carried upstream**. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

7.7 Computer Programs

A variety of computer programs are available to facilitate storm drain design. The use of any of these programs is acceptable, provided the program substantially conforms to the theory and methods described in HEC 22. StormCAD is the Department's preferred program used for storm drain design.

7.8 Additional Guidance

The components and guidelines listed below should be considered unless determined not to be applicable:

- In an effort to minimize excavation costs, a storm drain should be designed as close to the surface as possible while meeting minimum cover and/or hydraulic requirements.

- Tip: Coordinate with utility locations. Gravity systems such as sanitary sewers should be closely checked for conflicts. Pressure fed systems like water and gas can usually be routed to avoid the gravity flow systems.

- Drainage facilities upgrading in the project vicinity, but not vital to the project, shall be the responsibility of the controlling agency for that area. Such areas may include cities, counties, railroads, schools, private concerns, etc. The state may notify the appropriate agency of a deficiency found during the design process.

- Where the state highway right of way contains a deficient facility and its correction is not vital to the project under design, the situation will be duly noted and evaluated with regard to safety. Dependent upon the risk involved, a future project may be scheduled.

R7 Chapter 7 References

1. American Association of State Highway and Transportation Officials (AASHTO). 2014. Drainage Manual, 1st Ed.

2. American Concrete Pavement Association (ACPA). 2011. Concrete Pipe Design Manual.

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4. Schall, James D., Thompson, Philip L., Zerges, Steve M., Kilgore, Roger T., Morris, Johnny L. 2012, Hydraulic Design of Highway Culverts Third Edition, Hydraulic Design Series No. 5, FHWA-HIF-12-026. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.

5. Tennessee Department of Transportation (TDOT). 2013. Drainage Manual. Roadway Design Division.

Chapter 8: Culverts

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8.1 Introduction

This chapter provides design procedures for the hydraulic design of highway culverts that are based on FHWA Hydraulic Design Series No. 5 (HDS 5), *Hydraulic Design of Highway Culverts*.[8-6] (http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm)

This chapter includes the following:

- Results of the culvert analysis using the HY-8 culvert analysis software [8-4]
- Summary of the design philosophy contained in the AASHTO Highway Drainage Guidelines, Chapter 4 [8-1]

8.1.1 Definition

A culvert is a drainage structure primarily used to convey surface water through embankments that are often constructed in a variety of shapes, sizes, and materials. Culverts are defined according to their shape, size, material type, and usage. For example, a culvert can be defined as a single (CS), double (CD), triple (CT) or quadruple (CQ) 10 ft X 7 ft concrete box culvert (where 10 is the horizontal width and 7 is the vertical height), an 18 inch corrugated metal pipe (CMP) side drain pipe, or a 36 inch reinforced concrete pipe (RCP) cross drain pipe.

Culverts are distinguished from bridges in that they are usually covered with embankment material and are composed of structural material around the entire perimeter, although some are bottomless. Box, pipe, or arched culverts that have a clear span width of 20 ft or less, as measured parallel to the roadway centerline between the outermost hydraulic ends, are considered to be a culvert by definition.

For box, pipe, or arched culverts with a clear-span width greater than 20 ft, the culvert is defined as a bridge culvert and located in the bridge category for design criteria. For example, a double or CD 10 ft X 10 ft box culvert with a 1 ft wide center wall that has a total clear span width of 21 ft is considered a bridge culvert. Refer to Chapter 11 of this manual for more information on bridge design for a bridge culvert.

One exception to the 20 ft clear span width limit is a multi-barrel pipe culvert. Multi-barrel pipe culverts may exceed the 20 ft clear span width and still be called a culvert if the spacing between the culverts is greater than half a barrel diameter. Alternatively, a skewed (or angled) structure would be considered a bridge culvert when its clear-span width measured parallel to the roadway centerline is greater than 20 ft.

Hydraulic structures in this chapter as defined by their clear-span width criteria are designed hydraulically as a culvert and treated as such in this chapter.

8.1.2 Symbols

To provide consistency within this chapter and throughout this manual, the symbols given in Table 8.1 will be used. These symbols were selected because of their wide use in culvert publications.

Table 8.1 – Symbols and definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
AHW	Allowable HW	ft
B	Barrel width	in or ft
D	Culvert diameter or barrel height	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
g	Acceleration due to gravity	ft/s^2
H	Sum of $H_E + H_f + H_v$	ft
H_b	Bend head loss	ft
H_E	Entrance head loss	ft
H_f	Friction head loss	ft
H_L	Total energy losses	ft
H_o	Outlet or exit head loss	ft
H_v	Velocity head	ft
h_o	Hydraulic grade line height above outlet invert	ft
HW	Headwater depth (subscript indicates section)	ft
k_E	Entrance loss coefficient	Dimensionless
L	Length of culvert	ft
n	Manning's roughness coefficient	Dimensionless
P	Wetted perimeter	ft
Q	Rate of discharge (Flow)	ft^3/s
R	Hydraulic radius (A/P)	ft
S	Slope of culvert	ft/ft
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow with barrel full	ft/s
V_d	Mean velocity in downstream channel	ft/s
V_o	Mean velocity of flow at culvert outlet	ft/s
V_u	Mean velocity in upstream channel	ft/s
g	Unit weight of water	lb/ft^3
t	Tractive force	lb/ft^2

8.2 Design Guidelines

8.2.1 General Requirements

The following guidelines are provided for guidance in the design of culverts:

- All culverts shall be hydraulically designed by this guideline.

- HY-8, WSPRO and the HEC-RAS culvert module are the only computer programs allowed for the hydraulic analysis of a culvert. The FHWA HDS 5 Hydraulic Design of Highway Culverts is also acceptable and available from the FHWA website.

- In designing a replacement culvert where the existing structure has a span of 20 ft or greater measured perpendicular to flow, only the WSPRO or HEC-RAS bridge module should be used for hydraulic analysis. The existing and proposed structures should be analyzed using the same module.

- Survey information shall include topographic features, channel characteristics, high- water information, existing structures, and other related site-specific information.

- For projects funded with federal funds, Section 650.117 of 23 Code of Federal Regulations (CFR) 650A applies and requires that project plans for encroachment locations contain the following:

    1. 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) [8-3] (the largest storm event that may be reasonably estimated such as the 500-year storm event).

    2. The magnitude and water surface elevation of the base storm event, if larger than the overtopping storm. [8-3] *(The base storm event is the 100-year storm event).*

    Note: The overtopping storm event does not need to correspond with the design frequency for which the culvert is designed. The culvert should be designed for the event given in Section 8.2.2 *Design Storm Event.*

- If a long discharge easement off the ROW is required to obtain necessary cross-drain cover, then a higher roadway grade should be used if feasible.

- All new culverts (pipe and box culverts) shall be designed for a beveled edge or radius as shown in the Special and Standard Highway Drawings. Note that the grooved end (bell end), if left in place, may be a substitute for the bevel.

- Unless a specific material is specified, all calculations shall be performed assuming concrete will be used. If the contractor elects to use an alternate material, the structure must be checked and resized. The proper Manning's n for the culvert material (concrete, metal, plastic, etc.) must be used (see Section 8.2.9).

- The detail of documentation for each culvert site shall be commensurate with the risk and importance of the structure. Design data and calculations shall be assembled in an orderly fashion and retained for future reference.

- Any culvert spanning a clear distance of 20 ft or greater along the roadway centerline is to be classified as a bridge culvert in the plans, with the exception of multi-barrel pipe culverts (as noted in Section 8.1.1). See Chapter 11 of this manual for analyses pertaining to bridge culverts.

- For allowable end treatments for pipe culverts (see Section 8.2.9, Table 8.3).

8.2.2 Design Storm Event

Culverts are to be sized to accommodate the following storm events without exceeding the design storm headwater.

- Interstate and state routes: All culverts crossing interstate and state routes shall be designed to meet the headwater and roadway profile elevation criteria listed in Table 8.2 for the 50-year storm event. The 100-year storm is analyzed when FEMA requirements are a consideration. The 200-year check storm may be used for design if the risk is significant, but the 200-year design is not always attainable.

- Roads not designated as state routes: All culverts crossing a roadway not designated as an interstate or state route shall be designed to meet the minimum headwater and roadway profile elevation criteria listed in Table 8.2 for the design storm frequency based on ADT. Although Table 8.2 lists minimum design frequencies less than the 25-year storm event for roads with an ADT of 399 or less, the 25-year storm event is still recommended as a minimum design guideline unless there is information specific to the site that dictates a lower event design.

- Driveway pipe culverts: All driveway pipe culverts (side drain pipes) shall be designed for the 10-year frequency storm. All driveway pipes shall be checked to confirm that the headwater for the 50-year event does not violate the overtopping requirements for the adjacent roadway.

- Temporary cross drains: All temporary cross drains shall be designed based on a 5-year storm frequency.

It is important to note that the roadway will overtop at the nearest low point on the roadway, which does not necessarily correspond with the roadway elevation shown on the drainage cross section. Be aware the side ditch could overtop before the roadway. The design storm frequency and other criteria for culverts are also summarized in Table 8.2.

The definition of overtopping flood is defined by the Federal-Aid Highway Program Manual, Nov. 15, 1979 as "the flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief."

8.2.3 Allowable Headwater

The allowable headwater depth (HW_d), 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, and is limited by one or more of the following:

- To protect the roadway pavement, the minimum allowable freeboard shall be 1 ft as measured from the bottom of the subgrade to the design year HW elevation.
- The HW elevation should not be greater than the elevation at which flow diverts around the culvert.
- For streams with a FEMA designated floodway or in communities that participate in the NFIP, see Section 8.2.15 of this manual for guidance in establishing the HW elevation.

Design criteria for culverts are summarized in Table 8.2.

Table 8.2 – Culvert design criteria

Type	Item	ADT	Return Frequency Years	Check Frequency
Interstate & state highways	Bridge and roadway culverts	NA	50	200
Interstate & state highways	Cross drain pipes	NA	50	200
Interstate & state highways	Median ditches, inlets & storm drains	NA	50	200
Interstate & state highways	Lateral ditches[1], inlets[2] & storm drains[2]	NA	10	25
County/municipal collector or local road[3]	Bridge and roadway culverts or cross drain pipes	1-99[3]	1.5 to 25	5 to 50
County/municipal collector or local road[3]	Bridge and roadway culverts or cross drain pipes	100-399[3]	10 to 25	25 to 50
County/municipal collector or local road	Bridge and roadway culverts or cross drain pipes	400+	25	50

[1] Slope paved ditches should be designed for at least a 50-year return frequency because the liner can be lost if the ditch is overtopped.

[2] Use check storm for design at underpasses and depressed sections where water can only be removed through the storm drain system.

[3] Design flood should be commensurate with the type of road and risk the County/Municipality desires.

8.2.4 Tailwater Relationship

Tailwater relationships vary depending on the particular scenario. The two most common are for 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 bridge culverts, use headwater flood events for meeting standard design criteria and the backwater flood elevation from the dominant stream (downstream) for defining the road grade elevation if it exceeds the elevation from the headwater flooding analysis.

- For roadway culvert outfalls to main streams and tributaries, use Table 7-3, "Frequencies for Coincidental Occurrence," found in FHWA's Urban Drainage Design Manual, HEC 22 34rd Ed., Sept 2009.

- 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, see Chapter 11 of this manual.

8.2.5 Maximum Outlet Velocity / Energy Dissipators

The maximum velocity at the culvert outlet should be examined on a case-by-case basis, which may include reviewing permissible velocities based on the soils and/or vegetative cover at the outlet (see in Appendix E) or a sediment transport calculation examining the streambed shearing stress of the sediment. The culvert design methodology 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 discussed in Section 8.5 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:

- Modification of culvert design features such as flattening the slope
- Channel stabilization
- Energy dissipation

Scour holes at culvert outlets provide efficient energy dissipators. As such, outlet protection for the design storm event should be provided where the outlet scour hole depth computations (see Chapter 5 of HEC-14) indicate that the scour hole:

- Will undermine the culvert outlet
- May cause costly property damage
- Causes a nuisance effect (most common in urban areas)
- 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* (8-7) for scour computations and design of energy dissipators.)

8.2.6 Minimum Required Cover and Clearances

All pipes and most box culverts shall have a minimum cover of 1 ft. If minimum cover cannot be provided over a box culvert the designer should verify the culvert has been designed with adequate structural support to carry traffic. Cover should be checked based on concrete class as well. The minimum roadway clearance over a culvert shall be 1 ft measured from the bottom of the subgrade to the top of the culvert. Underground utilities shall have a minimum clearance of 18 inches, but they can go to 1 ft from the exterior crown of the culvert if approved by the State Utility Engineer. A 1ft minimum cover will be desired from the top of pipe to top of ground in areas where pipe is no longer under the roadway.

8.2.7 Culvert Extensions

All culvert extensions should be evaluated using the hydraulic principles discussed in this chapter. 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 shall only be extended with barrel sizes that are equal to or greater than the existing culvert barrel size. 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 shall be 30 degrees.

Extensions requiring multiple bends shall be limited to a 15 degree maximum deflection angle at each bend and a minimum distance of 20 ft before the next deflection.

Box culverts with a span or rise dimension less than four feet are excluded from new construction, but they may be used for extensions if the Engineer of Record determines it to be more cost effective than using a junction box and precast pipe. The State Bridge Engineer shall be consulted if there is a bend or any other situation other than a straight or continuous extension.

A circular pipe used to extend a box culvert shall have a diameter equal to or greater than the diagonal measurement of the box culvert. An arch pipe is preferred for extensions over round pipe.

8.2.8 Channel Changes

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

8.2.9 Pipe Culverts

New pipe culverts that cross under a roadway (cross drain) shall have a minimum diameter of 24 inches provided that the required amount of cover is achieved. New pipe culverts that cross under a driveway (side drain) shall have a minimum diameter of 18 inches provided that the required amount of cover is achieved. Equivalent arch pipes may be used to achieve proper cover.

For allowable end treatments for pipe culverts:

For cross drain pipe ends within the clear zone, use slope paved headwalls with beveled end cut. When slope paved headwalls are used, a grate will be required for cross drain pipe greater than 30 inches in diameter. Slope paved headwall or flared end sections without grates may be used in the clear zone when they are shielded from traffic by physical barriers.

For pipe ends outside the clear zone, alternate end treatments of slope paved headwall or flared end sections will be allowed. Grates will not be required regardless of size of pipe. Conventional headwall end treatment may be used in special cases.

For side drain pipe locations within the clear zone, use slope paved headwalls with beveled end cut. Grates will be required for side drain pipe within the clear zone that are greater than 24 inches in diameter. For pipe locations outside the clear zone, alternate end treatments of slope paved headwall or flared end sections will be allowed. No grates will be required for pipe located outside of the clear zone.

On two lane roadways, the same end treatment will be given to both ends of pipe.

On four lane roadways, place grates on traffic approach end only for side drains.

A maximum 6:1 slope will be used on pipe cuts within the clear zone. For the purpose of maintenance permits, the slope may be steepened to 3:1 on routes constructed to earlier design standards where constructing a slope of 6:1 would be impractical due to existing slope or ditch and elevation conditions at and around the proximity of a driveway.

It shall be the intent of the Department to use only concrete pipe for roadway pipe installations, including pipe extensions on all roads which comprise the State Highway System. Where “stack pipe” is required to connect inlets to junction boxes, 14 gauge, CCS, roadway pipe will be used. Where “wrap-around” pipe is required to handle median drainage down the fill slopes 14 gauge, CCS paved invert roadway pipe will be used. See Table 8.3 for a complete range of usage of all drawings for pipe end treatment on cross drain and side drain pipe.

Table 8.3 – Complete range of usage of all drawings available for pipe end treatment on cross drain and side drain pipe

Special Drawing No.	Side Drain Inside Clear Zone	Side Drain Outside Clear Zone	Cross Drain Inside Clear Zone	Cross Drain Outside Clear Zone	Median Crossover (4 Lane+)
FE-619	(6) (8)	(2)	(6)	(2)	
HW-614-B			(3)	(2)	
HW-614-SP	(5)	(2)	(1)		(7)

1. May be used on roadway pipe under intersecting side roads using side drain requirements.
2. No grates required.
3. Grates required for pipe greater than 30 inches in diameter.
4. For use in special cases.
5. Grates required for pipe greater than 24 inches in diameter.
6. May be used inside clear zone when shielded from traffic by physical barriers. No grates will be required, regardless of pipe size.
7. Grates required for pipe larger than 24 inches in diameter, pipe end treatment slopes shall be 10:1 regardless of pipe size.
8. May be used inside clear zone for maintenance permits where 3:1 slope is allowed as previously stipulated. Grates required for pipe larger than 24 inches in diameter.

For round storm sewer system pipes, Table 8.4 provides the Department's determination of the hydraulic equivalency of alternate pipe materials. The design shown in the plans will be based on the concrete pipe. If an alternate pipe type is allowed by specification, the size of the alternate pipe type supplied shall be determined based on the hydraulic equivalency in Table 8.4.

For projects on which alternate pipe types are allowed, sampling will be conducted at each location where cross-drain pipes are proposed per GFO 3-22.

An investigation will be conducted by Area personnel of existing drainage structures in similar geological areas to determine their age and condition. This should include evaluations of potential abrasion, pollution, and other physical factors which might affect the drainage structure. Written information of this investigation should be included in the materials write-up for the project.

In areas with apparent abrasion or erosion of the structure due to water laden with sand, gravel or stone, protection should be provided such as a paved invert or other method. Also, reduction of bedload upstream of the structure should be considered.

On low volume (250 ADT or less) secondary roads, uncoated structural pipe may be allowed.

Pipe culvert material alternates shall be as recommended by the Product Evaluation Board. These recommendations shall be shown in the plans. When alternate materials are used that are different from what is assumed in the design calculations, the contractor must perform a hydraulic analysis to account for the different roughness factors. Different materials may require different size structures.

Table 8.4 – ALDOT determination of the hydraulic equivalency of alternate pipe materials

	Concrete Pipe	HDPE Type S Pipe	HDPE Type S Pipe	HDPE Type C Pipe	Corrugated Metal (CMP)					Structural Plate		PVC Pipe	Concrete Pipe
							(Steel)	(Alum.)	Spiral Rib	(Steel)	(Alum.)		
Corrugations in.		Fill Hts<=10'	Fill Hts> 10'		2-2/3 x 1/2	3 x 1	5 x 1	6 x 1	All	6 x 2	9 x 2-1/2		
Design N Value	0.012	0.014	0.015	0.025	0.025	0.028	0.026	0.025	0.014	0.035	0.035	0.011	0.012
Pipe Size in.	12	15	15	18	18				15			12	12
	15	18	18	24	24				18			15	15
	18	24	24	30	24	30			24			18	18
	24	30	30	36	36	36	36		30			24	24
	30	36	36	42	42	42	42		36			30	30
	36	42	42	54	48	54	54	48	42	60	60	36	36
	42	48	48	60	60	60	60	60	48	66	66	42	42
	48	54	60		66	66	66	66	54	78	78	48	48
	54	60			72	78	78	72	60	84	84		54
	60				84	84	84	84	66	96	96		60
	66					96	90	90	72	102	102		66
	72					102	102	96	78	114	114		72
	78					108	108	108	84	120	120		78
	84					120	114	114	90	132	132		84
	90					126	126		96	138	138		90
	96					132	132		102	150	150		96
	102					144	138		114	156	156		102
	108								120	168	168		108
	114								126	174	174		114
	120								132	180	180		120
	126								138	192	192		126
	132								144	198	198		132
	138									210	210		138
	144									216	216		144
	150									228	228		150
	156									234	234		156
	162									246	246		162
	168									252	252		168
	174									264	264		174
	180									270	270		180

For cross drain pipes do an analysis with the N Factor given.
Concrete pipe with the socket end left on (not cut off) will have a much lower entrance head loss than thin walled pipe.
Flowlines cannot be changed to obtain cover if an equivalent sized pipe is selected.
Some pipe sizes are not shown because a smaller pipe size is equivalent to the concrete pipe.
Flowline changes will require a redesign of the system.

Pipe sizes on a horizontal row will carry at least as much flow as the concrete pipe size, but not as much flow as any pipes on the next lower horizontal row.

8.2.10 Box Culverts

New box culverts will have a minimum dimension of 4 ft in both height and width. For pre-existing box culverts having smaller dimensions, a culvert extension may be allowed if the Engineer of Record determines it to be more cost effective than using a junction box and precast pipe. The State Bridge Engineer shall be consulted if there is a bend or any other situation other than a straight continuous extension.

Multiple-barrel culverts shall fit within the natural dominant channel with only minor widening of the channel permissible in order to avoid conveyance loss through sediment deposit in some of the barrels.

8.2.11 Bottomless Culverts

Bottomless culverts are to be used in locations where it is necessary to maintain the natural streambed through the culvert to meet environmental regulatory requirements.

The footings for a bottomless culvert shall be placed below the streambed elevation on scour resistant material. The culvert foundations shall be placed deep enough to withstand the possible channel migration and scour. Due to the potential for scour problems at these sites, a scour analysis shall be performed for all bottomless culverts.

The following are possible alternates to using a bottomless culvert:

- Construct an embedded box culvert.
- Build a small bridge at the site.

8.2.12 Fall

When a culvert is depressed below the streambed at the inlet, the depression is called the Fall. This depression is used to exert more head on the throat section for a given headwater elevation. A hydrodynamic improvement is made to the culvert performance by providing a more efficient control inlet section, which is the throat of the Fall.

For culverts without tapered inlets, the Fall is defined as the depth from the natural stream bed at the face to the inlet invert. For culverts with tapered inlets, the Fall is defined as the depth from the natural stream bed at the face to the throat invert. When Fall is used, a detail should be placed on the plans so that the contractor will build it below the natural ground. For information concerning the design of an improved end treatment, see HDS 5.

8.2.13 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.

Culverts can be sized using the Federal Highway Administration's (FHWA) HY-8 computer model, the culvert design method given in Section 8.3, or computer programs approved by the Department.

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 shall first be determined by performing a backwater analysis using a HEC-RAS or WSPRO water surface profile computer model. See Section 8.2.4 for additional information on tailwater.

8.2.14 Hydraulic Reports

Culverts that meet any of the conditions given in Section 11.3.5 of this manual will require that a hydraulic study be completed. For hydraulic study guidelines, see Chapter 11 Sections 11.3.5, 11.3.6, and 11.3.7 of this manual.

If the project is a resurfacing project, it will not be necessary to show the hydraulic data or analyze the existing drainage structures that are to be extended, unless there is a history of flooding or some indication that the existing structure is undersized. Consideration should be given to replacing structures that have a history of flooding the roadway or the adjacent properties. The standard hydraulic data should be shown on the plans for structures to be replaced.

Widening projects, other than incidental widening such as turn lane shoulders, should be checked for hydraulic adequacy. If the original analysis is available, the check would be limited to determining if the factors used are still valid.

The designer should document for widening and resurfacing projects that the drainage for a project has been reviewed. The location and proposed improvements for inadequate drainage structures should be in the letter. If there are no existing drainage problems or history of flooding, it should be indicated in the letter.

8.2.15 Culverts Located Within a FEMA Flood Zone

If the culvert is located within a FEMA regulatory flood zone, FEMA guidelines must also be satisfied. See Chapters 2 and 11 of this manual for more information on FEMA regulations and hydraulic modeling.

8.3 Typical Information Needed for Design

Design data that is required for culvert design includes, but is not limited to, the following:

- Drainage Area
- Design Flow
- Headwater Depth
- Tailwater
- Roadway Data
- Culvert Data
- Stream Data
- Survey Data

Appendix F of this manual includes forms for design data documentation required for the design of culverts. With regard to the survey data requirements listed above, see the drainage section of ALDOT's Survey Requirements.(8-5)

8.4 Culvert Design Approach

Culvert flow may be non-uniform, gradually and rapidly varying, steady, or unsteady. A comprehensive analysis for these various flow scenarios would be time consuming and difficult. However, the FHWA has developed a design method that is straightforward and relatively easy to implement; the method involves evaluating different types of flow control for the culvert and designing based on the control that reflects the "minimum performance" or least efficient flow condition. For more detail relating to this design procedure and how it was developed, see FHWA HDS 5.

Using this design approach, flow through culverts has been classified on the basis of where the control section is located. A control section is a location where there is a unique relationship between the flow rate and the upstream water surface elevation. Many different flow conditions exist over time, but at a given time the flow is either governed by the inlet geometry (inlet control); or by a combination of the culvert inlet configuration, the characteristics of the barrel, and the tailwater (outlet control). Control may oscillate from inlet to outlet. That is, while the culvert may operate more efficiently at times (i.e., more flow for a given headwater level), it will never operate at a lower level of performance than calculated.

Design charts and nomographs that have been developed from hydraulic tests and theoretical calculations are provided for culvert design, see FHWA HDS 5. Computer programs such as HY-8, provided by the FHWA, have also been developed for culvert design and are available for download (http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/).

8.4.1 Types of Control

As previously stated, culverts may operate in either inlet or outlet control. Table 8.5 shows the factors that must be considered in culvert design for inlet and outlet control.

For inlet control, only the inlet area, the edge configuration, and the shape influence the culvert performance for a given headwater elevation. The headwater elevation is calculated with respect to the inlet invert, and the tailwater elevation has no influence on performance.

For outlet control, all of the factors listed in Table 8.5 affect culvert performance. Headwater elevation is calculated with respect to the outlet invert, and the difference between the energy grade line at the headwater and at the tailwater is the energy that carries the flow through the culvert.

Table 8.5 – Factors influencing culvert performance [(8-6)]

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 Elevation		X

Inlet Control

A culvert flowing in inlet control has shallow, high velocity flow categorized as supercritical. For supercritical flow, the control section is at the upstream end of the barrel (the inlet).

Figure 8.1 shows several different examples of inlet control flow. The type of flow depends on the submergence of the inlet and outlet ends of the culvert. In all of these examples, the control section is at the inlet end of the culvert. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet. Supercritical flow occurs in all of the barrels.

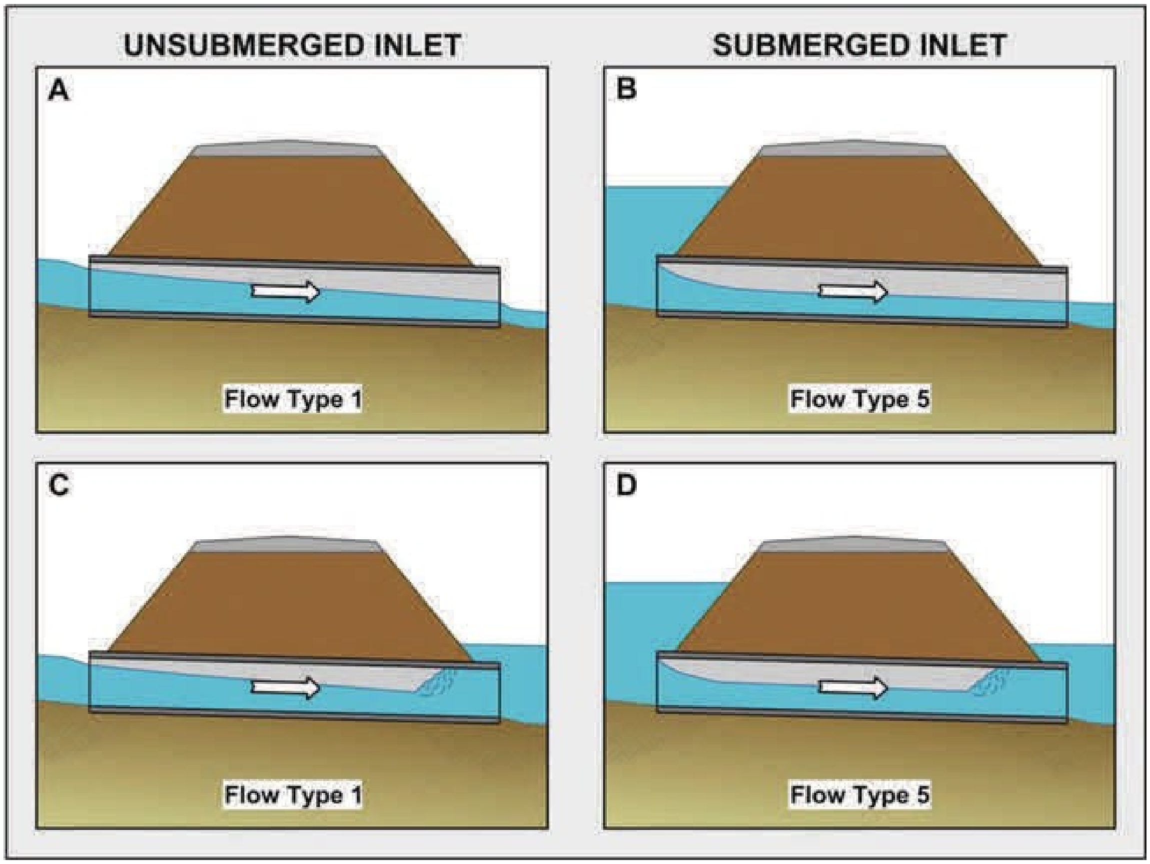


Figure 8.1 – Types of inlet control (8-6)

Culvert Factors Influencing Inlet Control

The following factors influence culverts operating in inlet control:(8-6)

- **Headwater** depth is measured from the invert of the inlet control section to the surface of the upstream pool.
- **Inlet area** is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area, but for tapered inlets the face area is enlarged, and the control section is at the throat.
- **Inlet edge configuration** describes the entrance type. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge.
- **Inlet shape** is usually the same as the shape of the culvert barrel; however, it may be enlarged as in the case of a tapered inlet. Typical shapes are rectangular, circular, and elliptical. Whenever the inlet face is a different size or shape than the culvert barrel, the possibility of an additional control section within the barrel exists.
- **Barrel Slope** influences inlet control performance, but the effect is small. Inlet control nomographs assume a slope of 2% for the slope correction term (0.5S for

most inlet types). This results in lowering the headwater required by .01D. In the computer program HY-8, the actual slope is used as a variable in the calculation.

Hydraulics of Inlet Control Culverts

Inlet control performance is defined by the three regions of flow, two of which are shown in Figure 8.2:

- Unsubmerged
- Transition
- Submerged

For low headwater conditions, as shown in Figure 8.1-A and Figure 8.1-C, the entrance of the culvert operates as a weir. A weir is an unsubmerged flow control section where the upstream water surface elevation can be predicted for a given flow rate.

For headwaters submerging the culvert entrance, as are shown in Figure 8.1-B and Figure 8.1-D, the entrance of the culvert operates as an orifice. An orifice is an opening, submerged on the upstream side, and flowing freely on the downstream side which functions as a control section. The flow transition zone between the low headwater (weir control) and the high headwater flow conditions (orifice control) is poorly defined as shown in Figure 8.2.

Headwater for inlet control can be determined using the inlet control nomographs found in HDS 5[8-6] for each type of culvert.

The type of inlet will affect the operation of a culvert when operating in inlet and outlet control. However, since the inlet is controlling the capacity of a culvert operating in inlet control (supercritical flow occurs in the barrel), the culvert entrance may be modified to improve the culvert performance. The four factors that affect culvert performance in inlet control are inlet edge condition, area, shape, and headwater. By making small modifications to these four factors, the capacity of a culvert may be increased dramatically. Culverts with these improvements are sometimes referred to as improved inlets.

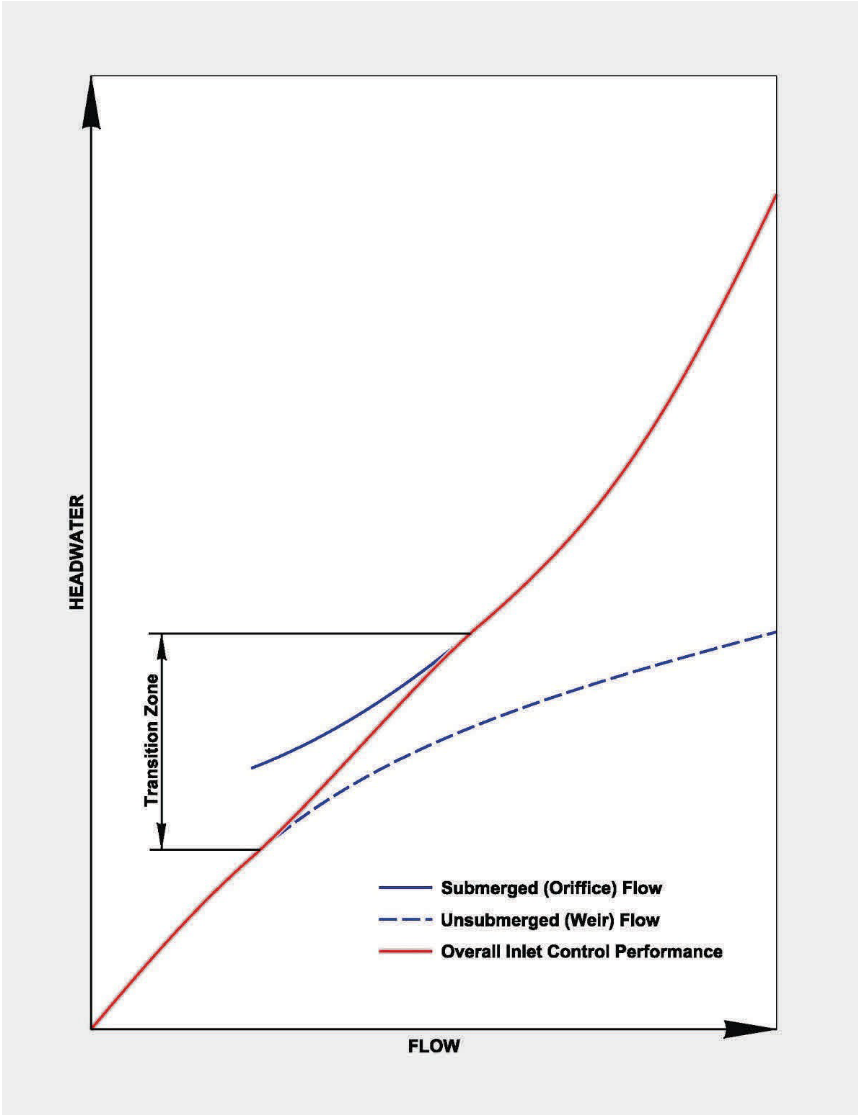


Figure 8.2 – Inlet control curves

Improved End Treatments for Inlets

All culverts operating in inlet control should be evaluated for improvements that consist of the following:

- Beveled-edged inlets
- Side-tapered inlets
- Slope-tapered inlets

Improved end treatments with an enlarged face, by means of a depression (Fall), create more head on the barrel or throat for a given headwater elevation. This causes culvert performance to increase. For further information regarding the design of improved end treatments, see HDS 5.[8-6] See Section 8.5.2 for dimensional limitations for improved inlets.

Economic considerations are important factors in determining the use of inlet improvement beyond the standard beveled edge. Such improvements should be evaluated comparing costs and benefits. Improved inlets are most cost effective on long culverts or to improve the flow of an existing inlet where the pipe can remain.

Outlet Control

A culvert flowing in outlet control will have relatively deep, low-velocity flow, termed subcritical flow or will be flowing full. For both subcritical flow and full barrel flow, the control is at the downstream end of the culvert (the outlet). In outlet control, the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control is located at the barrel exit or further downstream. All of the geometric and hydraulic characteristics of the culvert listed in Table 8.5 play a role in determining culvert capacity.

Figure 8.3 shows various culverts operating in outlet control. In all cases, the culvert is either flowing in subcritical flow or flowing full, and the control section is at the outlet of the culvert.

All of the factors influencing the performance of a culvert in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation affect culvert performance in outlet control (Table 8.5).

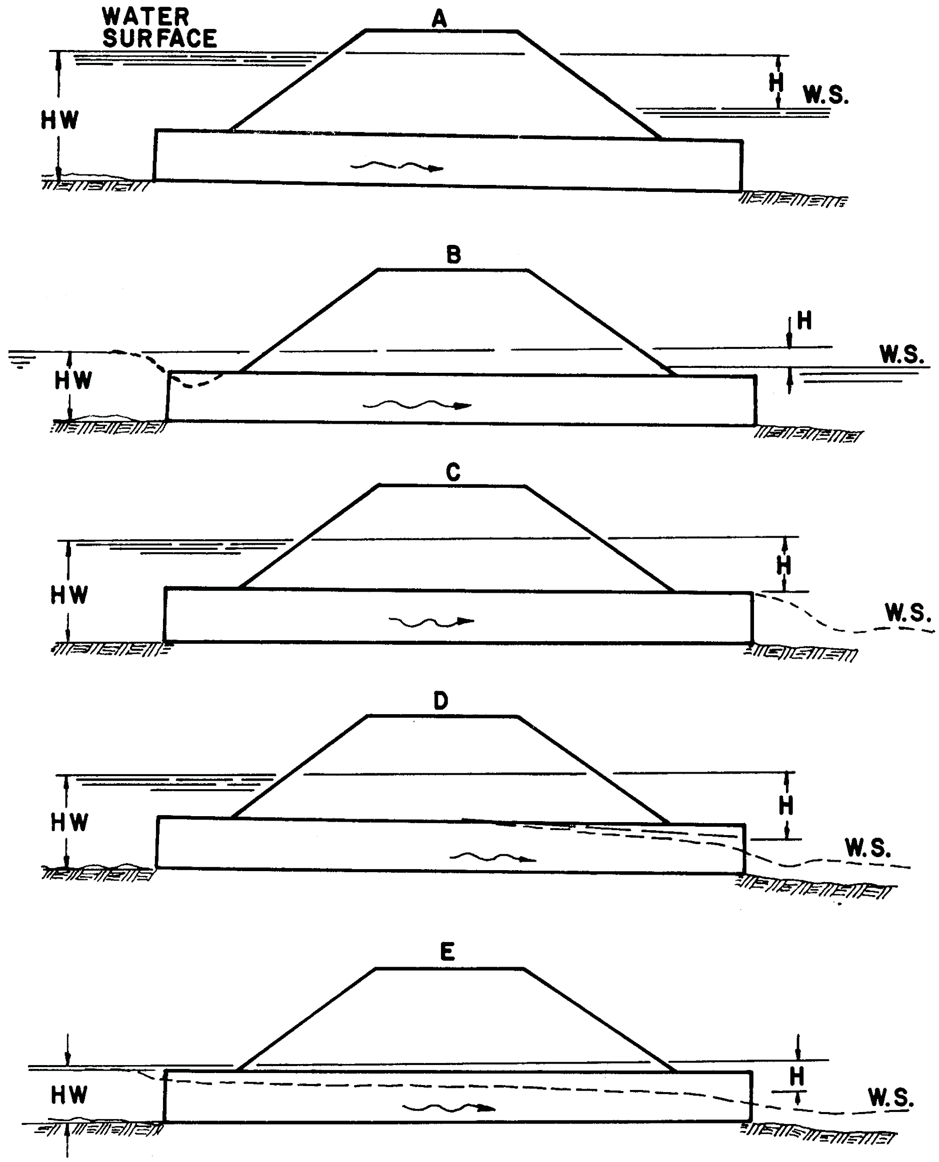


Figure 8.3 – Types of outlet control (8-6)

Culvert Factors Influencing Outlet Control

- **Barrel roughness** is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic roughness coefficient such as the Manning's n value.

- **Barrel area** is a function of the culvert dimensions. A larger barrel area will convey more flow.

- **Barrel shape** is function of culvert type and material. Based on the location of the center of gravity for a given area, a box is the most efficient barrel shape. The arch and the circle are examples of additional, but less efficient, shapes.

- **Barrel length** is the total culvert length from the entrance to the exit of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

- **Barrel slope** is the actual slope of the culvert barrel. The barrel slope is often the same as the natural stream slope. However, when the culvert inlet is raised or lowered, the barrel slope is different from the stream slope.

- **Tailwater elevation** is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation.

Hydraulics of Outlet Control Culverts

Full flow in the culvert barrel, as depicted in Figure 8.3-A, is the most applicable type of flow for describing outlet control hydraulics.

Outlet control flow conditions can be calculated based on energy balance. The total energy (H_L) required to pass the flow through the culvert barrel is made up of the following:

- Entrance loss (H_e)
- Friction losses through the barrel (H_f)
- Exit loss (H_o)

Other losses, including bend losses (H_b), losses at junctions (H_j), and losses at grates (H_g) should be included as appropriate (see Chapter 5 of HDS 5[8-6] for additional discussion of the bend and grate losses).

Entrance losses are a function of the velocity head in the barrel, and can be expressed as a coefficient times the velocity head.

$$H_e = k_e\left(\frac{V^2}{2g}\right) \quad (8.1)$$

Values of k_e based on various inlet configurations are given in Table 8.6.

Table 8.6 – Entrance loss coefficients [8-6]

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
• Projecting from fill, socket end (groove-end)	0.2
• Projecting from fill, sq. cut end	0.5
• Headwall or headwall and wingwalls	
○ Socket end of pipe (groove-end)	0.2
○ Square-edge	0.5
○ Rounded (radius = D/12)	0.2
• Mitered to conform to fill slope	0.7
• *End section conforming to fill slope	0.5
• Beveled edges, 33.7° or 45° bevels	0.2
• Side- or slope-tapered inlet	0.2
Pipe or Pipe-Arch, Corrugated Metal	
• Projecting from fill (no headwall)	0.9
• Headwall or headwall and wingwalls square-edge	0.5
• Mitered to conform to fill slope, paved or unpaved slope	0.7
• *End section conforming to fill slope	0.5
• Beveled edges, 33.7° or 45° bevels	0.2
• Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
• Headwall parallel to embankment (no wingwalls)	
○ Square-edged on 3 edges	0.5
○ Rounded on 3 edges to radius of D/12 or B12 or beveled edges on 3 sides	0.2
• Wingwalls at 30° to 75° to barrel	
○ Square-edged at crown	0.4
○ Crown edge rounded to radius of D/12 or beveled top edge	0.2
• Wingwall at 10° to 25° to barrel	
○ Square-edged at crown	0.5
• Wingwalls parallel (extension of sides)	
○ Square-edged at crown	0.7
• Side- or slope-tapered inlet	0.2

*__Note__: "End sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, these end sections are equivalent in operation to a headwall in inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

The friction loss in the barrel (H_f) is also a function of the velocity head. Based on Manning's equation, the friction loss is:

$$H_f = \left[\frac{K_U\ n^2\ L}{R^{1.33}}\right]\frac{V^2}{2g} \tag{8.2}$$

Where:

- K_U = 29 (English)
- n = Manning's roughness coefficient
- L = Length of the culvert barrel, ft
- R = Hydraulic radius of the full culvert barrel = A/p, ft
- A = Cross-sectional area of the barrel, ft^2
- p = Perimeter of the barrel, ft
- V = Velocity in the barrel, ft/s

The exit loss is a function of the change in velocity at the outlet of the culvert barrel. The downstream velocity is usually neglected, in which case the exit loss is equal to the full flow velocity head in the barrel as shown:

$$H_o = H_v = \frac{V^2}{2g} \tag{8.3}$$

By combining the sum of all losses, the Equation 8.4 for loss is obtained:

$$H = \left[1 + k_e + \frac{29\ n^2\ L}{R^{1.33}}\right]\frac{V^2}{2g} \tag{8.4}$$

It is important to note that the total available upstream energy (HW) includes the depth of the upstream water surface above the outlet invert and the approach velocity head. In most instances, the approach velocity is low, and the approach velocity head is neglected. However, it can be considered to be a part of the available headwater and used to convey the flow through the culvert.

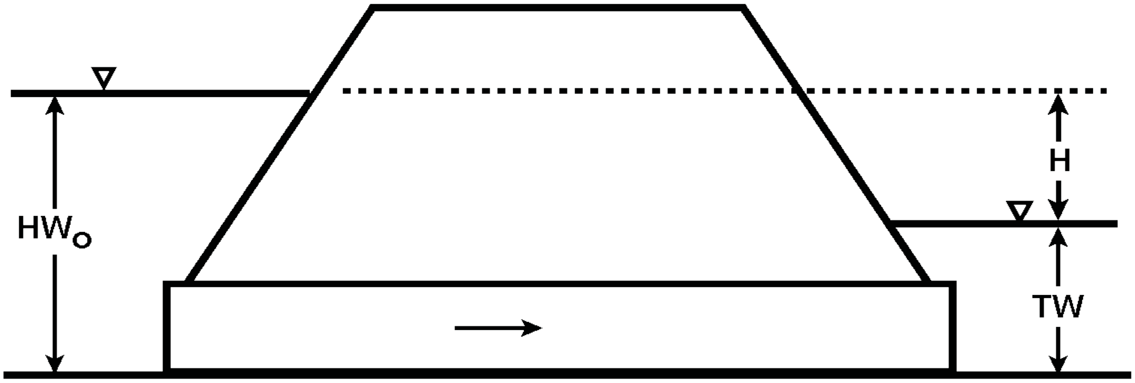


Figure 8.4 – Headwater based on outlet control analysis

Likewise, the velocity downstream of the culvert (V_d) is usually neglected. When both approach and downstream velocities are neglected, the Equation 8.5 is found:

$$HW_O = TW + H \tag{8.5}$$

In this case, H is the difference in elevation between the water surface elevation at the outlet (tailwater elevation) and the water surface elevation at the inlet (headwater elevation) as shown in Figure 8.4.

Equations 8.1 through 8.5 were developed for full barrel flow, shown in Figure 8.3-A. The equations also apply to the flow situations shown in Figures 8.3-B and C, which are effectively full flow conditions. Backwater calculations may be required for the part-full flow conditions shown in Figures 8.3-D and E. These calculations begin at the water surface at the downstream end of the culvert and proceed upstream to the entrance of the culvert. The downstream water surface is based on critical depth at the culvert outlet or on the tailwater depth, whichever is higher.

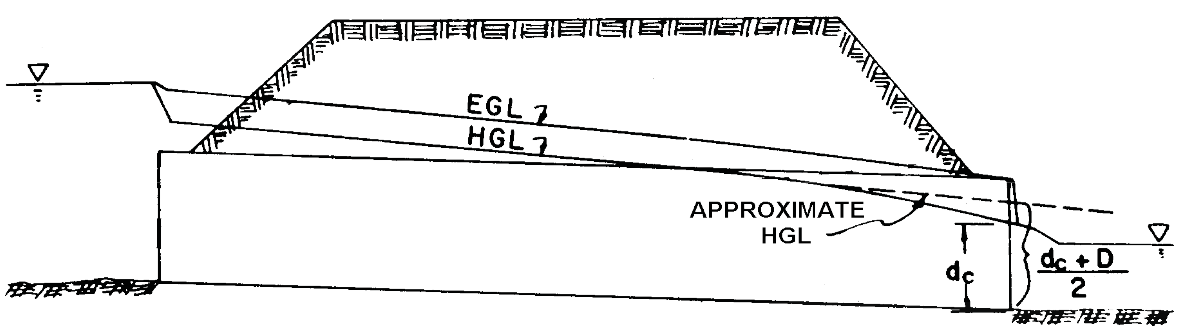


Figure 8.5 – Hydraulic grade line approximation

In order to avoid tedious backwater calculations, approximate methods have been developed to analyze part-full flow conditions. Based on numerous backwater calculations, it was found that a downstream extension of the full flow hydraulic grade line for the flow condition shown in Figure 8.5 pierces the plane of the culvert outlet at a point half-way between critical depth and the top of the barrel. If the tailwater exceeds $(d_c+D)/2$, then the tailwater depth should be used to set the downstream end of the extended full flow hydraulic grade line.

This approximate method works best when the barrel flows full over at least part of its length (Figure 8.5). When the barrel is partly full over its entire length, the method becomes increasingly inaccurate as the headwater decreases further below the top of the barrel at the inlet. Adequate results are obtained down to a headwater of 0.75D. For lower headwater depths, backwater calculations are required to obtain accurate headwater elevations.

The outlet control nomographs in HDS 5 provide solutions for Equation 8.5 for entrance, friction, and exit losses in full-barrel flow. Using the approximate backwater method, the losses (H) obtained from the nomographs can be applied for the part-full flow conditions shown in Figure 8.6. The losses are added to the elevation of the extended full flow hydraulic grade line at the barrel outlet in order to obtain the headwater elevation. The extended hydraulic grade line is set at the higher of $(d_c+ D)/2$ or the tailwater elevation at the culvert outlet. This new term is identified as h_o. See Equation 8.6. Again, the approximation works best when the barrel flows full over at least part of its length.

$$h_o = TW \text{ or } (d_c + D) / 2 \text{ whichever is greater} \qquad (8.6)$$

When culverts are on a grade as shown in Figure 8.3, then Equation 8.6 becomes:

$$HW_o = h_o + H - LS \qquad (8.7)$$

Remember, the elevation of the outlet control headwater is found from the following:

$$HW_o\ Elev = EL_o + h_o + H \qquad (8.8)$$

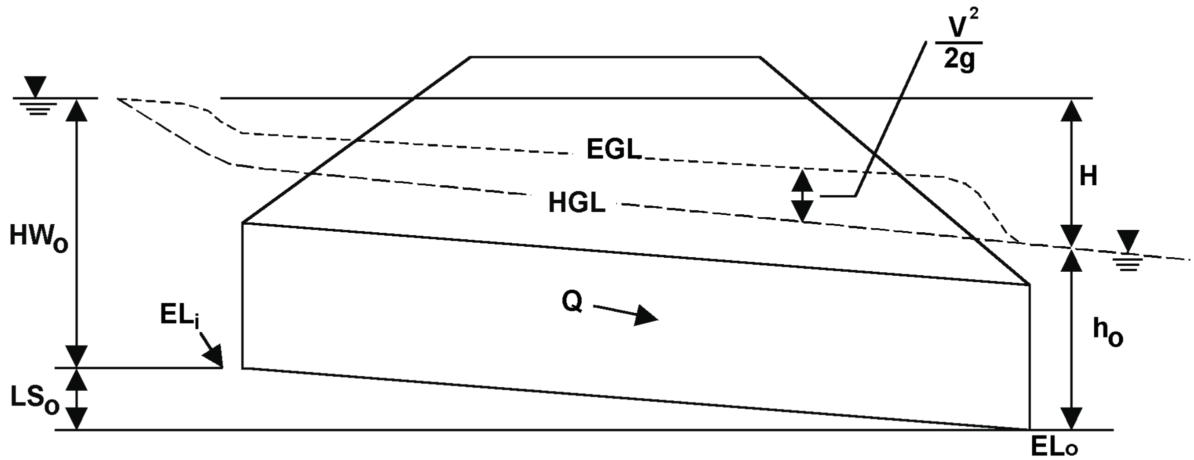


Figure 8.6 – Outlet control culvert on a grade

Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually result in outlet velocities which are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion.

Inlet Control Outlet Velocity

In inlet control, drawdown calculations may be necessary to determine the outlet velocity. These calculations begin at the culvert entrance and proceed downstream to the exit (HY-8 calculates outlet velocities using this procedure). The flow velocity is obtained from the flow and the cross-sectional area at the exit (Equation 8.2).

An approximation may be used to avoid drawdown calculations in determining the outlet velocity for culverts operating in inlet control. The water surface profile converges toward normal depth as calculations proceed down the culvert barrel. Therefore, if the culvert is of adequate length, normal depth will exist at the culvert outlet. Even in short culverts, normal depth can be assumed and used to define the area of flow at the outlet and obtain the outlet velocity (Figure 8.7). The velocity calculated in this manner may be slightly higher than the actual velocity at the outlet. Normal depth in common culvert shapes may be calculated using a trial and error solution of Manning's equation. The known inputs are flow rate, barrel resistance, slope, and geometry. Normal depths will typically be obtained using approved computer programs and may be checked from design aids provided in publications such as FHWA HDS 3.[8-2]

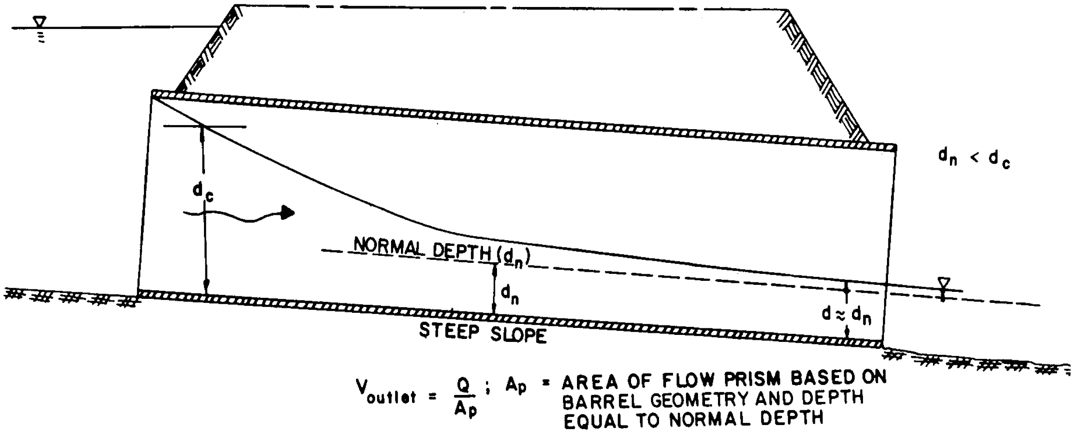


Figure 8.7 – Inlet control outlet velocity

Outlet Control Outlet Velocity

In outlet control, the cross-sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the conduit (Figure 8.8).

- Critical depth is used when the tailwater is less than the critical depth
- Tailwater depth is used when tailwater is greater than the critical depth, but below the top of the barrel
- Total barrel area is used when the tailwater exceeds the top of the barrel

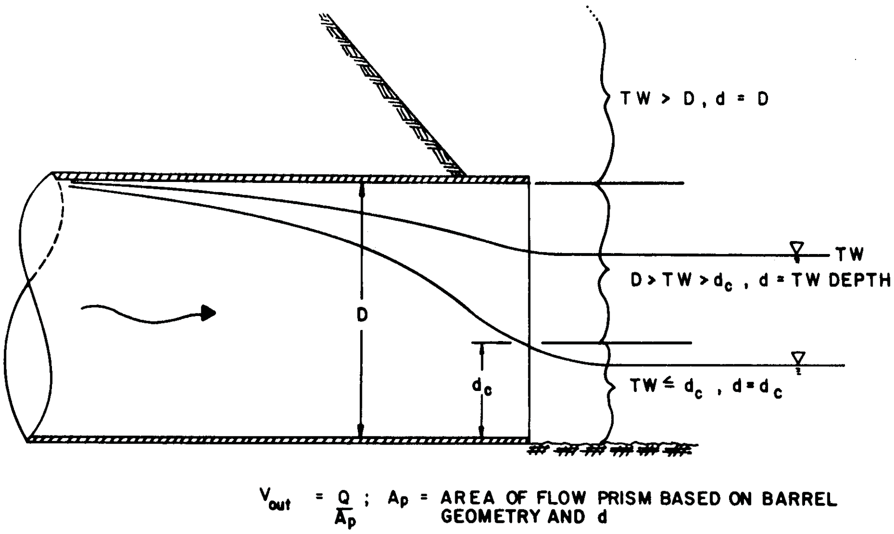


Figure 8.8 – Outlet control outlet velocity

Performance Curves

Performance curves are representations of flow rate versus headwater depth or elevation for a given flow. Due to the fact that a culvert has several possible control sections (inlet, outlet, and throat), a given installation will have a performance curve for each control section and one for roadway overtopping. The overall culvert performance curve is made up of the controlling portions of the individual performance curves for each control section. Figure 8.9 illustrates a performance curve for a culvert with roadway overtopping.

Using the combined culvert performance curve, the headwater elevation may be established for any flow rate or to visualize the performance of the culvert installation over a range of flow rates. When overtopping begins, the rate of headwater increase will flatten severely. The headwater will continue to rise very slowly from that point.

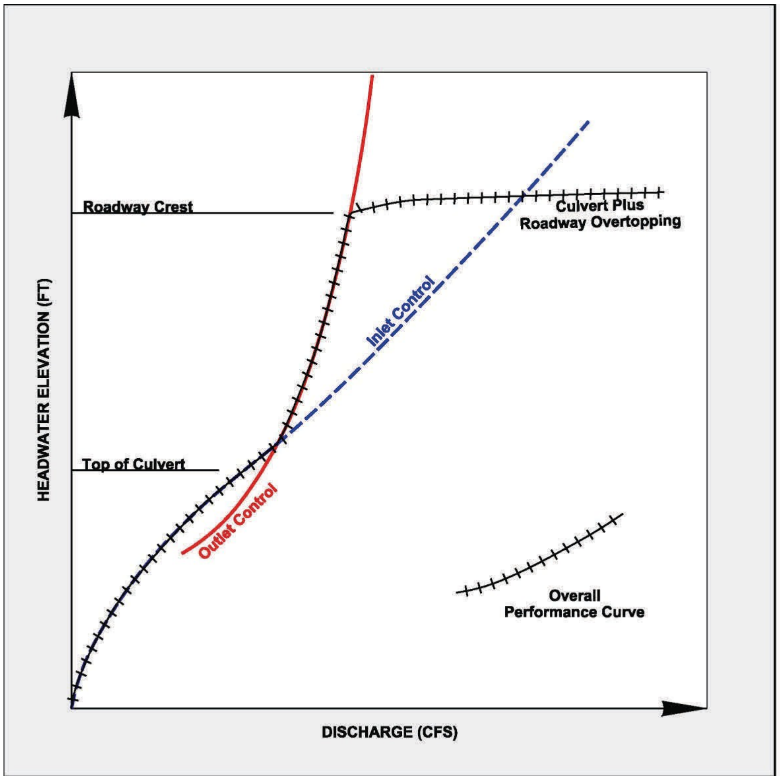


Figure 8.9 – Culvert performance curve with roadway overtopping [(8-6)]

Since improved inlets have more than one possible control section, always develop a performance curve as shown in Figure 8.10 that summarizes the culvert performance. Remember that the throat control curve should always be controlling at the design discharge. See HDS 5 [(8-6)] for more information.

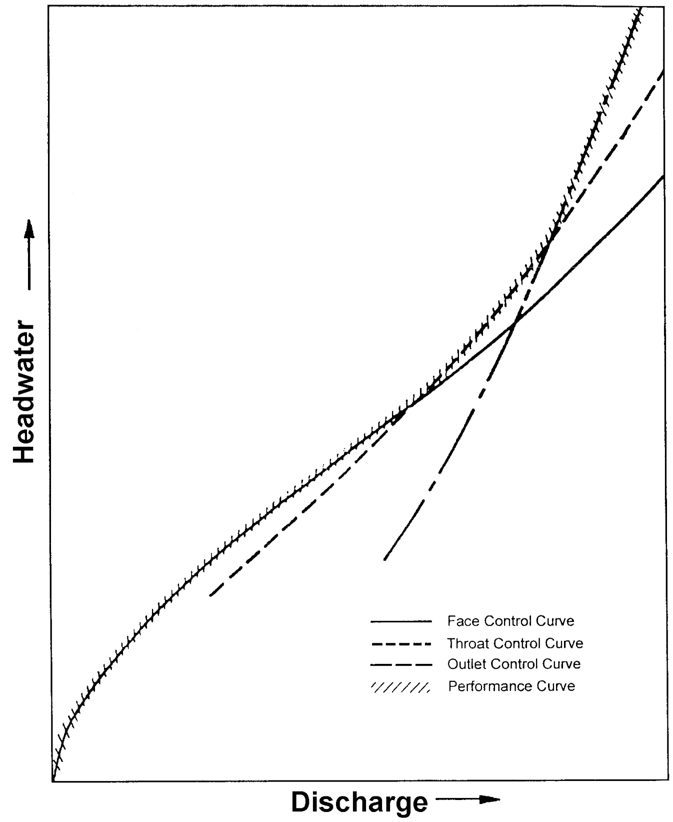


Figure 8.10 – Schematic of culvert performance curve with improved inlet

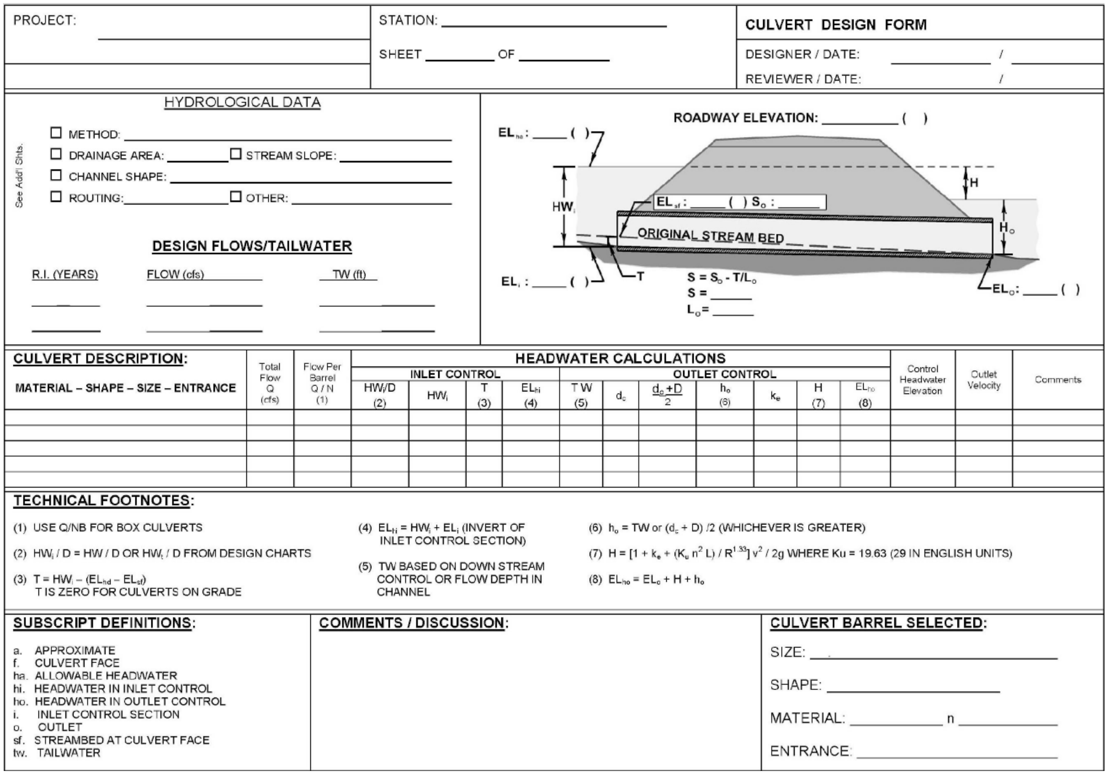
Constructing performance curves for culverts with tapered inlets helps to assure that the designer is aware of how the culvert will perform over a range of discharges. For high discharges, the outlet control curve may have a very steep slope which means that the headwater will increase rapidly with increasing discharge. Since there is a probability that the design discharge will be exceeded over the life of the culvert, the consequences of that event should be considered. This will help to evaluate the potential for damage to the roadway and to adjacent properties.

8.5 Culvert Design Method

The culvert design method presented here is a convenient and organized procedure for designing culverts, considering inlet and outlet control. While it is possible to follow the design method without an understanding of culvert hydraulics, this is not recommended. The result could be an inadequate and possibly unsafe structure.

8.5.1 Culvert Design Method

The culvert design form from HDS 5 [8-6] shown in Figure 8.11, has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data are also included. At the top right, there is a small sketch of the culvert with blanks for inserting important dimensions and elevations.



PROJECT:

STATION: ______ SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE: ______ / ______

REVIEWER / DATE: ______ / ______

HYDROLOGICAL DATA

See Add'l Shts.

- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ______
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

CULVERT DESCRIPTION: MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HW_i	T (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	Control Headwater Elevation	Outlet Velocity	Comments

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i / D = HW / D$ OR HW_i / D FROM DESIGN CHARTS

(3) $T = HW_i - (EL_{hd} - EL_{sf})$
T IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (K_u n^2 L) / R^{1.33}] v^2 / 2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
ha. ALLOWABLE HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

Figure 8.11 – Culvert design form from HDS 5

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet control and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert selected.

The first step in the design process is to summarize all known data for the culvert at the top of the culvert design form. This information will have been collected or calculated prior to performing the actual culvert design. The next step is to select a preliminary culvert material, shape, size, and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations. For additional information on completing the culvert design form, see HDS 5. (8-6)

8.5.2 Inlet Control Calculations

Conventional Culverts - Inlet Control Design Method

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration in inlet control. The approach velocity head may be included as part of the headwater, if desired. The inlet control nomographs in FHWA's HDS 5 are used in the design process. For the following discussion, refer to the schematic inlet control nomograph shown in Figure 8.12.

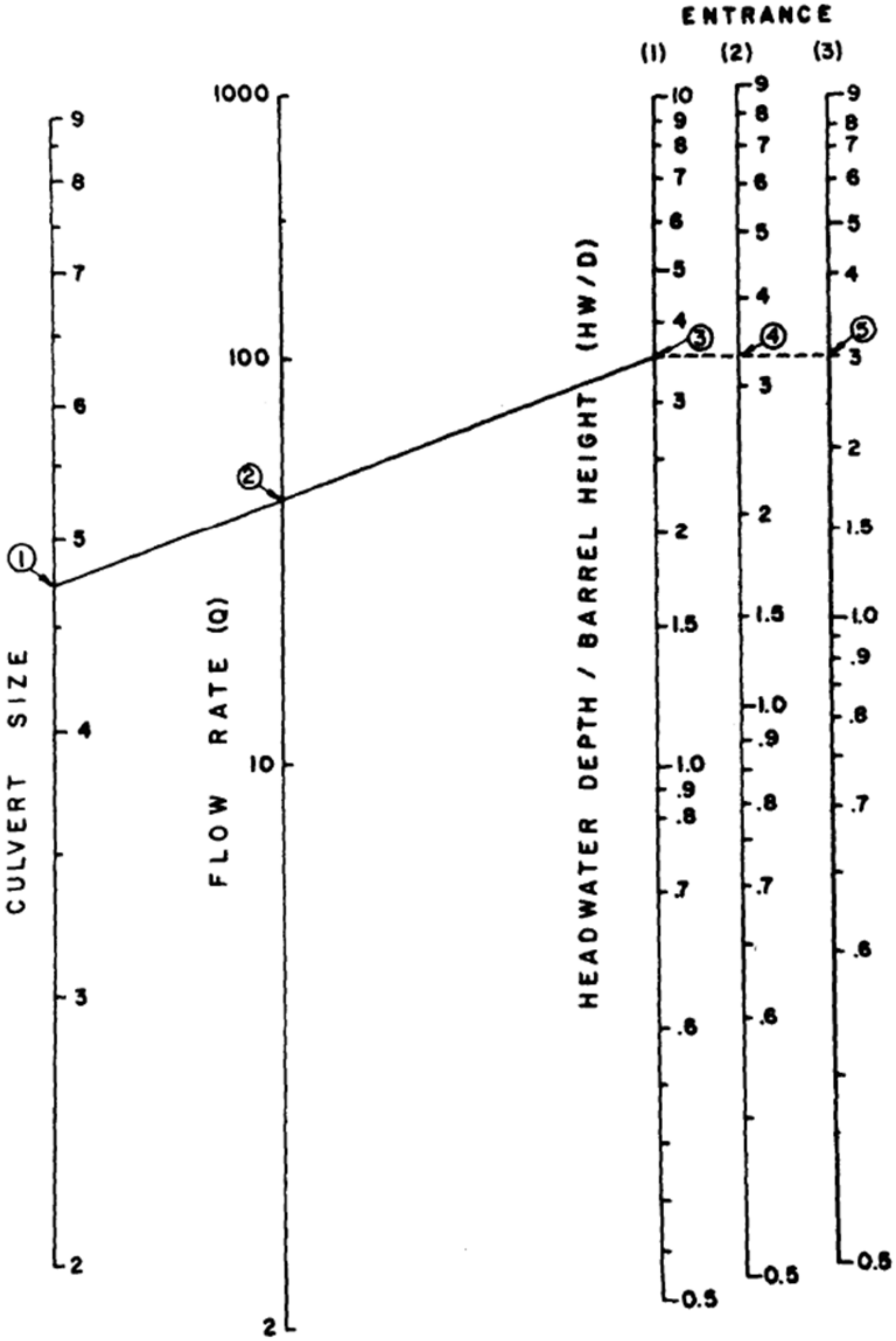


Figure 8.12 – Schematic of inlet control nomograph

- Locate the selected culvert size (point 1) and flow rate (point 2) on the appropriate scales of the inlet control nomograph. (Note that for box culverts, the flow rate per foot of barrel width is used.)

- Using a straight edge, carefully extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (HW/D) scale (point 3). The first HW/D scale is also a turning line.

- If another HW/D scale is required, extend a horizontal line from the first HW/D scale (the turning line) to the desired scale and read the result.
- Multiply HW/D by the culvert height, D, to obtain the required headwater (HW) from the invert of the control section to the energy grade line. If the approach velocity is neglected, HW equals the required headwater depth (HW_i). If the approach velocity is included in the calculations, deduct the approach velocity head from HW to determine HW_i.
- Calculate the required depression (Fall, or "T" as used in the culvert design form in Figure 8.11) of the inlet control section below the stream bed as follows:

$$HW_a = EL_a - EL_{sf} \tag{8.9}$$

$$Fall = HW_i - HW_a \tag{8.10}$$

Where:

HW_a =	Allowable headwater depth, ft
EL_a =	Allowable headwater elevation, ft
EL_{sf} =	Elevation of the streambed at the face, ft
HW_i =	Required headwater depth, ft

Possible results and consequences of this calculation are:

1. If the Fall is negative or zero, set Fall equal to zero.
2. If the Fall is positive, the inlet control section invert must be depressed below the streambed at the face by that amount, assuming that inlet control is maintained.
3. If the Fall is positive and greater than an acceptable value, select another culvert configuration and begin again.

Calculate the inlet control section invert elevation as follows:

$$EL_i = EL_{sf} - Fall \tag{8.11}$$

Where:

EL_i =	Invert elevation at the face of a culvert (EL_{sf}) or at the throat of a culvert with a tapered inlet (EL_t)

Improved Inlets - Design Methods

Tapered inlet design begins with the selection of the culvert barrel size, shape, and material. These calculations are performed using the culvert design form shown in Figure 8.11. The tapered-inlet design calculation form (Figure 8.13) and the design nomographs contained in FHWA's HDS 5 are used to design the tapered inlet. The result will be one or more culvert designs, with and without tapered inlets, all of which meet the site design criteria. The designer must select the best design for the site under consideration.

In the design of tapered inlets, the goal is to maintain control at the efficient throat section in the design range of headwater and discharge. This is because the throat section has the same geometry as the barrel, and the barrel is the most costly part where the use of a tapered inlet is justified. The inlet face is then sized large enough to pass the design flow without acting as a control section in the design discharge range. Some slight oversizing of the face is beneficial because the cost of constructing the tapered inlet is usually minor compared with the cost of the barrel where the use of tapered inlets is justified.

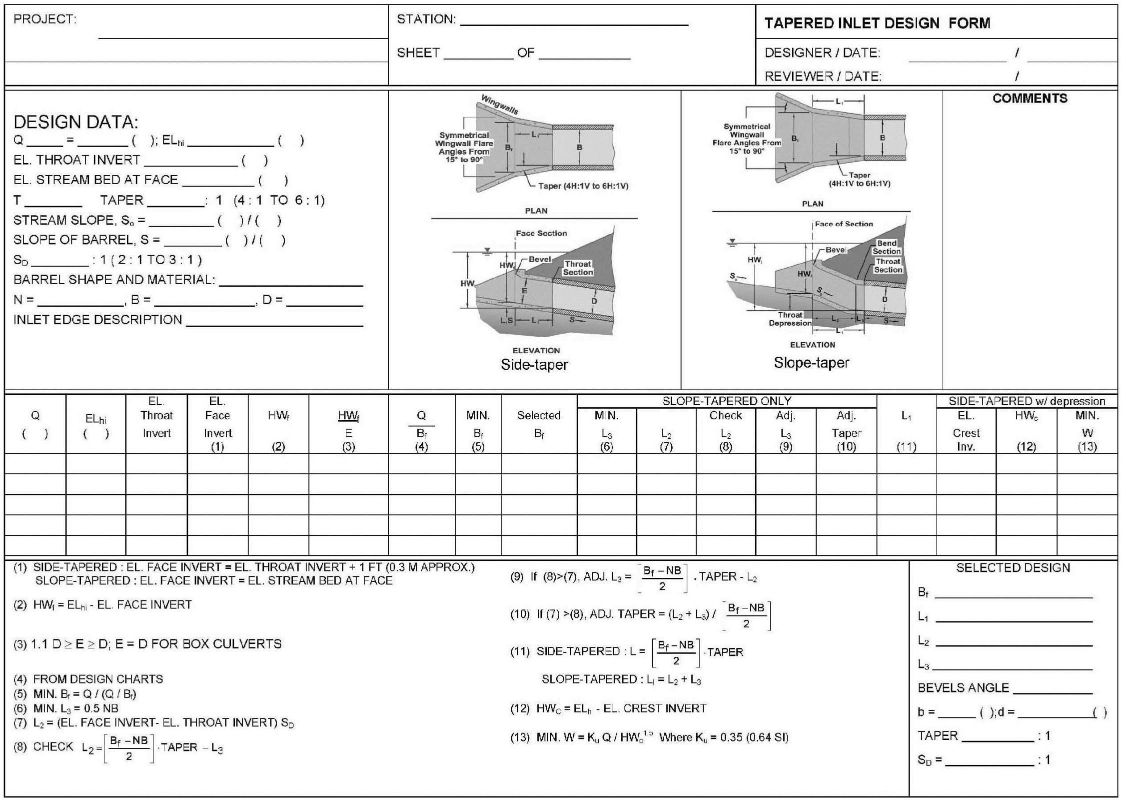
The required size of the face can be reduced by use of favorable edge configurations such as beveled edges on the face section. Design nomographs are provided for favorable and less favorable edge conditions.

The following steps outline the design process for culverts with tapered inlets. Steps 1 and 2 are the same for all culverts with and without tapered inlets.

1. **Preliminary Culvert Sizing**: Estimate the culvert barrel size to begin calculations.

2. **Culvert Barrel Design**: Complete the culvert design form (Figure 8.11). These calculations yield the required Fall at the culvert entrance. For the inlet control calculations, the appropriate inlet control nomograph is used for the tapered inlet throat. The required Fall is upstream of the inlet face section for side-tapered inlets and is between the face section and throat section for slope-tapered inlets. The culvert design form should be completed for all barrels of interest. Plot outlet control performance curves for the barrels of interest. Plot inlet control performance curves for the faces of culverts with non-enlarged inlets and for the throats of tapered inlets.

3. **Tapered Inlet Design**: Use the tapered inlet design form (Figure 8.13) for selecting the type of tapered inlet to be used and determining its dimensions.



PROJECT:

STATION: ______________ SHEET ______ OF ______

TAPERED INLET DESIGN FORM

DESIGNER / DATE: ________ / ________

REVIEWER / DATE: ________ / ________

DESIGN DATA:

Q ____ = ______ ( ); EL_{hi} ________ ( )

EL. THROAT INVERT ________ ( )

EL. STREAM BED AT FACE ________ ( )

T ______ TAPER ______ : 1 (4 : 1 TO 6 : 1)

STREAM SLOPE, S_o = ________ ( ) / ( )

SLOPE OF BARREL, S = ________ ( ) / ( )

S_D ______ : 1 (2 : 1 TO 3 : 1)

BARREL SHAPE AND MATERIAL: ____________

N = ________, B = ________, D = ________

INLET EDGE DESCRIPTION ____________

COMMENTS

Q ( )	EL_{hi} ( )	EL. Throat Invert	EL. Face Invert (1)	HW_f (2)	$\frac{HW_f}{E}$ (3)	$\frac{Q}{B_f}$ (4)	MIN. B_f (5)	Selected B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	SLOPE-TAPERED ONLY: L_2 (7)	SLOPE-TAPERED ONLY: Check L_2 (8)	SLOPE-TAPERED ONLY: Adj. L_3 (9)	SLOPE-TAPERED ONLY: Adj. Taper (10)	L_1 (11)	SIDE-TAPERED w/ depression: EL. Crest Inv.	SIDE-TAPERED w/ depression: HW_c (12)	SIDE-TAPERED w/ depression: MIN. W (13)

(1) SIDE-TAPERED : EL. FACE INVERT = EL. THROAT INVERT + 1 FT (0.3 M APPROX.)
SLOPE-TAPERED : EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi}$ - EL. FACE INVERT

(3) $1.1\,D \geq E \geq D$; E = D FOR BOX CULVERTS

(4) FROM DESIGN CHARTS

(5) MIN. $B_f = Q / (Q / B_f)$

(6) MIN. $L_3 = 0.5\,NB$

(7) L_2 = (EL. FACE INVERT- EL. THROAT INVERT) S_D

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_3$

(9) If (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] . TAPER - L_2$

(10) If (7) >(8), ADJ. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(11) SIDE-TAPERED : $L = \left[\frac{B_f - NB}{2}\right] \cdot TAPER$
SLOPE-TAPERED : $L_i = L_2 + L_3$

(12) $HW_C = EL_h$ - EL. CREST INVERT

(13) MIN. $W = K_u\,Q / HW_c^{1.5}$ Where $K_u = 0.35$ (0.64 SI)

SELECTED DESIGN

B_f ________

L_1 ________

L_2 ________

L_3 ________

BEVELS ANGLE ________

b = ____ ( ); d = ________ ( )

TAPER ________ : 1

S_D = ________ : 1

Figure 8.13 – Tapered inlet design form

To use the tapered inlet design form (Figure 8.13), perform the following steps:

1. **Complete Design Data.** Fill in the required design data on the top of the form.

    a. Flow, Q, is the selected design flow rate from the culvert design form (Figure 8.11).

    b. EL_{hi} is the inlet control headwater elevation.

    c. The elevation of the throat invert (EL_t) is the inlet invert elevation (EL_i).

    d. The elevation of the stream bed at the face (EL_{sf}), the stream slope (S_o), and the slope of the barrel (S).

    e. The Fall is the difference between the streambed elevation at the face and the throat invert elevation.

    f. Select a side taper (TAPER) between 4:1 and 6:1 and a Fall slope (S_f) between 1V:2H and 1V:3H. The TAPER may be modified during the calculations.

    g. Enter the barrel shape and material, the size, and the inlet edge configuration. Note that for tapered inlets, the inlet edge configuration is designated the "tapered inlet throat.

2. **Calculate the Face Width**.

   a. Enter the flow rate, the inlet control headwater elevation (EL_{hi}), and the throat invert elevation on the design forms. (For the slope-tapered inlet with mitered face, the face section is downstream of the crest. Calculate the vertical difference between the stream bed at the crest and the face invert (y), which includes part of the total inlet Fall.

   b. Perform the calculations resulting in the face width (B_f). Face control design nomographs are contained in FHWA's HDS 5.

   c. Note: When designing side-or slope-tapered inlets for box culverts with double barrels, the required face width derived from the design procedures is the total clear width of the face. The thickness of the center wall must be added to this clear width to obtain the total face width. No design procedures are available for tapered inlets on box culverts with more than two barrels.

3. **Calculate Tapered-Inlet Dimensions.** If the Fall is less than D/4 (D/2 for a slope-tapered inlet with a mitered face), a side-tapered inlet must be used. Otherwise, either a side-tapered inlet with a depression upstream of the face or a slope-tapered inlet may be used.

   a. For a slope-tapered inlet with a vertical face, calculate L_2, L_3, and the TAPER. (For the slope-tapered inlet with a mitered face, calculate the horizontal distance between the crest and the face section invert L_4. These dimensions are shown on the small sketches in the top center of the forms).

   b. Calculate the overall tapered inlet length, L_1.

   c. For a side-tapered inlet, check to assure that the Fall between the face section and the throat section is one foot or less. If not, return to step b. with a revised face invert elevation.

4. **Calculate the Minimum Crest Width.** For a side-tapered inlet with Fall upstream of the face, calculate the minimum crest width and check it against the proposed crest width. In order to obtain the necessary crest length for a depressed side-tapered inlet, it may be necessary to increase the flare angle of the wingwalls for the type of depression or to increase the length of crest on the sump for the design. It is important to note that the TAPER must be greater than 4:1.

5. **Fit the Design into the Embankment Section.** Using a sketch based on the derived dimensions and a sketch of the roadway section to the same scale, design a culvert that fits the site. Adjust inlet dimensions as necessary but do not reduce dimensions below the minimum requirements of the design form.

6. **Prepare Performance Curves.** Using additional flow rate values and the appropriate nomographs, calculate a performance curve for the selected face section. Do not adjust inlet dimensions at this step in the design process. Plot

the face control performance curve on the same sheet as the throat control and the outlet control performance curves.

7. **Enter Design Dimensions.** If the design is satisfactory, enter the dimensions at the lower right of the design form. Otherwise, calculate another alternative design by returning to step 3a.

Dimensional Limitations for Improved Inlets

The following dimensional limitations must be observed when designing tapered inlets using the design charts of this publication. Tapered inlets can only be used where the culvert width is less than three times its height, ($B < 3D$).

1. Side-Tapered Inlets.

    a. 4:1 < TAPER < 6:1

    b. Tapers less divergent than 6:1 may be used but performance will be underestimated.

    c. Wingwall flare angle range from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels (Figure 8.14).

    d. If a Fall is used upstream of the face, extend the barrel invert slope upstream from the face a distance of D/2 before sloping upward more steeply. The maximum vertical slope of the apron is: 1V:2H.

    e. $D < E < 1.1D$

2. Slope-tapered Inlets.

    a. 4:1 < TAPER < 6:1
    (Tapers > 6:1 may be used, but performance will be underestimated.)

    b. 3H:1V > Sf > 2H:1V
    If Sf > 3H:1V, use side-tapered design.

    c. Minimum L3 = 0.5B

    d. D/4 < Fall < 1.5D

        i For Fall < D/4, use side-tapered design

        ii For Fall < D/2, do not use the slope-tapered inlet with a mitered face

        iii For Fall > 1.5D, estimate friction losses between the face and the throat by using Equation 8.12 and add the additional losses to HW_t.

Where:

$$H_1 = \left[\frac{K_U\ n^2 L_i}{R^{1.33}}\right] \frac{Q^2}{2gA^2} \quad (8.12)$$

K_U	=	29 (English)
H_1	=	Friction head loss in the tapered inlet, ft
n	=	Manning's n for the tapered inlet material
L_i	=	Length of the tapered inlet, ft
R	=	Average hydraulic radius of the tapered inlet = $(A_f + A_t)/(P_f + P_t)$, ft
Q	=	Flow rate, ft^3/s
G	=	Gravitational acceleration, ft/s^2
A	=	Average cross sectional area of the tapered inlet = $(A_f + A_t)/2$, ft^2

3. Wingwall flare angles range from 15 degrees to 26 degrees with the top edge beveled or from 26 degrees to 90 degrees with or without bevels (Figure 8.14).

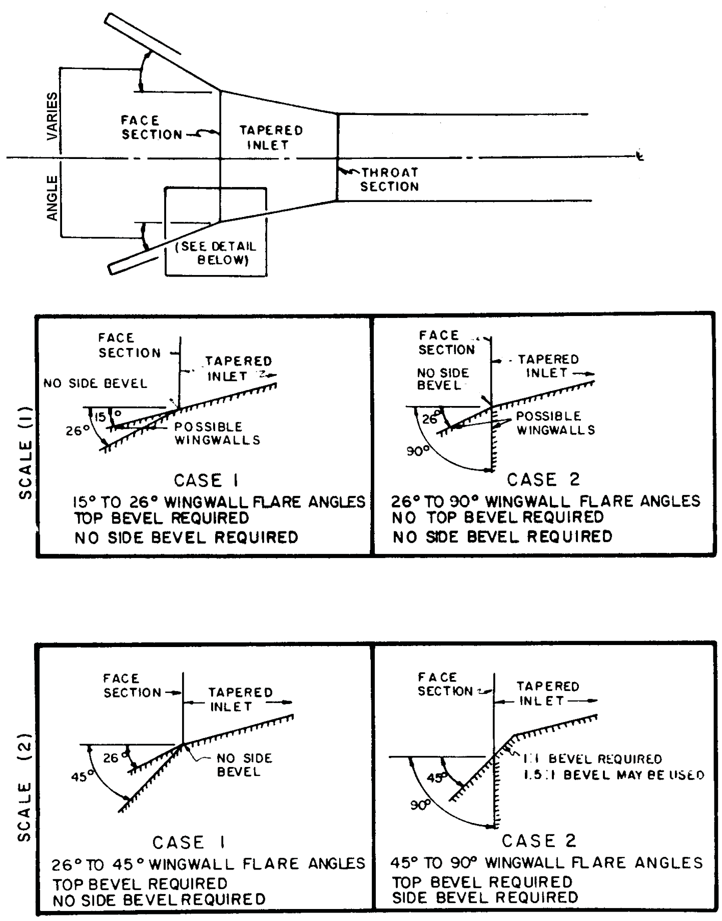


Figure 8.14 – Inlet edge conditions for rectangular tapered inlets

8.5.3 Outlet Control Calculations

The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. The approach and downstream velocities may be included in the design process, if desired. The critical depth charts and outlet control nomographs in FHWA's HDS 5 are used in the design process. For illustration, refer to the schematic critical depth chart and outlet control nomograph shown in Figures 8.15 and 8.16, respectively.

1. Determine the tailwater depth (TW) above the outlet invert at the design flow rate. This is obtained from backwater or normal depth calculations, from field observations or other method as appropriate from Section 8.2.4 Tailwater Relationship.

2. Using Figure 8.15 find the critical depth (d_c) by using the flow rate. Note: d_c cannot exceed D.

Note: The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for d_c > 0.9D consult a hydraulics handbook such as HDS 5. [8-6]

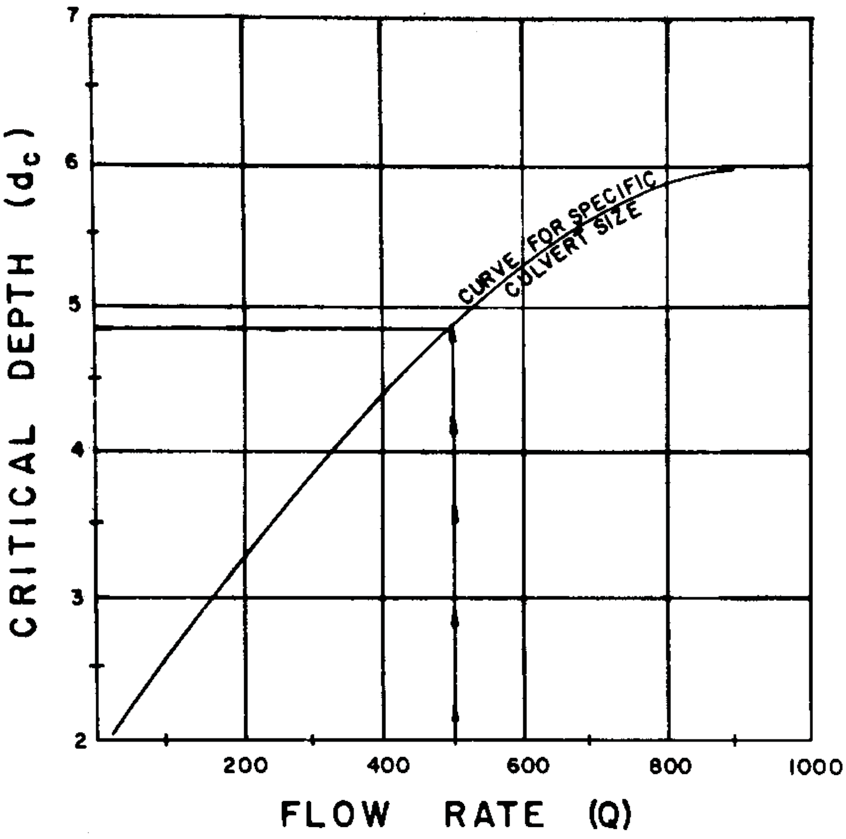


Figure 8.15 – Schematic of critical depth chart

3. Calculate $(d_c + D)/2$.

4. Determine the depth from the culvert outlet invert to the hydraulic grade line (h_o), h_o = TW or $(d_c + D)/2$ whichever is larger.

5. From Table 8.6, obtain the appropriate entrance loss coefficient, k_e, for the culvert inlet configuration.

6. Determine the losses through the culvert barrel, H, using the outlet control nomograph (Figure 8.16) or Equation 8.5 if outside the range of the nomograph.

7. Using a straight edge, connect the culvert size (point 1) with the culvert length on the appropriate k_e scale (point 2). This defines a point on the turning line (point 3).

8. Again using the straight edge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the head loss (H) scale. Read H. H is the energy loss through the culvert, including entrance, friction, and outlet losses.

Note: Careful alignment of the straightedge is necessary to obtain good results from the outlet control nomograph.

9. Calculate the required outlet control headwater elevation. Using Equation 8.13.

$$EL_{ho} = EL_o + H + h_o \qquad (8.13)$$

where EL_o is the invert elevation at the outlet.

10. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements provide limited benefit in outlet control.

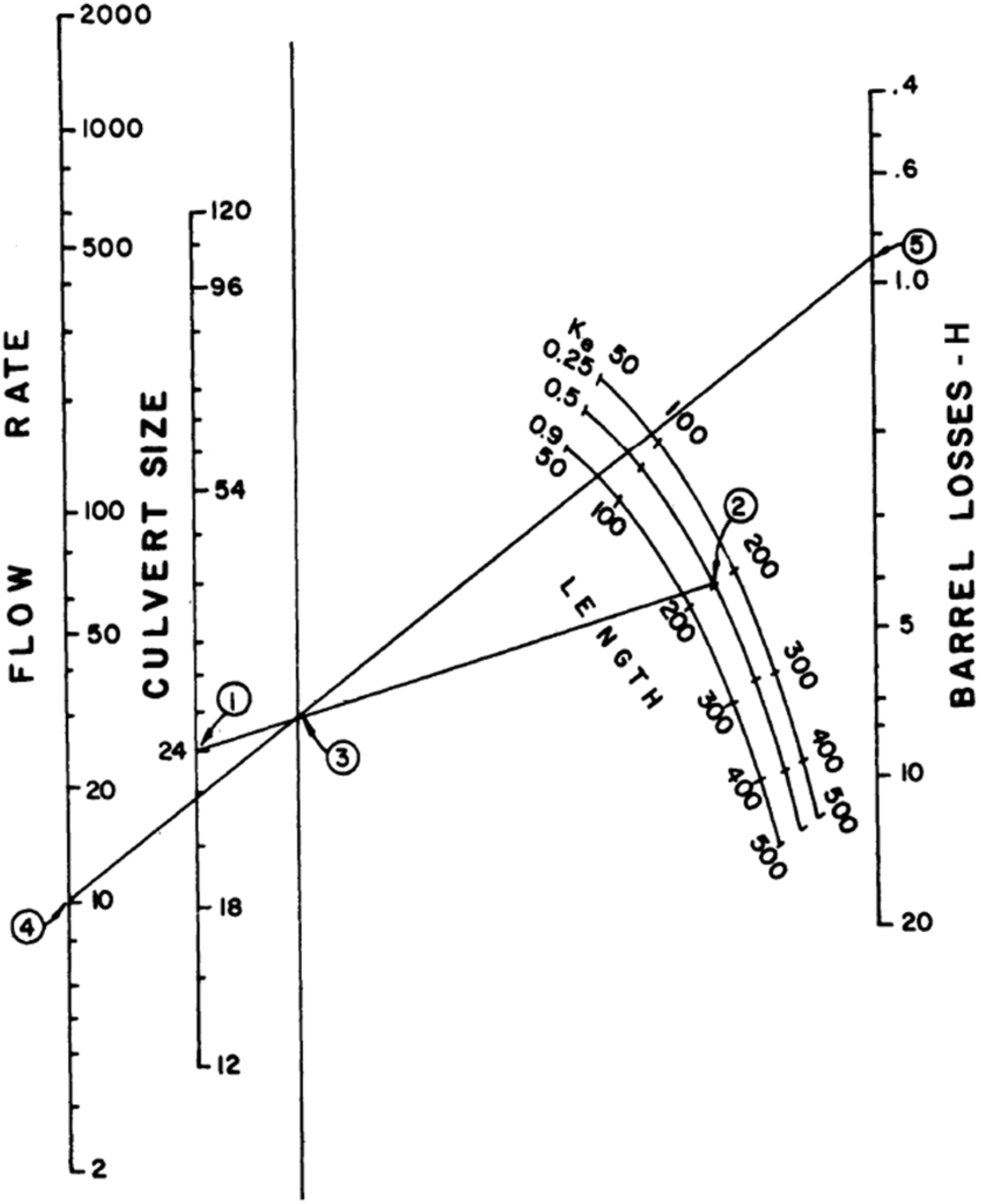


Figure 8.16 – Schematic of outlet control nomograph

8.5.4 Evaluation of Results

Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with the higher headwater for at least part of the time.

Special culvert installations such as culverts with safety grates, junctions, or bends are discussed in HDS 5, as well as unusual culvert configurations such as "broken-back" culverts, siphons, and low head installations.

A copy of the FHWA's culvert design form is provided in Appendix E of this manual to aid the designer. This form provides a convenient and organized way of keeping track of culvert design data and has been formulated to guide the designer through the design process.

8.5.5 Energy Dissipation

Erosion at culvert outlets is a common problem. Determination of the flow condition, scour potential, and channel erodibility should be standard procedure in the design of all highway culverts. Ultimately, the only safe procedure is to design on the basis that erosion at a culvert outlet and downstream channel will occur and must be protected against. See FHWA publication HEC 14 *Hydraulic Design of Energy Dissipators for Culverts and Channels* [8-7] and policies within this chapter for energy dissipation design guidance.

General Guidelines for Energy Dissipators

Energy dissipators should be considered for the following conditions:

1. The potential erosion at the culvert outlet will become a risk to the roadway itself or a downstream property.

2. Culvert outlet velocities are greater than 7 ft/s and dependent upon the erodibility of the soils at the outfall.

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.

General Design Procedure

The following method is intended to show the designer a general workflow process for a manual method of designing energy dissipators:

1. Locate the culvert's design data including survey information, design storm frequency, and all other pertinent hydraulic information (i.e., channel slope, culvert type, size, shape.)

2. Determine if an energy dissipator is warranted based on the previous section, *General Guideline for Energy Dissipators*.

3. Choose appropriate dissipator design options and begin designing each alternative.

4. Select the alternative that best fits the intended site while considering effectiveness and construction cost.

5. If a riprap apron is required, design the apron according to the guidelines in HEC 14. A riprap apron should only be used for median drains and small cross drains. Where other than minor damage could result from lack of dissipation, a dissipator from which an outlet design velocity can be calculated should be used.

6. Document all design, structural, and buoyancy calculations.

8.5.6 Culvert Outlet Velocity and Velocity Modification

The continuity equation (Equation 4.5 page 4-14 of this manual) can be used in all situations to compute culvert outlet velocity, either within the barrel or at the outlet. Given the design discharge, the designer should determine the flow area, which is a function of the type of control (outlet or inlet).

Culvert outlet velocity is one of the primary indicators of erosion potential. If the velocity is higher than the velocity in the downstream channel, measures to modify or reduce velocity within the culvert barrel should be considered.

However, the degree of velocity reduction is typically limited and must be balanced against the increased costs involved.

8.5.7 Outlet Velocity Considerations for Culverts on Mild Slopes

For culverts on mild slopes operating under outlet control with high tailwater depths (Figures 8.3a and 8.3b), the outlet velocity will be determined using the full area of the barrel. With this condition, it is possible to reduce the velocity by increasing the culvert size. Note that with high tailwater conditions, erosion may not be a serious problem since the ponded water will act as an energy dissipator; however, it will be important to determine if tailwater will always control, or if any of the other conditions shown on Figure 8.3 might occur.

When the discharge is high enough to produce a critical depth equal to the crown of the culvert barrel (Figure 8.3c), full flow will again occur, and the outlet velocity will be based on the area of the barrel. As before, the barrel size can be increased to achieve a reduction in velocity, but it will be necessary to evaluate if the increased size results in a flow depth below the crown, indicating less than full flow at the outlet. When this occurs, the area used in the continuity equation should be based on the actual flow area.

When culverts discharge with the critical depth occurring near the outlet (Figures 8.3d and 8.3e), increasing the barrel size will typically not significantly reduce the outlet velocity. Similarly, increasing the resistance factor will not affect outlet velocity since critical depth is not a function of n.

8.5.8 Outlet Velocity Considerations for Culverts on Steep Slopes

For culverts flowing on steep slopes with no tailwater (Figures 8.1a and 8.1c) the outlet velocity can be determined from normal depth calculations. With normal depth conditions on a steep slope, increasing the barrel size may slightly decrease the outlet velocity; however, calculations show that in reality, the slope is the driving force in establishing the normal depth. The velocity will not be significantly altered by doubling the culvert size/width. Thus, such an approach may not be cost effective. Some reduction in outlet velocity can be obtained by increasing the number of barrels, but this is also generally not cost effective.

Increasing the barrel resistance can significantly reduce outlet velocity and is an important factor in velocity reduction for culverts on steep slopes. The objective is to force full flow conditions near the outlet without creating additional headwater. HEC 14 discusses various methods of creating additional roughness (from changing pipe material to baffles and roughness rings) and details the appropriate design procedures.

8.5.9 Types of Energy Dissipation

Different stormwater outlets often require different methods of energy dissipation. This section of the chapter identifies alternate options for energy dissipators and provides a discussion on when they are warranted.

8.5.9.1 Hydraulic Jump Energy Dissipators

The hydraulic jump is a natural phenomenon which occurs when supercritical flow changes to subcritical flow (see Chapter 4 of this manual). This abrupt change in flow condition is accomplished by considerable turbulence and loss of energy, making the hydraulic jump an effective energy dissipation device. To better define the location and length of a hydraulic jump, standard design structures have been developed to force the hydraulic jump to occur. These structures typically use blocks, sills, or other roughness elements to impose exaggerated resistance to flow. Forced hydraulic jump structures applicable in highway engineering include the Colorado State University (CSU) rigid boundary basin, US Bureau of Reclamation (USBR) type IV basin, and the St. Anthony Falls (SAF) basin.

The CSU rigid boundary basin was developed from model study tests of basins with abrupt expansions (Figure 8.17 and 8.18); however, the configuration recommended for use is a combination flared-abrupt, expansion basin. The roughness elements are symmetrical about the basin centerline, and the spacing between the elements is approximately equal to the element width. Alternate rows of roughness elements are staggered. Riprap may be needed for a short distance downstream of the basin.

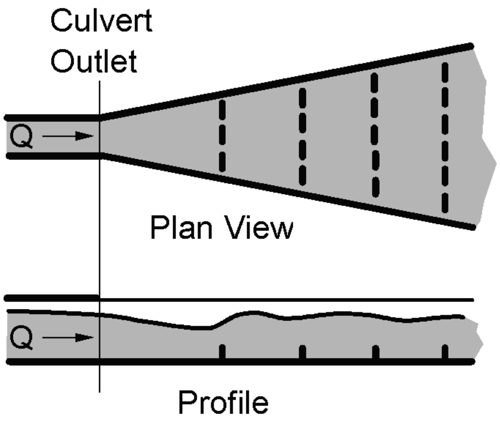


Figure 8.17 (left) – Schematic of CSU rigid boundary basin

Figure 8.18 (right) – Photo of CSU rigid boundary basin

The SAF stilling basin is a more generalized design that uses special appurtenances, chute blocks and baffle or floor blocks to force the hydraulic jump to occur (Figure 8.19 and 8.20). It is recommended for Froude numbers between 1.7 and 17. Similar to the CSU basin, the design criteria were developed from model study test results.

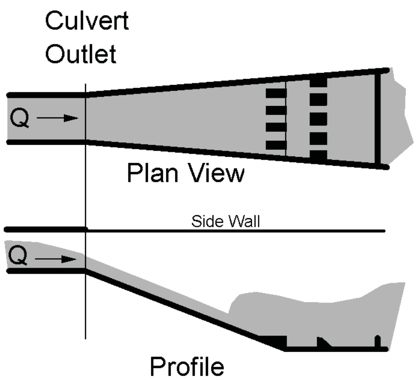


Figure 8.19 (left) – Schematic of SAF stilling basin

Figure 8.20 (right) – Photo of SAF stilling basin

8.5.9.2 Impact Basins

As the name implies, impact basins are designed with part of the structure physically blocking the free discharge of water. Water impacting on the basin structure dissipates energy and modifies the downstream flow regime.

Several types of impact basins include the Contra Costa Energy Dissipator, Hook type energy dissipator, and the USBR Type VI Stilling Basin.

The USBR Type VI impact basin is most commonly used in highway engineering (Figure 8.21 and 8.22). The structure is contained in a relatively small box-like structure which requires no tailwater for successful performance. The shape of the basin evolved from extensive tests that resulted in a design based around a vertical hanging baffle. Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies. Notches in the baffle provide a self-cleaning feature after prolonged nonuse of the structure. If the basin is full of sediment, the notches provide concentrated jets of water for cleaning. If the basin is completely clogged, the full discharge can be carried over the top of the baffle. Use of the basin is limited to installations where the velocity at the entrance of the basin does not exceed 24 ft/s and a discharge limit of 380 ft^3/s.

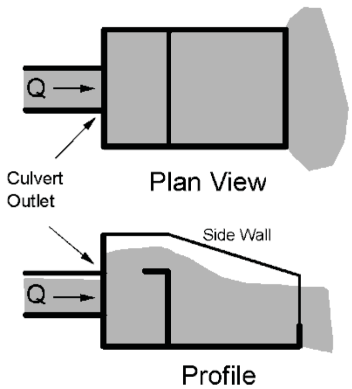


Figure 8.21 (left) – Schematic of USBR

Figure 8.22 (right) – Photo of Baffle-wall energy dissipator - USBR Type VI

8.5.9.3 Drop Structures with Energy Dissipation

Drop structures are commonly used for flow control and energy dissipation. Reducing channel slope by placing drop structures at intervals along the channel changes a continuous steeper sloped channel into a series of milder sloped reaches with vertical drops. Instead of slowing down and transferring high erosion producing velocities into lower non-erosive velocities, drop structures control the slope of the channel so that high velocities never develop. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by specially designed aprons or stilling basins.

Energy dissipation occurs through the impact of the falling water on the floor, redirection of the flow, and turbulence. The stilling basin used to dissipate excess energy can vary from a simple concrete apron to an apron with flow obstructions such as baffle blocks, sills, or abrupt rises. The length of the concrete apron required can be shortened by addition of these appurtenances. Figure 8.23 illustrates a straight drop stilling basin with floor blocks and an end sill. The design of this and other drop structure stilling basins is detailed in HEC 14.[8-7]

Figure 8.23 – Straight drop spillway stilling basin

8.5.9.4 Stilling Wells

Stilling wells dissipate kinetic energy by forcing flow to travel vertically upward to reach the downstream channel. The stilling well most commonly used in highway engineering is the USACE Stilling Well (Figure 8.24 and 8.25). Apply a stilling well where debris is not a serious problem. It will operate with moderate to high concentrations of sand and silt, but it is not recommended for areas where quantities of large floating or rolling debris are expected unless suitable debris-control structures are used. Its greatest application in highway engineering is at the outlets of storm drains and pipe down drains where little debris is expected. It is recommended that riprap or other types of channel protection be provided around the stilling well outlet.

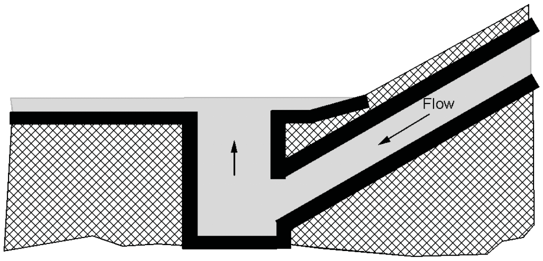


Figure 8.24 (left) – Schematic of USACE stilling well

Figure 8.25 (right) – Photo of USACE stilling well

8.5.9.5 Riprap Stilling Basins

Riprap stilling basins are commonly used at culvert outlets (Figure 8.26). The design procedure for riprap energy dissipators was developed from model study tests. The results of this testing indicated that the size of the scour hole at the outlet of a culvert was related to the size of the riprap, discharge, brink depth, and tailwater depth. The mound of rock material that often forms on the bed downstream of the scour hole contributes to dissipation of energy and reduction of the scour hole size. The general design guidelines for riprap stilling basins include preshaping the scour hole and lining it with riprap. Specific design criteria for the length, depth and width of the scour hole, and the entire basin, are provided in HEC 14.[8-7]

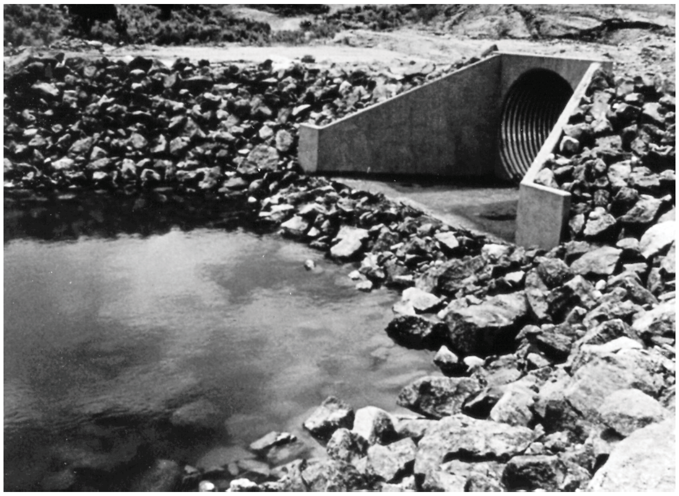


Figure 8.26 – Riprapped culvert energy basin

8.5.9.6 Contra Costa Energy Dissipator

This dissipator was developed to meet the following conditions: (1) to re-establish natural channel flow conditions downstream from the culvert outlet; (2) to have self-cleaning and minimum maintenance properties; (3) to drain by gravity when not in operation; (4) to be easily and economically constructed; and (5) to be applicable for a wide range of culvert sizes and operation conditions. The dissipator is best suited to small and medium size culverts of any cross section where the depth of flow at the outlet is less than the culvert height. It is applicable for medium and high velocity effluents. The dissipator design is such that the flow leaving the structure will be at minimum energy when in operation without tailwater. When tailwater is present, the performance will improve.

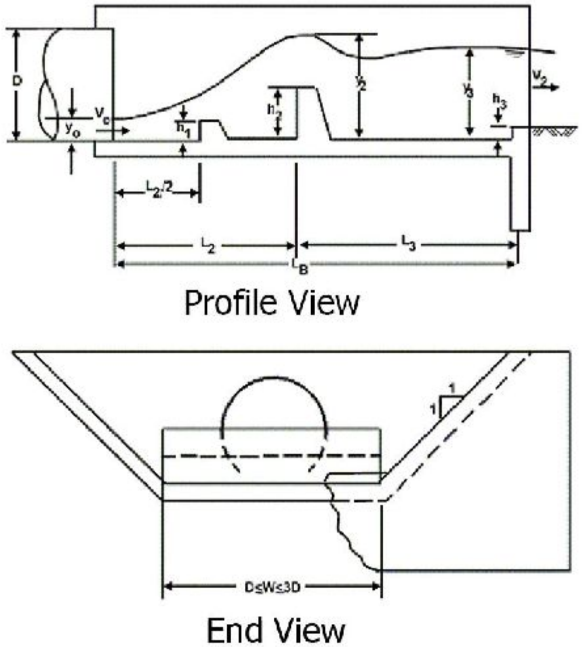


Figure 8.27 (left) – Schematic of Contra Costa Energy Dissipator

Figure 8.28 (right) – Photo of Contra Costa Energy Dissipator

8.6 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. This section provides some general information on the use of HY-8.

8.6.1 Culvert Design Using HY-8

Culvert design can be completed with HY-8. Dissipator designs with HY-8 are not acceptable unless confirmed with another method. Energy dissipation design should be based on FHWA publication HEC 14. [8-7] Table 8.7 provides guidelines for the use of various energy dissipators described in HEC 14. A performance curve is necessary for any energy dissipator design and analysis

HY-8 is a menu-driven culvert design program developed by the FHWA. The program allows the user to interactively enter, save, and edit data. The HY-8 program will compute the culvert hydraulics for circular, rectangular, elliptical, arch, and user defined geometry. The output from the HY-8 program can be printed out and incorporated directly into a hydraulic report.

The logic behind the HY-8 program is similar to that used in the culvert design method. The program calculates and compares the headwater elevations for inlet and outlet control. The program then selects the higher of the two elevations as the control elevation. The program incorporates the effects of tailwater when calculating these elevations. If the controlling headwater elevation results in overtopping of the roadway embankment, the program performs an overtopping analysis whereby the flow is balanced between the culvert discharge and the discharge over the overtopped structure or roadway.

There are five main groups of data to be entered into the program, which allows the user to edit the group fields all within one dialogue box. These groups are:

1. The discharge data
2. The tailwater data
3. The roadway data
4. The culvert data
5. The site data

Note: There are spreadsheets for riprap basins, Contra Costa basins, and USBR Type VI impact basins on the ALDOT Hydraulic section's internet site.

Table 8.7 – Energy Dissipator Limitations (source Table 1.1, HEC 14 (8-7))

Dissipator Type	Froude Number[1] Fr	Allowable Debris			Tailwater TW	Special Consideration
		Silt/Sand	Boulders	Floating		
Flow transitions	na	H	H	H	Desirable	na
Scour hole	na	H	H	H	Desirable	na
Hydraulic jump	>1	H	H	H	Required	na
Tumbling flow	>1	M	L	L	Not needed	$4\%<S_o<25\%$
Increased Resistance	na	M	L	L	Not needed	Check Outlet Control HW
USBR Type IX baffled apron	<1	M	L	L	Not needed	na
Broken-back culvert	>1	M	L	L	Desirable	na
Outlet weir	2 to 7	M	L	M	Not needed	na
Outlet drop/weir	3.5 to 6	M	L	M	Not needed	na
USBR Type III	4.5 to 17	M	L	M	Required	na
USBR Type IV	2.5 to 4.5	M	L	M	Required	na
SAF stilling basin	1.7 to 17	M	L	M	Required	na
CSU rigid boundary basin	<3	M	L	M	Not needed	na
Contra Costa basin	<3	H	M	M	<0.5D	na
Hook basin	1.8 to 3	H	M	M	Not needed	na
USBR Type VI impact basin	na	M	L	L	Desirable	$Q<400$ ft^3/s, $V<50$ ft/s
Riprap basin	<3	H	H	H	Not needed	na
Riprap apron	na	H	H	H	Not Needed	Culvert Rise ≤ 60 in
Straight drop structure	<1	H	L	M	Required	Drop< 15 ft
Box Inlet drop structure	<1	H	L	M	Required	Drop< 12 ft
USACE stilling well	na	M	L	N	Desirable	na

[1]Debris notes: N = none, L = low, M = moderate, H = heavy
na = not applicable.

R8 Chapter 8 References

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3. Federal Highway Administration (FHWA), Federal-Aid Policy Guide. “Highways.” Title 23 Code of Federal Regulations (CFR).

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6. Schall, James D., Thompson, Philip L., Zerges, Steve M., Kilgore, Roger T., Morris, Johnny L. 2012, Hydraulic Design of Highway Culverts Third Edition, Hydraulic Design Series No. 5, FHWA-HIF-12-026. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.

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Chapter 9: Post-Development Stormwater Management

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9.1 Introduction

The construction of ALDOT roads often requires some replacement of natural ground cover with more impervious surfaces as well as the alteration of natural topography. These changes can affect the hydrology of a drainage area with respect to runoff volume and peak flow rate. ALDOT manages post-development hydrology to the maximum extent practicable and in accordance with ALDOT's specific capabilities through post-construction stormwater management.

9.2 Post-Construction Stormwater Management Design Requirements

ALDOT's policy for implementing stormwater management,as well as, methods for determining the potential hydrologic impacts of development and the selection of BMPs to manage those impacts are detailed in this section.

9.2.1 ALDOT Post-Construction Policy

The guidelines given here should be followed during drainage design on all ALDOT projects requiring new development or re-development.

"New development" describes the creation of a new transportation facility or a new support facility that causes a ground disturbance of greater than one acre. "Re-development" with respect to transportation facilities describes non-maintenance work performed to or on an existing transportation facility that provides for an increased number of thru lanes of travel and causes a ground disturbance of greater than one acre. Work on an existing road that does not result in an additional thru lane does not constitute re-development. Re-Development with respect to support facilities describes non-maintenance work performed to or on an existing support facility that causes a ground disturbance of more than one acre.

Designers must provide features and practices that cause post-development hydrology to mimic pre-development hydrology of the site to the maximum extent practicable, working within the constraints of the project, at all locations of discharge. The basis for design to meet this requirement shall be small, frequent rain events up to and including the 95th-percentile rain event for the site. While working toward this design goal, initial consideration should be the use of decentralized practices and features near the source of the runoff. Design elements that utilize natural materials and processes will be considered whenever possible.

Small, frequent rain events are those storm events with rainfall depths up to and including the 95th-percentile event for a specific location. Pre-development and Post-development hydrology include both peak discharge and runoff volume. Pre-development hydrology is the existing hydrological condition of the site just prior to construction of the planned development or re-development.

The Chief Engineer may approve exceptions to this policy so long as downstream property will not be significantly impacted, and the bed and bank structure of receiving stream channels will not be significantly degraded by the increased stormwater discharge. Justification for an exception will be described and quantified in a written request to the Chief Engineer, including a description of the analysis and conclusions regarding downstream impacts.

9.2.2 Determining Post-Development Hydrology Changes

As indicated above, designers should provide features and practices that cause post-development hydrology to mimic pre-development hydrology of the site to the maximum extent practicable for applicable projects. To that end, the designer must be able to estimate the potential changes in hydrology caused by development. Below, guidance for drainage design using small, frequently occurring storms is provided for the designer. Runoff volume (in inches) is calculated using the 95^{th}-percentile rainfall event and a volumetric runoff coefficient. Peak discharge is calculated using the rainfall, basin area, modified curve number, and time of concentration. The modified curve number is determined using the rainfall and runoff volume. Peak discharge can be calculated by hand or through the use of various computer programs. Sample calculations for determining runoff and peak discharge have been included.

9.2.3 Design Storm

9.2.3.1 Design Storm

Small, frequently occurring storms account for a large proportion of the annual precipitation volume, and runoff from those storm events also significantly alter the discharge frequency, rate and temperature of the runoff (USEPA 2009). As indicated in the GFO 3-73, the Department will consider storm events with rainfall depths up to and including the 95th percentile rainfall event, as defined by USEPA (2009), for a specific location as being such small storm events. In turn, for stormwater runoff calculation, the design storm to be used in the analysis will be the 95th percentile rainfall event.

9.2.3.2 95th Percentile Rainfall Depths in Alabama

Estimation of the 95th percentile rainfall depths for all locations throughout the State was performed by the Department's Design Bureau according to the approach detailed in the MS4 Stormwater Management Program Plan. Figure 1 is the isohyetal map for the 95th percentile rainfall depths in Alabama generated using that approach.

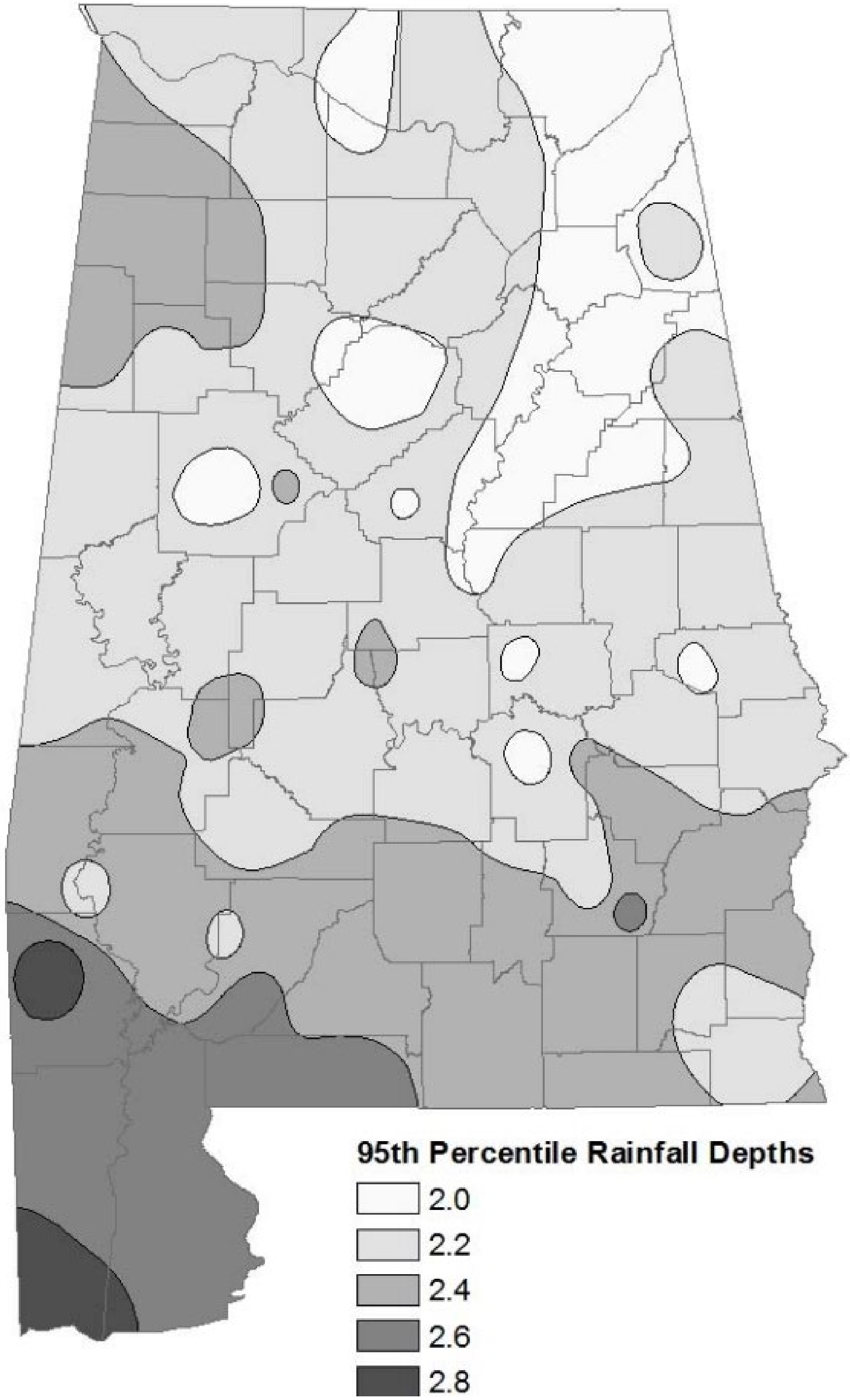

Figure 9.1 – Isohyetal map for the 95th percentile rainfall depths in Alabama

9.2.4 Stormwater Runoff Volume and Peak Discharge Calculation

9.2.4.1 NRCS Curve Number Method

The curve number (CN) method is the most commonly used tool for estimating runoff from rainfall excess. The method was developed by the USDA Natural Resources Conservation Service (NRCS, formerly SCS) and described in detail in Chapter 10 of the National Engineering Handbook, Part 630 - Hydrology (NEH 630) (USDA 2004). In this method, runoff is calculated based on precipitation, initial abstraction, and watershed storage. The curve number runoff equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad P > I_a \qquad (1)$$

$$Q = 0 \qquad P \leq I_a \qquad (2)$$

Q is runoff (in.)
P is design storm (in.)
I_a is initial abstraction (in.)
S is potential maximum retention (in.)

Initial abstraction (I_a) consists mainly of interception, infiltration, and depression storage. I_a can be highly variable but NRCS (USDA 2004) found that it can be empirically approximated by using the following formula:

$$I_a = 0.2\,S \qquad (3)$$

Therefore, the runoff equation becomes:

$$Q = \frac{(P - 0.2\,S)^2}{(P + 0.8\,S)} \qquad P > I_a \qquad (4)$$

where S is a function of CN:

$$S = \frac{1000}{CN} - 10 \qquad (5)$$

Therefore, runoff can be calculated using only the curve number and rainfall. Curve numbers are determined by land cover type, hydrologic condition, antecedent moisture condition (AMC), and hydrologic soil group (HSG). Curve numbers for various land covers based on an average AMC for annual floods and I_a = 0.2 S can be found in NEH

630 (USDA 2004). For watersheds having multiple land cover type and HSG, CN is weighted to get watershed CN, and the runoff is estimated using that weighted CN.

Despite its widespread use, the weighted CN method may not be appropriate for estimating runoff from smaller storm events because it can imply a significant initial loss that may not take place, as noted by Pitt (1999). Since all estimated 95th percentile storm events in Alabama are less than 3.0 inches (ranges from 2.0 to 2.8 inches), the design storm will be treated as a small storm. Therefore, the weighted CN will not be used to perform runoff volume and peak discharge calculations for the design storm. Instead, the CN will be modified using the methodology discussed in the following section.

9.2.4.2 Small Storm Hydrology Method

The Small Storm Hydrology Method (Pitt 1987) was developed to estimate the runoff volume from urban and suburban land uses for relatively small storm events. In this method, runoff is calculated using volumetric runoff coefficients. Pitt (2013) lists the runoff coefficients that are based on extensive field research conducted in the Midwestern U.S., the Southeastern U.S., and Ontario, Canada, over a wide range of land uses and storm events. Runoff coefficients for individual source areas generally vary with the rainfall amount. Larger storms have higher coefficients. The runoff coefficients for various source areas (Table 1) are derived using the original table from Pitt (2013).

Runoff is simply calculated by multiplying the rainfall amount by the appropriate runoff coefficient. Because the runoff relationship is linear for a given storm, a composite runoff coefficient (weighted average) can be computed for an area consisting of multiple land uses. Therefore, runoff is given by:

$$Q = P * R_{vc} \tag{6}$$

Q is runoff (in.)
P is the 95th percentile rainfall (in.)
R_{vc} is the composite runoff coefficient

The following equation is used to determine the stormwater runoff volume (V) in cubic feet:

$$V = \frac{P}{12} * R_{vc} * A * 43560 \tag{7}$$

V is runoff volume (ft^3)
A is drainage area (acres).

Using the rainfall amount and runoff, a corresponding modified CN can be computed utilizing the following equation:

$$CN = \frac{1000}{10 + 5P + 10Q - 10\sqrt{Q^2 + 1.25\ Q\ P}} \qquad (8)$$

Once the modified CN is computed, the time of concentration (t_c) can be computed based on methods identified in Chapter 15 of NEH 630 (USDA 2010) and peak discharge (Q_p) for the design storm can be computed. Procedures and sample calculations for stormwater runoff volume and peak discharge estimation are provided in the next subsection.

Table 9.1 – Source areas and corresponding R_v values for different rainfall amounts

Source Areas	Rainfall (inches)				
	2.0	2.2	2.4	2.6	2.8
Roof Areas					
Flat, Connected	0.90	0.91	0.91	0.92	0.93
Pitched, Connected	0.99	0.99	0.99	0.99	0.99
Flat or Pitched, Unconnected, A Soil	0.07	0.09	0.10	0.12	0.13
Flat or Pitched, Unconnected, B Soil	0.16	0.18	0.19	0.21	0.22
Flat or Pitched, Unconnected, C or D Soil	0.26	0.28	0.29	0.31	0.32
Parking and Storage Areas					
Paved, Connected	0.99	0.99	0.99	0.99	0.99
Unpaved, Connected	0.89	0.90	0.91	0.92	0.92
Paved or Unpaved, Unconnected, A Soil	0.07	0.09	0.10	0.12	0.13
Paved or Unpaved, Unconnected, B Soil	0.16	0.18	0.19	0.21	0.22
Paved or Unpaved, Unconnected, C or D Soil	0.26	0.28	0.29	0.31	0.32
Driveways or Sidewalks					
Connected	0.99	0.99	0.99	0.99	0.99
Unconnected, A Soil	0.07	0.09	0.10	0.12	0.13
Unconnected, B Soil	0.16	0.18	0.19	0.21	0.22
Unconnected, C or D Soil	0.26	0.28	0.29	0.31	0.32
Streets or Alley Areas					
Smooth textured	0.88	0.89	0.90	0.91	0.91
Intermediate or Rough Textured	0.84	0.85	0.86	0.87	0.88
Highway Areas					
Paved Lane and Shoulder	0.88	0.89	0.90	0.91	0.91
Undeveloped or Pervious Areas					
Undeveloped or Pervious Areas, A Soil	0.07	0.09	0.10	0.12	0.13
Undeveloped or Pervious Areas, B Soil	0.16	0.18	0.19	0.21	0.22
Undeveloped or Pervious Areas, C or D Soil	0.26	0.28	0.29	0.31	0.32
Residential Areas*					
Low Density, < 2 units / acre	0.26	0.28	0.29	0.31	0.32
Medium Density, between 2 and 6 units / acre	0.55	0.58	0.60	0.61	0.62
High Density, > 6 units / acre	0.99	0.99	0.99	0.99	0.99
Other Areas					
Commercial / Industrial	0.99	0.99	0.99	0.99	0.99
High Traffic Urban Paved Areas	0.98	0.98	0.98	0.99	0.99
High Traffic Urban Pervious Areas	0.55	0.58	0.60	0.61	0.62
Excavation or Embankment Construction	0.26	0.28	0.29	0.31	0.32

Connected - flows directly into the drainage system, or occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system.

Unconnected - drains over a pervious area as sheet flow, provided the impervious area is less than one-half the pervious area and the flow path through the pervious area is at least twice the impervious surface flow path. For unconnected flow use the R_v values associated with the appropriate soil type for pervious areas.

*Residential areas include buildings, driveways, yard and streets.

9.2.4.3 Calculation Procedures

Stormwater runoff volume and peak discharge can be estimated using the following procedure:

1. Determine the 95th percentile rainfall depth for the project location using the isohyetal map (Figure 1).
2. Delineate watershed boundaries and divide watershed into source areas based on its land use and soil type characteristics.
3. Assign runoff coefficients to source areas using Table 1 and compute the composite runoff coefficient (R_{vc}) by calculating a weighted average.
4. Compute runoff volume using Equations (6) and (7).
5. Compute modified CN using Equation (8).
6. Compute travel times and time of concentration using Velocity Method as described in Chapter 15 of NEH 630 (USDA 2010)
7. Calculate I_a/P using Equations (3) and (5).
8. Compute unit peak discharge (q_u) using Appendix I Figure I.2 or I.3.
9. Calculate peak discharge using Graphical Peak Discharge Method as described in TR-55 (USDA 1986)

Land use and soil data can be obtained from various online sources. A few example websites are provided below:

Land Use Data:

National Land Cover Database 2011 (NLCD 2011): NLCD 2011 is the most recent national land cover product, at the time of this manual, created by the Multi-Resolution Land Characteristics (MRLC) Consortium that has been applied consistently across the United States at a spatial resolution of 30 meters. Check the website for the most current products. Due to the coarser resolution of land use data for the purpose of this study, it is recommended that designers use recent aerial imagery to delineate land use for a given location manually and/or using GIS tools.

Aerial Imagery:

Aerial imagery is available online in ArcGIS or it can be downloaded from different sources:

USGS EarthExplorer: Aerial imagery of different types (high resolution orthoimagery, NAIP JPG2000, etc.) are available to download depending on selected location.

USGS National Map Viewer: 1-meter orthoimagery and other data can be downloaded from USGS National Map Viewer.

Soil Data:

The Soil Survey Geographic Database (SSURGO), operated by the USDA-NRCS, provides soil data and information produced by the National Cooperative Soil Survey. The information can be displayed in tables or as maps and is available for most areas in Alabama and other states. SSURGO map data can be viewed in the Web Soil Survey or downloaded in ESRI Shapefile format. The coordinate systems are geographic. Attribute data can be downloaded in text format that can be imported into a Microsoft Access database.

9.2.4.4 Sample Calculation (Example 1)

Using steps outlined in Section 9.2.6.3, the calculation of pre-development and post-development runoff volumes and peak discharges for the 95th percentile rainfall event in a watershed near Birmingham, Alabama is carried out below:

Pre-development Conditions

1. Determine the 95th percentile rainfall depth for the project location using the isohyetal map (Figure 1).

   95th percentile rainfall (P) = 2.0 in.

2. Delineate watershed boundaries and divide watershed into source areas based on its land use and soil type characteristics.

Manual delineation or automatic delineation using GIS tools can delineate watershed boundaries for a given outlet and can divide a watershed into grouped areas based on its land use and soil type characteristics.

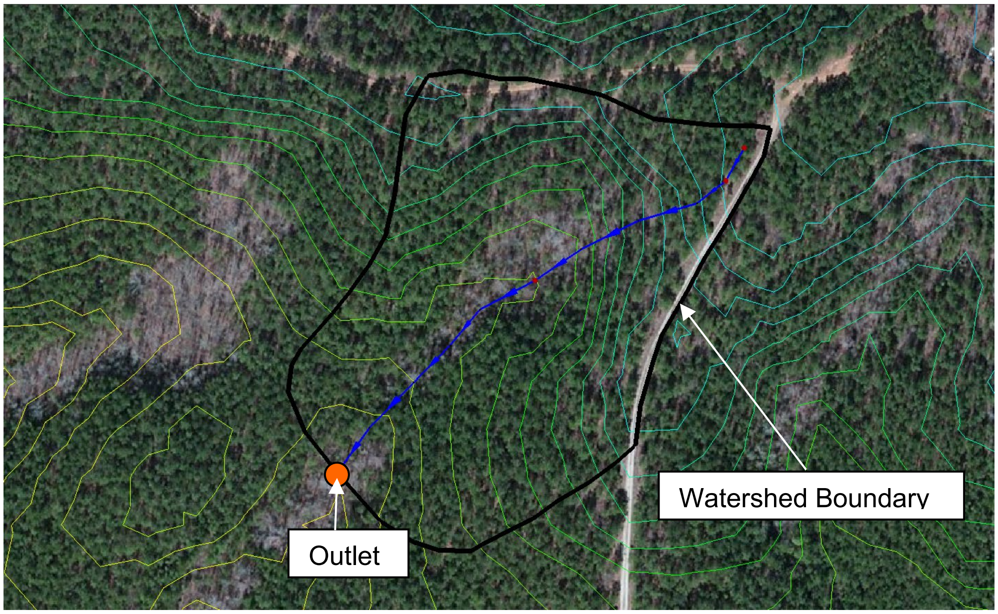

Figure 9.2 – Aerial photograph indicating an outlet and drainage boundary

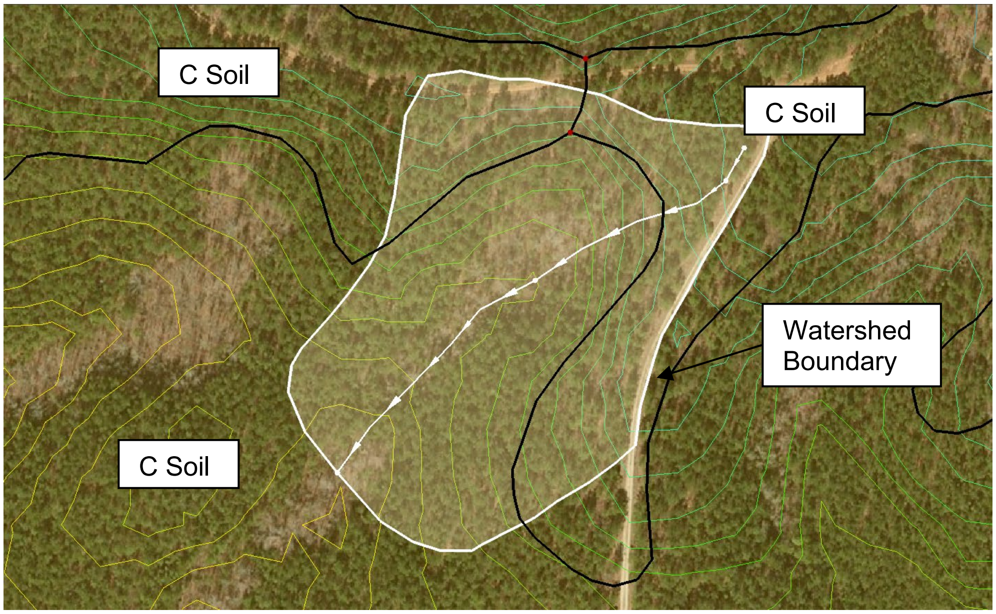

Figure 9.3 – Aerial photograph indicating drainage boundary and soil types

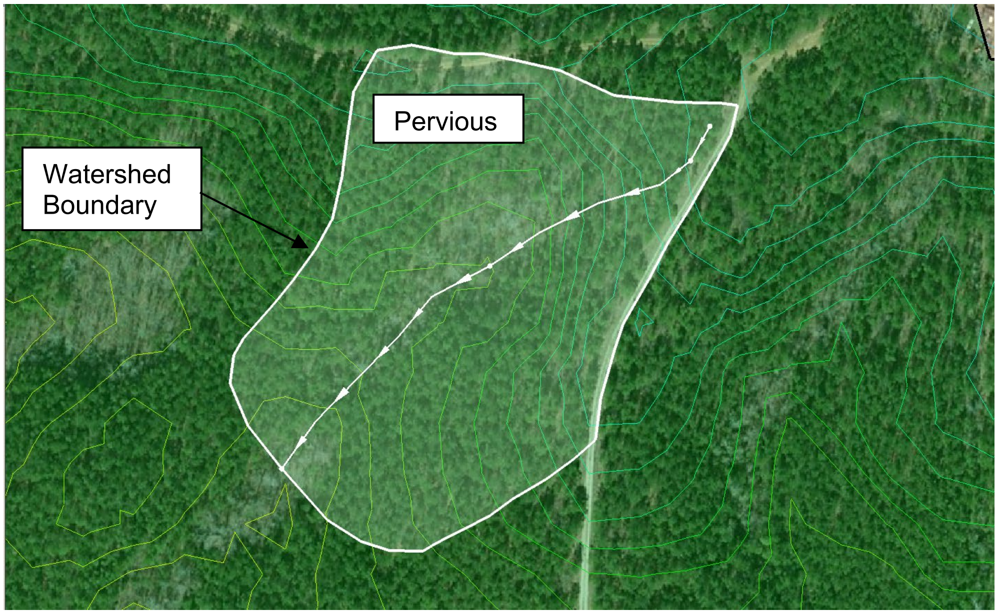

Figure 9.4 – Aerial photograph indicating drainage boundary and pre-development source areas

Table 2. Land use and soil type distribution of sample watershed in Birmingham, Alabama

	Land Use	Soil Type	Area in acres
1	Woods- Good	Type C	5.9

1. Assign runoff coefficient to source areas using Table 1 and compute the composite runoff coefficient (R_{vc}) by calculating a weighted average.

Table 3. Source areas and corresponding R_v

Source areas	Area (acres)	R_v (2 in)	Area * R_v
Woods (Pervious areas – clayey soils, HSG - C)	5.9	0.26	1.534
∑ A =	5.9	∑ (A*R_v)=	1.534

Composite runoff coefficient

$$R_{vc} = \frac{\sum A * R_v}{\sum A} = \frac{1.534}{5.9} = \mathbf{0.26}$$

2. Compute runoff volume using Equations (6) and (7).

$$Q = P * R_{vc} = 2 * 0.26 = \mathbf{0.52\ in.}$$

$$V = \frac{P}{12} * R_{vc} * A * 43560 = \frac{2}{12} * 0.26 * 5.9 * 43560 = \mathbf{11137\ ft^3}$$

3. Compute modified CN using Equation (8)

$$CN = \frac{1000}{10 + 5P + 10Q - 10\sqrt{Q^2 + 1.25\,Q\,P}}$$

$$CN = \frac{1000}{10 + 5 * 2 + 10 * 0.52 - 10\sqrt{0.52^2 + 1.25 * 0.52 * 2}} = \mathbf{79}$$

4. Compute travel time and time of concentration (t_c) using Velocity Method

9.2.4.4.1 SEGMENT 1 – SHEET FLOW

Travel time for sheet flow

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}} = \frac{0.007(0.4 * 50)^{0.8}}{(4.1)^{0.5}(0.029)^{0.4}} = 0.157\ hr = 9.4\ min$$

where overland roughness coefficient (n) = 0.4 (Light Woods) (Appendix I Table I.1),
flow length (L) = 50 ft,
2-year 24-hour rainfall (P_2) = 4.1 in., and
slope (S) = 0.029 ft/ft

9.2.4.4.2 SEGMENT 2 – SHALLOW CONCENTRATED FLOW

From Figure I.1 (in Appendix) based on ground cover (Forest) and slope (0.204), average flow velocity (v)

$$v = 2.516(S)^{0.5} = 2.516 * 0.204^{0.5} = 1.14\ ft/s$$

Travel time for shallow concentrated flow

$$T_t = \frac{L}{60\ v} = \frac{300}{60 * 1.14} = 4.4\ min$$

9.2.4.4.3 SEGMENT 3 – OPEN CHANNEL FLOW

For trapezoidal channel of width = 4 feet, flow depth = 0.4 feet (Grassed waterways, shallow concentrated flow, Figure I.1), and side slope (H:V)=3:1,

Area, $A = \frac{1}{2} * 0.4 * (6.4 + 4) = 2.08\ ft^2$

Wetted Perimeter, $P = 1.265 * 2 + 4 = 6.53\ ft$

Hydraulic Radius, $R = {}^{A}/_{P} = {}^{2.08}/_{6.53} = 0.319$

For open channel flow, velocity is estimated using Manning's equation:

$$v = \frac{1.49(R)^{\frac{2}{3}}(S)^{\frac{1}{2}}}{n} = \frac{1.49(0.319)^{\frac{2}{3}}(0.051)^{\frac{1}{2}}}{0.06} = 2.62\ \frac{ft}{s}$$

where channel roughness (n) = 0.06 and
slope (S) = 0.051 ft/ft

Travel time for open channel flow

$$T_t = \frac{L}{60\ v} = \frac{380}{60 * 2.62} = 2.4\ min$$

9.2.4.4.4 TIME OF CONCENTRATION

Table 4. Time of concentration calculation

Segment	Type of Flow	Length (ft)	Slope (ft/ft)	T_t (min)
1	Sheet	50	0.029	9.4
2	Shallow concentrated	300	0.204	4.4
3	Open channel	380	0.051	2.4

$$t_c = 9.4 + 4.4 + 2.4 = 16.2\ min = \mathbf{0.27\ hr}$$

5. Calculate I_a/P using Equations (3) and (5).

$$I_a = 0.2\ S = 0.2 * \left({}^{1000}/_{CN} - 10\right) = 0.2 * \left({}^{1000}/_{79} - 10\right) = 0.532$$

$$\frac{I_a}{P} = \frac{0.532}{2} = \mathbf{0.27}$$

6. Compute unit peak discharge (q_u) using Figure I.2 or I.3.

   q_u = 450 csm/in (From Appendix I Figure I.3 for t_c = 0.27 hr and I_a/P = 0.27)

7. Calculate peak discharge (Q_p) using Graphical Peak Discharge Method for pre-development conditions

   $$Q_p = q_u\, A\, Q\, F_p = 450 * 0.0092 * 0.52 * 1 = \mathbf{2.2\ cfs}$$

   where drainage area (A) = 0.0092 mi^2,
   runoff volume (Q) = 0.52 in., and
   F_p = 1 (From Appendix I Table I.2, no pond and swamp areas)

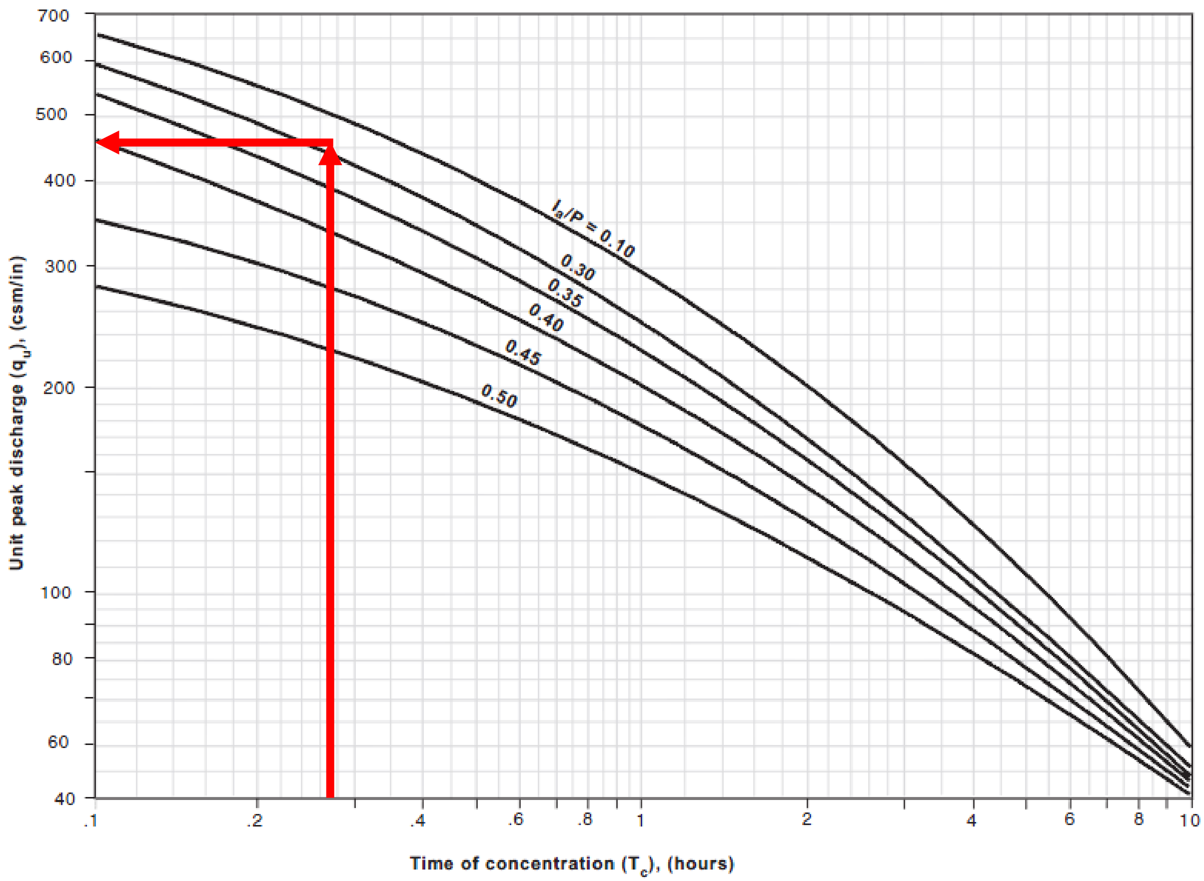

Figure 9.5 – Estimating unit peak discharge for type III rainfall distribution using Figure I.3

Post-development Conditions

1. Determine the 95th percentile rainfall depth for the project location using the isohyetal map (Figure 1).

   95th percentile rainfall (P) = 2.0 in.

2. Delineate watershed boundaries and divide watershed into source areas based on its land use and soil type characteristics.

Manual delineation or automatic delineation using GIS tools can delineate watershed boundaries for a given outlet and can divide a watershed into grouped areas based on its land use and soil type characteristics.

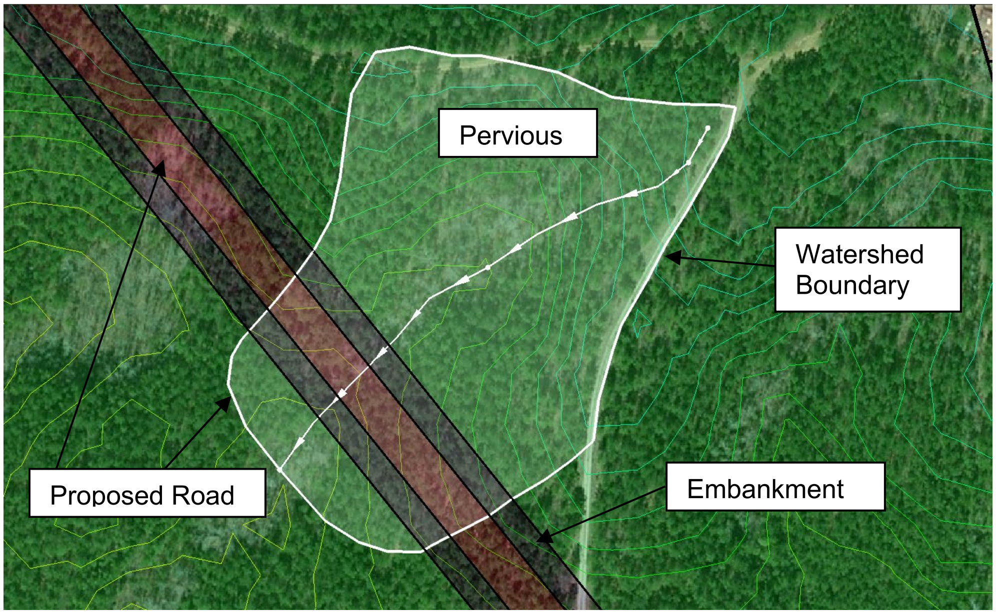

Figure 9.6 – Aerial photograph indicating drainage boundary and post-development source areas

Table 5. Land use and soil type distribution of sample watershed in Birmingham, Alabama

	Land Use	Soil Type	Area in acres	
			Pre	Post
1	Woods- Good	Type C	5.9	4.8
2	Compacted Embankment	Type C		0.5
3	Road/Highway	Type C	-	0.6

1. Assign runoff coefficient to source areas using Table 1 and compute the composite runoff coefficient (R_{vc}) by calculating a weighted average.

Table 6. Source areas and corresponding R_v

Source areas	Area (acres)	R_v (2 in)	Area * R_v
Woods (Pervious areas – clayey soils, HSG - C)	4.8	0.26	1.248
Compacted Embankment (Pervious, HSG - D)	0.5	0.26	0.130
Road (Paved freeway & shoulder, smooth)	0.6	0.88	0.528
∑ A =	5.9	∑ (A*R_v)=	1.906

Composite runoff coefficient

$$R_{vc} = \frac{\sum(A * R_v)}{\sum A} = \frac{1.906}{5.9} = \mathbf{0.32}$$

2. Compute runoff volume using Equations (6) and (7).

$$Q = P * R_{vc} = 2 * 0.32 = \mathbf{0.64\ in.}$$

$$V = \frac{P}{12} * R_{vc} * A * 43560 = \frac{2}{12} * 0.33 * 5.9 * 43560 = \mathbf{13707\ ft^3}$$

3. Compute modified CN using Equation (8).

$$CN = \frac{1000}{10 + 5P + 10Q - 10\sqrt{Q^2 + 1.25\ Q\ P}}$$

$$CN = \frac{1000}{10 + 5 * 2 + 10 * 0.64 - 10\sqrt{0.64^2 + 1.25 * 0.64 * 2}} = \mathbf{82}$$

4. Compute travel time and time of concentration (t_c) using Velocity Method

9.2.4.4.5 SEGMENT 1 – SHEET FLOW

Travel time for sheet flow

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}} = \frac{0.007(0.4 * 50)^{0.8}}{(4.1)^{0.5}(0.029)^{0.4}} = 0.157\ hr = 9.4\ min$$

where overland roughness coefficient (n) = 0.4 (Light Woods) (Appendix Table I.1),
flow length (L) = 50 ft,
2-year 24-hour rainfall (P_2) = 4.1 in., and
slope (S) = 0.029

9.2.4.4.6 SEGMENT 2 – SHALLOW CONCENTRATED FLOW

From Appendix Figure I.1 based on ground cover (Forest) and slope (0.204), average flow velocity (v)

$$v = 2.516(S)^{0.5} = 2.516 * 0.204^{0.5} = 1.14\ ft/s$$

Travel time for shallow concentrated flow

$$T_t = \frac{L}{60\ v} = \frac{300}{60 * 1.14} = 4.4\ min$$

9.2.4.4.7 SEGMENT 3 – OPEN CHANNEL FLOW

For trapezoidal channel of width = 4 feet, flow depth = 0.4 feet (Grassed waterways, shallow concentrated flow, Figure I.1), and side slope(H:V)=3:1,

Area, $A = \frac{1}{2} * 0.4 * (6.4 + 4) = 2.08\ ft^2$

Wetted Perimeter, $P = 1.265 * 2 + 4 = 6.53\ ft$

Hydraulic Radius, $R = {}^{A}/_{P} = {}^{2.08}/_{6.53} = 0.319$

For open channel flow, velocity is estimated using Manning's equation:

$$v = \frac{1.49(R)^{\frac{2}{3}}(S)^{\frac{1}{2}}}{n} = \frac{1.49(0.319)^{\frac{2}{3}}(0.051)^{\frac{1}{2}}}{0.06} = 2.62\ \frac{ft}{s}$$

where channel roughness (n) = 0.06 and
slope (S) = 0.051 ft/ft

Travel time for open channel flow

$$T_t = \frac{L}{60\ v} = \frac{380}{60 * 2.62} = 2.4\ min$$

9.2.4.4.8 TIME OF CONCENTRATION

Table 7. Time of concentration calculation

Segment	Type of Flow	Length (ft)	Slope (ft/ft)	T_t (min)
1	Sheet	50	0.029	9.4
2	Shallow concentrated	300	0.204	4.4
3	Open channel	380	0.051	2.4

$$t_c = 9.4 + 4.4 + 2.4 = 16.2\ min = \mathbf{0.27\ hr}$$

Calculate I_a/P using Equations (3) and (5).

$$I_a = 0.2\,S = 0.2 * \left({}^{1000}/_{CN} - 10\right) = 0.2 * \left({}^{1000}/_{82} - 10\right) = 0.439$$

$$\frac{I_a}{P} = \frac{0.439}{2} = \mathbf{0.22}$$

Compute unit peak discharge (q_u) using Figure I.2 or I.3.

q_u = 475 csm/in (From Appendix I Figure I.3 for t_c = 0.27 hr and I_a/P = 0.22)

5. Calculate peak discharge (Q_p) using Graphical Peak Discharge Method for post-development conditions

$$Q_p = q_u\,A\,Q\,F_p = 475 * 0.0092 * 0.64 * 1 = \mathbf{2.8\,cfs}$$

where drainage area (A) = 0.0092 mi^2,
runoff volume (Q) = 0.66 in., and
F_p = 1 (From Appendix I Table I.2, no pond and swamp areas)

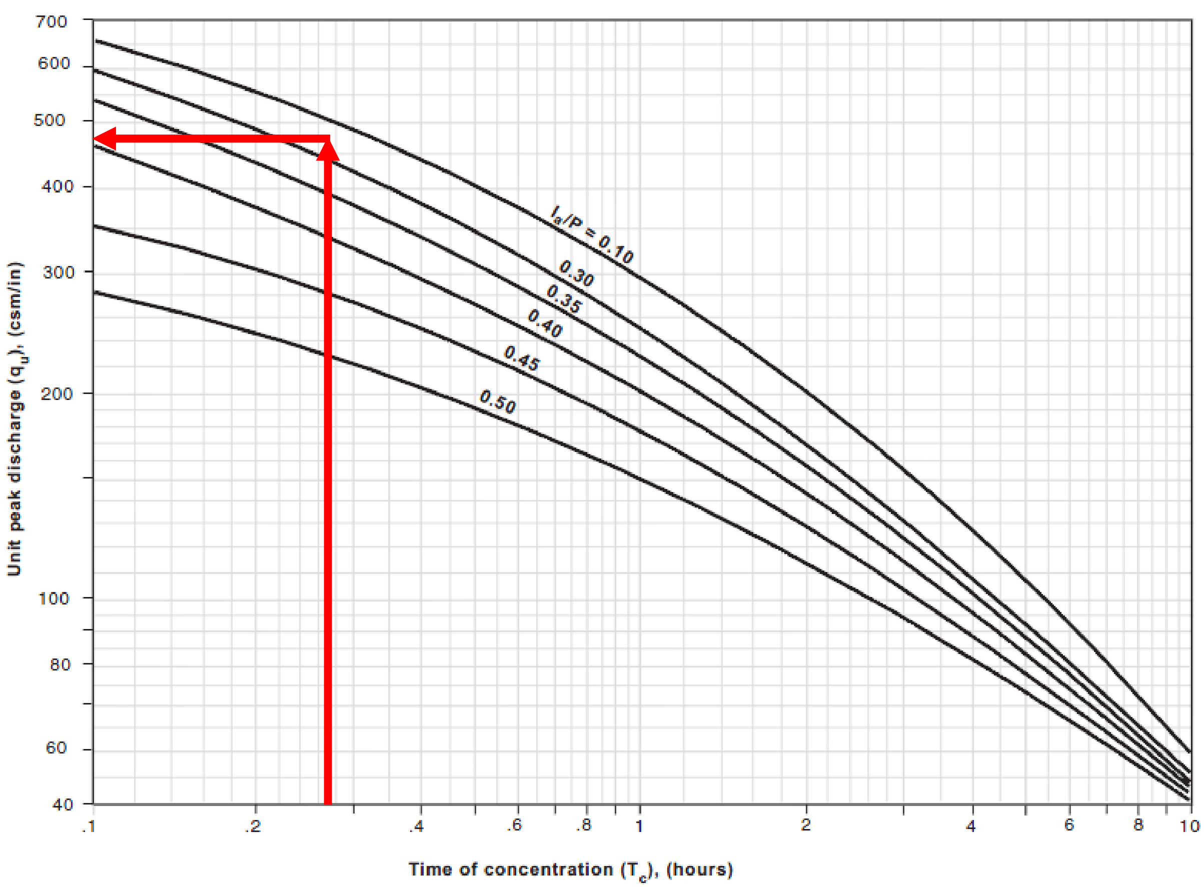

Figure 9.7 – Estimating unit peak discharge for type III rainfall distribution using Figure I.3

Summary of Results

Table 8. Comparison of pre-development and post-development runoff volumes and peak discharges

	Pre	Post
Runoff volume, Q (in.)	0.52	0.64
Runoff volume, V (ft^3)	11137	13707
Peak discharge, Q_p (cfs)	2.2	2.8

Post-development runoff volume has increased by 2570 ft^3 or 23% compared to pre-development runoff volume. Peak discharge has increased by 0.6 cfs or 27%. Since there is significant increase in runoff volume and peak discharge, runoff management practices will be required to maintain pre-development hydrology in accordance with GFO 3-72 (ALDOT 2014).

9.2.4.5 Sample Calculation (Example 2)

Using steps outlined in Section 9.2.6.3, the calculation of pre-development and post-development runoff volumes and peak discharges for the 95th percentile rainfall event for a watershed in Birmingham, Alabama is carried out below:

Pre-development Conditions

1. Determine the 95th percentile rainfall for project location using the isohyetal map.

   95th percentile rainfall (P) = 2.0 in.

2. Delineate watershed boundaries and divide watershed into source areas based on its land use and soil type character. Manual delineation or automatic delineation using GIS tools can delineate watershed boundaries for a given outlet and can divide a watershed into grouped areas based on its land use and soil type characteristics.

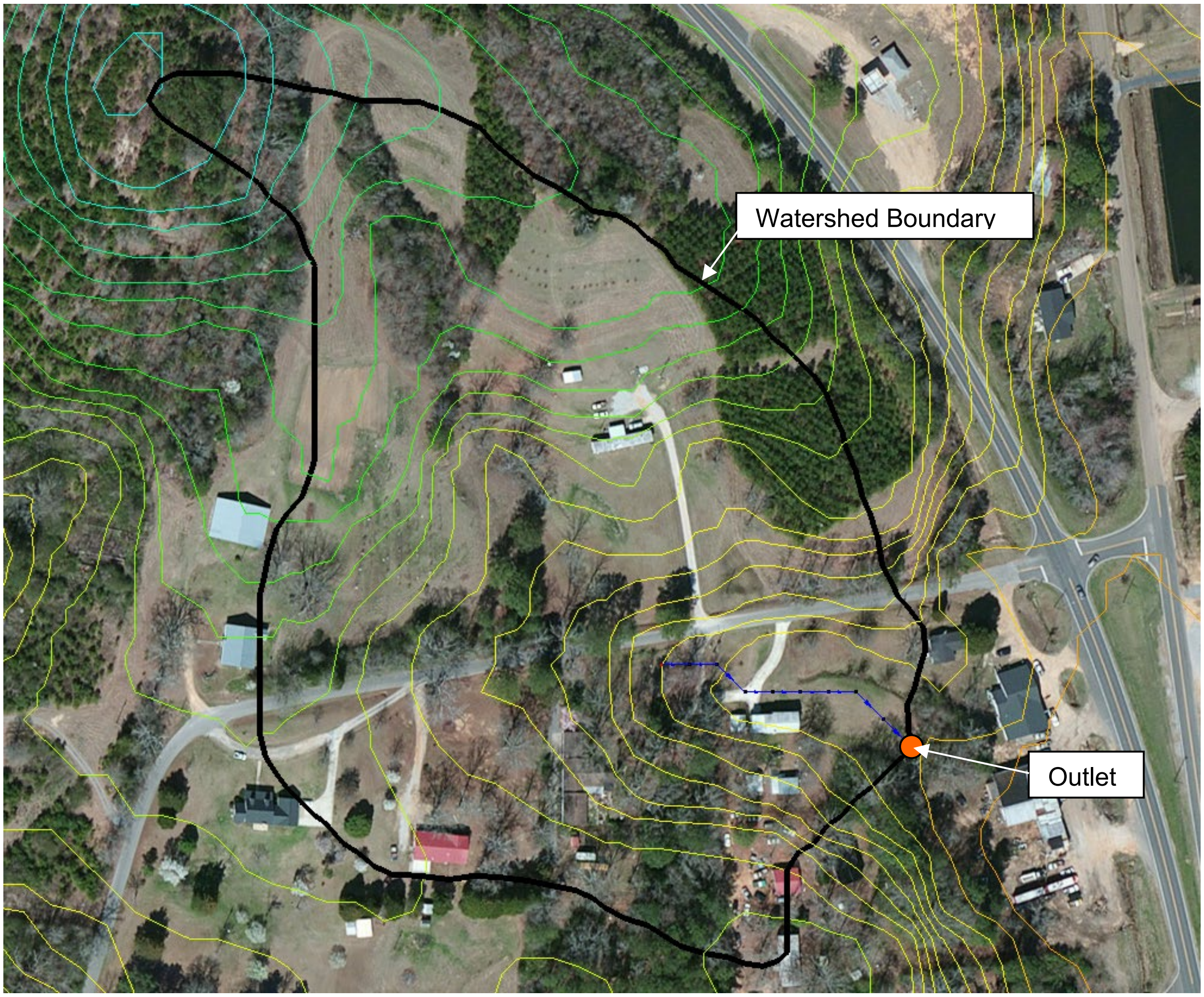

Figure 9.8 – Aerial photograph indicating an outlet and drainage boundary

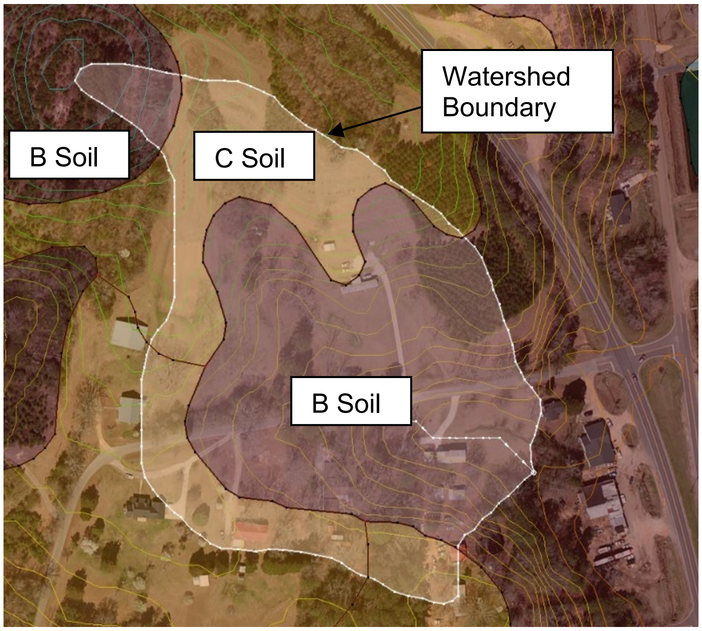

Figure 9.9 – Aerial photograph indicating drainage boundary and soil types

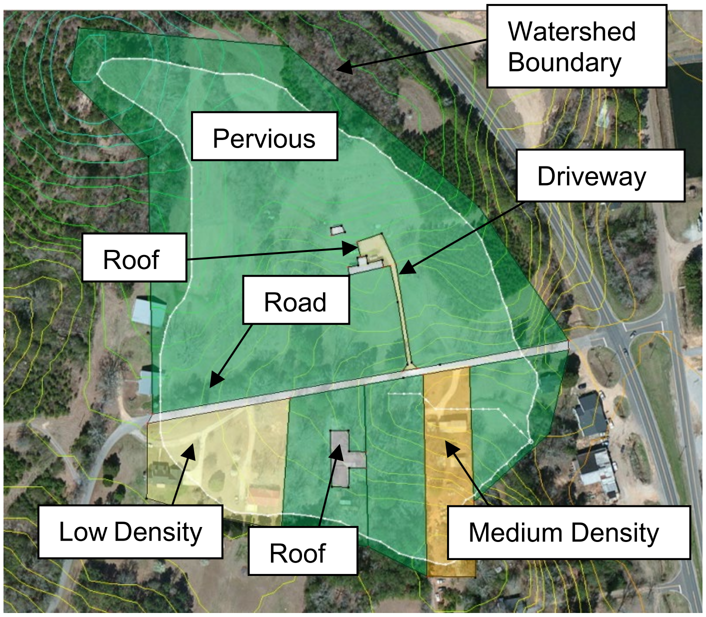

Figure 9.10 – Aerial photograph indicating drainage boundary and pre-development source areas

1. Assign runoff coefficient to source areas using Table 1 and compute the composite runoff coefficient (R_{vc}) by calculating a weighted average.

Table 9. Source areas and corresponding R_v

Source areas	Area (acres)	R_v (2 in)	Area * R_v
Undeveloped or Pervious Areas, B Soil	7.13	0.16	1.141
Undeveloped or Pervious Areas, C or D Soil	4.00	0.26	1.040
Streets, Intermediate or Rough Textured	0.32	0.84	0.269
Low Density, < 2 units / acre	1.12	0.26	0.291
Roof, Flat or Pitched, Unconnected, B Soil	0.15	0.16	0.024
Roof, Flat or Pitched, Unconnected, C or D Soil	0.03	0.26	0.008
Driveway or Sidewalk, Unconnected, B Soil	0.13	0.16	0.021
Driveway or Sidewalk, Unconnected, C or D Soil	0.02	0.26	0.005
Medium Density, between 2 and 6 units / acre	0.87	0.55	0.479
∑ A =	13.77	∑ (A*R_v)=	3.277

Composite runoff coefficient

$$R_{vc} = \frac{\sum A * R_v}{\sum A} = \frac{3.277}{13.77} = \mathbf{0.24}$$

2. Compute runoff volume using Equations (6) and (7).

$$Q = P * R_{vc} = 2 * 0.24 = \mathbf{0.48\ \mathit{in}.}$$

$$V = \frac{P}{12} * R_{vc} * A * 43560 = \frac{2}{12} * 0.24 * 13.77 * 43560 = \mathbf{23{,}993\ \mathit{ft}^3}$$

3. Compute modified CN using Equation (8)

$$CN = \frac{1000}{10 + 5P + 10Q - 10\sqrt{Q^2 + 1.25\,Q\,P}}$$

$$CN = \frac{1000}{10 + 5 * 2 + 10 * 0.48 - 10\sqrt{0.48^2 + 1.25 * 0.48 * 2}} = \mathbf{78}$$

4. Compute travel time and time of concentration (t_c)

9.2.4.5.1 SEGMENT 1 – SHEET FLOW

Travel time for sheet flow

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}} = \frac{0.007(0.4 * 43)^{0.8}}{(4.1)^{0.5}(0.026)^{0.4}} = 0.146\ hr = 8.8\ min$$

where overland roughness coefficient (n) = 0.4 (Light Woods) (Appendix I Table I.1),
flow length (L) = 43 ft,
2-year 24-hour rainfall (P_2) = 4.1 in., and
slope (S) = 0.026 ft/ft

9.2.4.5.2 SEGMENT 2 – SHALLOW CONCENTRATED FLOW

From Figure I.1 based on ground cover (Forest) and slope (0.072), average flow velocity (v)

$$v = 2.516(S)^{0.5} = 2.516 * 0.072^{0.5} = 0.68\ ft/s$$

Travel time for shallow concentrated flow

$$T_t = \frac{L}{60\ v} = \frac{328}{60 * 0.68} = 8.0\ min$$

9.2.4.5.3 SEGMENT 3 – OPEN CHANNEL FLOW

For trapezoidal channel of width = 5 feet, flow depth = 0.4 feet (Grassed waterways, shallow concentrated flow, Figure I.1), and side slope (H:V)=1:1,

Area, $A = \frac{1}{2} * 0.4 * (5.8 + 5) = 2.16\ ft^2$

Wetted Perimeter, $P = 0.57 * 2 + 5 = 6.13\ ft$

Hydraulic Radius, $R = {}^{A}/_{P} = {}^{2.16}/_{6.13} = 0.352$

For open channel flow, velocity is estimated using Manning's equation:

$$v = \frac{1.49(R)^{\frac{2}{3}}(S)^{\frac{1}{2}}}{n} = \frac{1.49(0.352)^{\frac{2}{3}}(0.056)^{\frac{1}{2}}}{0.05} = 3.52\ \frac{ft}{s}$$

where channel roughness (n) = 0.05 and
slope (S) = 0.056 ft/ft

Travel time for open channel flow

$$T_t = \frac{L}{60\ v} = \frac{971}{60 * 3.52} = 4.6\ min$$

9.2.4.5.4 TIME OF CONCENTRATION

Table 10. Time of concentration calculation

Segment	Type of Flow	Length (ft)	Slope (ft/ft)	T_t (min)
1	Sheet	43	0.026	8.8
2	Shallow concentrated	328	0.072	8.0
3	Open channel	971	0.056	4.6

$$t_c = 8.8 + 8.0 + 4.6 = 21.4\ min = \mathbf{0.36\ hr}$$

5. Calculate I_a/P using Equations (3) and (5).

$$I_a = 0.2\ S = 0.2 * \left({}^{1000}/_{CN} - 10\right) = 0.2 * \left({}^{1000}/_{78} - 10\right) = 0.564$$

$$\frac{I_a}{P} = \frac{0.564}{2} = \mathbf{0.28}$$

6. Compute unit peak discharge (q_u) using Figure I.2 or I.3.

q_u = 405 csm/in (From Figure I.3 for t_c = 0.36 hr and I_a/P = 0.28)

7. Calculate peak discharge (Q_p) using Graphical Peak Discharge Method for pre-development conditions

$$Q_p = q_u\ A\ Q\ F_p = 405 * 0.0215 * 0.48 * 1 = \mathbf{4.2\ cfs}$$

where drainage area (A) = 0.0215 mi^2,
runoff volume (Q) = 0.48 in., and
F_p = 1 (From Table I.2, no pond and swamp areas)

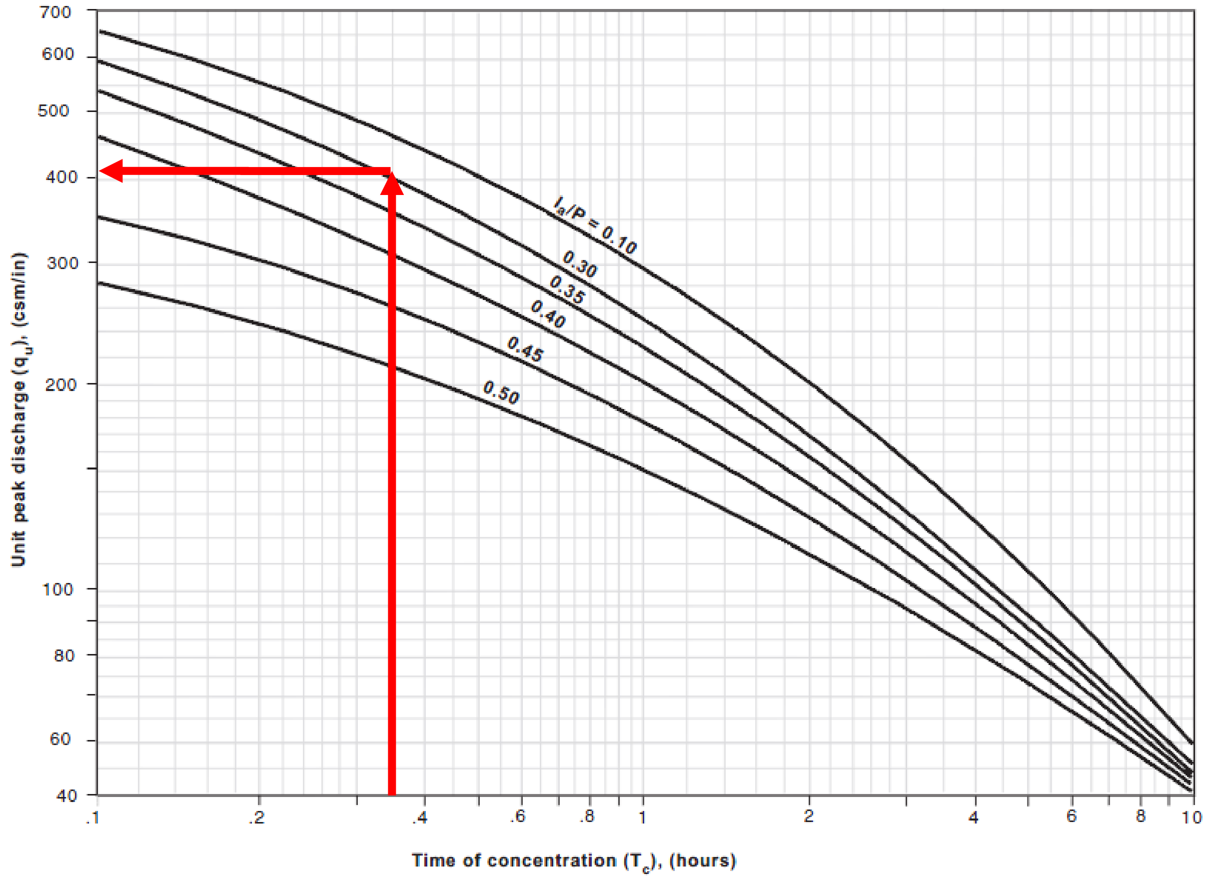

Figure 9.11 – Estimating unit peak discharge for type III rainfall distribution using Figure I.3

Post-development Conditions

1. Determine the 95th percentile rainfall for project location using the computer program described in Section 2.

   95th percentile rainfall (P) = 2.0 in.

2. Delineate watershed boundaries and divide watershed into source areas based on its land use and soil type characteristics.

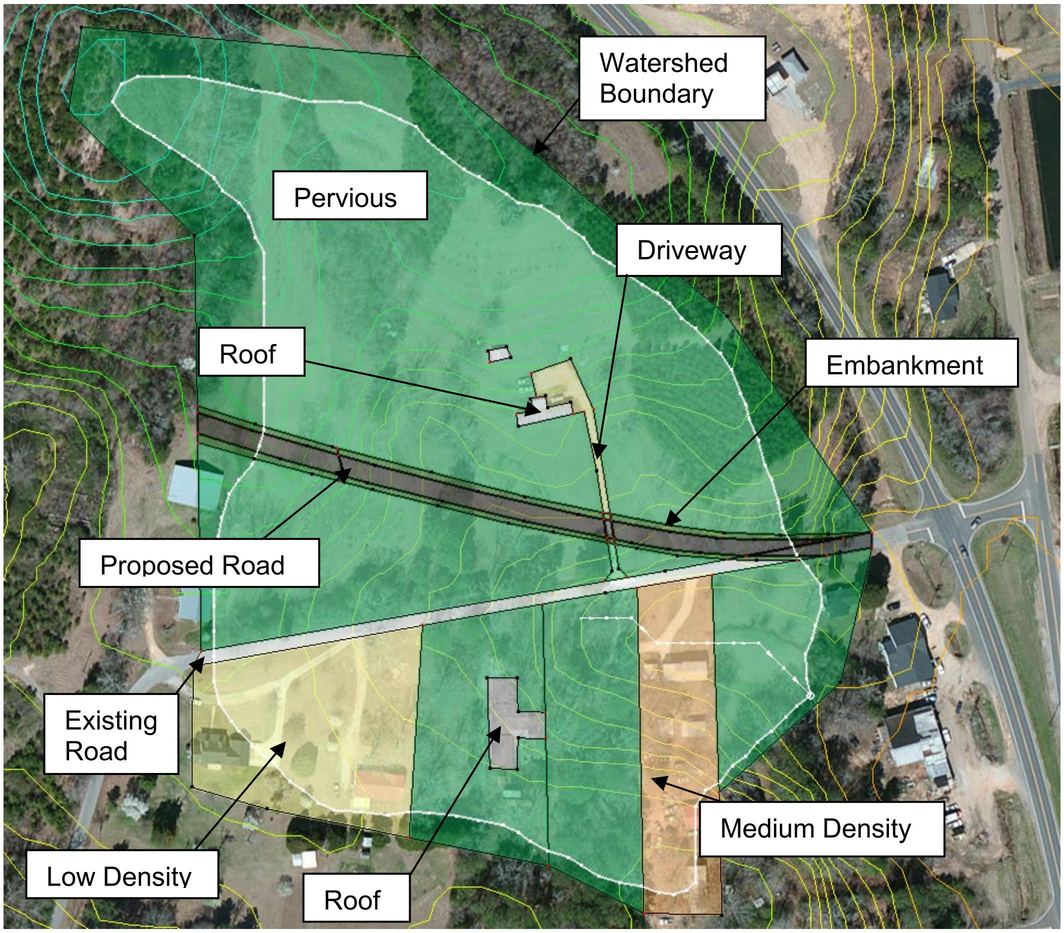

Figure 9.12 – Aerial photograph indicating drainage boundary and post-development source areas

1. Assign runoff coefficient to source areas using Table 1 and compute the composite runoff coefficient (R_{vc}) by calculating a weighted average.

Table 11. Source areas and corresponding R_v

Source areas	Area (acres)	R_v (2 in)	Area * R_v
Undeveloped or Pervious Areas, B Soil	6.58	0.16	1.053
Undeveloped or Pervious Areas, C or D Soil	3.81	0.26	0.991
Streets, Intermediate or Rough Textured	0.26	0.84	0.218
Low Density, < 2 units / acre	1.12	0.26	0.291
Roof, Flat or Pitched, Unconnected, B Soil	0.15	0.16	0.024
Roof, Flat or Pitched, Unconnected, C or D Soil	0.03	0.26	0.008
Driveway or Sidewalk, Unconnected, B Soil	0.12	0.16	0.019
Driveway or Sidewalk, Unconnected, C or D Soil	0.02	0.26	0.005
Medium Density, between 2 and 6 units / acre	0.87	0.55	0.479
Paved Lane and Shoulder	0.50	0.88	0.440
Excavation or Embankment Construction	0.31	0.26	0.081
∑ A =	13.77	∑ (A*R_v)=	3.608

Composite runoff coefficient

$$R_{vc} = \frac{\sum(A * R_v)}{\sum A} = \frac{3.608}{13.77} = \mathbf{0.26}$$

2. Compute runoff volume using Equations (6) and (7).

$$Q = P * R_{vc} = 2 * 0.26 = \mathbf{0.52\ \mathit{in}.}$$

$$V = \frac{P}{12} * R_{vc} * A * 43560 = \frac{2}{12} * 0.26 * 13.77 * 43560 = \mathbf{25{,}992\ \mathit{ft}^3}$$

3. Compute modified CN using Equation (8).

$$CN = \frac{1000}{10 + 5P + 10Q - 10\sqrt{Q^2 + 1.25\,Q\,P}}$$

$$CN = \frac{1000}{10 + 5 * 2 + 10 * 0.52 - 10\sqrt{0.52^2 + 1.25 * 0.52 * 2}} = \mathbf{79}$$

4. Compute travel time and time of concentration (t_c)

9.2.4.5.5 SEGMENT 1 – SHEET FLOW

Travel time for sheet flow

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}} = \frac{0.007(0.4 * 43)^{0.8}}{(4.1)^{0.5}(0.026)^{0.4}} = 0.146\ hr = 8.8\ min$$

where overland roughness coefficient (n) = 0.4 (Light Woods) (Appendix I Table I.1),
flow length (L) = 43 ft,
2-year 24-hour rainfall (P_2) = 4.1 in., and
slope (S) = 0.026

9.2.4.5.6 SEGMENT 2 – SHALLOW CONCENTRATED FLOW

From Figure I.1 based on ground cover (Forest) and slope (0.204), average flow velocity (v)

$$v = 2.516(S)^{0.5} = 2.516 * 0.072^{0.5} = 0.68\ ft/s$$

Travel time for shallow concentrated flow

$$T_t = \frac{L}{60\ v} = \frac{328}{60 * 0.68} = 8.0\ min$$

9.2.4.5.7 SEGMENT 3 – OPEN CHANNEL FLOW

For trapezoidal channel of width = 5 feet, flow depth = 0.4 feet (Grassed waterways, shallow concentrated flow, Figure I.1), and side slope (H:V)=1:1,

Area, $A = \frac{1}{2} * 0.4 * (5.8 + 5) = 2.16\ ft^2$

Wetted Perimeter, $P = 0.57 * 2 + 5 = 6.13\ ft$

Hydraulic Radius, $R = {}^{A}/_{P} = {}^{2.16}/_{6.13} = 0.352$

For open channel flow, velocity is estimated using Manning's equation:

$$v = \frac{1.49(R)^{\frac{2}{3}}(S)^{\frac{1}{2}}}{n} = \frac{1.49(0.352)^{\frac{2}{3}}(0.056)^{\frac{1}{2}}}{0.05} = 3.52\ \frac{ft}{s}$$

where channel roughness (n) = 0.05 and
slope (S) = 0.056 ft/ft

Travel time for open channel flow

$$T_t = \frac{L}{60\ v} = \frac{971}{60 * 3.52} = 4.6\ min$$

9.2.4.5.8 TIME OF CONCENTRATION

Table 12. Time of concentration calculation

Segment	Type of Flow	Length (ft)	Slope (ft/ft)	T_t (min)
1	Sheet	43	0.026	8.8
2	Shallow concentrated	328	0.072	8.0
3	Open channel	971	0.056	4.6

$$t_c = 8.8 + 8.0 + 4.6 = 21.4\ min = \mathbf{0.36\ hr}$$

5. Calculate I_a/P using Equations (3) and (5).

$$I_a = 0.2\ S = 0.2 * \left({}^{1000}/_{CN} - 10\right) = 0.2 * \left({}^{1000}/_{79} - 10\right) = 0.532$$

$$\frac{I_a}{P} = \frac{0.532}{2} = \mathbf{0.27}$$

6. Compute unit peak discharge (q_u) using Figure I.2 or I.3.

q_u = 407 csm/in (From Figure I.3 for t_c = 0.36 hr and I_a/P = 0.27)

7. Calculate peak discharge (Q_p) using Graphical Peak Discharge Method for post-development conditions

$$Q_p = q_u\ A\ Q\ F_p = 407 * 0.0215 * 0.52 * 1 = \mathbf{4.6\ cfs}$$

where drainage area (A) = 0.0215 mi^2,
runoff volume (Q) = 0.52 in., and
F_p = 1 (From Table I.2, no pond and swamp areas)

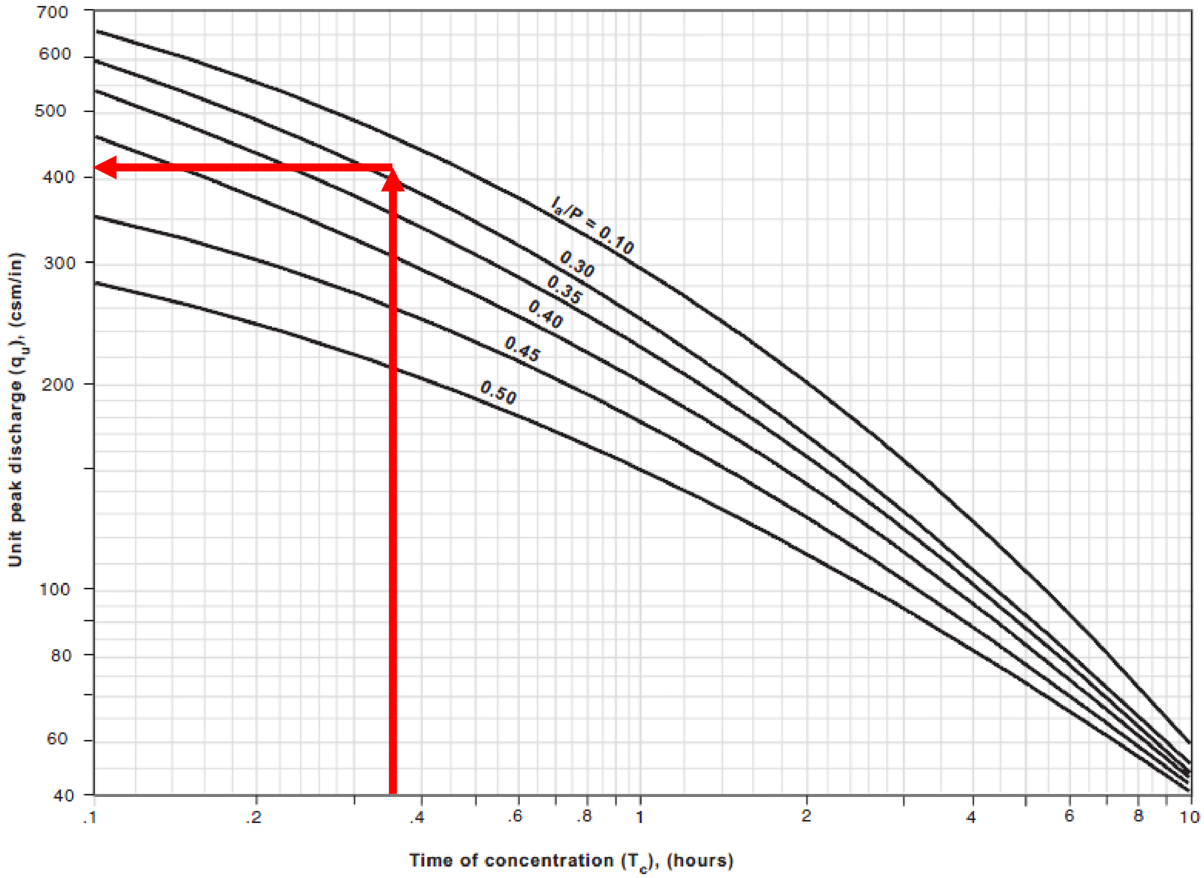

Figure 9.13 – Estimating unit peak discharge for type III rainfall distribution using Figure I.3

Summary of Results

Table 13. Comparison of pre-development and post-development runoff volumes and peak discharges

	Pre	Post
Runoff volume, Q (in.)	0.24	0.26
Runoff volume, V (ft^3)	23,993	25,992
Peak discharge, Q_p (cfs)	4.2	4.6

Post-development runoff volume has increased by 1,999 ft^3 or 8.3% compared to pre-development runoff volume. Peak discharge has increased by 0.4 cfs or 9.5%. Since there is significant increase in runoff volume and peak discharge, runoff management practices will be required to maintain pre-development hydrology in accordance with GFO 3-73 (ALDOT 2014).

9.2.5 Acceptable Computer Models

There is a wide variety of both public and private domain computer models available for performing stormwater calculations. The computer models use one or more calculation methodologies to estimate runoff characteristics. Below is a list of few widely used public domain models that use NRCS CN method (Table 14). Once a modified curve number is calculated from R_v coefficients, it can be used in one of the listed models to generate peak discharge.

Table 9.2 – List of acceptable public domain computer models

Program	Developer
HEC-1	U.S. Army Corps of Engineers
HEC-HMS	U.S. Army Corps of Engineers
SWMM	U.S. Environmental Protection Agency
WinTR-20	U.S. Department of Agriculture Natural Resources Conservation Service
WinTR-55	U.S. Department of Agriculture Natural Resources Conservation Service

9.3 Post-Construction BMP Selection

After determining the hydrology changes due to development as explained in Section 9.2 above, the designer should select BMPs that adequately manage the additional runoff volume and greater peak flow while accounting for constraints, such as available space and material costs.

The effectiveness of a given BMP is a function of the dimensions and other design specifications of the BMP as well as other variables, which may include the BMP's location on the project site, the rate at which the BMP receives runoff water, the rate at which water is discharged from the BMP, and subsurface soil characteristics. The safety of motorists and other citizens, the hydrologic sensitivity of a receiving water, and aesthetics also must be considered in the selection and placement of a BMP. Low-Impact Development (LID) and Green Infrastructure (GI) BMPs that utilize natural materials and processes (e.g., infiltration swale) should be considered whenever possible, in deference to the policy stated in Section 9.2, but non-LID/GI BMPs (e.g., detention pond) may be employed as warranted.

The designer may find it advantageous to coordinate with the Design Bureau Stormwater Section during post-construction BMP selection.

R9 Chapter References

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Chapter 10: Stream & Wetland Restoration Concepts

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10.1 Introduction

This chapter will discuss the relocation of streams as well as the restoration concepts related to streams and wetlands.

The designer should consult other chapters in this manual, as appropriate, for supporting information related to stream and wetland restoration concepts. For example, Chapter 4 presents general hydrology and hydraulic concepts, while Chapter 5 introduces stream topics such as: stream morphology, cross sections, Manning's n values, calibration, one-dimensional gradually varied flow profile analysis, and a few special analysis techniques. Following are the main topics presented in this chapter:

- Permitting requirements for stream and wetland restoration projects
- Natural stream studies and assessments of existing stream channels
- Guidance on stream restoration projects
- Guidance on wetland restoration projects

This chapter is not intended to be an all-encompassing guidance document on stream design or relocation, stream restoration, or wetland restoration. These types of design elements are part of a specialized field which requires an experienced designer. This chapter will present an overview of typical stream relocations and restoration concepts followed by an overview of wetland restoration design. It is recommended, however, that the designer consult outside references, as well as the various references cited throughout chapters 5 and 10, for actual stream relocation and wetland design procedures.

10.2 Permitting

Stream and/or wetland mitigation is often an applicable requirement under the CWA, Section 404, as administered by USACE, and Section 401, as administered by ADEM. USACE may require mitigation for the loss of streams and/or wetlands that occurs when highways and other facilities are constructed. Compensatory Mitigation for Losses of Aquatic Resources, (Federal Register 2008) issued by USACE and USEPA, defines the three compensation mechanisms that are used to mitigate for the loss of wetlands: permittee-responsible compensatory mitigation, mitigation banks, and in-lieu fee mitigation. Each type must have mitigation plans which include the same 12 fundamental components:

- Objectives
- Site selection criteria
- Baseline information (for impact and compensation sites)
- Credit determination methodology
- Mitigation work plan

- Maintenance plan
- Ecological performance standards
- Monitoring requirements
- Long-term management plan
- Adaptive management plan
- Financial assurances
- Site protection instruments (e.g., conservation easements)

Purchasing mitigation bank credits and permittee-responsible mitigation are the two mitigation methods available in Alabama. This chapter is intended to provide a general guideline on the subject and may also be useful when temporary impacts to streams and/or wetlands have been permitted and impacted areas must be returned to existing conditions prior to project completion.

Regulatory Agency Consultation and Permitting

Prior to the initiation of any activity within a stream or wetland, or the design of a mitigation plan, consultation with the appropriate regulatory agency must be conducted and appropriate permits, if any, need to be obtained. A list of the most commonly required permits/approvals and their appropriate regulating authority is provided below.

- Section 404 Permit – USACE
- Section 10 Permit – USACE
- Section 401 Certification (Required before 404) – ADEM

10.3 Stream Design and Restoration

10.3.1 Introduction

The general goal of stream design and restoration is to promote the use of ecological processes (physical, chemical, and biological) and minimally intrusive solutions to restore self-sustaining stream corridor functions. By developing and selecting appropriate alternatives and solutions, and making informed management decisions, a stream design and restoration plan can be generated. Designers may choose to reference one of the following technical documents related to stream stability and restoration/rehabilitation approaches:

- *Stream Restoration: A Natural Channel Design Handbook* [(10-3)]
- Hydraulic Design Series No. 6 (HDS-6), *Highways in the River Environment* [(10-7)]
- Hydraulic Engineering Circular No. 20 (HEC-20), *Stream Stability at Highway Structures* [(10-5)]
- *National Engineering Handbook, Part 654 Stream Restoration Design* [(10-12)]

- *Applied River Morphology* (10-8)
- *Hydraulic Design of Stream Restoration Projects* (10-11)
- *Stream Corridor Restoration: Principles, Processes, and Practices* (10-9)

Whether a highway project involves restoration or rehabilitation activities, the complexities of the stream corridor system need to be considered.

10.3.2 Definitions

The following definitions are provided as they apply to stream systems and their intended meaning within this chapter.

Stream: (In this chapter, also referred to as "natural stream" and assumes a stream is located in an undeveloped watershed.) 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.

Ephemeral Stream: A stream that has flowing water only during, and for a short duration after, precipitation events during a typical year is an ephemeral stream. 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.

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.

Perennial Stream: A perennial stream has flowing water year-round during a typical year. 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.

Restoration: The process of repairing damage to the diversity and dynamics of ecosystems. Ecological restoration is the process of returning an ecosystem as closely as possible to pre-disturbance conditions and functions. Implicit in this definition is that ecosystems are naturally dynamic. It is therefore not possible to recreate a system exactly. The restoration process reestablishes the general structure, function, and dynamics of the stream, but sustains the behavior of the ecosystem.

Rehabilitation: Rehabilitation is making the land useful again after a disturbance. It involves the recovery of ecosystem functions and processes in a degraded habitat. Rehabilitation does not necessarily reestablish the pre-disturbance condition, but it does involve establishing geological and hydrologically stable landscapes that support the natural ecosystem mosaic.

10.3.3 Natural Stream Design

As a general practice, the designer should make an effort to minimize or avoid impacts to streams. However, if stream restoration and/or rehabilitation is warranted, there are a wide range of design approaches available, ranging from relatively simple methods based on stream classification systems, to complex two- and three- dimensional numerical models that analyze water and sediment discharge conditions (reference Chapter 5 for more detailed information and references on these models). Simpler methods, including those based on stream classification concepts, do not include adequate consideration of hydraulic and sediment transport issues.

Engineering analysis of the hydraulic and sediment transport conditions in a restoration project is important to the long term success of a stream. Many restoration schemes emphasize more "natural" solutions (e.g., timber structures) that may be stable under normal flow conditions, but under flood conditions, suffer widespread failure. For channels in a truly "natural" environment, such failures may be of little consequence. However, for channels adjacent to highways, and particularly channels located in urban areas where significant infrastructure is at risk, such failures are not acceptable. In these situations, an engineering-based analysis is necessary to address all important issues, including an appropriate evaluation of sediment transport conditions.

10.3.3.1 Intent of Natural Stream Design

The general intent of natural stream design for a relocated stream reach is to preserve the conditions which exist within the larger stream system. The relocated reach should attempt to match, as closely as possible, the existing stream in terms of the following:

- Stream Planform
- Stream Vertical Profile
- Habitat Features
- Existing Floodplains

Each of these existing stream features is discussed in more detail in the following sections.

Stream Planform

Preserving the length and sinuosity, two main factors of the stream planform, is important in natural stream design. Maintaining the length of the stream is integral to maintaining the flood routing characteristics and stream profile. In addition to the meander characteristics, the designer should attempt to duplicate the existing sinuosity ratio (Figure 10.1), if present. Sinuosity is influenced and determined by the region of the state, similar to the ecoregions shown in Figure 10.4. For example, stream channels tend to be more sinuous in the coastal plains region than in the piedmont region, where streams have a greater number of riffles and shoals. Additionally, the proposed design should be based on the relationship between sinuosity and vertical stream structure, since pools tend to form in the outside portions of bends, while riffles tend to form in the straight sections between them.

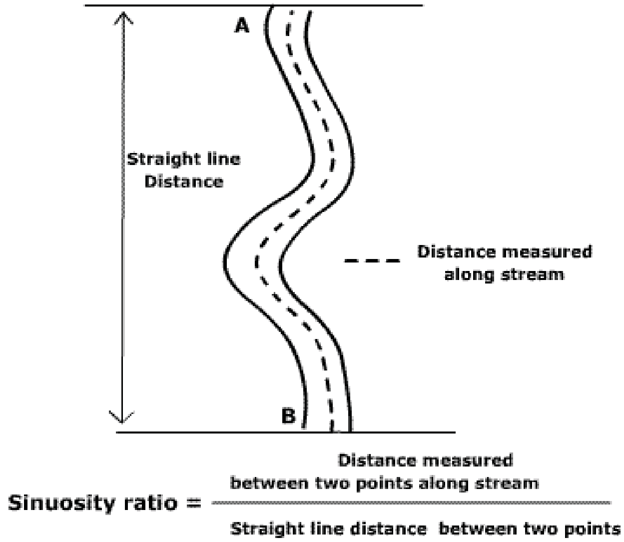

Figure 10.1- Sinuosity Ratio

Stream Vertical Profile

Vertical structures associated with natural channels are pools, riffles, runs, glides, and steps.

Pool and riffle structures connected by runs or glides are most often associated with alluvial streams on a sinuous alignment. As illustrated in Figures 10.2 and 10.3, the structure consists of a series of one or more deep pools interspersed with riffles composed of rock or gravel. These riffles and pools are connected by smooth, unbroken flow areas known as runs or glides. When an existing stream displays these types of vertical structures, the designer should examine the existing channel bottom profile and note the following:

- Length and depth of the pools
- Length and local slope of the riffles
- Gradient of the runs or glides

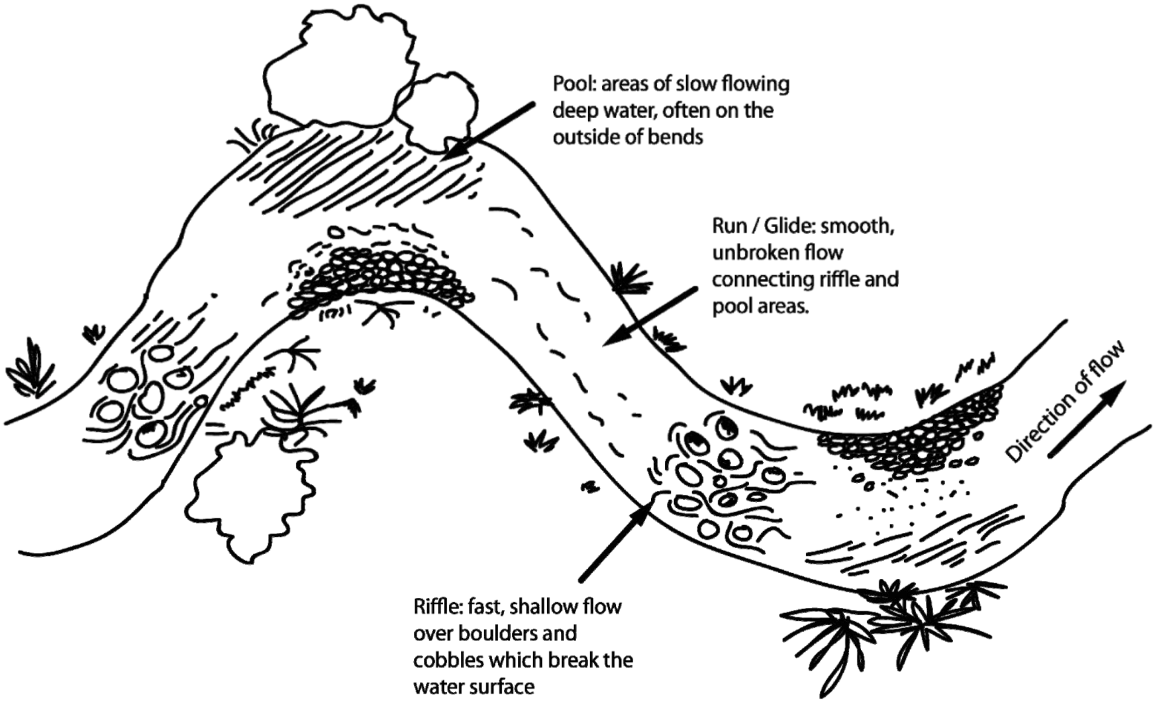

Figure 10.2 - The Stream Reach
Reference: West Virginia Department of Environmental Protection

Figure 10.3 - Stream Structure Example
Reference: Daphne, Alabama

Step structures are most often associated with steep natural threshold streams flowing through boulders or bedrock. They consist of a series of short comparatively flat reaches followed by steep drops. When an existing stream displays this type of vertical structure, the designer should note the following:

- Length and local slope of each step
- Drop height between steps
- Materials (bedrock, boulders, etc.) forming each step

The designer may also need to evaluate the overall floodplain (valley) slope in addition to the local channel slope. These two slopes can be different for streams with a high degree of sinuosity. The floodplain slope is necessary for determining flood elevations for large discharge events. On the other hand, the local channel slope is used to determine the channel forming discharge which is then used for a number of design parameters, including the selection and design of mitigation practices.

Existing Floodplains

A stable natural stream usually consists of a channel section that conveys low flows and overbanks, which will convey flows when the stream is at its bankfull elevation. This is typically a 1 to 2-year recurrence interval. Where this situation exists, the goal of the natural stream design for a relocated channel should be to maintain the existing stream cross section. While it is recognized that this may not be practical in all situations, this would include duplicating the existing top of bank elevations, as well as the floodplain widths. As a rule of thumb, the floodplain width is preferred to be five to ten times the width of the bankfull elevation width.

There may be a temptation by the designer to increase the size of the channel in order to decrease the required size of the floodplain. However, this is not recommended since it may lead to stability problems, especially for alluvial streams. In addition, there may be ecological impacts if the frequency of flooding on the overbanks is reduced.

As required by FEMA, the designer should check that the flood elevations in the proposed condition do not exceed the flood elevations in the existing condition for both the relocated reach and upstream of the project site. The designer should conduct hydraulic analyses for both the existing and proposed conditions to check flood elevations for both the design discharge and the 100-year discharge. These analyses should assume that floodplain conditions, including riparian vegetation, are the same in the proposed condition as they are in the existing condition.

10.3.3.2 Recommended Design Approach

The first step in a channel restoration project is to identify the problems observed in the reach of concern. The stream reconnaissance techniques and field checklists provided in FHWA's publication, HEC-20, [10-5] support a determination of the nature and extent of the observed problems. A rapid assessment methodology, such as that presented in HEC-20, Appendix D, [10-5] can help in evaluating the severity of the problem.

To determine the cause of the stream instability, a qualitative assessment of important geomorphic factors (reference HEC-20 [10-5] Chapter 2) can provide an initial indication, although a more detailed analysis which follows the Level 1 and Level 2 procedures will be required (reference HEC-20 [10-5] Chapter 3). Understanding land use change in the contributing watershed and its effects on the delivery (both timing and quantity) of water and sediment to the stream system is critical in identifying the complex interrelationships that are responsible for stream instability.

To develop a restoration solution for a degraded stream, it is often useful to review the existing stream system and a variety of stream channel classifications based on planform, bed form, bed materials, bank materials, sediment load, and hydraulic and geomorphic parameters to determine potential stream types consistent with watershed and valley features. In addition, a successful restoration project will require developing a stable form for the stream, considering the existing hydrologic and sediment regime. The designer must develop a stream that is stable laterally (in planform) and vertically (in profile).

The AASHTO publication, Highway Drainage Guidelines, [10-1] contains detailed guidelines for stream modification and mitigation practices, particularly regarding aquatic habitat and wetland functions. The AASHTO publication, Drainage Manual, [10-2] recommends a number of strategies to develop channel mitigation geometries when disturbance of a channel is determined to be unavoidable. The Drainage Manual suggests three alternatives, along with conceptual sketches, for maintaining a stream's functional value. These alternatives include: grade control structures, fish habitat structures, and bendway bank protection.

The ultimate test of restoration design is the ability of the reconfigured channel to achieve a state of dynamic equilibrium considering the size and volume of sediment delivered from upstream. The sediment continuity concept, presented in HEC-20, [10-5] can be used for a preliminary evaluation of stream system stability; however, a more detailed model may be required for large rivers or complex projects.

In terms of analytical complexity, an intermediate approach based on application geomorphology, channel forming discharge analysis, one-dimensional hydraulic analysis, and sediment transport calculations is provided in the United States Army Corps of Engineers (USACE) publication, Hydraulic Design of Stream Restoration Projects.[10-11] This comprehensive methodology recognizes that regardless of the goals of the rehabilitation project, the fundamentals of planning activities should be followed, including the following general steps:

- Preliminary planning to establish the scope, goals, preliminary objectives, and general approach for restoration.
- Baseline assessments and inventories of project location to assess the feasibility of preliminary objectives, to refine the approach to restoration, and to provide for the project design.
- Design restoration projects to reflect objectives and limitations inherent to the project location.
- Evaluate construction to identify, correct, or accommodate for inconsistencies with project design.
- Monitor parameters important for assessing goals and objectives of restoration.
- Based on these guidelines, a systematic approach to initiating, planning, analyzing, implementing, and monitoring stream restoration and rehabilitation projects can be developed.

10.4 Wetland Restoration/Mitigation

As noted in Section 10.2, purchasing mitigation bank credits and permittee-responsible mitigation are the two available methods for obtaining stream credits in Alabama. If the Department has indicated wetland restoration will be acceptable for the project, the designer should consider the following information. As stated in the beginning of this chapter, wetland restoration projects, often referred to as wetland mitigation, are a multi-disciplinary undertaking, requiring successful solutions to problems of hydrology, vegetation, soil, wildlife habitat, and pollutant/flood abatement in order to address Section 404 regulations. The lead role in design and implementation of wetland mitigation projects is usually a wetland specialist, who may employ the expertise of other specialists such as hydrologists, botanists, foresters, landscapers, construction engineers, soil scientists, and wildlife biologists. A description of each expert's role and various specific construction techniques required for a wetland mitigation project are beyond the scope of this chapter. The goal of this section, therefore, is to present highlights of the subject that will inform a hydraulics professional on aspects that should be given consideration. For further information on wetland design, the reader is referred to the following list of references:

- Planning Hydrology for Constructed Wetlands (10-6)
- Wetland Delineation Manual (10-10)
- HDS-2 Highway Hydrology – Chapter 9 (10-4)

Desired mitigation functions might include the following: special habitat for a targeted wildlife species, flood storage within a flood prone watershed, sediment and stormwater pollutant trapping within an impaired watershed, protection from erosion in areas affected by tides and currents, groundwater recharge or discharge areas, and/or recreational and educational values. An interdisciplinary approach to wetland design provides for the development of desirable functional goals and success criteria that meet regulatory standards.

Considerations for the following wetland parameters should be made by the interdisciplinary design team:

- Site selection (based on USACE district requirements)
- Wetland types (replacement in kind)
- Suitable soils (developing hydric soils)
- Vegetation (consider wetland type and ecoregion of the state, Figure 10.4)
- Hydrology
- Water balance (Water Budget)
- Water control structures (to allow variable depths)
- Construction constraints (site access, seasonal construction period, adherence to plans)

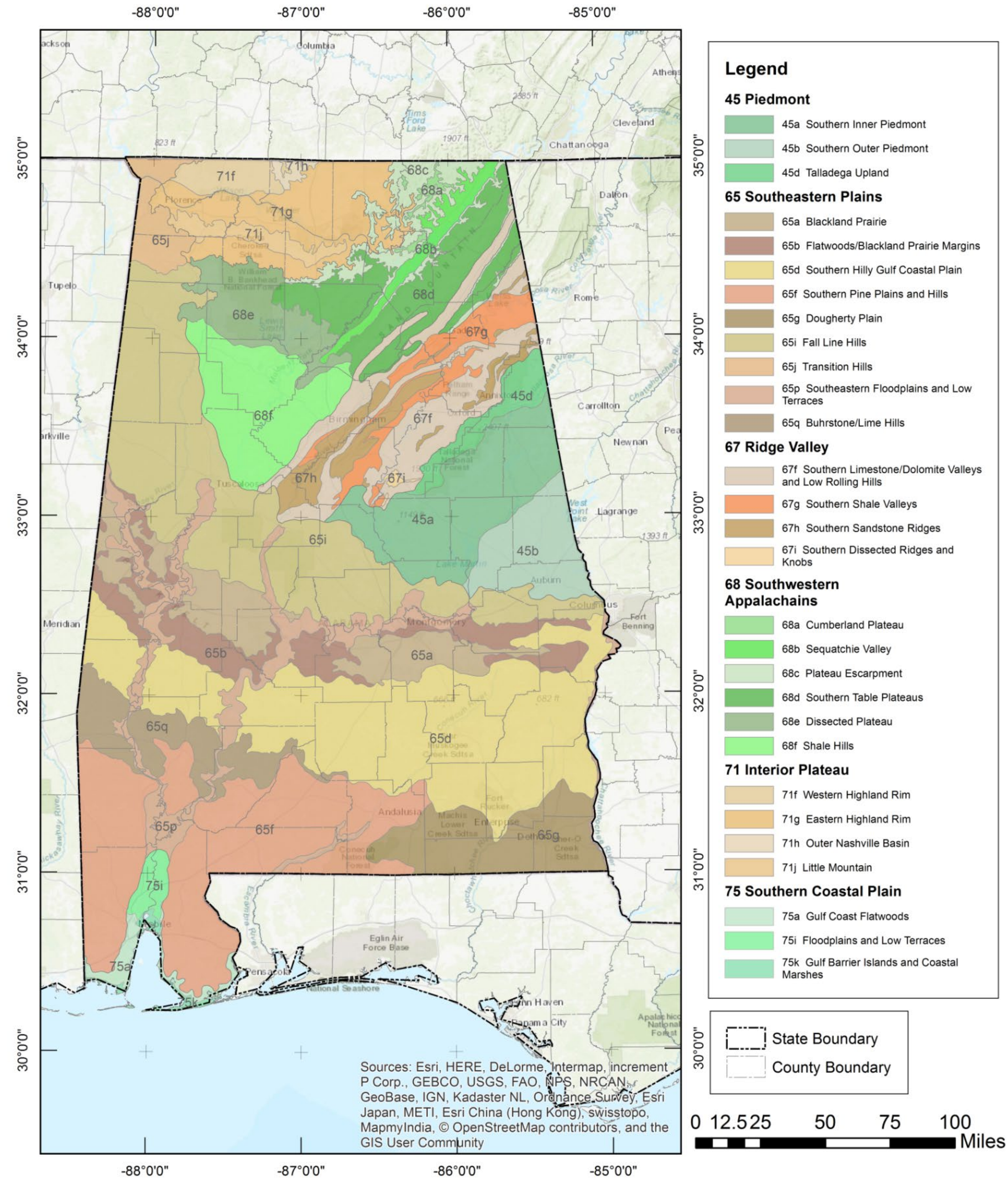

Figure 10.4 - Level III and IV Ecoregions of Alabama Reference: EPA

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Chapter 11: Requirements for Hydraulic Design Studies

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List of Tables

Hydrologic and Hydraulic (H&H) Requirements for Bridges and Selected Bridge Culvert Sites

11.1 Design Criteria

All drainage structures will be designed to minimize flood hazards to pass flood flows across the right-of-way with due consideration given to the risk to the facility, to structures in the floodplain affected by the facility, to the traveling public, and to environmental impacts. Floodplains are the low areas bordering a stream that are subject to inundation by floods and the term is used in this chapter generally and to refer to specific flood boundaries such as the 100-year floodplain. This chapter provides hydraulic design criteria for all existing and/or proposed river and tidal bridge sites and for culverts that meet any of the following conditions:

- Existing or proposed bridge culverts that have a total span length along the roadway centerline of 20 feet or more (length including supports, undercopings of abutments, spring lines of arches, or extreme ends of openings for multiple boxes).
- All sites located on streams where the 100-year floodplain has been delineated on FEMA maps
- All sites located on streams that are named on county and/or USGS maps
- All sites that have a significant risk associated with the project such as existing or potential flooding problems
- All sites that are affected by downstream constrictions, obstructions, or abnormal flood stages (backwater) from another stream

Study requirements for these major culverts are provided in Section 11.3.3.

11.1.1 Design Frequencies and Freeboards

Note: Unless otherwise specified, freeboard refers to the vertical clearance between the bridge superstructure at its lowest point, and the flood stage elevation. Freeboard requirements will not apply to bridge culverts. However, bridge culverts will be subjected to allowable headwater requirements as outlined in Chapter 8, Section 8.2.3 of this manual.

Riverine Bridge Replacements / New Locations

All bridges will be sized to convey the design flood without causing significant damage to the highway, the stream, or other property. The design flood will be conveyed only through the bridge opening(s), while larger floods may be conveyed over the roadway and through the bridge opening(s).

1. Interstate
    a. The design flood is the 50-year frequency flood discharge.
    b. A minimum of 2 feet of freeboard above the design flood stage is required.
2. Roads Designated as State Routes
    a. The design flood is the 50-year frequency flood discharge.
    b. A minimum of 2 feet of freeboard above the design flood stage is required.
3. Roads Not Designated as State Routes (local roads)
    a. The design flood will be based on average daily traffic (ADT) as follows:

Design Traffic (ADT[1]) Frequency	Minimum Design Flood
1 - 99	1.5-25 Year[2]
100 – 399	10-25 Year[2]
400 –	25 Year

[1] Average daily traffic – projected 20-year volume.
[2] Design flood should be commensurate with the type road and risk the county/municipality desires.

    b. A minimum of 2 feet of freeboard above the design flood stage is required for roads having an ADT of 400 vehicles or greater.
4. Special Conditions for Freeboard Waivers

For the above-mentioned highways, there can be special circumstances in which raising the road grade would cause an increase in the upstream water surface profile in the vicinity of the highway project. Both the impact on nearby residents/development and/or FEMA restrictions may necessitate the need to maintain the existing roadway profile grade. In these circumstances, engineers may apply for a waiver to decrease or even eliminate freeboard for the new bridge construction. This design flood should be approved by the State Bridge Engineer with concurrence by the local city or county government. If the site has an ADT of under 400 vehicles, no waiver is needed.

5. Additional Design Frequency And Freeboard Considerations
    a. The design flood may require the roadway to be overtopped with interruptions to traffic due to a low roadway profile. In this circumstance, the design flood can have a smaller recurrence interval. This design flood should be approved by the State Chief Engineer with concurrence by the local city or county government.

b. Engineering judgment may be used for bridge sites where flood stages are affected by backwater from another stream. An example may be where a bridge crossing is located just upstream from the stream's confluence with a larger river system. Case in point, a facility's finished grade may be designed using the higher flood stage caused by backwater from the larger river downstream with the minimum freeboard and the bridge opening designed for headwater flooding (without the backwater) for velocity or, if the facility is on the secondary system then it may be designed without considering the backwater from the larger stream.

c. If the bridge is over a major lake or reservoir where there is boat traffic, the desirable grade should be set so that there is at least 8 feet of freeboard above the maximum operating pool. 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.

d. If debris is a problem at the site, the above-required minimum clearances may be increased with the concurrence of the State Bridge Engineer.

e. 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.

Note: See Guidelines for Operations (GFO 3-39) for additional information concerning design frequencies for bridge openings and scour evaluations.

Widened and Parallel Bridges

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

2. The guidelines for widened bridges are outlined below:

   a. It is desirable for widened bridges to follow the design criteria for bridge replacements.

   b. At a minimum, the bottom elevation of the widened superstructure should usually approximate the existing bottom of superstructure elevation, thereby not reducing the existing area of bridge opening. This minimum design is only considered if no scour or flooding problems exist and the potential for any significant problems seems low.

11.1.2 Discharge Determination

1. For rural drainage basins, use USGS publication, *Magnitude and Frequency of Floods in Alabama, 2003*: U.S. Geological Survey Scientific Investigations Report 2007–5204[11-6] to determine the various flood-peak discharges for the project site. The regional flood frequency relations and applicable gauge data (if

available) should be used to obtain an improved estimate as outlined in this publication. Updated gauge information can also be obtained at http://waterdata.usgs.gov/al/nwis/rt

2. For urban drainage basins, use USGS publication, *Magnitude and Frequency of Floods for Urban Streams in Alabama, 2007*: U.S. Geological Survey Scientific Investigations Report 2010–5012[11-7] to determine the various flood-peak discharges at the project site.

3. For small streams in rural drainage basins, use USGS publication, *Magnitude and Frequency of Floods on Small Rural Streams in Alabama*: U.S. Geological Survey Scientific Investigations Report 2004–5135[11-8]. These equations are especially recommended for rural streams having a drainage area of less than five square miles.

11.1.3 Flood Stages

1. When a USGS gauge is located at or near the bridge site of interest, recorded peak flows and associated flood stages should be used to calibrate a hydraulic model. This data can be obtained from either of the USGS offices in Montgomery or Tuscaloosa or at http://al.water.usgs.gov/. In addition, if in the engineer's judgment, reliable high water information at or near the site is available and the flood frequency of the applicable flood can be determined, the hydraulic model can be calibrated using this information.

2. For sites where reliable flood stage information is not available, the applicable hydraulic model should be used to determine the various flood stages at the project site. Additionally, historical high-water information (flood marks) obtained from field site investigations, local residents, old highway plans, etc. should be used as guidance for such hydraulic models.

11.1.4 Backwater

Backwater is measured relative to the natural water surface elevation without the effect of the bridge at the approach cross section.

Note: The U.S. Army Corps of Engineers Hydrologic Engineering Center has conducted research concerning the location of the approach and exit sections in the hydraulic computer model. The conclusions and recommendations from this study are contained in the HEC-RAS Hydraulic Reference Manual,[11-13] Appendix B, *Flow Transitions in Bridge Backwater Analysis*, and should be used in determining the locations of the approach and exit sections in the HEC-RAS or WSPRO computer models.

1. The 100-year backwater should be limited to 1 foot above the unconstricted or natural 100-year water surface profile. This should be true for all sites unless there is a stricter guideline in place for the site. See Guidelines for Operation (GFO 3-60) for detailed guidance concerning compliance with FEMA flood plain regulations when backwater limitations are exceeded.

Note: This backwater value should include effects from the proposed roadway in the case of a longitudinal encroachment on the floodplain.

2. The engineer may determine that the above limitation in Requirement 1 is not practical for bridge replacement projects. In this case, the 100-year backwater elevation may exceed 1 foot above the unconstricted or natural 100-year water surface profile, but it may not be higher than the existing condition backwater value.

   Note: 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 for the above limitation in Requirement 1. The waiver of the above limitation in Requirement 1 necessitates the approval of the Department.

   Note: Example conditions where the limitation in Section 11.1.4 Requirement 1 would be waived are as follows:

   > *Due to shallow flow in the overbank area where additional span lengths and/or overflow structures do not significantly reduce the velocity and backwater values; and where the existing structure creates a significant amount of backwater and storage upstream of the roadway and sizing the proposed structure to meet Requirement 1 would adversely affect downstream development (and result in a bridge design that is not cost effective).*

   Justification for the waiving of 11.1.4 Requirement 1 should be clearly shown in the hydrologic and hydraulic study. In all cases, the drainage structure should be sized so that the drainage structure and roadway are protected against failure during design flood events for backwater, velocity, and scour.

3. 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. The guidelines contained in Section 11.3.1, paragraph 8.b, Widened and Parallel Bridges, are recommended to be followed to minimize increases in backwater due to the proposed construction.

4. In addition to the above limitations, bridges located within areas covered by FEMA studies will be sized to satisfy FEMA requirements. See Chapter 2 of this manual, Agency Coordination and Regulations.

5. Future development, current conditions, and past historical flooding conditions in the upstream and downstream floodplains should be considered for all cases.

11.1.5 Flow Velocities

Flow velocities within the bridge opening should be limited to minimize scour in the overbank portion of the opening. Acceptable 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. Box culverts should be sized with acceptable flow velocities to minimize potential scour.

Note: As a general rule to minimize scour and backwater, the mean velocity values for a bridge opening should not exceed 4.5 feet per second (for design year flood event) unless site conditions and engineering judgment dictate otherwise. Excavation of bridge opening to improve velocities will require a waiver if it goes below natural ground. Bridge culverts should have a targeted mean velocity of around 5.5 feet per second at the culvert outlet. In the event that a substantial amount of rock is present in the channel bed, the targeted mean velocity for both bridges and culverts can be significantly higher.

11.1.6 Bridge Scour

A scour analysis should be performed for all bridges using the methods in the latest version of the FHWA HEC-18, [11-2] *Evaluating Scour at Bridges*. General contraction and local (pier) scour calculations should also be performed. The design flood for Interstate Highways is the 100-year flood unless overtopped by a smaller flood. Scour for these sites should also be computed for the 500-year flood unless overtopped by a smaller flood. The design flood for State and U.S. Highways is the 100-year flood unless overtopped by a smaller flood. Scour for these sites should also be computed for the 200-year flood unless overtopped by a smaller flood. The design flood for scour for local roads is the 50-year flood or the overtopping flood if it is less than or equal to the 50-year flood. Scour should also be computed for the 100-year flood for local roads unless overtopped by a smaller flood. In the Black Prairie Region of the state, techniques and methods outlined in USGS publication *Clear-Water Contraction Scour at Selected Bridge Sites in the Black Prairie Belt of the Coastal Plain in Alabama, 2006*: U.S. Geological Survey Scientific Investigations Report 2007–5260[11-9] may be used to provide guidance in determining estimated scour values.

11.1.7 Bridge Abutment Protection

Spill-through type abutments with a 2:1 slope normal to the end bent are used for new bridges. The bridge end(s) will be located such that the toe of the spill-through slope(s) is setback at least 10 feet from any point along the channel bank(s). Greater setbacks may be dictated by other hydraulic factors and needs. Riprap protection for these abutments should be sized using the method shown in the latest version of the FHWA HEC-23,[11-10] *Bridge Scour and Stream Instability Countermeasures*. The 100-year flood should be used for this design. This riprap protection should be entrenched 2 feet below the natural ground line. The riprap protection should be extended a minimum distance of 20 feet behind the end of the abutments (see ALDOT Special Drawing RR-610). A riprap apron with a width equal to twice the 100-year flood flow depth in the overbank area (8-foot minimum to 25-foot maximum) should be used to protect the abutment toes. The riprap apron should not extend beyond the top of the channel bank. The riprap depth should be a minimum of two feet beneath the natural groundline at the abutments. Suitable geotextile is required under the riprap. The five classes of approved riprap are presented in Table 11.1.

Table 11.1 ALDOT Approved Riprap Classes

<table>
<tr><th rowspan="2">Weight Range (LBS)</th><th rowspan="2">Class</th><th rowspan="2">“n” Value</th><th colspan="2">D50 Size (Mean Stone Size)</th><th colspan="2">Maximum Size</th><th rowspan="2">Minimum Thickness of Layer (Feet)</th><th rowspan="2">Maximum Velocity (Ft/Sec)</th><th rowspan="2">Exact LBS/CY[9]</th><th rowspan="2">Estimated Used LBS/CY[10]</th></tr>
<tr><th>Weight (LBS)</th><th>Diameter (Feet)</th><th>Weight (LBS)</th><th>Diameter (Feet)</th></tr>
<tr><td>10-100</td><td>1</td><td>0.0381</td><td>50</td><td>0.8</td><td>100</td><td>1.1</td><td>1.7</td><td>8</td><td rowspan="3">3200</td><td rowspan="5">3800</td></tr>
<tr><td>10-200</td><td>2</td><td>0.0395</td><td>80</td><td>1.0</td><td>200</td><td>1.3</td><td>2.0</td><td>9</td></tr>
<tr><td>25-500</td><td>3</td><td>0.0413</td><td>200</td><td>1.3</td><td>500</td><td>1.8</td><td>2.7</td><td>10</td></tr>
<tr><td>50-1000</td><td>4</td><td>0.0436</td><td>500</td><td>1.8</td><td>1000</td><td>2.3</td><td>3.5</td><td>12</td><td rowspan="2">3402</td></tr>
<tr><td>2000-[11]</td><td>5</td><td>0.0454</td><td>1000</td><td>2.3</td><td>2000</td><td>2.8</td><td>4.2</td><td>14</td></tr>
<tr><td colspan="11">165 LBS/CF = 4455 LBS/CY = Solid Rock</td></tr>
</table>

[9] From Materials & Tests “Aggregate Unit Masses”.
[10] We normally use 3800 LBS/CY for estimating all classes of riprap.
[11] ALDOT Standard Specifications Section 814 states 2000 LBS and down
[12] For velocities that exceed 14 feet/second, consider using an energy dissipator

11.1.8 Guide Banks

Guide bank calculations should be performed as shown in the latest version of the FHWA HEC-23, (11-10) *Bridge Scour and Stream Instability Countermeasures,* and should be based on the 100-year flood. Guide banks are not required to be built where the calculated length is less than 100 feet. Based on FHWA practice, the Department recommends a maximum length guide bank of 150 feet be built where the calculated length exceeds150 feet. ALDOT’s “Special and Standard Highway Drawings” include a 100 foot, 125 foot, and 150 foot design option.

Note: 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.

11.1.9 Detour Structures

Where detour structures are required, these structures need to be sized to maintain traffic during the new construction. The detour structure may be a bridge, extension of a proposed culvert, or metal pipes. In certain cases, traffic can also be maintained by staged construction of the proposed bridge. Since the detour structure is usually sized to convey a smaller flood than the adjacent highway bridge, the detour structure should be placed downstream of the roadway bridge unless conditions warrant otherwise. These conditions include, but are not limited to, adverse downstream channel geometry, conflicts with utilities, conflicts with houses, buildings or other structures, and wetland or other environmental issues. It is assumed that the detour bridge will be centered about the channel and/or aligned with the existing bridge opening. The detour structure will be sized on a reduced design flood (typically 2-year flood, but possibly larger flood event) to be determined on a case-by case basis by local personnel.

11.1.10 Longitudinal Roadway Encroachments

Since longitudinal encroachments into the base floodplain (100-year floodplain) and floodway by new and widened roadways can 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 widened and parallel roadways, and new locations:

1. For roadway widening projects, the typical section should be set to avoid or minimize the placement of additional roadway fill within the adjacent base floodplain.

2. For parallel roadway projects, the new roadway should be placed to avoid or minimize longitudinal encroachments on the base floodplains.

3. New location 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.

11.1.11 Hydraulic Modeling Floodplain Constrictions/Obstructions and Abnormal Flood Stage Conditions

Effects from natural or man-made conditions may affect the flood stages at the crossing site. These effects should be taken into account when modeling and analyzing the hydraulic conditions. The hydraulic engineer should identify and include any of these conditions in the hydraulic model. Following are some examples:

1. Roadway and railroad stream crossings

2. Longitudinal roadway encroachments (see Section 11.1.10 of this manual)

3. Natural narrowing of the floodplain

4. Fill that has been placed within the floodplain

5. Reservoirs, dams, and levee structures

6. Buildings and other ineffective or blocked flow areas

7. Confluence with another stream

All the above items should be taken into account when modeling such streams.

Normal Water Surface Profile Run. This computer run includes any floodplain constriction or obstruction that controls or affects the flood stages at the project site with the normal flood flows along the stream reach. This computer run is the basic run in all hydraulic studies.

Abnormal Flood Stage Run. This computer run includes any backwater effects from a natural or man-made condition that causes flood stages at the project crossing that are not due only to the normal flood flows along the stream reach. For example, an abnormal flood stage may result when the studied stream is a tributary to another river or stream, and the flood flows along this river or stream cause flood stage elevations to rise at the project site.

A reservoir that affects the flood stages at the project site can be considered a normal or abnormal flood stage condition depending on the situation. If the flood stages at the project site are controlled at all times by the reservoir, this is considered a normal flood condition. If the flood stages at the project site are only controlled a portion of the time by the reservoir, then this would be considered an abnormal flood stage condition.

If the proposed bridge site is affected by abnormal flood stages that result in higher flood stages and lower flood flow velocities than a normal flood condition, the bridge is to be designed to provide the required freeboard above the abnormal flood stage elevations. In addition, the bridge is also to be designed for the higher flood flow velocities that occur without the effects of the abnormal flood stages.

11.2 Design Data Required

11.2.1 Required Data from Project Manager

1. Three sets of preliminary proposed roadway plans. These plans are to include, but are not limited to, the following information:

    a. A cover/title sheet with the project number, PE number, route number, traffic data, and location map.

    b. Typical sections of bridges and roadways.

    c. Plan and profile sheets should depict the highway and floodplain limits. The scale should either be 1 in. = 50 ft. horizontal, 1 in. = 5 ft. vertical, or 1 in. = 100 ft. horizontal, 1 in. = 10 ft. vertical. A larger sheet (an extended roll) may be used if required for a wide floodplain. The plan and profile sheets should include the following information:

        1) Existing and proposed profile grade data with vertical curve data complete with point of vertical intersection (PVI) stations, elevations, grades and vertical curve lengths.

        2) Bearing along tangent section of the construction centerline.

        3) Horizontal curve data complete with point of intersection (PI) station and maximum super-elevation rate.

4) Transition stations from normal crown section to full super-elevation section.

5) Location of existing bridge(s) and roadway; begin and end bridge stations, including location and size of any relief structures.

6) Benchmark information; location of benchmarks in stations and offsets; physical description of benchmarks; benchmark elevation; benchmark datum. Unless otherwise specified by the State, the designer will use NAD83 (2011) datum for horizontal control and NAVD88 (based upon latest Geoid) for vertical control. Since these surveys originate and terminate at points with datum adjusted Alabama State Plane Coordinates, all computed coordinates will be datum adjusted NAD83(2011) Alabama State Plane Coordinates, U.S. Survey Foot, East or West Zone. No further datum adjustment is required.

7) Benchmarks should be set no further than 1,000 ft if possible along the survey and near all major structure sites and major intersections. All benchmarks will be permanent in nature and are to be fully described (from Survey Requirements Design Bureau – Location Section version 2.0).

8) Plot of stream traverse on plan sheet.

9) Plan view should show location of the downstream floodplain profile (distance downstream, angle(s), stations, etc. as related to roadway alignment). All topography including top of the stream's banks, scour holes, etc.

10) Profile view should show plot of cross-sections (downstream floodplain, existing bridge opening (including low steel), and three line profile for proposed alignment). Profile view should also include a streambed profile for sites having a drainage area of less than 30 square miles.

    Note: The downstream floodplain section should be taken far enough downstream to ensure the cross-section is on natural ground (not in side ditches or on roadway embankment or in scour holes). This section should run from high ground to high ground and should define the channel and all other abrupt breaks. This section should be plotted on the plan and profile sheet and if possible using the roadway stationing. As far as high ground is concerned, the ends of the section should be at least ten (10) feet above the channel bank elevations. This is a rule-of-thumb and may or may not apply at every site. If the floodplain profile is not within close proximity of the bridge, a profile of natural ground just beyond (downstream) the road side ditches is required or cross sections of the road in the vicinity of the bridge. Additionally, the streambed profile should be taken at least 500 feet upstream and 500 feet downstream of the proposed structure.

2. Quad map showing location of the stream especially for projects on new location. Projects on new location should have the alignment accurately plotted on the quad map. This diminishes errors that have been associated with the location maps typically used on the title sheet.

3. A minimum of five color photos of the site showing the upstream channel, downstream channel, downstream floodplain (left & right overbank), and existing bridge. These photos help in the estimation of the roughness coefficients used in the hydraulic model and in the documentation of the project.

   Note: Data should be submitted in hard or paper format along with the corresponding computer program files either by CD or electronically.

4. A copy of the HYD-100, 101, 102 and 103 forms. For a template copy go to Appendix F.

11.2.2 Reference Publications for Design Guidance

1. FHWA HEC-18,[11-2] Evaluating Scour at Bridges

2. FHWA HEC-20,[11-11] Stream Stability at Highway Structures

3. FHWA HEC-23,[11-10] Bridge Scour and Stream Instability Countermeasures

4. FHWA HEC-25,[11-5] Highways in the Coastal Environment

5. USGS Scientific Investigations Report 2007–5204, Magnitude and Frequency of Floods in Alabama, 2003[11-6]

6. USGS Scientific Investigations Report 2010-5012, Magnitude and Frequency of Floods for Urban Streams in Alabama, 2007[11-7]

7. USGS Scientific Investigations Report 2004-5135, Magnitude and Frequency of Floods on Small Rural Streams in Alabama[11-8]

8. USGS Scientific Investigations Report 2007-5260, Clear-Water Contraction Scour at Selected Bridge Sites in the Black Prairie Belt of the Coastal Plain in Alabama, 2006[11-9]

9. USGS Water-Resources Data Alabama Water Year

10. The user manuals for the respective computer models

11. FEMA Flood Insurance Studies

12. FHWA Hydraulics of Bridge Waterways [11-4]

13. U.S. Army Corps of Engineers, Shore Protection Manual [11-12]

11.2.3 Maps

1. USGS contour maps
2. County maps
3. Bathymetric maps

11.2.4 Other Plans, Reports, and Miscellaneous Data

1. The existing bridge and roadway plans
2. The bridge maintenance file for the existing structure
3. Previous hydraulic studies done by the Department, U.S. Army Corps of Engineers, FEMA and the USGS
4. Aerial photos

11.2.5 Regulations and Design Guides

1. The proposed bridge widths are to be determined using roadway widths including graded shoulders from AASHTO's *A Policy on Geometric Design of Highways and Streets*, Latest Version.
2. Federal-Aid Policy Guide, NS 23 CFR 650A, Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency (FEMA). See Chapter 2 of this manual.

11.2.6 Department Acceptable Computer Models

1. HEC-RAS (USACE)
2. WSPRO (FHWA)
3. FESWMS (FHWA)
4. HY-8 (FHWA)
5. SRH2D (USBR)

11.2.7 The Internet

Note: The Internet is an important tool that should be used for gathering information that previously was available only through publications or various agencies. The USGS, FEMA, the USACE, NOAA, TVA and the FHWA are among the agencies that have internet web sites. Internet sites, in many cases, will have the most updated information that can be used in the performance of hydrologic and hydraulic studies.

Information, data, and publications from the above agencies may be found at the following websites:

1. USGS: http://pubs.er.usgs.gov/
2. FEMA: https://www.fema.gov/flood-insurance/outreach-resources/publications
3. USACE: http://www.publications.usace.army.mil
4. NOAA: https://library.noaa.gov/Research-Tools/E-Resources/NOAA-Publications
5. TVA: https://www.tva.com/
6. FHWA: http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm?archived=true

11.3 Design Methods/Procedures – Hydrologic and Hydraulic (H&H) Studies

11.3.1 Methods/Procedures – All Riverine Bridge Projects

Note: The following methods/procedures are for bridge requirements, new locations, and both widened and parallel bridges unless otherwise noted.

1. The following hydraulic computer models are approved by the Department to be used when tidal flow is not present:

    a. USACE computer model HEC-RAS. The WSPRO bridge routine is the preferred option for bridge hydraulic analyses. One of the other bridge options may be more appropriate for specific site conditions and can be used. The HEC-RAS Hydraulic Reference Manual provides guidance on selecting a bridge modeling approach for specific site conditions.

    b. FHWA computer model WSPRO.

    c. The Finite Element Surface Water Modeling System (FESWMS) two-dimensional computer model. This model can be used in cases where there is a large amount of two-dimensional flow and the hydraulic engineer considers the WSPRO and/or HEC-RAS computer models to be inadequate for the conditions. Cases where this program can be used include a skewed crossing of a wide floodplain, a wide floodplain requiring multiple bridges, very wide floodplains, or if there is significant lateral flow in the vicinity of the bridge (such as close proximity to a meander bend, or a stream junction immediately upstream).

    d. Sedimentation and River Hydraulics (SRH2D) computer model. This two-dimensional computer model can be used in lieu of FESWMS for the floodplain conditions listed in item (c) above.

e. For bridge sites with a drainage area of 10 square miles or less, a box culvert alternate may be considered. Two culvert computer models are accepted: (1) the FHWA HY-8 computer model for box culverts is to be used in conjunction with the results from the WSPRO computer model; and (2) the HEC-RAS computer model.

f. For regulatory FEMA hydraulic models produced from the USACE software HEC-2, HEC-RAS may be used to duplicate the current regulatory FIS hydraulic model from HEC-2 to produce the floodway and profile runs.

Note: The HEC-RAS computer model with the WSPRO bridge routine, or the WSPRO computer model is to be used for the bridge hydraulic analysis unless special floodplain conditions exist which warrant the use of other bridge routines within HEC-RAS or other computer models. Computer models, other than those listed above, may be considered for special floodplain conditions.

2. Investigate the flood history of the stream. Sources for this information include, but are not limited to the following:

    a. USGS gauge records

    b. Existing bridge and maintenance files

    c. Previous studies done by ALDOT, USACE, FEMA, and the USGS

    d. Information from local residents

    e. Information from the local government

    f. Information from local Department personnel

3. Investigate the bridge site scour history. The following are some sources of information:

    a. Bridge inspection and maintenance files.

    b. Comparison of the original bridge plan and profile with the currently surveyed profile.

    c. Aerial photographs taken over as long a time span as available. Based on this information, an indication of the long-term channel stability and aggradation or degradation can be estimated. An evaluation of the performance of the existing bridges can also be made.

4. Determine the project site hydrology for the bridge.

    a. Use USGS topographical data (maps) or GIS spatial data to determine the drainage basin area for the project site. Determine the land usage from the most current aerial photography. A site visit will be required to confirm land use information. **Note:** USGS StreamStats can be used to perform a check for both basin drainage area and development attributes but should not be used in place of the previously stated methods/options.

b. Determine the discharges at the project site for the various flood frequencies. Refer to Section 4.1.1.1 for the appropriate method.

c. Estimate the average hydraulic slope at the site using USGS topographical, field survey, GIS spatial terrain data (i.e., LiDAR data), or an existing FEMA study.

d. Estimate Manning's n values for the channel and floodplain areas for the study reach. Manning's n values should be determined from the results of the site inspection and compared with the table values and photographs from the following publications:

    1) FHWA, Hydraulics of Bridge Waterways, March 1978 (11-4)
    2) USGS Water Supply Paper 1849, Roughness Characteristics of Natural Channels (11-3)
    3) USGS Water Supply Paper 2339, Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains (11-1)

5. Field Inspection of the project site.

    a. The hydraulic engineer performing the study and computer modeling should perform a site visit and inspection of the bridge site(s). For every hydraulic study it is strongly recommended that a thorough site inspection be conducted. Site inspections should include an evaluation of the survey depicted in plan/profile sheets in both floodplain and channel regions as well as in bridge openings. Geometric features depicted in these surveys should reasonably match what is seen in the field. The engineer should attempt to locate any evidence of high water in the vicinity of the bridge. High-water marks, lodged drift in bridge members, information from local residents, etc. are important data in determining the reliability of a hydraulic model. Site visits are the best resource when attempting to determine existing scour issues, drift issues, previous overtopping of embankments, and bent spacing and placement. Site visits also provide the best opportunity to observe nearby land cover and assess Manning's n for the hydraulic model.

    b. In addition, the following site conditions should be noted:

        1) Buildings or structures in the floodplain that may be subject to flooding
        2) Evidence of past channel migration or potential for future migration
        3) Channel bank stability both upstream and downstream from project site

4) Channel bed stability and consistency (slope changes, head cuts, etc.) throughout reach

c. During the field inspection, stream crossings immediately upstream or downstream of the project site on the same stream may be visited and the performance of the structures noted.

6. Determine the extent of survey data.

   a. The hydraulic engineer is to determine the extent of survey data required to accurately model the project site based on the requirements from the ALDOT Survey Requirements (latest version). Please refer to this document for the required survey information.

7. Hydraulic Analysis

   a. The hydraulic computer model is to be used to determine the natural, existing, and proposed flow conditions at the site. The 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals are to be modeled for the project site. The design flood and 100-year flood should be modeled for both existing and proposed structures for mean velocity and backwater estimations. The design flood is conveyed through the bridge opening, while floods greater than the design flood may be conveyed over the roadway and through the bridge opening.

   b. When a USGS gauge is located at or near the bridge site of interest, recorded peak flows and associated flood stages should be used to calibrate the hydraulic model. This data can be obtained from the USGS offices in Montgomery or Tuscaloosa or at http://al.water.usgs.gov/. If reliable high-water information at or near the site is available and the flood frequency of the applicable flood can be determined, the computer model should be calibrated using this information.

   c. If the drainage area is less than 10 square miles, a box culvert alternate may be considered. The natural or unconstricted high-water profiles should be developed using WSPRO or HEC- RAS. Two culvert computer models are acceptable: (1) the FHWA HY-8 computer model; and (2) the culvert routine within the HEC-RAS computer model.

   d. For projects with existing and/or proposed multiple bridges/culverts within the same floodplain, the WSPRO or HEC-RAS computer models can be used to size and analyze these drainage structures at crossings where two-dimensional computer models may not be necessary. The culvert analysis provided by WSPRO and HEC-RAS for these multiple drainage structure conditions are acceptable without running the HY-8 computer model.

   e. If the project is within an area covered by a FEMA study, FEMA guidelines should also be satisfied. See Chapter 2, *Agency Coordination and Regulations*.

f. When bridge modeling within HEC-RAS and analyzing low flow of a given crossing, calculate or run *momentum and energy* for low flow and check use highest energy answer.

g. For any effective model which has been geo-referenced, geo-reference all new cross-sections added to the model.

8. Hydraulic Design of Bridge

   a. Bridge Replacements/New Locations

      1) Establish the orientation of the bridge substructure by determining the flood flow angle. This should be based on topographic maps, aerial photographs, and the site inspection. If FESWMS is used, it will compute the velocity vectors, which will show the flood flow angle directly.

      2) Spill-through abutments with a 2:1 slope normal to the end bent are used for new bridges. The toe of the bridge abutment should be placed a minimum of 10 feet from the top of channel bank.

      3) In cases where the approaching channel bends before crossing under the bridge, the toes of the bridge abutments should be placed further away from the top of bank to avoid direct overbank flow from the channel, if practical.

      4) If the bridge is located in or near a channel bend, the possibility of channel migration increases. The toes of the bridge abutment should be placed far enough back so that any channel migration would not reach them during the lifetime of the bridge; 75 years is a minimum lifetime of the bridge. The rate and direction of channel migration can be predicted by comparing historic and recent aerial photography (HEC-20).

      5) The proposed bridge length should be set as the minimum length structure which has acceptable backwater and flow velocities as per the guidelines in Sections 11.1.4 and 11.1.5 of the design criteria section of this manual.

      6) The minimum bridge elevation should be set so that the proposed bridge superstructure will meet the clearance requirements specified in Section 11.1.1 of this manual while keeping the proposed profile as close to the existing profile as possible. The profile grade along the centerline of the proposed bridge should be set so that the bridge will drain surface flow. Avoid flat grades and the placement of the low point of a vertical curve on a bridge or approach slab.

7) The profile grade along the proposed roadway shall be set to meet the requirements as specified in Section 11.1.1.

8) When determining span lengths, the span over the channel should be set first. If practical, the channel should be completely spanned. The substructure should be offset far enough from the channel banks so that the banks will not be impacted during construction. For intermediate bents, this means that a minimum clearance of 10 feet should be maintained from the top of bank to the centerline of the bent.

   Note: the environmental document for the project should be checked to see if language is included conveying a commitment to span the channel.

9) Where intermediate bents must be located within the stream channel, they should be aligned with the channel flow. Tower bents should not be located within the channel or at the channel banks.

10) For ease of structural design and repetition in fabrication, the use of equal span lengths is recommended while following sound hydraulic design practices.

11) For bridge replacement projects, the existing bridge deck and substructure is removed as per the specifications. Any existing roadway fill within the proposed bridge opening is removed down to the original/natural groundline. If the new roadway and bridge is along a shifted or new alignment, the existing bridge and roadway fill is removed to natural groundline beyond the opening of the new bridge.

   Note: Exceptions to the above existing bridge and roadway removal will be made if the proposed bridge is along a new or shifted alignment and the existing bridge is declared historical; or the county wants to maintain the existing bridge and assume all liability for the structure. Even in these cases, the existing roadway fill may have to be totally or partially removed for hydraulic purposes.

12) For new parallel bridges, it is desirable to align the proposed abutments and intermediate bents. If conditions warrant, the span arrangement for each bridge can be varied to adhere to the recommendations listed in this section.

b. Widened and Parallel Bridges

   1) In general, the above recommendations in Section 11.3.1, paragraph 1.a for bridge replacements are employed where applicable.

2) When paralleling an existing bridge, approximate the existing low chord elevation, span lengths, and skew.

3) When paralleling an existing bridge, align the proposed and existing substructure.

4) Some common complications and solutions are as follows:

    1. If the bridge widening is significant and the existing bents do not align with the flood and/or channel flow, the widened bents can be skewed to match the flood and/or channel flow. Similarly, bents for a parallel bridge may also be skewed to match the flood/channel flow.

    2. The span arrangement for the parallel bridge can be varied from the existing bridge to adhere to the recommendations in Section 11.3.1 paragraph 8.a.

    3. If the existing low chord elevation does not provide the required clearance over the design year floods and/or the backwater/velocity/scour values indicate that a longer/higher structure is needed, the following steps should be taken:

        a. The bridge history should be investigated, and maintenance records should be reviewed for any past or existing scour problems at the site. The engineer should perform a site inspection to observe any existing or possible future problems.

        b. If no scour or flooding problems exist and the potential for any significant problems seems low, the engineer can opt to widen the structure in-kind with no major changes. It is desirable that new parallel bridge should provide the same freeboard as that required of new bridges.

        c. If the existing structure appears to be undersized based on hydraulic calculations and current design standards, and if there is evidence of flooding and visible scour problems, then the engineer must make the necessary adjustments to the existing structure until it is hydraulically sufficient. These options can include jacking the superstructure of the existing bridge, adding spans to the existing bridge, adding an overflow structure, or replacing the existing structure. A cost comparison is desirable to determine the most cost-effective alternative.

5) It is desirable for the proposed widened and/or parallel bridge abutments to clear the channel by the minimum distance specified in Section 11.3.1, paragraphs 8.a and 8.b. If this clearance cannot be achieved by widening or paralleling in-kind, the following options should be considered:

    1. For a bridge widening, the end bent(s) can be skewed away from the channel.

    2. The proposed widened and/or parallel bridge(s) can be lengthened, placing the end bent(s) farther away from the channel to obtain this clearance.

6) The possibility of replacing the existing bridge with a more cost-effective structure should be checked if the following is evident:

    1. The computer model indicates that the existing bridge is undersized or significantly oversized.

    2. Extensive repairs to the existing bridge are required.

    3. Box culvert alternative

        a. Box culverts should be considered at sites having favorable floodplain conditions. Favorable conditions would include a well-defined channel that does not accumulate a substantial amount of silt or debris in the culvert barrels or carry appreciable amounts of drift. Additionally, a waiver from the State Chief Engineer must be obtained if a precast box culvert is not used.

        b. Culverts should not be placed at locations with unfavorable conditions such as swampy areas, sites that are frequently affected by abnormal stage conditions (backwater), sites where beaver dams are prevalent, or sites that historically have had large amounts of debris in the channel.

        c. General design considerations for using a box culvert alternative are as follows:
            i. Culvert width is set by matching the width of structure to the top width of the channel (at a minimum).
            ii. Design the culvert to flow full at the outlet for the design year flood.
            iii. The culvert should be sized to provide acceptable flow velocities and backwater values.

iv. ALDOT standard sizes and skews for box culverts should be used

v. Acceptable outlet velocities should be determined by comparison with the natural channel velocities and existing drainage structure velocities and should follow the guidance previously provided in Section 11.1.5.

vi. The type of soil at the site (erodible, poor soils, muck, etc.) should be considered.

vii. A cost comparison between using a box culvert versus a bridge should be performed to support the final hydraulic structure selection.

d. Environmental considerations may preclude construction of a box culvert and this will be identified in the project's environmental documentation.

9. Scour Analysis

a. A scour analysis will be performed for all bridges, using the methods shown in the latest version of FHWA's HEC-18,[11-2] Evaluating Scour at Bridges. The latest version of FHWA's HEC-20,[11-11] Stream Stability at Highway Structures should also be consulted regarding aggradation, degradation, and channel lateral migration considerations. Contraction and local (pier) scour calculations should be performed. The design flood for Interstate Highways is the 100-year flood unless overtopped by a smaller flood. Scour for these sites should also be computed for the 500-year flood unless overtopped by a smaller flood. The design flood for State and U.S. Highways is the 100-year flood unless overtopped by a smaller flood. Scour for these sites should also be computed for the 200-year flood unless overtopped by a smaller flood. The design flood for scour for local roads is the 50-year flood or the overtopping flood if it is less than or equal to the 50-year flood. Scour should also be computed for the 100-year flood for local roads unless overtopped by a smaller flood. In the Black Prairie Region of the state, techniques and methods outlined in USGS publication Clear-Water Contraction Scour at Selected Bridge Sites in the Black Prairie Belt of the Coastal Plain in Alabama, 2006: U.S. Geological Survey Scientific Investigations Report 2007–5260 may be used to provide engineering guidance in determining estimated scour.

b. One of the primary locations where scour occurs at a bridge site is at the abutments. This is primarily due to a large discharge in the overbank area that is re-directed (contracted) toward the channel/bridge area. Guide banks can be considered for protection against this type of scour. All bridge abutments should be protected from scour by riprap or other means. The proposed bridge opening(s) should be sized to minimize the possibility of abutment and overbank scour. Due to the over prediction of the present abutment scour equations, and with the approval of the FHWA, the Department designs and protects the bridge abutments with riprap and riprap aprons as specified in Section 11.1.7, in lieu of using the results from the abutment scour calculations.

c. If the bridge is located on or near a channel bend, the possibility of channel migration is increased. Placing the bridge foundations deep enough to withstand possible migration and channel scour is recommended. The bridge abutments should be placed far enough back so that any channel migration would not reach them during the lifetime of the bridge (75 years at a minimum). Channel stabilization should be considered using the methods in FHWA's HEC-23. (11-9)

10. Relief/Overflow Structures

a. Relief or overflow openings may be needed on streams with wide floodplains. The purpose of additional openings in the floodplain is to pass a portion of the flood flow when there is a major flood event

b . Basic objectives in choosing the location of relief openings include:

1) Maintenance of flow distribution and flow patterns

2) Accommodation of relatively large flood conveyances in the floodplain

3) Avoidance of floodplain flow along the roadway embankment for long distances

4) Crossing of secondary or tributary channels

c. Relief structures should be considered for wide floodplains with a large amount of two-dimensional flow (i.e. skewed crossings)

11. Cost Analysis

a. Cost estimates should be calculated for all proposed drainage structure alternatives. The most cost effective, hydraulically adequate alternate should be chosen.

12. Risk Assessment

a. When the bridge hydraulic design is selected, a risk assessment should be performed to determine the need for an alternative design approach. The risk assessment involves questions that will determine the need for a risk analysis. See the risk assessment form in Appendix F.

13. Channel Changes

    a. It may be desirable in some instances to construct a channel change to improve the hydraulic performance of the structure. Several options should be considered and coordinated with the Environmental Technical Section and Roadway Design. Channel changes should be avoided if at all possible.

    b. If channel realignment is required, refer to Chapter 5 for guidance on the design.

14. Wetlands/Environmental Concerns

    a. Environmental concerns and/or extensive mitigation requirements may require bridges in lieu of box culverts to span wetland areas that have been delineated by the Environmental Technical Section.

15. Hydraulic Data Submittal

    a. Drainage area at the site

    b. Historic high-water (flood of record) data including the elevation of flood stage, the date of occurrence, and the source of the data.

    c. Hydraulic data for the 50-, 100-, 200-, and 500-year floods (flood stage elevations with associated discharges) should be included for Interstate, State, and U.S. Highway projects. Hydraulic data for the 25-, 50-, 100-, 200-, and 500-year floods (flood stage elevations with associated discharges) should be included for local highway projects. These flood stage elevations should be taken from the downstream face of the constricted section of the proposed bridge run in the HEC-RAS model, or from the full valley, un-constricted section of the WSPRO model. For all types of roads mentioned above, the overtopping flood with upstream stage should be included.

       Note: In cases where roadway overtopping occurs at lower flood stages than listed above, hydraulic data for the smaller flood events should be included for the site.

    d. Mean flow velocities through the bridge opening for the design year and 100-year flood or the overtopping flood if less than 100-year flood; for tidal bridges, these velocities would be the maximum velocities for the above floods.

    e. Backwater values for the design year and 100-year floods, or the overtopping flood if less than 100-year flood.

       Note: For bridges with abnormal flood stage conditions (backwater), hydraulic data should be shown for the normal and abnormal stage conditions.

16. Abutment riprap detail at the end bents.

    a. Refer to Appendix H for the general list of content items needed for a riverine hydrologic and hydraulic study and examples of the Department's H&H reports. Also refer to ALDOT's Special and Standard Highway Drawing RR-610.

11.3.2 Methods/Procedures – All Tidal Projects

The following methods/procedures are for bridge replacements, new locations, widened/parallel bridges, culvert replacements, and extensions, unless otherwise noted. In general, the methods and procedures for riverine projects should be followed to analyze headwater flooding scenarios for the site where the bay/delta is experiencing low tide. Additionally, the Storm Tide Data Tool found in Appendix H may be used to set finish grades for most bridge decks in the Mobile Delta if riverine flooding does not produce a higher stage/elevation. Exceptions to this are listed below:

> **Note:** The Interstate 10 and Battleship Parkway (U.S. Highway 90/98) crossings of Mobile Bay, as well as Perdido Pass (State Highway 182), Little Lagoon Bridge (State Highway 182), and the Dauphin Island Bridge (State Highway 193) are the exceptions to the above-mentioned procedures for tidal areas. These sites should be handled on a case-by-case basis and will likely require a complex two-dimensional hydraulic/storm tide model for site analysis.

Refer to Appendix H for the general list of content items needed for a tidal hydrologic and hydraulic study and for the Storm Tide Data Tool mentioned above.

> **Note:** The Storm Tide Data Tool found in Appendix H (Figure H.1 & H.2) was developed from extensive data compilation (high-water marks) from the two largest hurricanes of record experienced in the Mobile Bay (Frederic in 1979 and Katrina in 2005). This storm tide profile represents the highest surge elevation experienced on either side of Mobile Bay during these storms referenced to a bay centerline (stationed baseline extending from southern tip of Ft. Morgan northward to the Interstate 65 crossing of the Delta).

11.3.3 Methods/Procedures – All Riverine Major Culvert Projects

Note: The following methods and procedures are for culvert replacements, new locations, and culvert extensions that do not involve an existing or proposed bridge and are non-tidal. If an existing or proposed bridge is involved, see Section 11.3.1. If the site is tidal, see Section 11.3.2.

The methods and procedures in this section and in Section 11.3.4 are for culverts that meet any of the following conditions:

1. Existing or proposed culverts that meet the criteria given in Section 11.1

2. All sites located on streams where the 100-year floodplain has been delineated on FEMA maps

3. All sites located on streams that are named on county and/or USGS maps

4. All sites that have a significant risk associated with the project such as existing or potential flooding problems

5. All sites that are affected by downstream constrictions/obstructions or abnormal flood stages from another stream

The following hydraulic computer models are approved by the Department:

1. The FHWA HY-8 culvert analysis model.

2. The USACE computer model HEC-RAS.

3. The FHWA computer model WSPRO.

4. For regulatory FEMA hydraulic models produced from the USACE software HEC-2, HEC-RAS may be used to duplicate the current regulatory FEMA hydraulic model from HEC-2 to produce the floodway and profile runs.

Note: Computer models other than those listed above may be considered for special floodplain conditions.

1. Investigate the flood history of the stream. Sources for this information include, but are not limited to the following:

    a. USGS gauge records

    b. Existing culvert and maintenance files (The Maintenance Bureau maintains electronic files for culverts with spans of 20 feet or more)

    c. Previous studies done by ALDOT, USACE, FEMA and the USGS information from local residents

    d. Information from the local government and information from local ALDOT personnel

2. Investigate the culvert site history. Some sources of information are:

    a. The culvert inspection and maintenance files

    b. A comparison of the original culvert plan and profile with the currently surveyed profile. Based on this information, an indication of the long-term channel stability and aggradation or degradation can be estimated. An evaluation of the performance of the existing culvert can also be made

3. Determine the project site hydrology for the culvert.

    a. The same procedure outlined in Section 11.3.1 paragraph 4 for riverine bridge projects should be followed to determine the hydrologic characteristics for the culvert project location.

4. Provide a field inspection of the project site.

    a. The hydraulic engineer performing the study and computer modeling should visit the culvert site(s) and perform a site inspection. During the field inspection, the engineer should evaluate the following:

        1) Characteristics and hydraulic properties of the stream

        2) Performance of the existing culvert (if applicable)

        3) Channel and floodplain geometrics

        4) Adequacy and accuracy of the survey data

        5) Stream drift potential

        6) Manning's n-values

        7) Presence of head cuts in reach

        8) Condition of trees on/near top of bank

    b. In addition, the following site conditions should be noted:

        1) Buildings or structures in the floodplain that may be subject to flooding

        2) Scour and/or undermining problems at the existing culvert (if applicable)

        3) Evidence of past channel migration or potential for future migration

    c. During the field inspection, stream crossings immediately upstream or downstream of the project site on the same stream may be visited and the performance of the structures noted

5. Determine the extent of the survey data.

    a. The hydraulic engineer should determine the extent of the survey data required to accurately model the project site based on the requirements from the ALDOT Design Bureau Manual On Survey Requirements. Please refer to this manual for the required survey information.

    b. See the Hydraulics/Drainage Collection in the ALDOT Design Bureau Manual On Survey Requirements (latest version) for a detailed listing of the minimum survey data required.

6. Perform a hydraulic analysis.

   a. The hydraulic computer model should be used to determine the existing and proposed conditions at the site. The 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year floods should be modeled for the project site. The design flood should be conveyed through the culvert opening, while floods greater than the design flood may be conveyed over the roadway and through the culvert opening.

   b. The FHWA HY-8 computer culvert analysis model may be used to calculate tailwater solutions using the irregular channel option and to size a culvert and set the minimum roadway grade for the project site if the following criteria are met:

      1) The channel is uniform and the channel slope is constant.

      2) Tailwater at the site is not affected by downstream conditions such as another roadway crossing, a natural constriction of the channel and/or floodplain, or a confluence with another stream.

      3) If accurate tailwater elevations are required due to the risk associated with a project such as existing or potential upstream flooding problems, the engineer may choose to use the more detailed analysis described in item (c) below.

   c. For all other project sites the natural or unconstricted high-water profiles should be developed using the WSPRO or HEC-RAS computer models. The results of these computations should be used to determine the various tailwater heights, size the culvert, and evaluate the culvert hydraulic performance using HY-8. Another option is to use the culvert routine within HEC-RAS. The HEC-RAS computer model contains an option for an arch type bottomless bridge culvert.

   d. If the project is located within a FEMA study area, FEMA guidelines must also be satisfied. See Chapter 2, *Agency Coordination and Regulations*.

7. Consider hydraulic design guidelines for culverts.

   a. Culvert Replacements

      1) In general, box culverts are placed at sites which have favorable floodplain conditions, such as in a well-defined channel and where excessive silt is not likely to accumulate in the culvert barrels or carry appreciable amounts of drift. For this reason, culverts are generally not placed in swampy areas or sites that are frequently affected by abnormal stage conditions (backwater), locations where beaver dams are prevalent, or sites that historically have had large amounts of debris in the channel. Additionally, a waiver from the State Chief Engineer must be obtained in the event that a precast box culvert is not used.

i. Design criteria in Section 11.1 of this chapter should be followed where applicable to culverts.

ii. ALDOT standard sizes and skews for concrete box culverts are to be used

iii. Culvert width is normally set by matching the top width and profile of the channel and designing the culvert to flow full for the design year flood. Design flood frequencies are established in the Section 11.1 of this chapter.

iv. Culverts should if possible be sized to provide acceptable flow velocities and backwater values.

v. Profile grades along the proposed roadway should be set to meet the requirements as specified in Section 11.1.1.

2) Culvert Extensions

i. In general, the above recommendations (Section 11.3.3 paragraph 7a) for culvert replacements apply where applicable.

ii. A hydraulic analysis is required for culvert extensions because length is a factor if the culvert flow is under outlet control. In addition, an upstream extension may affect headwater if the culvert is under inlet control.

iii. The culvert history should be investigated. The maintenance records should be reviewed for any past or existing scour problems at the site. The engineer should perform a site inspection to observe any existing or possible future scour and/or flooding problems.

iv. If there is evidence of flooding and/or scour problems or if the culvert extension/proposed roadway work is so significant that the calculations indicate a larger structure is required, the engineer must make the necessary adjustments to the existing structure until it is hydraulically sufficient. These adjustments can include adding barrels to the existing culvert or replacing the existing culvert.

v. The possibility of replacing the existing culvert with a more cost-effective structure should be checked if extensive repairs to the existing culvert are required.

3) Bottomless Culverts

Bottomless culverts are used where the natural streambed is kept intact for ecological and environmental concerns. The culverts most often used in these cases are concrete box culverts and arch type bridge culverts. These bottomless arch structures can also be used as an alternate to a standard box culvert or small bridge for non-environmental reasons (typically site aesthetics). These applications are for sites with solid rock stream bottoms.

8. Use wingwalls and toe walls.

Wingwalls should be used to retain and protect the highway embankment and toe walls should be used to protect the culvert from stream degradation and prevent undermining of the structure.

9. Perform a cost analysis.

Cost estimates should be calculated for all proposed drainage structure alternatives. The most cost effective, hydraulically adequate alternative should be chosen.

10. Perform a risk assessment.

A risk assessment will be performed to determine if an alternative design approach should be considered. The risk assessment includes questions that will determine the need for a risk analysis. See the risk assessment chart in Appendix F.

11. Consider channel changes.

Refer to Section 11.3.1 paragraph 13 regarding the criteria for channel improvements.

12. Provide roadway plan information.

a. Bridge culvert information to be shown on the roadway plans includes, but is not limited to the following:

1) Plan and elevation view of the proposed bridge culvert. The culvert size, length, location, and invert elevations should be shown.

2) Approximate original groundline should be shown in the elevation view.

3) Historic high-water (flood of record) data including: elevation of high-water, date of occurrence, and source of data.

4) Hydraulic data for the 50-, 100-, 200-, and 500-year floods (flood stage elevations with associated discharges) should be included for Interstate, State, and U.S. Highway projects. Hydraulic data for the 25-, 50-, 100-, 200-, and 500-year floods (flood stage elevations with associated discharges) should be included for local highway projects. All flood stage elevations given are at the outlet of the proposed culvert.

   Note: In cases where roadway overtopping occurs at lower flood stages than listed above, hydraulic data for the smaller flood events should be included for the site.

5) Design year headwater elevation and 100-year headwater elevation or the overtopping flood headwater elevation if less than 100-year flood.

6) Drainage area at the site.

7) Type and size of the detour structure.

8) Proposed grade data.

9) Horizontal curve data.

10) Bearing along the construction centerline.

11) Benchmark data.

12) Traffic data.

13) Utilities, existing and proposed.

14) A construction sequence is required if stage is constructed.

15) North arrow.

16) Flow direction arrow; for tidal sites, ebb tide and flood tide directions should be shown.

17) Destination arrows.

18) Title block information, which includes route name and number, stream name, county, PI number, and the date drawn.

b. Refer to Appendix H for the general list of content items needed for a riverine hydrologic and hydraulic major study.

11.3.4 Hydrologic and Hydraulic (H&H) Study Procedures – Design/Bridge Bureau and Consultant Responsibilities

General Guidelines

All Hydraulic Studies

1. A hydrologic and hydraulic study should be performed for a project site that involves an existing or proposed bridge.

2. The units for the hydrologic and hydraulic study should be consistent with the proposed roadway plans.

In-House Hydraulic Studies

1. The Hydraulics Section of the Bridge Bureau is responsible for performing the hydrologic and hydraulic studies for all bridge replacement and widened/parallel bridge projects, as well as new locations where bridges are proposed.

2. The Roadway Design Section is responsible for performing roadway culvert and pipe studies in which a flood profile model such as WSPRO or HEC-RAS is not required. The Hydraulics Section in the Bridge Bureau is available for guidance for culvert studies that involve the use of programs such as WSPRO or HEC-RAS.

Consultant Projects

Consultant Responsibilities

The consultant is responsible for the following:

1. Sizing the most cost effective drainage structure in accordance with the design criteria, procedures and guidelines contained within this manual.

2. Proficiency in the knowledge and use of all required computer models, as well as the required methods, procedures, calculations, publications, and design criteria contained within this manual.

3. Obtaining or requesting from the Design Services Engineer/Region Consultant Engineer/project manager any survey data that is required to accurately model the project site, depending on the contract. In addition, it is desirable that the consultant be proficient in the use of InRoads.

4. Investigating the bridge site history by searching the electronic files for the existing bridges maintained in the Maintenance Bureau. These electronic files often contain old hydraulic studies, bridge foundation investigations, and existing bridge plan sheets that may be useful in assessing scour or debris problems. For later existing studies, bridge foundation investigations, and bridge

plan sheets that may have been done but are not contained in the Maintenance Bureau's electronic files, the consultant should contact the Design Services Engineer/Region Consultant Engineer/project manager for assistance.

5. Obtaining or requesting any profile grade change(s) from the Design Services Engineer/Region Consultant Engineer/project manager that is required for the project to meet the guidelines contained within this manual. If the consultant is also producing the roadway plans for the project, setting the profile grade(s) to meet these guidelines is the consultant's responsibility.

6. Obtaining or requesting any horizontal alignment change(s) from the Design Services Engineer/Region Consultant Engineer/project manager that would enable the bridge to be built more efficiently or would limit encroachment on channels and/or floodplains.

7. Obtaining approval from the Bridge Hydraulic Section before using a computer model other than the HEC-RAS or the WSPRO model for non-tidal conditions.

8. Obtaining approval from the project engineer for a channel change.

   Note: Due to the extensive mitigation required for channel changes, approval for a channel change is extremely unlikely.

9. Obtaining approval from the State Bridge Engineer/project engineer before proceeding with plans for bridge widening projects that replace or significantly change the existing bridge.

10. Sizing a drainage structure for a site within a FEMA regulatory floodway which meets the Department's, the affected community's and FEMA's standards and approval. The consultant should provide the necessary forms, floodway and flood profile computer modeling, and other supporting documentation as required for approval. Prior to the approval of the hydraulic study by the Department, the consultant should assist in performing the necessary community and /or FEMA coordination.

    Note: All supporting documentation, along with copies of correspondence and approvals from the community and/or FEMA should be provided to the Department for their records and use.

11. For local transportation projects involving state and federal funds where the consultant has performed a hydraulic study for the community, the consultant, at a minimum, should provide the Department with a copy of a letter of concurrence from the community and approval from FEMA (if required) along with the hydrologic/hydraulic report summarizing the results of the study.

12. Making any necessary adjustments and/or corrections to the hydrologic and hydraulic study, computer models, and FEMA documentation as required as a result of reviews, field inspections, bridge stakeouts, and/or bridge foundation investigations.

13. Providing the Department with an electronic copy of the final hydrologic and hydraulic study and FEMA package (if applicable) including all model input and output files.

In addition to the items above, the cover sheet of the completed hydrologic and hydraulic study must state "Hydraulic study prepared by" and must include the signature, date, and PE stamp for the engineer who prepared the study.

Common Omissions and Points of Emphasis

1. Sizing of proposed bridges for replacement and new location projects.

    a. In many cases, a proposed bridge opening has been sized to approximate or to be slightly larger than the existing structure. This bridge may or may not be the minimum length bridge that is needed at the site. In other cases, a proposed bridge is sized that can be reduced in length due to a lack of potential upstream flooding problems or very low proposed backwater and flow velocity values.

    b. The proposed drainage structure should be sized as the minimum length bridge, smallest culvert, or most cost-effective combination of drainage structures that have acceptable backwater and velocity values that fits the stream geometry and meets applicable FEMA requirements while adhering to the procedures, guidelines and design criteria of this manual.

    c. The minimum length bridge that can be placed at a site due to the channel geometry is specified in Section 11.3.1 paragraphs 8.a and 8.b. If this minimum length bridge has acceptable backwater and flow velocities and meets applicable FEMA requirements, then this is the proposed bridge length that should be chosen. If not, the bridge length should be increased if feasible until acceptable backwater and flow velocities are achieved.

    d. The 100-year backwater should be limited to 1 foot above the unrestricted or natural 100-year water surface profile. As a general rule to minimize scour and backwater, the mean velocity values for a bridge opening should not exceed 4.5 feet per second. Bridge culverts should have a targeted mean velocity of around 5.5 feet per second at the culvert outlet. In the event that a substantial amount of rock is present in the channel bed, the targeted mean velocity for both bridges and culverts can be significantly higher based on existing velocities.

    e. The reason(s) for choosing the proposed drainage structure should be clearly stated in the written report**.** Example justifications are: "The 240 ft long bridge was chosen as the replacement structure for this site, because it was the minimum length bridge that has acceptable backwater and channel velocities."; or "The 240 ft long bridge was chosen as the replacement structure for this site, because it was the minimum length bridge that aligns well with the approach channel geometry and has acceptable backwater and channel velocities."

2. Model all floodplain constrictions/obstructions and abnormal flood stage conditions that affect the project site (see Section 11.1.11). The consultant is responsible for recognizing and identifying these conditions at the outset of the project. The costs for modeling these conditions should be included in the initial work order. If the consultant is responsible for providing any additional survey information that is required to model the project site, these costs should be included as well. The drainage structure should be modeled and sized based on these conditions.

3. The hydraulic engineer performing the study and computer modeling should visit the project site and perform a site inspection.

4. Box culvert alternatives may be considered at all sites with a drainage area of 10 square miles or less. The results of this consideration are to be included in the hydraulic study. If it is determined that a box culvert will be hydraulically satisfactory at the project site, the final decision as to whether a box culvert or bridge will be used should be based on a cost comparison. This cost comparison is to be included in the hydraulic study. Additionally, a waiver from the State Chief Engineer must be obtained in the event that a precast box culvert is not used. The computer modeling for the culvert and bridge alternates should be included along with hydraulic tables showing the results for both alternates. The reasons that the proposed drainage structure was chosen or eliminated from consideration should be included in the written report of the hydraulic study.
    a. ALDOT standard size and skew concrete box culverts should be used at proposed culvert sites.
    b. Environmental considerations and/or unfavorable floodplain conditions may preclude a box culvert alternative at a site. If this is the case, no computer modeling is necessary for the box culvert option. The reasons for this determination should be stated in the written hydraulic report.

5. If a bridge is required to be constructed at a site due to environmental considerations, written documentation should to be included in the hydraulic study. This documentation should state the reasons that a box culvert should not be constructed at the project site. In addition, any limitations placed on the location of the abutment and/or intermediate bents for the proposed bridge should be included in this documentation.

6. Errors that should be checked for in the hydrologic and hydraulic studies include negative backwater values, and/or flood flow velocities through the bridge opening that are less than the flow velocities for the natural, unconstricted conditions. When a constriction, such as a roadway, is placed into a floodplain, it will not lower the upstream water surface elevation from the natural conditions, nor will the flow velocities through the constricted bridge opening be less than the natural condition flow velocities. Stream channel improvements that extend a significant distance upstream and downstream of the crossing could reduce the water surface elevations. Due to environmental concerns, channel improvements to this extent are rare. Model parameters should be rechecked if either of the conditions is observed in the model results.

7. The Department’s Bridge Inventory Number(s) (BINs) should be included in all appropriate documentation relating to the project including the final hydrologic and hydraulic report for the site.

Review of Consultant Hydrologic and Hydraulic (H&H) Studies

Note: The Department’s review of the consultant's work shall not relieve the consultant of the responsibility and accountability for sizing drainage structures in accordance with the design criteria, procedures, and guidelines contained within this manual. Furthermore, the Department’s review is not intended to be used as a quality control device by the consultant. The Department’s review is cursory and may note obvious discrepancies. The parameters and values used to model the site are not thoroughly checked. Specific numbers are, in general, not checked.

R11 Chapter 11 References

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Chapter 12: Bridge Deck Drainage Systems

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12.1 Introduction

Bridge deck drainage is similar to that for a curbed roadway section. It can be less efficient because deck cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging due to debris.

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. Intercepted runoff should be collected by means of inlets and conveyed within a storm sewer system to the proper stable designed outlet. For minimal intercepted flow, gutter turnouts may be used to direct the runoff to an adjacent road side ditch.

The bridge deck drainage system should be designed to convey water and keep it from contacting the structural components of the bridge in order to prevent deterioration from runoff pollutants.

12.2 Design Guidelines

FHWA Hydraulic Engineering Circular No. 21 (HEC-21), *Design of Bridge Deck Drainage, May 1993*, [(12-1)] should be referenced for bridge deck drainage design procedures and example problems. The following is a summary of design guidelines for bridge deck drainage systems.

12.2.1 General Design Criteria

The designer should follow the basic rules listed below to eliminate and/or minimize bridge deck drainage problems.

- Superelevation transitions, flat grades, and sag vertical curves should be avoided, if practical, on bridges. The minimum desirable longitudinal grade for bridge deck drainage is 0.5%. Where flat grades are necessary, the designer should provide scupper spacing for adequate drainage.

- Gutter flow drainage from the upslope roadway should be collected before it reaches the bridge deck.

- Runoff from bridge decks should be collected immediately if practical after it flows onto the subsequent roadway section where larger grates and inlet structures can be used.

- Typical practice is to provide a 2% pavement cross slope for travel lanes. Cross slope should be increased to 2.5% in areas where an increase is practicable and justified. On multi-lane roadways, the cross slope may be broken at 0.5% intervals not to exceed 4% on any lane. Steeper cross slopes (4% maximum) should be considered for roadways draining more than three travel lanes in the same direction or in a 4-lane divided section where the gutter grade is less than 0.5%.

- Ideally, the longitudinal slope of the bridge deck should be steep enough to satisfy the gutter-spread requirements without the need for scuppers or a closed conveyance system on the structure.

- For long span bridges, it is desirable to set the proposed profile in a crest vertical curve with the high point occurring in the center of the bridge.

12.2.2 Design Spread and Frequency

Criteria for design spread and frequency are listed below:

- The Rational Method should be used for computing runoff for bridge decks.

- Rainfall spread on the bridge (spread from gutterline) should be limited to the shoulder area during the design storm.

- The design storm should be the 10-year frequency except that the 50-year frequency storm should be used for bridges located where the low point of a sag vertical curve occurs on the bridge. Any exceptions to this policy will require the approval of the State Bridge Engineer.

12.2.3 Bridge Deck Drainage Systems

Four-inch diameter scuppers should be used to provide deck drainage unless otherwise directed by the Hydraulic Engineer. The scuppers should be spaced at 5’ (maximum) centers along both gutterlines in normal crowned section with eight to ten foot shoulders and 4’ along the lower gutterline in a superelevated section unless otherwise directed by the Hydraulic Engineer. Scuppers should be omitted over pier caps, roadway lanes, and railroad beds. Larger scuppers or deck drain inlets may be required if the above design is not adequate or if a closed system is required.

Water from bridge decks should be allowed to fall freely to the ground through deck scuppers and open joints, except over streets and railroad. Closed drainage systems should only be used in special conditions where water draining from the deck is not permitted (Example: Sensitive Features).

12.3 Information Needed for Design

- Preliminary proposed roadway plans

- Preliminary bridge layout

- FHWA HEC-21

12.4 Design Methods and Procedures

- The roadway engineer should consider drainage early in the design phase. By avoiding superelevation transition, flat grades, and sag vertical curves on bridges, inlets on bridges can often be eliminated. Adequate cross slope should be provided on the bridge section so that the water flows quickly toward the drain.

- The roadway engineer should calculate the gutter flow drainage from the upslope roadway using the Rational Method as shown in Chapter 6, *Pavement Drainage*.

- The roadway engineer should place and size one or more drainage structures to collect the gutter flow drainage from the upslope roadway before it reaches the bridge deck. See Chapter 6 of this manual.

- The hydraulic engineer should determine if the standard bridge deck drain systems described in Section 12.2.3 are adequate. **The engineer should take into account that bridges located over railroads, roadways, and other sensitive features may not have any open deck drains incorporated into the structure.**

- The roadway engineer should place and size one or more drainage structures to collect runoff from the bridge deck immediately after it flows onto the subsequent roadway section (see Chapter 6 of this manual).

- If the hydraulic engineer determines that the standard open deck drains are inadequate for the bridge, the methods in HEC-21 should be used to size an adequate deck drain system. A catalog from an approved supplier should be used to select a bridge drain system that will be satisfactory both hydraulically and structurally. The hydraulic engineer and bridge structural engineer should meet and decide, on a case-by-case basis, which deck drain system is the best for the bridge.

12.5 Analysis of Circular Scuppers

The flow in circular scuppers can be estimated using Equation 6.12 provided in Chapter 6, which is included as follows for convenience:

$$Q_i = EQ \qquad (6.12)$$

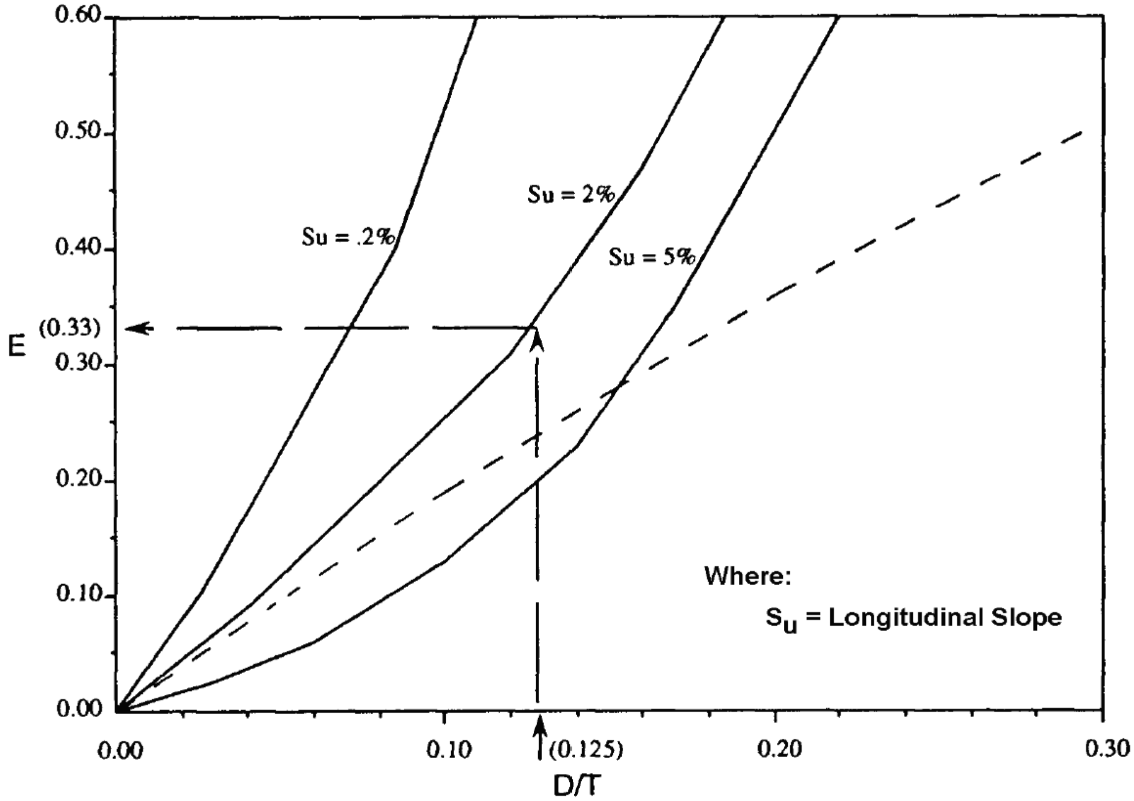
Where:

Q_i = Flow intercepted by the circular scupper inlet, ft^3/s

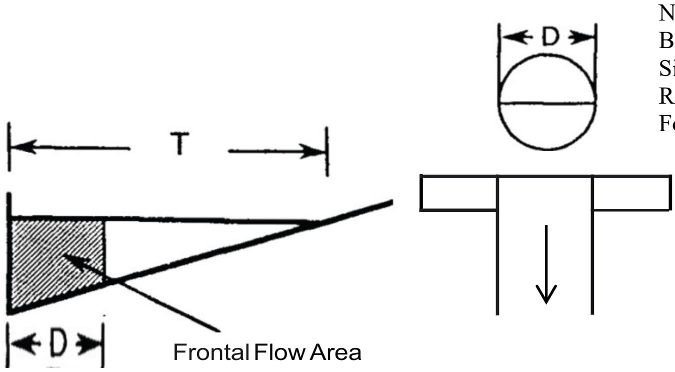
E = Efficiency

Q = Flow in the gutter for a given width of spread, ft^3/s

The efficiency (E) of circular scuppers to be used with Equation 6.12 is given by Figure 12.1.



———	Empiric curves based on laboratory data by Anderson, 1973
— — —	100% of frontal flow area of width, D



Note: One cross
Bar did not
Significantly
Reduce efficiency
For D = 4 inches

EXAMPLE

Given:

D = 6 in.; T = 4 ft.
Su = 2%

Then:

D/T = .125
E = 0.33

Figure 12.1 – Efficiency curves for circular scuppers(12-1)

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Appendix A: Acronyms

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A list of stormwater management terms used for state highways is provided below:

AASHTO	American Association of State Highway Transportation Officials
ACPA	American Concrete Pipe Association
ADT	Average Daily Traffic
AHW	Allowable Headwater
AOP	Aquatic Organism Passage
ASTM	American Society for Testing and Materials
BIN	Bridge Identification Number
BMP(s)	Best Management Practice(s)
CAD	Computer-Aided Design (software)
CE	Categorical Exclusion
CFR	Code of Federal Regulations
CLOMR	Conditional Letter of Map Revision
CN	Curve Number
CWA	Clean Water Act
DFIRM	Digital Flood Insurance Rate Maps
DNR	Department of Natural Resources
DTM	Digital Terrain Model
EA	Environmental Assessment
EGL	Energy Grade Line
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency (Federal)
ADEM	Alabama Department of Environmental Management (State)
FEIS	Final Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Studies
ALDOT	Alabama Department of Transportation

GDCP	Guide for Developing Construction Plans
GI	Green Infrastructure
GIS	Geographic Information System
GPS	Global Positioning System
HDS	Hydraulic Design Series
HGL	Hydraulic Grade Line
HW	Headwater
IDF	Intensity-Duration-Frequency
LID	Low Impact Development
LOMR	Letter of Map Revision
LRFD	Load and Resistance Factor Design
MDM	Model Drainage Manual
MEP	Maximum Extent Practicable
MS4	Municipal Separate Storm Sewer Systems
NAVD	North American Vertical Datum
NEPA	National Environmental Protection Act
NFIP	National Flood Insurance Program
NGS	National Geodetic Survey
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
NWP	Nationwide Permit
O&M	Operation and Maintenance
OGFC	Open Graded Friction Course
PDF	Portable Document Format
PE	Professional Engineer
PCN	Pre-Construction Notification
PI	Point of Intersection

PSC	Prestressed Concrete
PVC	Polyvinyl Chloride
PVI	Point of Vertical Intersection
QA	Quality Assurance
QC	Quality Control
RCP	Reinforced Concrete Pipe
ROW	Right-of-Way
SCS	Soil Conservation Service
SDH	Single Design Hydrograph
TMDL	Total Maximum Daily Load
TVA	Tennessee Valley Authority
TW	Tailwater
USACE	United States Army Corps of Engineers
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

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Appendix B: FEMA Agency Coordination, Regulations, and Documentation

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FEDERAL-AID POLICY GUIDE
September 30, 1992, Transmittal 5 NS 23 CFR 650A
Attachment 2

NON-REGULATORY SUPPLEMENT ATTACHMENT

OPI:HNG-31

PROCEDURES FOR COORDINATING HIGHWAY ENCROACHMENTS ON FLOODPLAINS WITH FEDERAL EMERGENCY MANAGEMENT AGENCY (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. Most NFIP communities have established a permit requirement for all development within the base (100 year) floodplain. Consistency with NFIP standards is a requirement for Federal-aid highway actions involving regulatory floodways. The community, by necessity, is the one who must submit proposals to FEMA for amendments to NFIP ordinances and maps in that community should it be necessary. Determination of the status of a community's participation in the NFIP and review of applicable NFIP maps and ordinances are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

Where NFIP maps are available, their use is mandatory in determining whether a highway location alternative will include an encroachment on the base floodplain. Three types of NFIP maps are published: (1) a Flood Hazard Boundary Map (FHBM), (2) a Flood Boundary and Floodway Map (FBFM), and a Flood Insurance Rate Map (FIRM). A FHBM is generally not based on a detailed hydraulic study and, therefore, the floodplain boundaries shown are approximate. A FBFM, on the other hand, is generally derived from a detailed hydraulic study and should provide reasonably accurate information. The hydraulic data from which the FBFM was derived is available through the regional office of FEMA. This is normally in the form of computer input data cards for calculating water surface profiles. The FIRM is generally produced at the same time using the same hydraulic model and has appropriate rate zones and base flood elevations added.

Communities in the regular program of the NFIP generally have had detailed flood insurance studies performed. In these communities the NFIP map will be a FIRM and in the majority of cases, a regulatory floodway is in effect.

Communities in the emergency program of the NFIP usually have not had a detailed flood insurance study completed and, usually, only limited floodplain data is available. In this case the community NFIP map will be a FHBM and there will not be a regulatory floodway.

Other possibilities are: (1) the community is not in a FEMA identified flood hazard area and thus there is no NFIP map, (2) a FHBM, FIRM, or FBFM is available but the community is not participating in the NFIP, (3) a community is in the process of converting from the emergency program to the regular program and a detailed flood insurance study is underway, or (4) a community is participating in the regular program, the NFIP map is a FIRM, but no regulatory floodway has been established.

Information on community participation in the NFIP is provided in the "National Flood Insurance Program Community Status Book" which is published bi-monthly for each State and is available through the Headquarters of FEMA.

Coordination With FEMA

It is intended that there should be highway agency coordination with FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances which would ordinarily require coordination with FEMA are:

1. A proposed crossing encroaches on a regulatory floodway and, as such, would require an amendment to the floodway map,

2. A proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated and the maximum 1 foot increase in the base flood elevation would be exceeded,

3. A local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are underway,

4. A local community is participating in the emergency program and base flood elevation in the vicinity of insurable buildings is increased by more than 1 foot. (Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to base flood elevations as a result of highway construction.)

The draft EIS/EA should indicate the NFIP status of affected communities, the encroachments anticipated and the need for floodway or floodplain ordinance amendments. Coordination means furnishing to FEMA the draft EIS/EA and, upon selection of an alternative, furnishing to FEMA through the community a preliminary site plan and water surface elevation information and technical data in support of a floodway revision request as required. If a determination by FEMA would influence the selection of an alternative, a commitment from FEMA should be obtained prior to the FEIS or FONSI. Otherwise this later coordination may be postponed until the design phase.

For projects that will be processed with a categorical exclusion, coordination may be carried out during design. However, the outcome of the coordination at this time could change the class of environmental processing.

Highway Encroachments Which Are Consistent With Regulatory Floodways In Effect

In many situations it is possible to design and construct highways in a cost-effective manner such that their components are excluded from the floodway. This is the simplest way to be consistent with the standards and should be the initial alternative evaluated. If a project element encroaches on the floodway but has a very minor effect on the floodway water surface elevation (such as piers in the floodway), the project may normally be considered as being consistent with the standards if hydraulic conditions can be improved so that no water surface elevation increase is reflected in the computer printout for the new conditions. Examples of such improvement could be the clearing of vegetative cover (reducing roughness) or earthwork (increasing flow area) in the overbank regions of the bridge opening. Such improvements should be within the project right-of-way and should meet USACE permit compliance in the event the site is in a designated wetland.

Revision of Regulatory Floodway So That Highway Encroachment Would Be Consistent

Where it is not cost-effective to design a highway crossing to avoid encroachment on an established floodway, a second alternative would be a modification of the floodway itself. Often, the community will be willing to accept an alternative floodway configuration to accommodate a proposed crossing provided NFIP limitations on increases in the base flood elevation are not exceeded. This approach is useful where the highway crossing does not cause more than a 1 foot rise in the base flood elevation. In some cases, it may be possible to enlarge the floodway or otherwise increase conveyance in the floodway above and below the crossing in order to allow greater encroachment. Such planning is best accomplished when the floodway is first established. However, where the community is willing to amend an established floodway to support this option, the floodway may be revised.

The responsibility for demonstrating that an alternative floodway configuration meets NFIP requirements rests with the community. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Floodway revisions must be based on the hydraulic model which was used to develop the currently effective floodway but updated to reflect existing encroachment conditions. This will allow determination of the increase in the base flood elevation that has been caused by encroachments since the original floodway was established. Alternate floodway configurations may then be analyzed.

Base flood elevation increases are referenced to the profile obtained from either the original model or the corrected effective model (if changes have occurred).

Data submitted to FEMA in support of a floodway revision request should include:

1. Copy of current regulatory Flood Boundary Floodway Map, showing existing conditions, proposed highway crossing and revised floodway limits.

2. Copy of computer printouts (input, computation, and output) for the duplicated original 100-year base flood model and original 100-year floodway model.

3. Copy of computer printouts (input, computation, and output) for the corrected effective100-year base flood model and current 100-year floodway model. **Note:** This is only needed if changes in the flood plain in the vicinity of the project have taken place since the original model was completed.

4. Copy of computer printouts (input, computation, and output) for the proposed conditions 100-year base flood and floodway model.

5. Copy of engineering certification is required for work performed by private subcontractors.

The existing and proposed conditions computer data required above should extend far enough upstream and downstream of the floodway revision area in order to tie back into the original floodway and profiles using sound hydraulic engineering practices. This distance will vary depending on the magnitude of the requested floodway revision and the hydraulic characteristics of the stream.

A floodway revision will not be acceptable if development that has occurred in the existing flood fringe area since the adoption of the community's floodway ordinance will now be located within the revised floodway area unless adversely affected adjacent property owners are compensated for the loss.

If the input data representing the original hydraulic model is unavailable, an approximation should be developed. A new model should be established using the original cross-section topographic information, where possible, and the discharges contained in the Flood Insurance Study which establish the original floodway. The model should then be run confining the effective flow area to the currently established floodway and calibrate to reproduce within 0.10 foot, the "With Floodway" elevations provided in the Floodway Data Table for the current floodway. Floodway revisions may then be evaluated using the procedures outlined above.

Floodway Encroachment Where Demonstrably Appropriate

When it would be demonstrably inappropriate to design a highway crossing to avoid encroachment on the floodway and where the floodway cannot be modified such that the structure could be excluded, FEMA will approve an alternate floodway with backwater in excess of the 1 foot maximum only when the following conditions have been met:

1. A location hydraulic study has been performed in accordance with "Location and Hydraulic Design of Encroachments on Floodplains" (23 CFR 650, Subpart A) and FHWA finds the encroachment is the only practicable alternative.

2. The constructing agency has made appropriate arrangements with affected property owners and the community to obtain flooding easements or otherwise compensate them for future flood losses due to the effects of the structure.

3. The constructing agency has made appropriate arrangements to assure that the National Flood Insurance Program and Flood Insurance Fund do not incur any liability for additional future flood losses to existing structures which are insured under the Program and grandfathered in under the risk status existing prior to the construction of the structure

4. Prior to initiating construction, the constructing agency provides FEMA with

revised flood profiles, floodway and floodplain mapping, and background technical data necessary for FEMA to issue revised Flood Insurance Rate Maps and Flood Boundary and Floodway Maps for the affected area upon completion of the structure.

Highway Encroachment On A Floodplain With A Detailed Study (FIRM)

In communities where a detailed flood insurance study has been performed but no regulatory floodway designated, the highway crossing should be designed to allow no more than a 1 foot increase in the base flood elevation based on technical data from the flood insurance study. Technical data supporting the increased flood elevation should be submitted to the local community and FEMA for their files. Where it is demonstrably inappropriate to design the highway crossing and meet backwater limitations the procedures outlined under:

> *Floodway Encroachment Where Demonstrably Appropriate* should be followed in requesting a revision of base floodplain reference elevations.

Highway Encroachment On A Floodplain Indicated On An FHBM

In communities where detailed flood insurance studies have not been performed, the highway agency must generate its own technical data to determine the base floodplain elevation and design encroachments in accordance with 23 CFR 650A. Base floodplain elevations should be furnished to the community, and coordination carried out with FEMA as outlined previously where the increase in base flood elevations in the vicinity of insurable buildings exceeds 1 foot.

Highway Encroachment On Unidentified Floodplains

Encroachments which are outside of NFIP communities or NFIP identified flood hazard areas should be designed in accordance with 23 CFR 650A of the Federal Highway Administration. The NFIP identified flood hazard areas are those delineated on an FHBM, FBFM or FIRM.

To Obtain FEMA Publications

1. **FEMA Flood Map Service Center**

   https://msc.fema.gov/portal

2. **Alabama Office of Water Resources**

   https://www.adeca.alabama.gov/Divisions/owr/floodplain/Pages/default.aspx

   Note: Current effective hydraulic models can be requested from Alabama OWR

3. **National Flood Insurance Program Community Status Book**

   Write to FEMA, 500 "C" Street, SW., Room 431, Insurance Operations, Washington, D.C. 20472 and request to be placed on the appropriate State mailing list.

4. **Flood Insurance Study Report and/or FBFM**

   Write to FEMA, 500 "C" Street, SW., State and Local Programs Room 418, Washington, D.C. 20472 request:

   (a) For future studies,

   To be placed on mailing list to receive all studies and maps as they are completed for a State.

   (b) For completed studies,

   (1) The study for a particular community (provide number).

   (2) All the studies for a particular State. You will receive about 50% of the completed studies to date.

5. FHBM or FIRM for a particular community with ID number,

   (a) Call NFIP contractor (800) 638-6620, (800) 492-6605(MD), 897-5900 in D.C., or

   (b) Write NFIP, P.O. Box 34604, Bethesda, Maryland 20034.

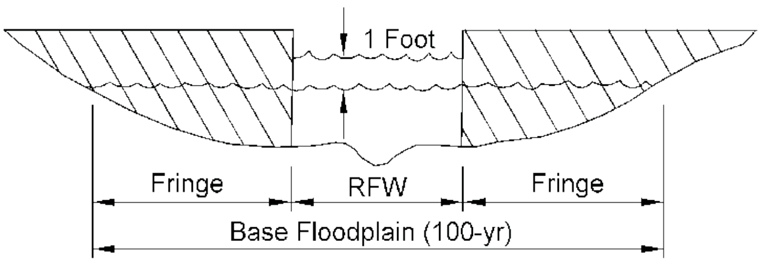
Briefing FHWA/FEMA Coordination Procedures

The procedures divide highway encroachments on floodplains into six categories:

1. **Consistent with a Regulatory Floodway (RFW)**

   (a) Applicable to 5,000 communities (county or city) which are in the FEMA regular flood insurance program

   (b) Community prohibits development in RFW, but allows development that is flood proofed in fringe



   (c) Highways are consistent by not increasing backwater

      (1) Bridging RFW and
      (2) Excluding fill from RFW

2. **Consistent by Revision of RFW**

   (a) Same as 1
   (b) Same as 1
   (c) Same as 1
   (d) If community and FEMA agree, RFW can be shifted

3. **On RFW where demonstrably appropriate**

   (a) Same as 1
   (b) Same as 1
   (c) Highways can increase backwater if:
      (1) Little or no risk to development can be demonstrated, and
      (2) Community and FEMA concur

4. **On floodplain shown on Flood Insurance Rate Map (FIRM)**

   (a) Applicable to 2,000 communities in regular insurance program
   (b) No RFW has been developed, but flood elevations have
   (c) Community controls development within FIRM
   (d) Highway encroachment should cause less than 1 foot of backwater

5. **On floodplain shown on Flood Hazard Boundary Map (FHBM)**

    (a) Applicable to 13,000 communities, 10,000 in emergency insurance program
    (b) No RFW or flood elevations have been developed
    (c) Community controls development within FHBM
    (d) Highway encroachment should cause less than 1 foot of backwater if insurable buildings are present

6. **On unidentified floodplains**

    (a) Floodplain is not shown on FIRM or FHBM

    (b) Floodplain is therefore outside of the 20,000 flood prone areas in the U.S. that are of concern of FEMA

    (c) Apply FHPM 6-7-3-2, Location and Hydraulic Design of Encroachments on Floodplains

Section 2 - Definitions of NFIP Terminology

Frequently used terms related to NFIP compliance are defined below.

The BASE FLOOD is the flood having a 1% chance of being equaled or exceeded in a given year. This is often referred to as the 100-year flood.

The BASE FLOOD ELEVATION is the water surface elevation at a given location associated with the base flood.

The BASE FLOOD PROFILE is the water surface profile along a stream associated with the base flood.

The COMMUNITY is the local entity (city or county government) with jurisdiction for floodplain administration under the NFIP.

A CONDITIONAL LETTER OF MAP REVISION (CLOMR) is a letter issued by FEMA that approves a proposed project. The letter states that the project will result in the specified changes to the base flood elevations, floodway elevations, floodplain limits, and floodway boundaries if constructed as shown.

The request for a CLOMR is made by the Community.

An ENCROACHMENT in the context of this manual is a placement of embankment fill or structure within the floodplain and/or floodway so as to affect or alter flow conditions.

A FLOOD INSURANCE RATE MAP (FIRM) is an official map of a community showing the delineation of the area Special Flood Hazard Area, along with insurance risk premium zones applicable to the community. Some FIRM's include contours of the Base Flood Elevations in areas where detailed hydraulic studies have been made.

The FLOODPLAIN is the land area inundated by the base flood. Also referred to as the SPECIAL FLOOD HAZARD AREA (SFHA).

The FLOODWAY is a portion of the floodplain that must be reserved in order to prevent activities that would cumulatively cause an increase in the base flood profile of more than a designated height. The designated height is never more than a foot, but in some communities can be less than a foot. Also referred to as the REGULATORY FLOODWAY or DESIGNATED FLOODWAY. This term applies only to floodplains within which a floodway has been officially established.

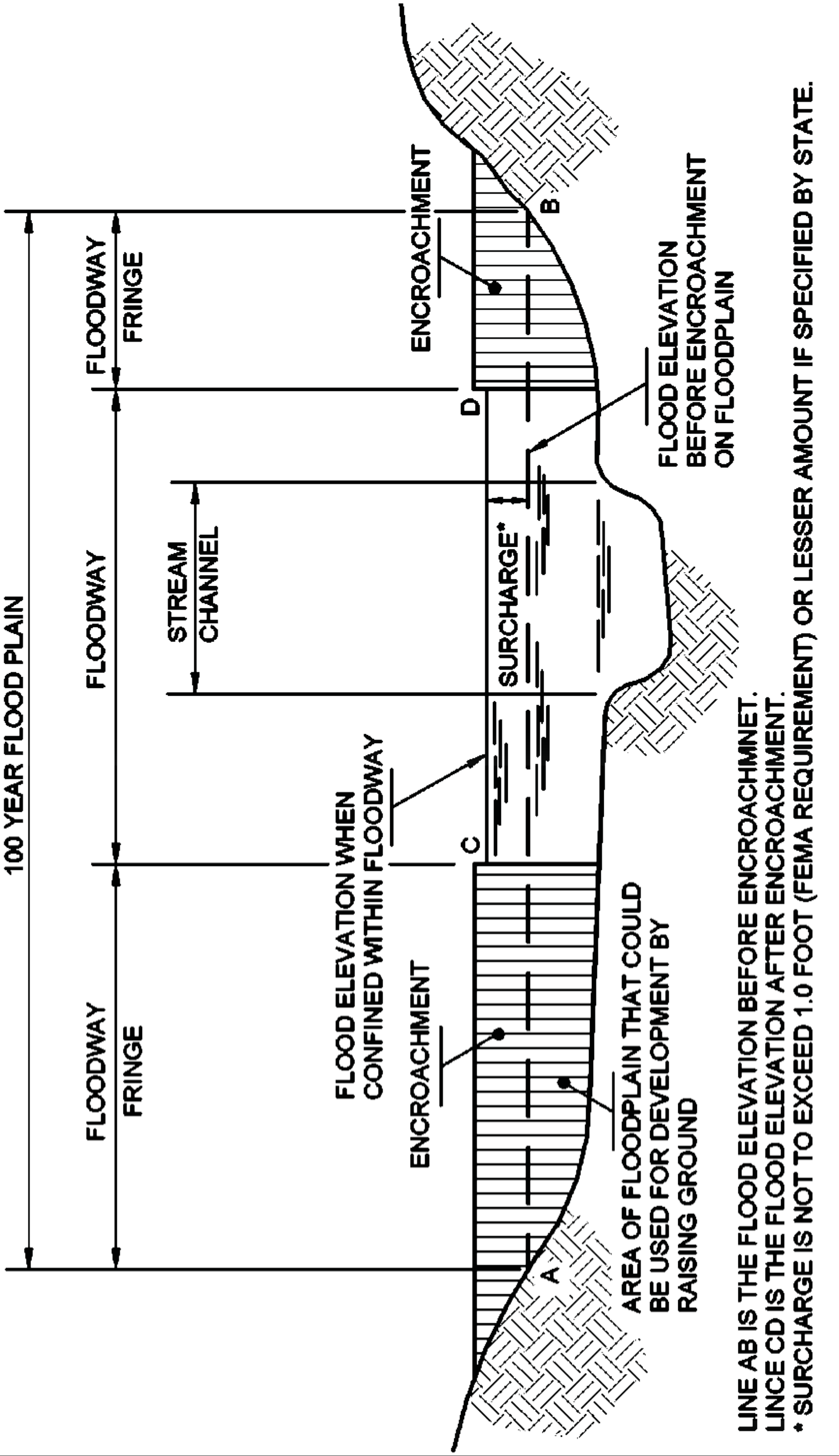
The FLOODWAY FRINGE is the portion of the floodplain that lies outside of the floodway. This term applies only to floodplains within which a floodway has been officially established.

A LETTER OF MAP REVISION (LOMR) is a letter issued by FEMA that revises the base flood elevations, floodway elevations, floodplain limits, and floodway boundaries for a given stream reach, based on documentation of changed or updated physical conditions. The request for a LOMR is made by the Community.

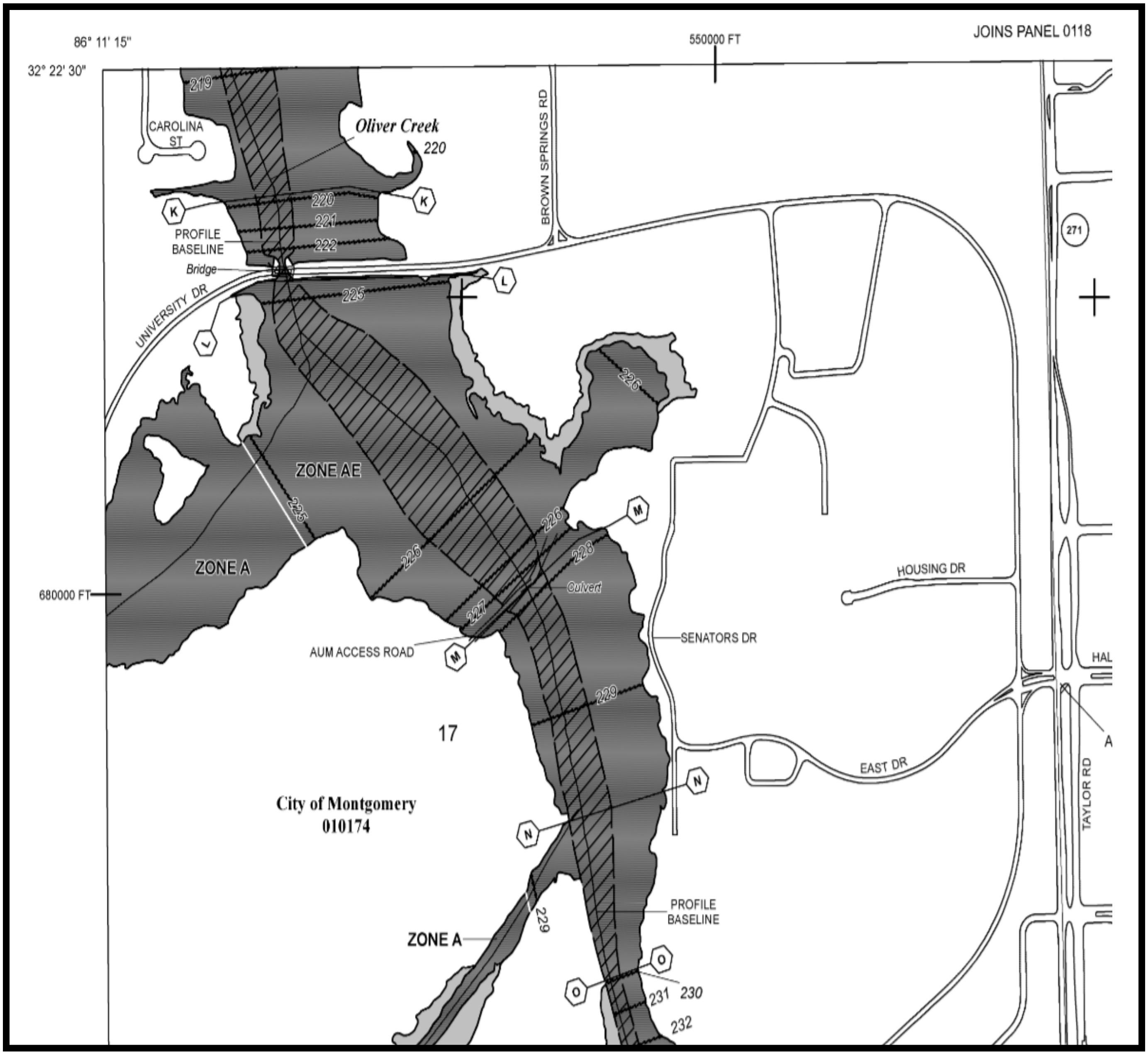
A NO-RISE certificate is a document submitted to the Community, with attached hydraulic computations, affirming that the proposed encroachment will not cause an increase in the base flood profile, the floodway width, or the floodway profile. See Appendix F for a sample.

The FLOODWAY ELEVATION is the water surface elevation resulting from encroachment in the floodplain to the designated floodway boundaries.

Section 3 - FEMA Floodway Encroachment Figure



Section 4 – Sample FEMA Floodway Map



Section 5 – Sample FEMA Floodway Table

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY (FEET NAVD)	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE (FEET)
Oliver Creek								
A	7,834[1]	134	887	6.0	171.6	168.6[2]	169.3	0.7
B	10,055[1]	831	1,926	2.2	171.6	171.4[3]	172.4	1.0
C	12,322[1]	78	698	6.0	178.0	178.0	178.6	0.6
D	14,025[1]	69	696	6.0	182.6	182.6	182.6	0.0
E	15,672[1]	181	749	4.4	188.0	188.0	188.1	0.1
F	17,830[1]	217	1,041	4.3	195.4	195.4	196.1	0.7
G	19,894[1]	45	566	8.0	200.8	200.8	201.4	0.6
H	20,533[1]	33	276	16.3	204.7	204.7	205.1	0.4
I	21,486[1]	155	931	4.1	212.4	212.4	213.4	1.0
J	23,717[1]	238	1,174	3.3	218.1	218.1	219.1	1.0
K	24,616[1]	180	758	5.1	220.1	220.1	221.1	1.0
L	24,983[1]	107	723	5.3	224.4	224.4	224.5	0.1
M	26,837[1]	254	813	3.1	227.6	227.6	228.5	0.9
N	27,858[1]	226	689	1.6	229.3	229.3	230.3	1.0
O	28,558[1]	80	161	6.7	229.9	229.9	230.2	0.3
P	29,430[1]	53	200	5.4	234.1	234.1	234.9	0.8
Q	29,953[1]	44	260	4.2	236.9	236.9	237.0	0.1
Oliver Creek Tributary								
A	203[4]	30	123	11.5	188.0	182.8	182.8	0.0
B	1,997[4]	27	127	11.2	191.2	191.2	191.2	0.0
C	2,354[4]	24	123	11.5	195.7	195.7	195.7	0.0
D	4,236[4]	31	106	10.3	204.5	204.5	204.5	0.0
E	4,886[4]	20	90	12.1	208.8	208.8	208.9	0.1
F	5,247[4]	30	151	8.1	214.1	214.1	214.4	0.3
G	6,777[4]	21	104	13.3	218.2	218.2	218.3	0.1
H	7,294[4]	16	87	12.5	220.5	220.5	221.1	0.6

[1]Stream distance in feet above confluence with Tallapoosa River
[2]Elevation computed without consideration of overflow effects from Tallapoosa River
[3]Elevation computed without consideration of backwater effects from Tallapoosa River
[4]Stream distance in feet above confluence with Oliver Creek

TABLE 7

FEDERAL EMERGENCY MANAGEMENT AGENCY

MONTGOMERY COUNTY, AL
AND INCORPORATED AREAS

FLOODWAY DATA

OLIVER CREEK – OLIVER CREEK TRIBUTARY

Section 6 – Sample No-Rise Certification Letter

Engineering "No-Rise" Certification

Catoma Creek

Bridge Replacement

U.S. Highway 331

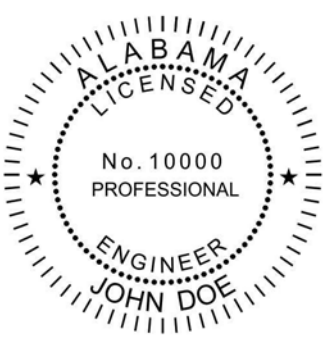
Montgomery County, Alabama

This is to certify that I am a duly qualified engineer licensed to practice in the State of Alabama. It is to further certify that the attached technical data supports the fact that the proposed construction of the replacement bridge over Catoma Creek at the U.S. Highway 331 crossing will not create any increase to the 100-year base flood elevations, floodway elevations, and floodway widths at published sections in the Flood Insurance Study for the City of Montgomery, Alabama, dated January 7, 2015 and will not create any increase in 100-year flood and floodway elevations and floodway widths at unpublished cross-sections in the vicinity of the project.

________________________ ______________________________

DATE SIGNATURE

SEAL:



Section 7 – Sample Letters to the Community and FEMA

ALABAMA DEPARTMENT OF TRANSPORTATION
1409 Coliseum Boulevard
Montgomery, Alabama 36110

Telephone: 334.242.6311

Kay Ivey
Governor

John R. Cooper
Transportation Director

(DATE)

Project ___________________________

PI No. ____________________________

(Name)
City or County Manager/Engineer (Note: The appropriate Community official varies)
(Address)

Dear_____________________________,

This project consists of the replacement of the existing XX ft wide by XX ft long bridge on __________________________________Over ______________________________with parallel XX ft wide by XX ft long bridges. This site crosses the regulatory floodway established for _________________located in Unincorporated_______________County.

The required HEC-RAS models along with supporting technical data for the proposed project is included in the attached documentation. The results show that the proposed construction will not increase the floodway widths or elevations from the existing conditions.

Included in this documentation for your use and files are:

1. A floodway map showing the location of the proposed site;
2. Tables showing the results of the floodway calculations;
3. A detailed explanation of the floodway calculations;
4. A preliminary bridge layout;
5. A set of roadway plans;
6. Hard copies of the required HEC-RAS models; and
7. A computer disk with the required HEC-RAS models.

The proposed bridge construction is consistent with the regulatory floodway at this site since the proposed construction will not increase the floodway widths or elevations from the existing conditions. In accordance with Section NS 23 CFR 650A of the Federal-Aid Policy Guide, coordination with FEMA will not be required.

A letter of concurrence from your community is required since this project crosses a regulatory floodway. Please review the enclosed information and send your letter of concurrence to this office at your earliest convenience.

This project is presently scheduled to be let to construction in __________________. If you have any questions and/or comments, please contact _____________________of the __________________________Office at telephone number ____________________.

Attachments

c:

ALABAMA DEPARTMENT OF TRANSPORTATION
1409 Coliseum Boulevard
Montgomery, Alabama 36110

Telephone: 334.242.6311

Kay Ivey
Governor

John R. Cooper
Transportation Director

(DATE)

(Name)
City or County Manager/Engineer (Note: The appropriate Community official varies)
(Address)

RE: Project No. __________
_________ County
PE Reference # 1000____
(Project Description)

Dear _________,

This project consists of the replacement of the existing 24 ft wide by 60 ft long bridge on ___ _ over _____ with parallel 38 ft wide by 100 ft long bridges. This site crosses the regulatory floodway established for ______ located in Unincorporated ___ County.

The required HEC-2 models along with supporting technical data for the proposed project is included in the attached documentation. The results show that the proposed construction will not increase the floodway widths or elevations from the existing conditions.

Included in this documentation for your use and files are:

1. A floodway map showing the location of the proposed site;
2. Tables showing the results of the floodway calculations;
3. A detailed explanation of the floodway calculations;
4. A preliminary bridge layout;
5. A set of roadway plans;
6. Hard copies of the required HEC-2 models; and
7. A computer disk with the required HEC-2 models.

The proposed bridge construction is consistent with the regulatory floodway at this site since the proposed construction will not increase the floodway widths or elevations from the existing conditions. In accordance with Section NS 23 CFR 650A of the Federal-Aid Policy Guide, coordination with FEMA will not be required.

A letter of concurrence from your community is required since this project crosses a regulatory floodway. Please review the enclosed information and send your letter of concurrence to this office at your earliest convenience.

Project # ________
________ County
Reference # 1000_____
(Date)

This project is presently scheduled to be let to construction in _____ . If you have any questions and/or comments, please contact ____ at (___) ___-____ or (email).

Sincerely,

____
(NAME)

Attachments

c:

ALABAMA DEPARTMENT OF TRANSPORTATION
1409 Coliseum Boulevard
Montgomery, Alabama 36110

Telephone: 334.242.6311

Kay Ivey
Governor

John R. Cooper
Transportation Director

(DATE)

(Name)
City or County Manager/Engineer (Note: The appropriate Community official varies)
(Address)

RE: Project No. __________
_________ County
PE Reference # 1000____
(Project Description)

Dear _________,

This project consists of the replacement of the existing 24 ft wide by 60 ft long bridge on ___ _ over _____ with a 38 ft wide by 130 ft long bridge. This site crosses the regulatory floodway established for ______ located in Unincorporated ___ County. The proposed bridge does not encroach horizontally or vertically on the existing regulatory floodway at this site.

Included in this documentation for your use and files are:

- A floodway map showing the location of the proposed site;
- The published floodway tables for the stream reach;
- A preliminary bridge layout; and
- A set of roadway plans.

Since the regulatory floodway width of 60 ft at the crossing site is cleared by the toe of endroll to toe of endroll width of the proposed 130 ft long bridge, and the 100-year floodway elevation is cleared by the proposed superstructure, there is no encroachment on the existing regulatory floodway.

The proposed bridge construction is consistent with the regulatory floodway at this site due to the bridging and excluding of fill from the existing floodway. In accordance with Section NS 23 CFR 650A of the Federal-Aid Policy Guide, coordination with FEMA will not be required.

A letter of concurrence from your community is required since this project crosses a regulatory floodway. Please review the enclosed information and send your letter of concurrence to this office at your earliest convenience.

Project # ________
________ County
Reference # 1000_____
(Date)

This project is presently scheduled to be let to construction in _____ . If you have any questions and/or comments, please contact ____ at (___) ___-____ or (email).

Sincerely,

____
(NAME)

Attachments

c:

ALABAMA DEPARTMENT OF TRANSPORTATION

1409 Coliseum Boulevard
Montgomery, Alabama 36110

Telephone: 334.242.6311

Kay Ivey
Governor

John R. Cooper
Transportation Director

(DATE)

(Name)
City or County Manager/Engineer (Note: The appropriate Community official varies)
(Address)

RE: Project No. __________
_________ County
PE Reference # 1000____
(Project Description)

Dear _________,

This project consists of the replacement of the existing 24 ft wide by 60 ft long bridge on ___ _ over _____ with a 38 ft wide by 130 ft long bridge. This site crosses the regulatory floodway established for ______ located in Unincorporated _________ County.

Due to several errors in the original ______ floodway run, several of the Flood Insurance Study published widths and elevations were found to be incorrect. Corrections to the original floodway model, along with the addition of four surveyed cross sections at the project site yielded the base floodway run. The proposed bridge and roadway were then inserted into the base floodway run, yielding the proposed floodway model. The results show that the proposed construction does not increase the floodway widths or elevations from the base run (corrected existing conditions).

Included in this documentation for your use and files are:

- A floodway map showing the location of the proposed site and the corrected floodway;
- The published floodway tables for the stream reach;
- Tables showing the results of the floodway calculations;
- A detailed explanation of the floodway calculations;
- A preliminary bridge layout;
- A set of roadway plans;
- Hard copies of the required floodway models; and
- A computer disk with the required floodway models.

As stated above, the results show that the published existing floodway is incorrect due to technical errors in the original model. The proposed bridge construction is consistent with the corrected regulatory floodway at this site since the proposed construction will not increase the floodway widths or elevations from the corrected existing conditions (base run). Since the proposed construction will have no impacts on the corrected existing floodway widths and elevations, in accordance with Section NS 23 CFR 650A of the Federal-Aid Policy Guide, ALDOT coordination with FEMA will not be required.

Project # ________
________ County
Reference # 1000_____
(Date)

A letter of concurrence from your community is required since this project crosses a regulatory floodway. Please review the enclosed information and send your letter of concurrence to this office at your earliest convenience.

his project is presently scheduled to be let to construction in _____ . If you have any questions and/or comments, please contact ____ at (___) ___-____ or (email).

Sincerely,

____
(NAME)

Attachments

c:

ALABAMA DEPARTMENT OF TRANSPORTATION

1409 Coliseum Boulevard
Montgomery, Alabama 36110

Telephone: 334.242.6311

Kay Ivey
Governor

John R. Cooper
Transportation Director

(DATE)

(Name)
City or County Manager/Engineer (Note: The appropriate Community official varies)
(Address)

RE: Project No. __________
_________ County
PE Reference # 1000____
(Project Description)

Dear _________,

This project consists of the replacement of the existing 24 ft wide by 60 ft long bridge on ___ _ over _____ with a 38 ft wide by 130 ft long bridge. This site crosses the regulatory floodway established for ______ located in Unincorporated _____ County.

The proposed construction at this site increases the floodway elevations at published sections A, B and C in excess of 0.1 ft. This construction does not cause more than a 1.0 ft rise in the existing 100-year base flood elevation. The existing floodway width at section B is increased from 150 to 200 ft.

Included in this documentation for your use and files are:

1. Floodway map showing the location of the proposed site and the corrected floodway
2. Published floodway tables for the stream reach
3. Tables showing the results of the floodway calculations
4. Detailed explanation of the floodway calculations
5. Preliminary bridge layout
6. Set of roadway plans
7. Hard copies of the required floodway models
8. Computer disk with the required floodway models

Please review the enclosed documentation, and if acceptable, a letter of concurrence from your community is required since this project crosses a regulatory floodway. Please send your letter of concurrence to the Alabama's Office of Water Resources with a copy to this office at your

Project # ________
________ County
Reference # 1000_____
(Date)

earliest convenience. OWR's address is listed below:

Alabama Office of Water Resources
Attn : MT-2 LOMC Coordinator
401 Adams Avenue
Montgomery, AL 36104

This project is presently scheduled to be let to construction in _____ . If you have any questions and/or comments, please contact ____ at (___) ___-____ or (email).

Sincerely,

____
(NAME)

Attachments

c:

ALABAMA DEPARTMENT OF TRANSPORTATION
1409 Coliseum Boulevard
Montgomery, Alabama 36110

Telephone: 334.242.6311

Kay Ivey
Governor

John R. Cooper
Transportation Director

(DATE)

(Name)
Alabama Office of Water Resources
401 Adams Avenue
Montgomery, AL 36104

Attn: MT-2 LOMC Coordinator

RE: Project No. __________
_________ County
PE Reference # 1000____
(Project Description)

Dear _________,

This project consists of the replacement of the existing 24 ft wide by 60 ft long bridge on ___ _ over _____ with a 38 ft wide by 130 ft long bridge. This site crosses the regulatory floodway established for ______ located in Unincorporated ___ County.

The proposed construction at this site increases the floodway elevations at published sections A, B and C in excess of 0.1 ft. This construction does not cause more than a 1.0 ft rise in the existing 100-year base flood elevation. The existing floodway width at section B is increased from 150 to 200 ft.

Included in this documentation for your use and files are:

1. Floodway map showing the location of the proposed site and the corrected floodway
2. Published floodway tables for the stream reach
3. Tables showing the results of the floodway calculations
4. Detailed explanation of the floodway calculations
5. Preliminary bridge layout
6. Set of roadway plans
7. Hard copies of the required floodway models
8. Computer disk with the required floodway models

A letter of concurrence for this project from (name of affected community) has been requested.

This project is presently scheduled to be let to construction in _____ . If you have any questions and/or comments, please contact ____ at (___) ___-____ or (email).

Project # ________
________ County
Reference # 1000_____
(Date)

Sincerely,

____
(NAME)

Attachments

c:

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Appendix C: Designer's Checklist for Project Documentation

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APPENDIX C

Roadway Design/Region Checklist
For
Project Documentation
Check All Appropriate Items

Date: ______________________________

Project number: ______________________

County: ____________________________

PI number: __________________________

Design Office: ______________________

Consultant: _________________________

Designer: __________________________

Let date: ___________________________

Reference Data

Maps
☐USGS quad map
☐DOT map
☐Local zoning map
☐Flood hazard delineation
☐Floodplain delineation
☐Local land use
☐Soils map
☐Geological map
☐Aerial photos

External Agency Studies
☐USACE floodplain info
☐NRCS watershed studies
☐Local watershed mgmt.
☐USGS gages & studies
☐Interim floodplain studies
☐Water resource data
☐Regional planning data
☐Forestry service
☐Utility company plans

Internal Source Studies
☐Quarterly reports
☐Hydraulics section records
☐District drainage records
☐Flood records

Hydraulic Design

Hydraulic Design
☐ Calibration of Highwater Data
☐ Discharge and frequency of highwater elevation
☐ Influences responsible for highwater elevation
☐ Analyze hydraulic performance of existing facility for minimum flow through (100 year)
☐ Analyze hydraulic performance of proposed facility for minimum flow through (100 year)

Design Appurtenances
☐ Dissipators
☐ Riprap
☐ Erosion & sediment control
☐ Fish & wildlife

Technical Aids
☐ ALDOT Drainage Manual
☐ ALDOT and FHWA Directives

Computer Programs
☐ USACE HEC-RAS
☐ HY-8
☐ WSPRO
☐ Visual Urban
☐ HYDRAIN
☐ _______________
☐ _______________
☐ _______________
☐ _______________
☐ _______________

Appendix D: Manning's Tables

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Manning's “n” for Closed Conduits Flowing Partly Full

Type of Channel and Description	Minimum	Normal	Maximum
Brass, Smooth	0.009	0.010	0.013
Steel			
Lockbar and welded	0.010	0.012	0.014
Riveted and spiral	0.013	0.016	0.017
Cast Iron			
Coated	0.010	0.013	0.014
Uncoated	0.011	0.014	0.016
Wrought Iron			
Black	0.012	0.014	0.015
Galvanized	0.013	0.016	0.017
Corrugated Metal			
Subdrain	0.017	0.019	0.021
Storm Drain	0.021	0.024	0.030
Lucite	0.008	0.009	0.010
Glass	0.009	0.010	0.013
Cement			
Neat Surface	0.010	0.011	0.013
Mortar	0.011	0.013	0.015
Concrete			
Culvert, straight and free of debris	0.010	0.011	0.013
Culvert with bends, connections, and some debris	0.011	0.013	0.014
Finished	0.011	0.012	0.014
Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
Unfinished, steel form	0.012	0.013	0.014
Unfinished, smooth wood form	0.012	0.014	0.016
Unfinished, rough wood form	0.015	0.017	0.020
Wood			
Stave	0.010	0.012	0.014
Laminated, treated	0.015	0.017	0.020
Clay			
Common drainage tile	0.011	0.013	0.017
Vitrified sewer	0.011	0.014	0.017
Virtified sewer with manholes, inlet, etc.	0.013	0.015	0.017
Virtified Subdrain with open joint	0.014	0.016	0.018
Brickwork			
Glazed	0.011	0.013	0.015
Lined with cement mortar	0.012	0.015	0.017
Sanitary sewers coated with sewage slime with bends and connections	0.012	0.013	0.016
Paved invert, sewer, smooth bottom	0.016	0.019	0.020
Rubble masonry, cemented	0.018	0.025	0.030

[Chow, 1959] Chow, V.T., 1959, Open Channel Hydraulics, McGraw-Hill Book Company, NY.

Manning's “n” for Corrugated Metal Pipe

Type of Pipe and Diameter	Unpaved	25% Paved	Fully Paved
Annular 2.67 x 2 in. (all diameters)	0.024	0.021	0.021
Helical 1.50 x 1/4 in.:			
8 inch diameter	0.012		
10 inch diameter	0.014		
Helical 2.67 x 2 in.:			
12 inch diameter	0.011		
18 inch diameter	0.014		
24 inch diameter	0.016	0.015	0.012
36 inch diameter	0.019	0.017	0.012
48 inch diameter	0.020	0.020	0.012
60 inch diameter	0.021	0.019	0.012
Annular 3 x 1 in. (all diameters)	0.027	0.023	0.012
Helical 3 x 1 in.:			
48 inch diameter	0.023	0.020	0.012
54 inch diameter	0.023	0.020	0.012
60 inch diameter	0.024	0.021	0.012
66 inch diameter	0.025	0.022	0.012
72 inch diameter	0.026	0.022	0.012
78 inch & larger	0.027	0.023	0.012
Corrugations 6 x 2 in.:			
60 inch diameter	0.033	0.028	
72 inch diameter	0.032	0.027	
120 inch diameter	0.030	0.026	
180 inch diameter	0.028	0.024	

[AISI, 1980] American Iron and Steel Institute (AISI), 1980. Modern Sewer Design, Washington D.C.

*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.

Manning's "n" for PE and PVC Pipe

Type of Pipe and Description	Minimum	Maximum
Corrugated Polyethylene (PE) with smooth inner walls [a,b]	0.009	0.015
Corrugated Polyethylene (PE) with corrugated inner walls [c]	0.018	0.025
Polyvinyl Chloride (PVC) with smooth inner walls [d,e]	0.009	0.011

[a] Barfuss, Steven and J. Paul Tullis. Friction factor test on high density polyethylene pipe. Hydraulics Report No. 208. Utah Water Research Laboratory, Utah State University. Logan, Utah. 1988.

[b] Tullis, J. Paul, R.K. Watkins, and S. L. Barfuss. Innovative new drainage pipe. Proceedings of the International Conference on Pipeline Design and Installation, ASCE. March 25-27, 1990.

[c] Barfuss, Steven and J. Paul Tullis. Friction factor test on high density polyethylene pipe. Hydraulics Report No. 208. Utah Water Research Laboratory, Utah State University. Logan, Utah. 1994.

[d] Neale, L.C. and R.E. Price. Flow characteristics of PVC sewer pipe. Journal of the Sanitary Engineering Division, Div. Proc 90SA3, ASCE. pp. 109-129. 1964.

[e] Bishop, R.R. and R.W. Jeppson. Hydraulic characteristics of PVC sewer pipe in sanitary sewers. Utah State University. Logan, Utah. September 1975.

Manning's 'n' Values for Channels

Type of Channel and Description	Minimum	Normal	Maximum
A. Natural Streams			
1. **Main Channels**			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.070	0.100	0.150
2. Flood Plains			
a. Pasture no brush			
1) Short grass	0.025	0.030	0.035
2) High grass	0.030	0.035	0.050
b. Cultivated areas			
1) No crop	0.020	0.030	0.040
2) Mature row crops	0.025	0.035	0.045
3) Mature field crops	0.030	0.040	0.050
c. Brush			
1) Scattered brush, heavy weeds	0.035	0.050	0.070
2) Light brush and trees, in winter	0.035	0.050	0.060
3) Light brush and trees, in summer	0.040	0.060	0.080
4) Medium to dense brush, in winter	0.045	0.070	0.110
5) Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1) Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2) Same as above, but heavy sprouts	0.050	0.060	0.080
3) Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4) Same as above, but with flow into branches	0.100	0.120	0.160
5) Dense willows, summer, straight	0.110	0.150	0.200
3. Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

[Chow, 1959] Chow, V.T., 1959, Open Channel Hydraulics, McGraw-Hill Book Company, NY.

(Continued) Manning's 'n' Values

Type of Channel and Description	Minimum	Normal	Maximum
B. Lined or Built-Up Channels			
1. Concrete			
a. Trowel finish	0.011	0.013	0.015
b. Float Finish	0.013	0.015	0.016
c. Finished, with gravel bottom	0.015	0.017	0.020
d. Unfinished	0.014	0.017	0.020
e. Gunite, good section	0.016	0.019	0.023
f. Gunite, wavy section	0.018	0.022	0.025
g. On good excavated rock	0.017	0.020	
h. On irregular excavated rock	0.022	0.027	
2. Concrete bottom float finished with sides of:			
a. Dressed stone in mortar	0.015	0.017	0.020
b. Random stone in mortar	0.017	0.020	0.024
c. Cement rubble masonry, plastered	0.016	0.020	0.024
d. Cement rubble masonry	0.020	0.025	0.030
e. Dry rubble on riprap	0.020	0.030	0.035
3. Gravel bottom with sides of:			
a. Formed concrete	0.017	0.020	0.025
b. Random stone in mortar	0.020	0.023	0.026
c. Dry rubble or riprap	0.023	0.033	0.036
4. Brick			
a. Glazed	0.011	0.013	0.015
b. In cement mortar	0.012	0.015	0.018
5. Metal			
a. Smooth steel surfaces	0.011	0.012	0.014
b. Corrugated metal	0.021	0.025	0.030
6. Asphalt			
a. Smooth	0.013	0.013	
b. Rough	0.016	0.016	
7. Vegetal lining	0.030		0.500

[Chow, 1959] Chow, V.T., 1959, Open Channel Hydraulics, McGraw-Hill Book Company, NY.

(Continued) Manning's 'n' Values

Type of Channel and Description	Minimum	Normal	Maximum
C. Excavated or Dredged Channels			
1. Earth, straight and uniform			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. Earth, winding and sluggish			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble side	0.028	0.030	0.035
e. Stony bottom and weedy banks	0.025	0.035	0.040
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline-excavated or dredged			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. Rock cuts			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. Channels not maintained, weeds and brush			
a. Clean bottom, brush on sides	0.040	0.050	0.080
b. Same as above, highest stage of flow	0.045	0.070	0.110
c. Dense weeds, high as flow depth	0.050	0.080	0.120
d. Dense brush, high stage	0.080	0.100	0.140

[Chow, 1959] Chow, V.T., 1959, Open Channel Hydraulics, McGraw-Hill Book Company, NY.

Appendix E: FHWA Culvert Design Form and Permissible Velocity Tables

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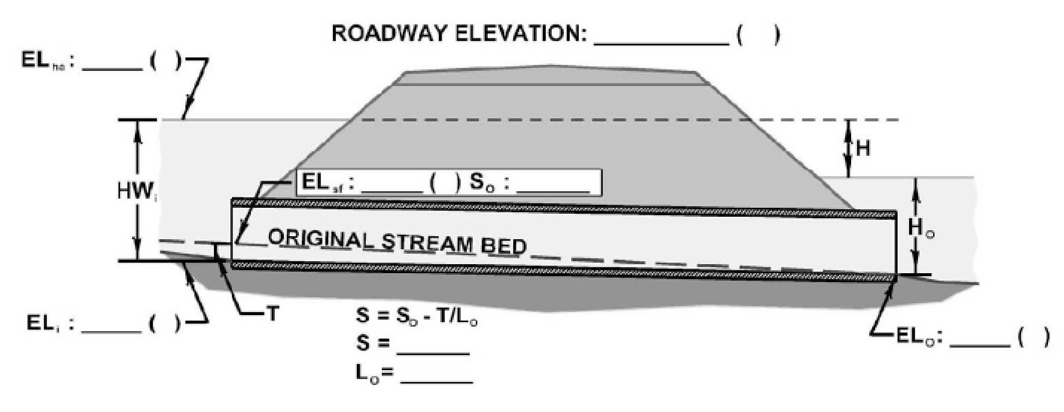
PROJECT:	STATION: ____________________	**CULVERT DESIGN FORM**
	SHEET ________ OF __________	DESIGNER / DATE: __________ / ________
		REVIEWER / DATE: /

HYDROLOGICAL DATA

See Add'l Shts.

- ☐ METHOD: ______________________________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: __________
- ☐ CHANNEL SHAPE: ______________________________
- ☐ ROUTING: ____________ ☐ OTHER: ______________



DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

CULVERT DESCRIPTION: MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	HEADWATER CALCULATIONS: INLET CONTROL				OUTLET CONTROL							Control Headwater Elevation	Outlet Velocity	Comments
			HW_i/D (2)	HW_i	T (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW_i / D = HW / D OR HW_f / D FROM DESIGN CHARTS

(3) $T = HW_i - (EL_{hd} - EL_{sf})$
T IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or $(d_c + D)$ /2 (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (K_u n^2 L) / R^{1.33}] v^2 / 2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
ha. ALLOWABLE HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____________________

SHAPE: ____________________

MATERIAL: __________ n __________

ENTRANCE: ____________________

Maximum permissible velocities in erodible channels, based on uniform flow in continuously wet, aged channels[1]

	Maximum permissible velocities for		
	Clear Water (fps)	Water Carrying Fine Silts (fps)	Water Carrying Sand and Gravel (fps)
Fine sand (non-colloidal)	1.5	2.5	1.5
Sandy loam (non-colloidal)	1.7	2.5	2.0
Silt Loam (non-colloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic Ash	2.5	3.5	2.0
Fine Gravel	2.5	5.0	3.7
Stiff Clay (very colloidal)	3.7	5.0	3.0
Graded loam to cobbles (non-colloidal)	3.7	5.0	5.0
Graded silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial Silts (non-colloidal)	2.0	3.5	2.0
Alluvial Silts (colloidal)			
Coarse Gravel (non-colloidal)	4.0	6.0	5.5
Cobbles and Shingles	5.0	5.5	6.5
Shales and Hard Pans	6.0	6.0	5.0

[1]As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926, for channels with straight alignment. For sinuous channels multiply allowable velocity by 0.95 for slightly sinuous, by 0.9 for moderately sinuous channels and by 0.8 for highly sinuous channels.

Maximum permissible velocities in channels lined with uniform stands of various grass covers, well maintained[1, 2]

	Maximum permissible velocities on		
	Slope Range Percent	Erosion Resistant Soils (fps)	Easily Eroded Soils (fps)
Bermuda grass	0-5	8	6
	5-10	7	5
	Over 10	6	4
Buffalo grass	0-5	7	5
Kentucky bluegrass	5-10	6	4
Smooth brome	Over 10	5	3
Grass Mixture	0-5[2]	5	4
	5-10[3]	4	3
Lespedeza Sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0-5[3]	3.5	2.5
Common Lespedeza[5]	0-5[4]	3.5	2.5

[1] From *Handbook of Channel Design for Soil and Water Conservation*.

[2] Use velocities over 5 fps, only where good covers and proper maintenance can be obtained.

[3] Do not use on slopes steeper than 10 percent.

[4] Not recommended on slopes steeper than 5 percent.

[5] Annuals, used on mild slopes or as temporary protection until permanent covers are established.

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Appendix F: HYD Forms

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Alabama Department of Transportation
Design Bureau
Location Information – Office

Project No: ____________________________ Date: ____________________________
Division: ______________ County: ______________ Prepared By: ______________
Section: ______________ Township: ______________ Range: ______________
Over (River, Creek, Branch, Ditch): ____________________________
Highway or Road No. ______________ Station No. ______________

A. Flood Studies

1. Any flood zoning (FIS, etc.)? Yes ________ No ________
2. Type of Study: ____________________________
3. Comments: ____________________________
4. Governing community has policy or guideline: Yes ______________ No ______________
   Comments: ____________________________

B. Environmental Considerations

1. List commitments in environmental documents which affect hydraulic design. None __________, or
   Comments: ____________________________

C. Traffic Related Evaluations

1. Present Year: ____________ Traffic Count: __________ A.D.T. % Trucks ____________
2. Design Year: ____________ Traffic Count: __________ A.D.T. % Trucks ____________
3. Emergency Route: __________ School Bus Route: ____________ Mail Route: ____________
4. Detour Available: Yes ________ No ________ Length of Detour: ____________
5. Design Speed: ____________ 6. Can Route be Closed? Yes ________ No ____________
6. Comments: ____________________________
7. Interstate, Freeway, Arterial, Collector, Local, Other. (Please Circle One)
   Comments: ____________________________

8. Existing Roadway:

(a) Pavement Type & Width: ________________ (b) Shoulder Type & Width: ____________________

(c) Curb & Gutter: Yes _________ No _________ (d) No. Lanes: ___________________________

(e) Median: Yes _________ No _________ Type & Width of Median: ________________________

(f) Description of Existing:

____

____

____

(g) Total Roadway Width: ________________ Ft.

9. Other Remarks:

____

____

____

____

____

____

____

____

____

Alabama Department of Transportation
Section I.
Location Information – Field Party

Project No: ________________________________ Date: ________________________________

Division: __________________________________ Prepared By: __________________________
Chief of Survey Party

CPMS No: ______________________________

County: _____________ Section: _________ Township: __________ Range: __________

State Highway, or County Road No: ___________________________ Station: _____________

Over: (River ☐, Creek ☐, Branch ☐, or Drainage Ditch ☐)

(If a named River, Stream, or Tributary, indicate Name):

____

A. Description of Stream Channel

1. Natural Stream Bottom Slope: ______________________________ ft/ft.

2. Material in Stream Bottom: (Check All that Apply)

   Mud ☐ Silt ☐ Clay ☐ Sand ☐ Gravel ☐ Cobbles ☐

   Boulders ☐ Soft Solid Rock ☐ Stratified Rock ☐ Hard Rock ☐

   Silt Sedimentation ☐ Deposition of Large Stones ☐

3. Material in Stream Banks: (Check All that Apply)

   Mud ☐ Silt ☐ Clay ☐ Sand ☐ Gravel ☐ Cobbles ☐

   Boulders ☐ Soft Solid Rock ☐ Stratified Rock ☐ Hard Rock ☐

   Silt Sedimentation ☐ Deposition of Large Stones ☐

4. Are Fish or Aquatic Organisms visible? Yes ☐ No ☐

5. Are Banks Scouring? Yes ☐ No ☐

   In Which Direction? Upstream ☐ Downstream ☐

6. Material in Flood Plain: (Check All that Apply)

   Mud ☐ Silt ☐ Clay ☐ Sand ☐ Gravel ☐ Cobbles ☐

   Boulders ☐ Soft Solid Rock ☐ Stratified Rock ☐ Hard Rock ☐

   Silt Sedimentation ☐ Deposition of Large Stones ☐

7. Is Bottom Aggrading (filling)? ☐ Degrading (Deepening)? ☐

8. Vegetation in and Along Channel:

____

9. Vegetation in Flood Plain:

____

10. Presence of Features that Might Affect Discharge, Flood, Tailwater, or Headwater Elevations:

a. Levees:

____

b. Upstream Diversions (e.g. Sinkhole, millrace, irrigation channel):

____

c. Backwater from Another Source: (Check One)

Stream ☐ Pipe or Culvert ☐ Bridge ☐ Dam ☐

Other ☐ None ☐

(If Other, List):

____

List Backwater Elevation: ______________ Date Measured: ___________________

d. Other Observed Features:

____

e. Does stream carry a large amount or accumulation of sand, or debris (fragments of rock, driftwood, etc.)?

____

f. Downstream drainage structures which can affect Tailwater: (List All)

(If descriptions of additional structures are required, attach copies of this section.)

1) Type of Structure: ____________ 2) Distance Downstream: ____________

3) Condition of Structure: Good ☐ Fair ☐ Poor ☐

4) Size: ____________________ 5) Material: __________________________

6) Inlet Flowline: ____________ 7) Outlet Flowline: ____________________

g. Other Influences:

____

B. Existing Structures

1. Is Scour indicated near structure? Yes ☐ No ☐

2. Alignment and General Description of Structure:

   a. Skew Angle: ____________ Lt. ☐ (or) Rt. ☐; Ahead ☐, (or) Back ☐

   b. Shape: Circular pipe ☐ Arch pipe ☐ Other ☐ Box Culvert ☐ Bridge ☐

      (If Other, Describe): ____

   c. Material: Concrete ☐ Steel/Aluminum ☐ Brick & Mortar ☐

      Stone ☐ Plastic ☐

      If Steel/Aluminum, or Plastic: Smooth ☐ Corrugated ☐

   d. Size or Waterway Opening of Structure:

      Span: ____________ Rise: ____________

   e. Condition of Structure: Good ☐ Fair ☐ Poor ☐

3. Elevation of:

   a. Low Superstructure (Bridge): __________________

   b. Crown of Pipe: ______________________________

   c. Inside Top of Culvert: _________________________

   d. Flowlines of:

      i) Pipe:

      Inlet: _____________ Outlet: ______________

      ii) (Bridge or Culvert):

      Indicate material: Natural Channel Bottom ☐ (or) Structure ☐

      Inlet: _____________ Outlet: ______________

4. Is a Dissipater Present? Yes ☐ No ☐

   If yes, Indicate Type: Riprap ☐ Concrete ☐ Other ☐

   If Other, Describe: ____

5. Overtopping Location and Elevation: (Choose Only One)

Roadway ☐ Ditch Berm ☐ Watershed Divide ☐ Emergency Relief Structure ☐

Station _____________ Elevation: _____________

6. Roadway Width:

Shoulder-Shoulder ☐ (or) Curb-Curb ☐ : _____________ Ft.

7. Are <u>any</u> parts of the Existing Roadway Fill sections acting as a dam for standing water?

Yes ☐ No ☐

8. Centerline Elevation of Structure at Centerline of Stream: __________________

C. Property Susceptible to Flooding

(If descriptions of additional structures are required, attach copies of this section.)

1. Location of Structure:
   a. Station: __________________________
   b. Type of Structure: _________________
   c. Description: ____

2. Floor Elevation: ________________________

3. Upstream Land Use:

____

4. Downstream Land Use:

____

5. Probable (Anticipated) Changes:

____

D. Historical Highwater (H.W.) or Flood Information

(Please record more than one source if information can be obtained.)

1. Source of Information:

____

____

____

2. Elevation of H.W.

Indicate: Field Measurement ☐ (or) Flood Information ☐: Elev.______________________

Source: ____

3. Date of H.W., Flood, or Floods (if it can be determined):______________________

4. Estimated Allowable H.W.: ______________________

5. Damage from Previous Floods (if available):

____

____

____

Closing Remarks: ____

____

____

Alabama Department of Transportation
Preliminary Risk Assessment For
Flood Plain Encroachment

Project No: ________________________________ Date: ______________________
Division: ___________________ Highway or Road No. ________ Station No. ________________
County: _____________ Section: _____________ Township: ___________ Range: ______________
Over River □, Creek □, Branch □, Ditch □ ____
Name of Stream: ____

1.1 Approximate Fill Height ____ *feet*

1.2 Estimated Structure Cost $ ____

1.3 (a) Overtopping: Elevation ____________ Discharge _______________ cfs
Return Period ________ yrs.

2.0 Are significant embankment/pavement repair costs likely at this location?
Yes __________ (explain below)
No ___________ (proceed)
Unsure (explain below)

Explanation: ____
____
____
____
____

3.0 Risk of Flood Damages:

3.1 Are there significant flood damages prior to design?
Yes __________ (explain below)
No ___________ (proceed)
Unsure (explain below)

Explanation: ____
____
____
____
____

3.2 Are there significant flood damages after design?
Yes __________ (explain below)
No ___________ (proceed)
Unsure (explain below)

Explanation: ____
____
____
____
____

4.0 Are there any additional factors to be considered in the assessment process?

Yes __________ (explain below)
No ___________ (proceed)
Unsure _ (explain below)

Explanation: ____

5.0 Are risks significant in relation to capital costs? (Adjacent property, structures, etc.).

Yes __________ (explain below)
No ___________ (proceed)
Unsure _ (explain below)

Explanation: ____

6.0 Should risk analysis be investigated?

Yes __________ (explain below)
No ___________ (proceed)
Unsure _ (explain below)

Explanation: ____

6.1 Mark out the statements below to indicate your opinion:

The risks associated with this encroachment are/are not acceptable.
Capital costs are/are not excessive.
Further studies involving risk are/are not necessary.

Name of Person Completing Form: _________________________ Date: _______________
Person Reviewing Form: ___________________________________ Date: _______________

Alabama Department of Transportation
Request for (Design) (Approval) Secondary Road System

Location

1. County________________ Sec. _________ T'ship _________ Range _________ Proj. No. _____________
2. Over (River, Creek, Branch, Ditch) ____
3. Road System ________________ Sta. Existing Struct. ___________ Sta. Proposed Struct. ______________

General Data

4. Drainage area ____________ Character ________________ Approx. length & Width __________________
5. Extreme highwater: Date of occurrence _______________ Info from _________________________________
   Elev. Near site ________________ Elev. Downstream ______________ Elev. Upstream ________________
   (Water elev. ________________ on date of survey ________________) Fall in stream ________________ ft.
6. List buildings in flood plain _______________ Location _______________ Floor Elev. __________________
7. Is excessive local scour probable? Yes _______ No _______ Explain _________________________________
   Probable max. depth of scour below streambed? ____
8. Is stream deepening or filling? _________________________ Approx. amount per year ________________
9. Is stream widening? ____ (Show direction, rate, & amount)
10. Does the stream carry appreciable amount of large driftwood? ____________________________________
    ____
11. Bench mark No., Location, Elevation ____

Present or Old Structure

12. Superstructure Type ____ Skew angle ________________
13. Substructure Type ____
14. Span lengths _______________________ Roadway width ______________ Type of Floor ________________
15. Grade elev. At entr. ____ Date built _________________
16. Condition of superstructure ____
17. Condition of substructure ____
18. Remarks:

    ____
    ____
    ____
    ____
    ____
    ____
    ____
    ____

Proposed Structure

19. Superstructure: Type ____ Skew angle _____________
20. Substructure: Type _______________________________ List fill if RCB or Pipe _________________
21. Span lengths: _______________________________ Total Length ___________________________
22. Bridge width ________ Approach Roadway: Width & type of surface ___________ SH to SH _______
23. Grade elev. _____________ Abutment footing elev. ____________ Pier footing elev. _____________
24. Length and type of piling: Abutments _______________________ Piers _______________________
25. Design highwater: Elev. ________ Frequency ________ Yrs.; Bridge Waterway Area _________ sq. ft.
    Discharge _______________ cfs
26. What provision is made for overflow? ____
27. Can channel be cleared to provide more waterway? _______________________________________
28. Disposition of existing structure ____
29. Traffic count __________ ADT Year of ADT _________ ADT Estimated ____ or Observed ________
30. Remarks:

____

____

Notes and recommendations by: ______________________________ Date: ______________________

County Engineer

The submittal of a bridge type structure will include a right angle valley section. This section should be taken downstream from the crossing. It will be noted whether it is an average section or a control section. Enough ground shots will be taken to outline the valley to an elevation well above extreme highwater. Special care will be taken to accurately outline the main channel. Each shot will be identified; that is (Fp) flood plain, (TB) top of bank, (ES) edge of stream, etc. Suggest Manning's coefficients and photos of the channel and flood plain.

Remarks:
____
____
____
____
____

Plat of Drainage Area

The drainage area is to be platted as completely and accurately as possible and to the largest practicable scale on a separate sheet. Use a defined scale, as 1" equals ¼, ½, 1, or 2 miles, and indicate what scale has been used. In addition to the outlines of the watershed, indicate the positions of the streams and, roughly, the character of the soil, and the relative locations of the steep and flat portions. Whenever practicable, the above information should be secured by going over the area either on foot, or in a car. For most watersheds, the information may be secured from the best existing data, soil map, U.S.G.S. maps, and Bulletin No. 7-1.H.R.B.

Remarks:
____
____
____
____

Give additional information by reference to Marginal number on other sheet.
____
____

Marginal No.
____
____
____
____
____
____

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Appendix G: Rational Method Example and USGS Alabama Hydrograph Method

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Hydrologic Calculations

Rational Method Example

Example of a rational equation computation for a small rural watershed on CR- 12 west of Newtonville in Fayette County:

The drainage area is measured as 17.5 acres on the Newtonville topographic map on Terrain Navigator Pro as a route. The route was then translocated to the web soil survey by the directions above. The soil map unit legend shows three map units over the drainage area with the number of acres of each. The three map units are subdivided into an association and two complexes so the percentage of effective area of the major soils were evaluated (the minor components are small and are not significant). The AOI inventory report gives further data such as the soil profiles and HSG's of each soil group. Although the area is mostly wooded, the rolling and hilly slopes with HGS's of C and D gave a soils C value of 0.47

The impervious area is essentially half the roadway width times the length or

$$1130 \text{ ft} \times 22/2 \text{ ft} = 12430 \text{ ft}^2 = 0.285 \text{ acres.}$$

The impervious area, although small, was enough to raise the C value on this small watershed from 0.47 to 0.48.

Two flow paths were possible candidates for the longest t_c:

- One from the left peak had a flowtime of 6.56 mins
- One from the right peak had a flowtime of 6.98 mins

Which will result in the t_c = 6.98 mins for the drainage area.

The return frequency for a county road with an Average Daily traffic (ADT) of 400 or greater is normally 25 years, and the check frequency is 50 years. The Tuscaloosa rainfall gage is used as the reference station.

For 25 year return period: $I = 26.27/\ (t_c + 1.40)^{0.5429} = 8.28$ in. /hr

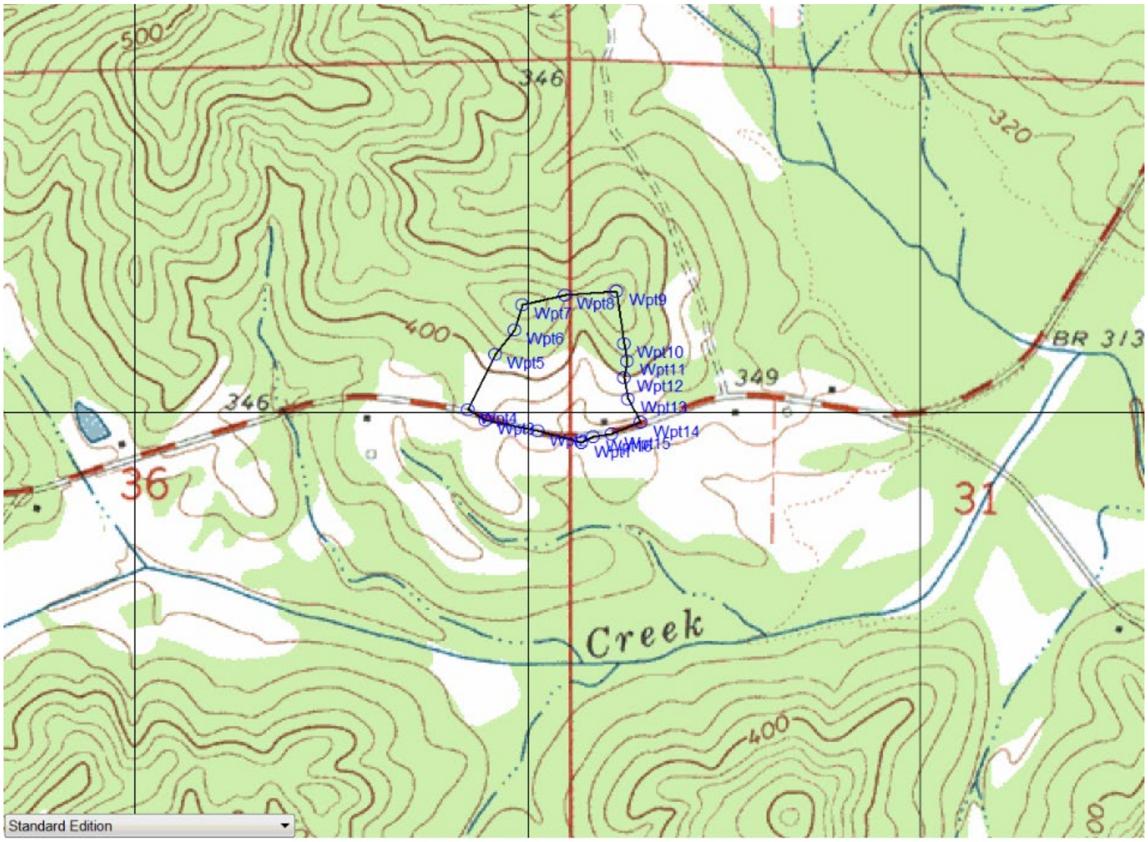
$$Q_{25} = CIA = 0.48 \times 8.28 \times 17.5 = 69.6 \text{ cfs}$$

The computer spreadsheets for example 1 for the 25 year storm shows an intensity of 8.29 in. / hr and a Q_{25} of 69.6 cfs.

For the 50 year return period: $I = 29.02/\ (t_c + 1.42)^{0.54160} = 9.16$ in. / hr

$$\boxed{Q_{50} = CIA = 0.48 \times 9.16 \times 17.5 = 77.0 \text{ cfs}}$$

Area of Interest drawn as route on TNP



Soil Map from WSS

Map Unit Legend

Fayette County, Alabama (AL057)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
RtE	Smithdale-Luverne association, 12 to 35 percent slopes	8.2	47.1%
ShC2	Shubuta-Boswell complex, 6 to 10 percent slopes, eroded	4.7	27.0%
ShD2	Shubuta-Boswell complex, 10 to 15 percent slopes, eroded	4.5	25.9%
Totals for Area of Interest		**17.5**	**100.0%**

Soil Map Unit Descriptions from WSS

Fayette County, Alabama

RtE—Smithdale-Luverne association, 12 to 35 percent slopes

Map Unit Setting

National map unit symbol: 2sh4l
Elevation: 250 to 500 feet
Mean annual precipitation: 48 to 69 inches
Mean annual air temperature: 51 to 70 degrees F
Frost-free period: 230 to 290 days
Farmland classification: Not prime farmland

Map Unit Composition

Smithdale and similar soils: 50 percent
Luverne and similar soils: 35 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Smithdale

Setting

Landform: Hillslopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Loamy fluviomarine deposits derived from sedimentary rock

Typical profile

A - 0 to 6 inches: fine sandy loam
Bt1 - 6 to 45 inches: sandy clay loam
Bt2 - 45 to 85 inches: sandy loam

Properties and qualities

Slope: 12 to 35 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.60 to 2.00 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Available water storage in profile: High (about 9.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: B
Hydric soil rating: No

Description of Luverne

Setting

Landform: Hillslopes

Hydro 13 Composite C Worksheet

Runoff Coefficient C

For map units with multiple major components

Project No.:	Example 1			
County	Fayette	Latitude °	Longitude °	Elev. (ft)
Location	Newtonville	33.5329	87.8407	354
Highway	Co Rd 12			
Station				
Date Run	9/27/2016			
Run By	dr			

Soil C estimation

A. Soil C value (use soil survey map)

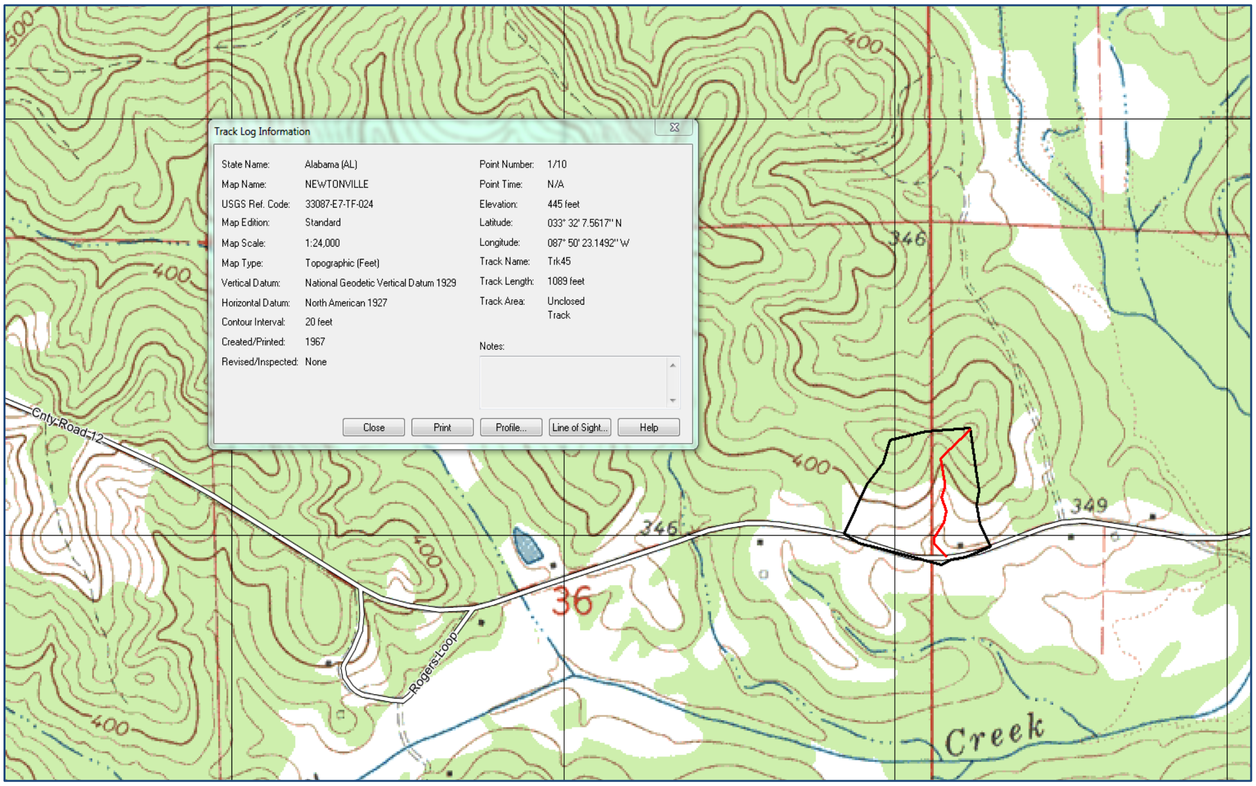
Soil map code	Soil map area (A) acres	Subarea (A_s) acres	Land Cover	Soil Description	Slope %	Hydrologic soil group	Soil group C value	line CxA acres	Composite Soil C ∑(CxA)/A)
RtE	8.20		woods	Smithdale-Luverne association	10 to 35	B & C			
		4.10	woods	50% Smithdale	10 to 35	B	0.40	1.640	
		2.87	woods	35% Luverne	10 to 35	C	0.55	1.579	
ShC2	4.70		woods	Shubuta-Boswell complex, eroded	6.to 10	C & D			
		2.82	woods	60% Shubuta (luverne) and similar soils	6.to 10	C	0.40	1.128	
		0.71	woods	15% Boswell and similar soils	6.to 10	D	0.50	0.355	
ShD2	4.50		woods/grass	Shubuta-Boswell complex, eroded	10 to 15	C & D			
		2.25	woods/grass	50% Shubuta (luverne) and similar soils	10 to 15	C	0.53	1.193	
		0.68	woods/grass	15% Boswell and similar soils	10 to 15	D	0.60	0.408	∑(CxA)/A)
∑A	17.40	13.43					∑ CxA	6.302	0.469

Composite C estimate

B. Entrance area & composite C (Weighted average of soil C and impervious surfaces C which flow to inlet).

Total area	Total area A	Impervious Area	Impervious Area IA	Soil area	Impervious C	Composite Soil C	Impervious CxA	Soil CxA	∑ CxA	Runoff Coefficient C
ft^2	acres	ft^2	acres	acres			acres	acres	acres	∑(CxA)/A)
757944	17.40	12430	0.285	17.11	0.9	0.47	0.257	8.031	8.288	0.48

TNP Drainage Basin & Flow Path for Tc



Hydro 13 Rational Method Worksheet

Rational Method Runoff Values

Project No.	Example 1	Drainage Area, A	17.5 acres
County	Fayette	Runoff Coefficient, C	0.48
Highway	Co Rd 12	Time of Concentration, t_c	6.98 min
Station			
Stream	ditch		
Date Run	9/27/2016		
Run By	dr		

		Latitude°	Longitude°	Elev. (ft)
Location	Newtonville	33.5329	87.8407	354
Reference Rainfall Gage	**Tuscaloosa Oliver Dam**	**33.2097**	**87.5936**	**152**

Intensity Formula: $I = a / (t_c + b)^m$

Recurrence Interval Year	t_c min	Y intercept a in./hr	addend b min	Exponent m	Intensity I in./hr
2	6.98	16.52	1.53	0.55050	5.08
5	6.98	20.35	1.63	0.55219	6.20
10	6.98	22.93	1.52	0.54782	7.10
25	6.98	26.27	1.40	0.54285	8.28
50	6.98	29.02	1.42	0.54160	9.16
100	6.98	31.78	1.42	0.54199	10.03
200	6.98	34.00	1.31	0.53887	10.88

Equation Rainfall Intensities, in./hr

Recurrence Interval

t_c min	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr	200 yr
5	5.88	7.16	8.21	9.59	10.60	11.60	12.60
10	4.30	5.25	6.01	7.01	7.76	8.49	9.20
15	3.53	4.31	4.93	5.75	6.37	6.97	7.55
30	2.47	3.02	3.46	4.04	4.49	4.91	5.32
60	1.71	2.09	2.40	2.81	3.12	3.41	3.70

Recurrence Interval Year	**Peak Q ft^3/sec**
2	**42.69**
5	**52.06**
10	**59.64**
25	**69.59**
50	**76.98**
100	**84.23**
200	**91.37**

Hydro 13 Kirpich Time of Concentration Worksheet

Kirpich Time of Concentration, t_c, Calculator

t_c is longest flow time of all flow paths

t_c limits: $5 \le t_c \le 60$ min

Project	Example 1
County	Fayette
Location	Newtonville
Highway	Co. Rd. 12
Station	
Date	9/27/2016
Run By	dr

Latitude °	Longitude °	Elev. (ft)
33.5331	87.8429	353

t_c = 6.975 min

Kirpich t_c Modifiers

Overland Flow on:		Channels:	
Bare earth	1.0	Well defined natural	1.0
Grassed surfaces	2.0	Mowed grass roadside	1.0
Concrete surfaces	0.4	Concrete	0.2
Asphaltic surfaces	0.4		

Flow Path 1

	Overland Segment 1	Channel Segment 2	Segment 3	Segment 4	Segment 5	∑
Highest El. (ft)	445	428				
Lowest El. (ft)	428	354				
Δ Elev. (ft)	17	74				91
Δ Length (ft)	287	801				1088
Slope, S (ft/ft)	0.059	0.092				
Segment time (min)	1.808	3.358				
Surface	woods	natural				
t_c multiplier	2	1				
Modified time (min)	3.616	3.358				6.975

Flow Path 2

	Overland Segment 1	Channel Segment 2	Segment 3	Segment 4	Segment 5	∑
Highest El. (ft)	441	421				
Lowest El. (ft)	421	354				
Δ Elev. (ft)	20	67				87
Δ Length (ft)	275	772				1047
Slope, S (ft/ft)	0.073	0.087				
Segment time (min)	1.617	3.344				
Surface	woods	natural				
t_c multiplier	2	1				
Modified time (min)	3.233	3.344				6.577

Kirpich Time of Concentration, t_c, Calculator

t_c is longest flow time of all flow paths

t_c limits: $5 \le t_c \le 60$ min

Project	Example 1
County	Fayette
Location	Newtonville
Highway	Co. Rd. 12
Station	
Date	9/27/2016
Run By	dr

Latitude °	Longitude °	Elev. (ft)
33.5331	87.8429	353

t_c = 6.975 min

Kirpich t_c Modifiers

Overland Flow on:		Channels:	
Bare earth	1.0	Well defined natural	1.0
Grassed surfaces	2.0	Mowed grass roadside	1.0
Concrete surfaces	0.4	Concrete	0.2
Asphaltic surfaces	0.4		

Flow Path 3

	Channel Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	∑ line
Highest El. (ft)						
Lowest El. (ft)						
Δ Elev. (ft)						
Δ Length (ft)						
Slope, S (ft/ft)						
Segment time (min)						
Surface						
t_c multiplier						
Modified time (min)						

Flow Path 4

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	∑
Highest El. (ft)						
Lowest El. (ft)						
Δ Elev. (ft)						
Δ Length (ft)						
Slope, S (ft/ft)						
Segment time (min)						
Surface						
t_c multiplier						
Modified time (min)						

NOAA Atlas 14 Intensity Coefficients for 23 Gage Stations

(Intensities used in the ALDOT spreadsheet)

		NOAA Atlas 14		Volume 9		2013 Intensity Coefficients		I = a/(t +b)^m			
		Year	a	b	m			Year	a	b	m
Station:	ANDALUSIA	2	21.61	1.72	0.55968	Station:	ANNISTON	2	16.54	1.73	0.56056
	3 W	5	29.14	2.24	0.59349		METRO AP	5	19.73	1.63	0.55541
Latitude:	31.3067˚	10	32.84	2.09	0.59197	Latitude:	33.5872˚	10	22.52	1.59	0.55155
Longitude:	86.5222˚	25	37.02	2.10	0.57899	Longitude:	85.8556˚	25	26.09	1.47	0.54276
Elevation:	250'	50	36.67	1.47	0.54719	Elevation:	594'	50	28.10	1.19	0.52963
		100	35.46	0.84	0.51066			100	30.34	0.97	0.51913
		200	35.18	0.43	0.48060			200	32.83	0.87	0.51052
		Year	a	b	m			Year	a	b	m
Station:	BIRMINGHAM	2	17.65	1.83	0.56594	Station:	BRIDGEPORT	2	16.01	2.04	0.58013
	WSFO	5	20.80	1.68	0.55858	Latitude:	34.9786˚	5	20.04	2.06	0.58242
Latitude:	33.4667˚	10	23.78	1.65	0.55746	Longitude:	85.8008	10	23.52	2.08	0.58394
Longitude:	86.8333˚	25	27.66	1.58	0.55236	Elevation:	670'	25	28.45	2.08	0.58519
Elevation:	744'	50	30.13	1.36	0.54465			50	32.63	2.13	0.58702
		100	33.04	1.33	0.53988			100	36.85	2.14	0.58801
		200	36.59	1.44	0.53930			200	40.75	2.03	0.58584
		Year	a	b	m			Year	a	b	m
Station:	DAUPHIN	2	21.09	1.13	0.52234	Station:	DOTHAN	2	25.13	2.59	0.61670
	ISLAND #2	5	24.73	0.99	0.51478	Latitude:	31.1942˚	5	28.86	2.44	0.60560
Latitude:	30.2500˚	10	27.66	0.93	0.50789	Longitude:	85.3708˚	10	29.47	1.70	0.57981
Longitude:	88.0833˚	25	31.21	0.73	0.49726	Elevation:	275'	25	33.47	1.79	0.57153
Elevation:	8'	50	34.53	0.71	0.49409			50	35.19	1.61	0.55658
		100	36.16	0.45	0.48085			100	36.50	1.38	0.54071
		200	38.91	0.37	0.47535			200	37.13	1.07	0.52140

		NOAA Atlas 14		Volume 9		2013 Intensity Coefficients		I = a/(t +b)^m		
		Year	a	b	m			Year	a	b	m
Station:	EUFAULA	2	22.23	2.39	0.60408	Station:	EVERGREEN	2	21.56	1.88	0.57156
Latitude:	31.8667˚	5	26.50	2.33	0.60001	Latitude:	31.4449˚	5	27.25	2.12	0.58636
Longitude:	85.1500˚	10	29.48	2.24	0.59153	Longitude:	86.9532˚	10	30.33	2.03	0.57981
Elevation:	200'	25	32.44	1.89	0.57467	Elevation:	290'	25	32.90	1.65	0.55941
		50	35.19	1.88	0.56652			50	33.46	1.23	0.53384
		100	36.37	1.50	0.54963			100	35.79	1.30	0.52173
		200	37.39	1.23	0.53306			200	33.84	0.48	0.48219
		Year	a	b	m			Year	a	b	m
Station:	FLORENCE	2	17.92	2.08	0.58518	Station:	FT PAYNE	2	19.78	2.53	0.61106
Latitude:	34.8000˚	5	23.02	2.30	0.59742	Latitude:	34.4406˚	5	24.84	2.59	0.61512
Longitude:	87.6833˚	10	27.72	2.46	0.60733	Longitude:	85.7236˚	10	28.94	2.55	0.61544
Elevation:	581'	25	34.00	2.52	0.61253	Elevation:	917'	25	35.05	2.53	0.61630
		50	39.94	2.62	0.62172			50	40.27	2.59	0.61833
		100	46.79	2.90	0.63094			100	46.01	2.75	0.62098
		200	54.50	3.07	0.64071			200	49.90	2.50	0.61335
		Year	a	b	m			Year	a	b	m
Station:	HAMILTON	2	16.23	1.61	0.55036	Station:	HUNTSVILLE	2	16.58	1.81	0.56394
	3 S	5	19.83	1.63	0.55424		INTNL AP	5	21.12	1.88	0.57259
Latitude:	34.0967˚	10	23.37	1.70	0.56032	Latitude:	34.6439˚	10	25.79	2.09	0.58439
Longitude:	87.9914˚	25	29.37	1.91	0.57313	Longitude:	86.7861˚	25	32.81	2.34	0.59807
Elevation:	435'	50	33.97	1.93	0.57779	Elevation:	624'	50	37.67	2.32	0.60042
		100	39.54	2.04	0.58599			100	42.93	2.34	0.60316
		200	46.43	2.29	0.59636			200	51.42	2.81	0.61923

NOAA Atlas 14 Volume 9 2013 Intensity Coefficients I = a/(t +b)^m

		Year	a	b	m			Year	a	b	m
Station:	JACKSON	2	20.71	1.64	0.55298	Station:	LIVINGSTON	2	17.40	1.57	0.55326
Latitude:	31.5250˚	5	24.60	1.54	0.54922	Latitude:	32.5811˚	5	20.83	1.56	0.55233
Longitude:	87.9278˚	10	27.21	1.41	0.53876	Longitude:	88.1897˚	10	23.27	1.50	0.54750
Elevation:	220'	25	30.22	1.16	0.52193	Elevation:	128'	25	26.88	1.50	0.54563
		50	31.97	0.94	0.50503			50	29.15	1.37	0.54124
		100	32.20	0.36	0.47930			100	31.36	1.27	0.53703
		200	34.03	0.26	0.46597			200	34.45	1.46	0.53913
		Year	a	b	m			Year	a	b	m
Station:	MOBILE	2	23.56	1.72	0.55912	Station:	MONTGOMERY	2	19.07	1.96	0.57744
Latitude:	30.6833˚	5	27.54	1.60	0.55133		WB	5	22.79	1.95	0.57670
Longitude:	88.0333˚	10	30.14	1.37	0.53948	Latitude:	32.3833˚	10	25.44	1.87	0.57129
Elevation:	10'	25	34.27	1.24	0.52940	Longitude:	86.3000˚	25	28.68	1.74	0.56171
		50	36.68	1.01	0.51749	Elevation:	256'	50	31.17	1.75	0.55504
		100	39.47	0.89	0.50921			100	31.96	1.27	0.53814
		200	43.47	0.93	0.50792			200	33.75	1.16	0.52899
		Year	a	b	m			Year	a	b	m
Station:	MOUNT	2	22.51	1.73	0.55856	Station:	ONEONTA	2	18.29	2.00	0.57704
	VERNON	5	25.80	1.49	0.54615	Latitude:	33.9478˚	5	22.08	1.97	0.57712
Latitude:	31.0881˚	10	28.14	1.23	0.53370	Longitude:	86.4692˚	10	25.49	1.99	0.57763
Longitude:	88.0258	25	31.17	0.97	0.51586	Elevation:	892'	25	31.00	2.08	0.58317
Elevation:	172'	50	33.36	0.80	0.50212			50	35.67	2.18	0.58762
		100	36.53	0.87	0.49543			100	40.74	2.24	0.59277
		200	36.08	0.21	0.46668			200	45.11	2.14	0.59161

		NOAA Atlas 14		Volume 9		2013 Intensity Coefficients		I = a/(t +b)^m	
		Year	**a**	**b**	**m**			**Year**	**a**	**b**	**m**
Station:	OPELIKA	2	18.22	1.91	0.57632	Station:	PRATTVILLE	2	19.37	2.13	0.58487
Latitude:	32.6592˚	5	22.61	2.06	0.58263	Latitude:	32.4833˚	5	23.79	2.21	0.59113
Longitude:	85.4492˚	10	26.22	2.15	0.58720	Longitude:	86.4833˚	10	26.63	2.14	0.58575
Elevation:	640'	25	30.49	2.13	0.58617	Elevation:	302'	25	29.95	1.95	0.57517
		50	33.47	2.04	0.58434			50	32.12	1.76	0.56549
		100	36.67	2.09	0.58323			100	35.03	1.83	0.56192
		200	39.78	2.10	0.58249			200	35.79	1.42	0.54463
		Year	**a**	**b**	**m**			**Year**	**a**	**b**	**m**
Station:	THOMASVILLE	2	21.51	2.11	0.58262	Station:	TROY	2	20.03	1.95	0.57337
Latitude:	31.9172˚	5	25.10	1.98	0.57698	Latitude:	31.8075˚	5	22.33	1.59	0.55374
Longitude:	87.7347˚	10	27.39	1.80	0.56630	Longitude:	85.9722˚	10	24.56	1.47	0.54423
Elevation:	390'	25	29.69	1.46	0.54703	Elevation:	542'	25	26.11	1.00	0.51910
		50	31.86	1.46	0.53696			50	27.84	0.93	0.50655
		100	32.44	1.04	0.51710			100	28.98	0.73	0.49091
		200	32.96	0.74	0.49822			200	29.23	0.26	0.46981
		Year	**a**	**b**	**m**						
Station:	TUSCALOOSA	2	16.52	1.53	0.55050						
	OLIVER DAM	5	20.35	1.63	0.55219						
Latitude:	33.2097˚	10	22.93	1.52	0.54782						
Longitude:	87.5936˚	25	26.27	1.40	0.54285						
Elevation:	152'	50	29.02	1.42	0.54160						
		100	31.78	1.42	0.54199						
		200	34.00	1.31	0.53887						

USGS Alabama Hydrograph

An average dimensionless Alabama hydrograph is obtainable by using *Estimating Flood Hydrographs and Volumes for Alabama Streams, 1988: USGS Water Resources Investigations Report 88-4041*. The report provides a method to estimate flood hydrographs, volumes, and lag-times for both rural and urban streams with drainage areas less than 500 square miles. Figure G.1 illustrates a plot of the dimensionless hydrograph:

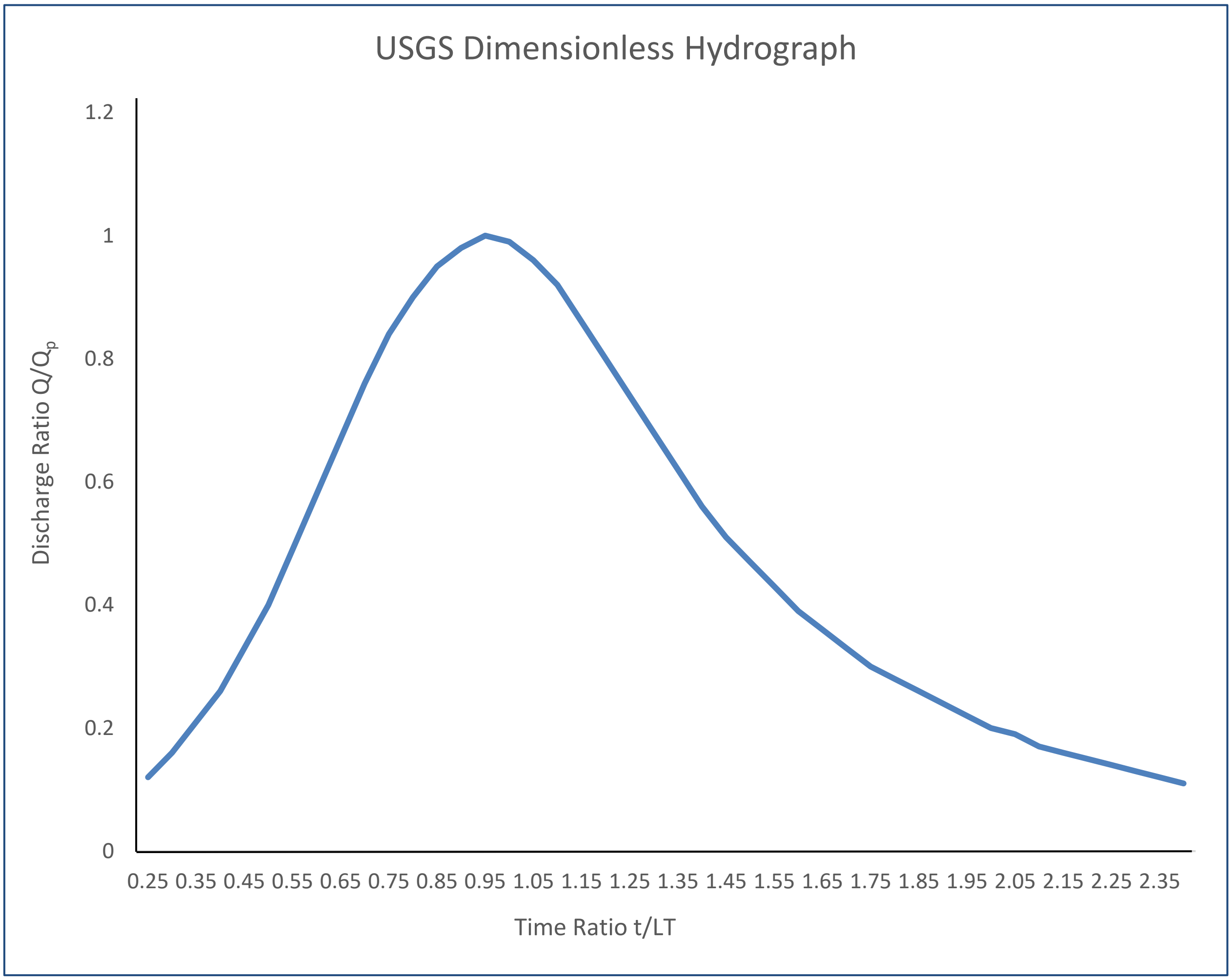


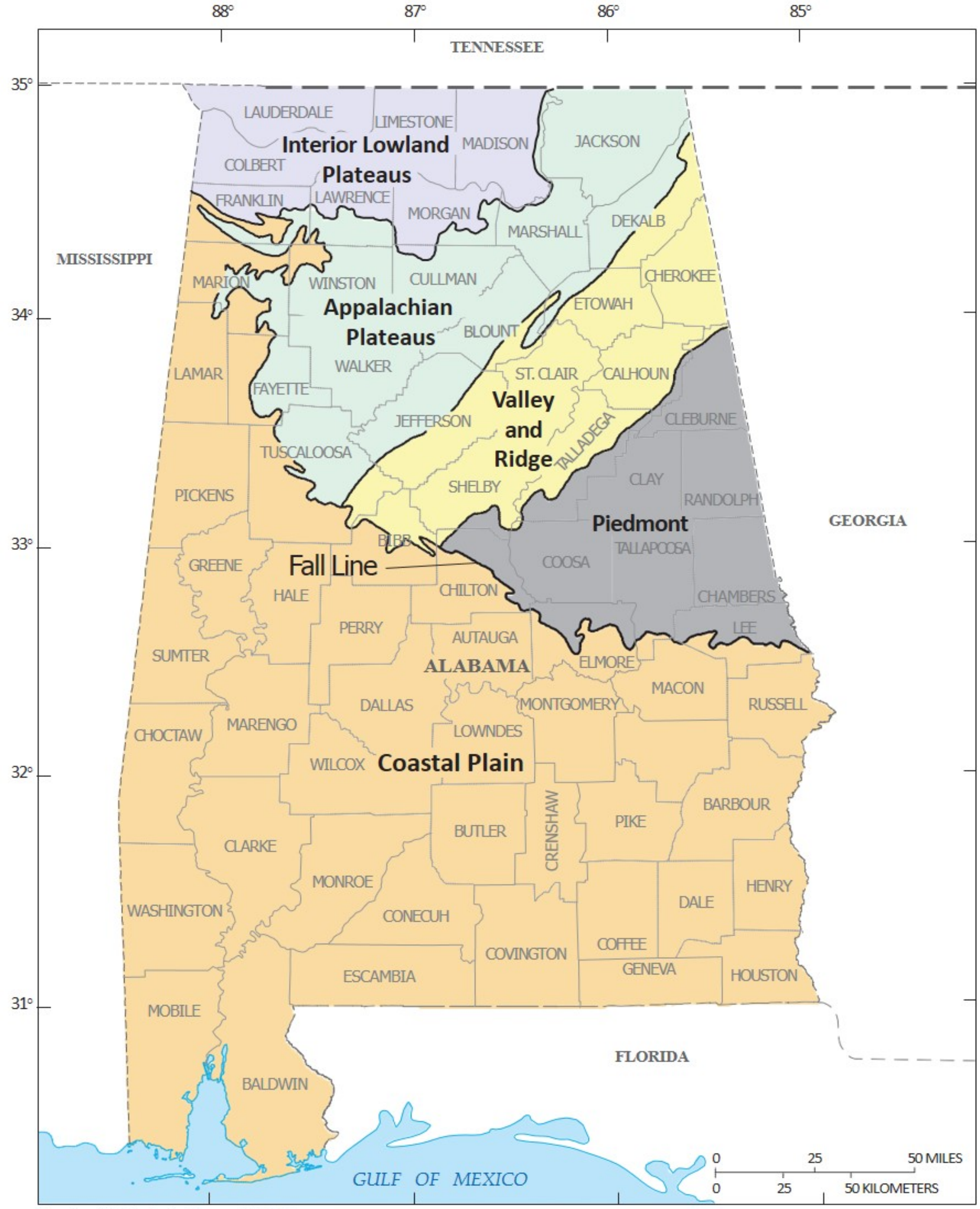
Figure G.1: **USGS Dimensionless Hydrograph**

The hydrograph is a representation of flow versus time. The Time Ratio, shown on the y-axis of the hydrograph, consists of the comparison between travel time and Lag-Time (LT). LT is the time, in hours, from the centroid of the rainfall excess to the centroid of the resultant runoff hydrograph. Table G.1 shows the Time and Discharge Ratios of the Dimensionless Hydrograph.

Table G.1: Time and Discharge Ratios (Inman 1986)

Time Ratio (t/LT)	Discharge Ratio (Q/ Q_p)	Time Ratio (t/LT)	Discharge Ratio (Q/ Q_p)	Time Ratio (t/LT)	Discharge Ratio (Q/ Q_p)
0.25	0.12	1.00	0.99	1.75	0.30
0.30	0.16	1.05	0.96	1.80	0.28
0.35	0.21	1.10	0.92	1.85	0.26
0.40	0.26	1.15	0.86	1.90	0.24
0.45	0.33	1.20	0.80	1.95	0.22
0.50	0.40	1.25	0.74	2.00	0.20
0.55	0.49	1.30	0.68	2.05	0.19
0.60	0.58	1.35	0.62	2.10	0.17
0.65	0.67	1.40	0.56	2.15	0.16
0.70	0.76	1.45	0.51	2.20	0.15
0.75	0.84	1.50	0.47	2.25	0.14
0.80	0.90	1.55	0.43	2.30	0.13
0.85	0.95	1.60	0.39	2.35	0.12
0.90	0.98	1.65	0.36	2.40	0.11
0.95	1.00	1.70	0.33		

The rural LT equations are different north and south of the coastal plain. The fall line separates the coastal plain from the other four physiographic provinces of the state. Figure G.2 depicts the location of the fall line:



Base from U.S. Geological Survey digital data,
1:100,000, Universal Transverse Mercator Projection,
Zone 16.

Figure G.2: Locations of Physiographic Provinces in Alabama

The urban lag-time equations additionally depend upon the percent imperviousness (IA) to compute lag time. The IA (%) for urban lag time can be computed on the IA & PD sheet also found in the Hydraulic Sections spreadsheet. Table G.2 shows the LT equations separated by the fall Line in Alabama:

Table G.2: Lag-time Equations

Lag-time Area	Equation	SE_E
North of the Fall Line (Rural)	$LT = 2.66\ A^{0.46}\ S^{-0.08}$	31.6
South of the Fall Line (Rural)	$LT = 5.06A^{0.50}\ S^{-0.20}$	31.2
Statewide (Urban)	$LT = 2.85A^{0.295}\ S^{-0.183}\ IA^{-0.122}$	21.0

The standard error of estimate (SE_E) is the standard deviation of the differences between station data and the corresponding values computed from the regression equation.

The hydrograph is used with the computed LT for the regression equations, or the time of concentration for the rational method. The equations all depend upon Area (A) and the 10 to 85 percent channel slope (S). The slope of the channel, expressed as feet/mile, is measured between points 10 and 85 percent of the distance that has the longest flow time from the outlet to the basin divide. Table G.3 shows the limits of the A and S variables used in the lag-time equations.

Table G.3: Limits of the Lag-time Equations

North of the Fall Line (Rural)			
Variable	**Minimum**	**Maximum**	**Units**
A	0.59	481.0	Square miles
S	5.20	296.2	Feet per mile
South of the Fall Fine (Rural)			
A	1.11	485.0	Square miles
S	4.20	83.3	Feet per mile
Statewide (Urban)			
A	0.16	41.8	Square miles
S	10.6	295.6	Feet per mile
IA	8.40	42.9	Percent

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Appendix H: Additional Bridge Information

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Section 1 - Contents List for Riverine Hydrologic and Hydraulic Study

1. **Cover Sheet**. The following information should be shown:
   a. Project number, BIN, Route and stream name;
   b. Signature and date; and
   c. For Consultant projects, the Hydrologic and Hydraulic Study should be stamped and signed by a registered Professional Engineer
2. **Hydrologic and Hydraulic Report**. See Section 3 of Appendix H for *Example Hydraulic Reports*. Include the description of the project, description of the existing structure(s), the methods of analysis along with determination of boundary conditions, and the conclusions and results for the project. Report should include discussion of how the hydraulic analysis was performed (i.e., headwater flooding only, if analysis did not consider the effects of backwater by inflow of tributaries or other encroachments downstream).

   Note: The proposed drainage structure(s) should be sized as the minimum length bridge, or smallest culvert, or most cost-effective combination of drainage structures that have acceptable backwater and velocity values, meets FEMA requirements if applicable, while adhering to the procedures, guidelines, and design criteria of this manual.
3. **Site Inspection**. A site inspection should be performed with the results included in the study. This site inspection should include the date of the site inspection, detailed descriptions of the existing channel, upstream and downstream floodplains, existing bridges and/or culverts, development/houses near the site, and any scour, erosion or debris problems, etc.
4. **Scour calculations for bridges.** Should include a scour plot with a table of estimated scour depths (See Section 11.1.6).
5. **Precast Culvert Waiver Documentation**. If a precast culvert alternate is not allowed, a waiver from the Chief Engineer must be obtained. This documentation should state the reason(s) that a precast box culvert should not to be used at the site (See GFO 3-21).
6. **Hydraulic Table(s)**. Hydraulic reports should present tables listing the natural (unconstricted) flood stages and peak flows for the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals. The design-year and 100-year hydraulic properties (flood stages, mean velocities in bridge opening, area of hydraulic opening, and backwater) should be presented for the existing, and proposed conditions along with any applicable structure alternates. Flood stages at the structure (downstream face of bridge or culvert) and the unconstricted and constricted flood stages at the upstream approach section should also be included in report tables. If multiple openings and/or weir flow is present, flow distribution through the structure(s) and over the roadway should also be included. If the site is affected by abnormal flood stages (backwater), separate tables should be shown for the design-year and 100-year headwater floods and backwater flood stages.

   Note: This table is separate from the hydraulic computer model generated tables. This table contains all of the above-specified hydraulic values that can easily be compared for each flood magnitude and condition (I.E., existing, proposed, and alternates). See the hydraulic table example contained in this appendix.
7. **Drainage Calculations.** The drainage area and flood discharges should be shown.
8. **Copies of Gage Data** used if applicable (or other supporting data).

9. **Guide Bank Calculations** (bridge only). If applicable

10. **Riprap Calculations**. If applicable

11. **Risk Assessment Form**.

12. **Bridge Clearance Determination.** The vertical clearance between the water-surface and the low girder is referred to as freeboard. Freeboard is used with the girder and bridge deck thicknesses to set the minimum finished grade. Freeboard should be 2 feet above the design-year flood measured at the downstream side of the bridge. If abnormal flood stages (backwater) are present, adjustments/increases to finished grade and freeboard should be considered.

13. **Bridge Culverts** will be subjected to allowable headwater requirements as outline in Chapter 8, Section 8.2.3.

14. **Roadway Plan Sheets.** Copies of the plan and profile sheets, along with the title and typical section sheets should be included.

    Note: If the proposed drainage structure is a box culvert, a sketch of the culvert placement should be shown on the applicable plan and profile sheet.

15. **USGS Quadrangle Map**. With the project location marked.

16. **Computer Data**.

    Input and Output of the hydraulic computer model for the following:

    a. Natural (unconstricted), existing, and proposed conditions;

        Note: Natural conditions for bridge replacements and widened/parallel bridges refers to natural unconstricted conditions at the project site. This computer run removes the existing roadway and structure (bridge or culvert) at the project site. Other structures and constrictions upstream and downstream of the project site remain in the model.

        The WSPRO model provides this unconstricted natural run automatically.

    b. Applicable alternates; and

    c. Detour structure (if applicable).

    If the WSPRO model is used, include the following input and output from the computer run in the study at a minimum:

    a. The input data;
    b. The final iteration showing the water surface profiles through the stream reach for all required floods; and
    c. The computation of the sub-area properties used in the various hydraulic calculations (for scour computations).

    If the HEC-RAS model is used, include the following input and output from the computer run in the study at a minimum:

    a. The report showing all input data;

    b. The schematic plan view of the stream reach showing the location of the cross sections;

    c. The standard profile output tables, numbers 1 and 2;

d. The cross section profile table including the bridge or culvert;

e. The bridge or culvert output tables;

f. The cross section output tables for the bridge or culvert; and

g. The scour calculations and results for the proposed bridge.

The above output tables should include the natural (w/o structure or roadway at the project site) conditions, as well as the existing and proposed bridge conditions for the various required flood discharges as applicable.

Note: Consultants are required to include a computer disk with the above runs for the Department's use.

18. Flood Insurance Study Information.

If the site is located within a FEMA regulatory floodway, the following information is required to be placed within the study:

a. An explanation of any required modification and/or corrections to the 100-year base flood profile or floodway model;

b. The floodway map with the site marked and any modification delineated;

c. Floodway data tables for the original (published), corrected effective (existing conditions), and proposed conditions models;

d. Base flood profile runs including for the 10-, 50-, 100-, and 500-year floods; and floodway run input files for the 100-year flood

Note: Consultant's should include two computer disks with the above runs for the Department's use and distribution.

Section 2 - Contents List for Tidal Hydrologic and Hydraulic Study

1. **Cover Sheet**. The following information should be shown:
   a. Project number, BIN, Route and stream name;
   b. Signature and date.
   c. For Consultant projects, the Hydraulic Study should be stamped and signed by a registered Professional Engineer.
2. **Hydraulic and Hydrologic Report**. See Section 3 of Appendix H for *Example Hydraulic Reports*. Include the description of the project, description of the existing structure(s), the methods of analysis along with the determination of the boundary conditions, and the conclusions and results for the project. Report should include discussion of how the hydraulic analysis was performed (i.e., headwater flooding only, if analysis did not consider the effects of backwater by inflow of tributaries or other encroachments downstream).

   Note: The proposed drainage structure(s) should be sized as the minimum length bridge, or smallest culvert, or most cost-effective combination of drainage structures that have acceptable backwater and velocity values, fits the channel geometry, meets FEMA requirements if applicable, while adhering to the procedures, guidelines, and design criteria of this manual.
3. **Site Inspection**. A site inspection should be performed with the results included in the study. This site inspection should include the date of the site inspection, detailed descriptions of the existing channel, upstream and downstream floodplains, existing bridges and/or culverts, development/houses near the site, and any scour, erosion or debris problems, etc.
4. **Scour Report and Calculations**. The scour analysis should be done for the floods listed in Chapter 11, Section 11.1.6 for upland riverine flooding only, assuming the Mobile Bay is at low tide conditions. The four Mobile Bay sites specifically mentioned in Chapter 11, Section 11.3.2, will have scour analyses that include both headwater flooding and storm tide surge flooding scenarios. The scour table should include general contraction, local (pier) and total scour for these floods.
5. **Hydraulic Table**. Hydraulic reports should present tables listing the natural (unconstricted) flood stages and peak flows for the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals. The design-year and 100-year hydraulic properties (flood stages, mean velocities in bridge opening, area of hydraulic opening, and backwater) should be presented for the existing, and proposed conditions along with any applicable structure alternates. Flood stages at the structure (downstream face of bridge or culvert) and the unconstricted and constricted flood stages at the upstream approach section should also be included in report tables. If multiple openings and/or weir flow is present, flow distribution through the structure(s) and over the roadway should also be included. If the site is affected by abnormal flood stages (backwater), separate tables should be shown for the design-year and 100-year headwater floods and backwater flood stages.

   Note: This table is separate from the hydraulic computer model generated tables. This table contains all of the above-specified hydraulic values that can easily be compared for each flood magnitude and condition (I.E., existing, proposed, and alternates). See the hydraulic table example contained in this appendix.

6. **Drainage Calculations.** The riverine drainage area and the upland flood discharge should be shown. The high and low elevations should be shown at the project site. These elevations should be given to the project datum.
7. **Copies of Gage Data** used (or other supporting data). Copies of the publications, information and methods used to determine the normal and storm surge tidal conditions at the project site should be provided. Tide gage data should be included in the study. The various storm hydrographs should be shown. The National Geodetic Survey provides information on Tidal Benchmarks and conversions between tidal datums (e.g. mean low water) and fixed datums (NGVD-29 and NAVD-88).
8. **Guide Bank Calculations**. If applicable
9. **Riprap Calculations**. If applicable
10. **Risk Assessment Form**.
11. **Clearance Determination**. Clearance determination will be determined by the Department on a case by case basis for all tidally influenced sites.
12. **Roadway Plan Sheets.** Copies of the plan and profile sheets, along with the title and typical section sheets should be included.
13. **USGS Quadrangle Map**. With the project location marked. Copies of the contour and hydrographic maps showing the extent of the study grid should be included. Cross sections used in the computer model should be shown and labeled on these maps.
14. **Computer Data**.

    Input and Output of the hydraulic computer model for the following:

    a. Existing and proposed conditions
    b. Detour structure (if applicable)

    If the WSPRO model is used, include the following input and output from the computer run in the study at a minimum:

    a. The input data;
    b. The final iteration showing the water surface profiles through the stream reach for all required floods; and
    c. The computation of the sub-area properties used in the various hydraulic calculations (for scour computations).

    If the HEC-RAS model is used, include the following input and output from the computer run in the study at a minimum:

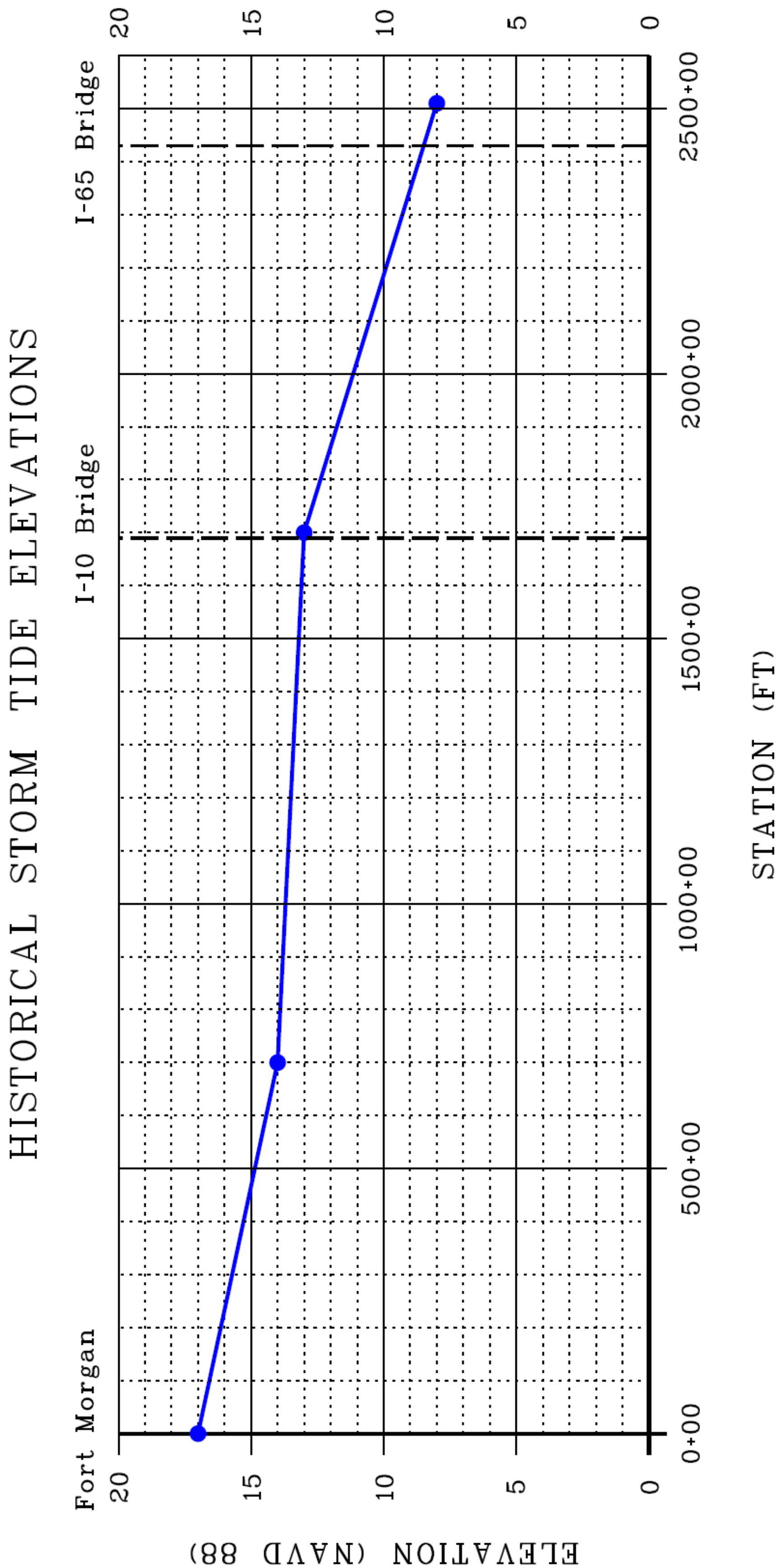
    a. The report showing all input data;
    b. The schematic plan view of the stream reach showing the location of the cross sections;
    c. The standard profile output tables, numbers 1 and 2;

d. The cross section profile table including the bridge or culvert;
e. The bridge or culvert output tables;
f. The cross section output tables for the bridge or culvert; and
g. The scour calculations and results for the proposed bridge.

The above output tables should include the natural (w/o structure or roadway at the project site) conditions, as well as the existing and proposed bridge conditions for the various required flood discharges as applicable.

Note: Consultants are required to include a computer disk with the above runs for the Department's use.

Figure H.2 Historical Storm Tide Elevations



Section 3 - Example Hydraulic Reports

Note: The following pages contain sample written hydraulic reports.

For Consultant projects, a registered professional engineer is required to stamp, sign, and date the cover sheet of the hydraulic report.

PROJECT NO. BR-0014(536)
BRIDGE REPLACEMENT ON SR-14
BLUBBER CREEK
PICKENS COUNTY

The proposed project is located approximately one and one-half miles northwest of the City of Aliceville in Pickens County (fig 1). The proposed bridge is will replace the existing bridge in-kind on the existing alignment. A temporary diversion will be constructed just downstream of the existing bridge. Typical section for the project is forty-four feet (44') shoulder to shoulder. This site was visited on October 27, 2016 with personnel from West Central Region – Tuscaloosa Area (WCR-TA), and Bridge Bureau.

The in-place bridge length on SR-14 and other pertinent data are listed in Table 1. The superstructure of the bridge includes steel girders with a concrete deck. The substructure includes vertical concrete abutments on steel piles and steel pile bents for the interior supports. All supports appear to be embedded in a marl or soapstone. According to ALDOT BrM, this structure was built in 1950 and has a sufficiency rating of 38.7.

Bridge Length (ft.)	Bridge Width (ft.)	Span Arrangement	BIN
272.0'	29.2'	8@34'	003677

Table 1. Existing bridge on SR-14 over Blubber Creek, Pickens County.

Blubber Creek is a small size, perennial creek with a wide flood plain (fig. 2). The drainage area for the stream at the crossing is 16.8 square miles. The channel is roughly fifty-five feet wide and eight feet deep. The channel boundaries are alluvial. Trees and thick undergrowth line the channel banks and cover the floodplain. This is a normal crossing and the stream is meandering along this reach with a sharp bend located just upstream.

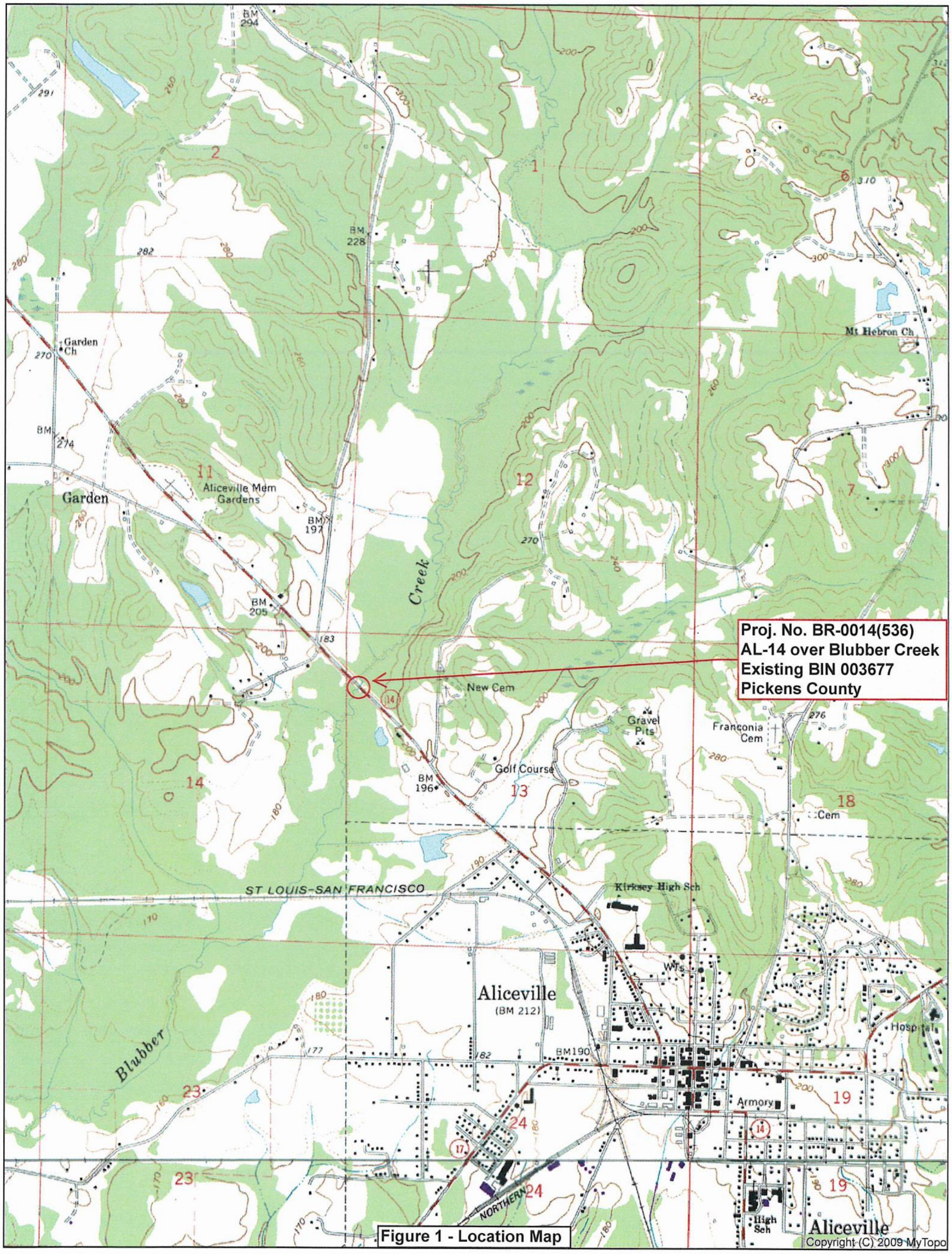


Figure 1 - Location Map

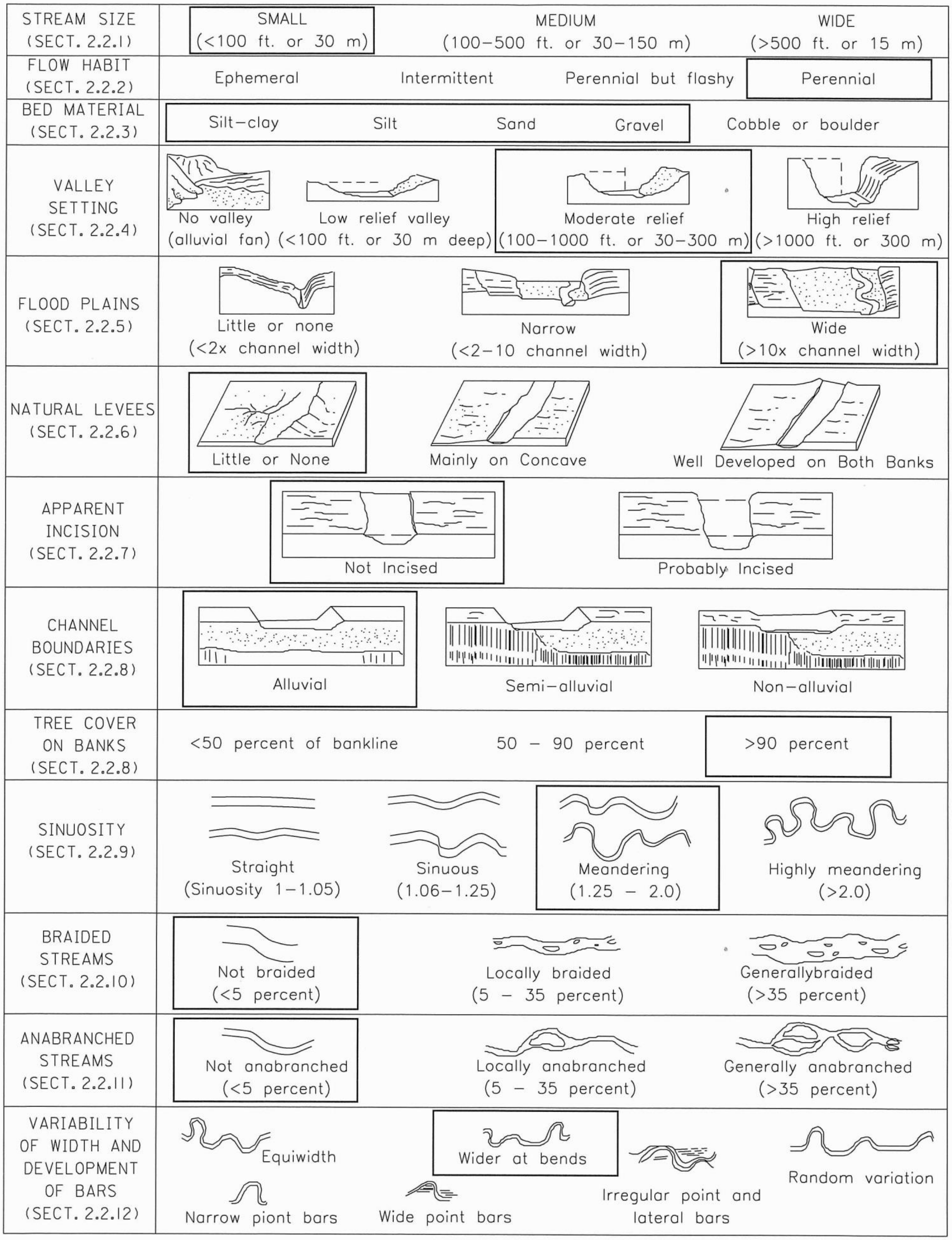


FIGURE 2. Geomorphic factors that affect stream stability (Adapted from [1]).

A flood frequency relation was developed for the site using procedures as described in the USGS report 2007-5204, *Magnitude and Frequency of Floods in Alabama*, 2003, by T.S. Hedgecock and Toby D. Feaster. The peak discharges for selected recurrence intervals are given in Table 2.

A stage-discharge relationship was developed just downstream of the site using the computer model WSPRO, a model for water surface profile computations. The model will analyze flow through bridges and over roadways. The West Central Region provided the downstream valley section used in the analysis. The roughness values were determined during the site inspection. Stages or water surface elevations for the selected recurrence intervals are also given in Table 2.

Recurrence Interval (year)	Discharge (ft^3/s)	Water Surface Elevation (ft)
50	3940	176.6
100	4655	176.9
200	5440	177.3
500	6635	177.8

Table 2. Peak discharges and downstream stages for Blubber Creek at SR-14, Pickens County.

Recommendations and hydraulic data for the proposed replacement bridge are given below:

7@40' = 280' AASHTO Girder Bridge
BIN: 021299
Begin Bridge Sta: (+/-) 714+60
End Bridge Sta: (+/-) 717+40
Minimum Bridge Elev. At C/L: 182.1 ft
Skew: 0°
Bridge Width: 44 ft curb to curb

The backwater computed for the 50-year flood is eight tenths (0.8') of a foot and nine tenths (0.9') of a foot for the 100-year flood. The average velocity through the bridge for the 50 and 100-year floods are 3.7 and 4.0 feet per second. These values are acceptable for this site. The channel banks should be plated with Class II riprap if disturbed during construction.

This reach of Blubber Creek is designated as a special flood hazard area, “Zone A” on the Flood Insurance Rate Map (FIRM) for the City of Aliceville in Pickens County dated September 17, 2010 (fig 3). As part of the National Flood Insurance Program (NIFP), the community is required to adopt minimal standards for floodplains. The West Central Region – Tuscaloosa Area (WCR-TA) will need to coordinate this project with the community floodplain administrator regarding the road and bridge construction in the floodplain. No coordination with the Federal Emergency Management Agency (FEMA) should be required since the proposed bridge will provide the same if not more opening and conveyance as the existing bridge and the proposed structure will not increase the base flood stage.

The hydraulic analysis for this site was done for headwater flooding only and does not take into consideration the effects of backwater caused by inflow of tributaries or other encroachments downstream of the project.

________________ ________________________

Date

John Doe, PE
Hydraulic Engineer

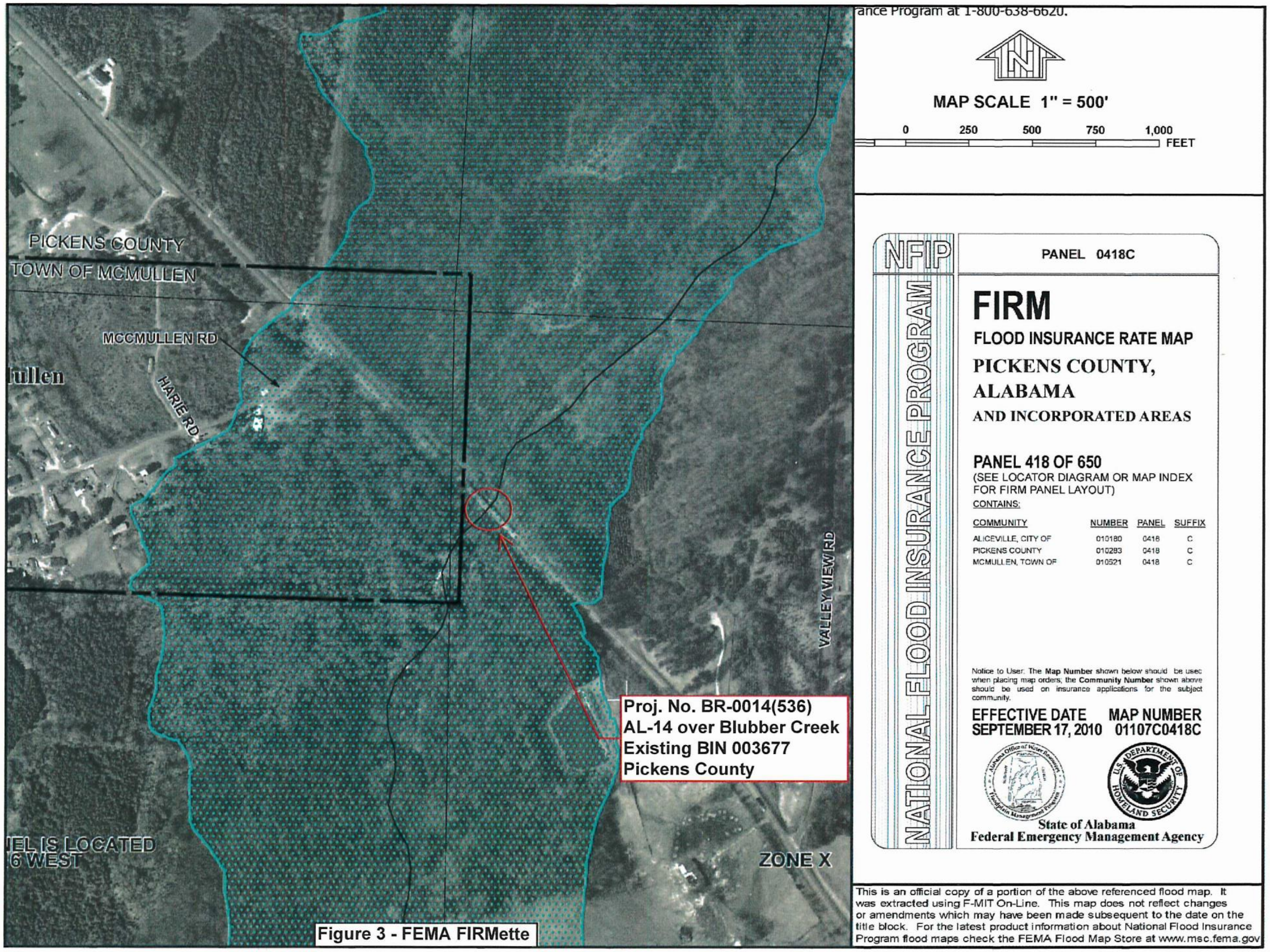


Figure 3 - FEMA FIRMette

Example Bridge Culvert Hydraulic Report

PROJECT NO.BR-0051(512)
BRIDGE REPLACEMENT ON SR-51
ROBINSON CREEK
LEE COUNTY

The proposed project is located approximately two miles south of the City of Opelika in Lee County (fig 1). Proposed plan is to replace the existing bridge on SR-51 over Robinson Creek on the existing alignment with an on-site diversion. Typical section for the project is forty-four feet (44') shoulder to shoulder. This site was visited on October 4, 2017 with personnel from Southeast Region – Troy Area (SER-TA) and Bridge Bureau.

The in-place bridge length on SR-51 and other pertinent data are listed in Table 1. The superstructure of the bridge includes steel girders with a concrete deck. The substructure consists of steel piles at the center support and vertical concrete abutments. According to ALDOT BrM, this structure was built in 1939 and has a sufficiency rating of 32.3.

Bridge Length (ft)	Bridge Width (ft)	Span Arrangement	BIN
51.9'	25.3'	1@27'-1@24.9'	002013

Table 1. Existing bridge on SR-51 over Robinson Creek, Lee County.

Robinson Creek is a small size, perennial stream with a wide floodplain (fig. 2). The drainage area for the stream at this crossing is 0.85 square miles. The channel is approximately twenty feet wide and two to five feet deep. Trees and thick undergrowth line the channel banks and cover the floodplain. The channel boundaries are semi-alluvial and the stream is meandering throughout this reach.

A flood frequency relation was developed for the site using procedures as described in the USGS report 2010-5012, *Magnitude and Frequency of Floods for Urban Streams in Alabama*, 2007, by T.S. Hedgecock and K.G. Lee. Robinson Creek is impacted by urbanization with development occurring throughout the entire drainage basin. Percent development was estimated at sixty percent (60%) using aerial maps and the National Land Cover Dataset (NLCD).

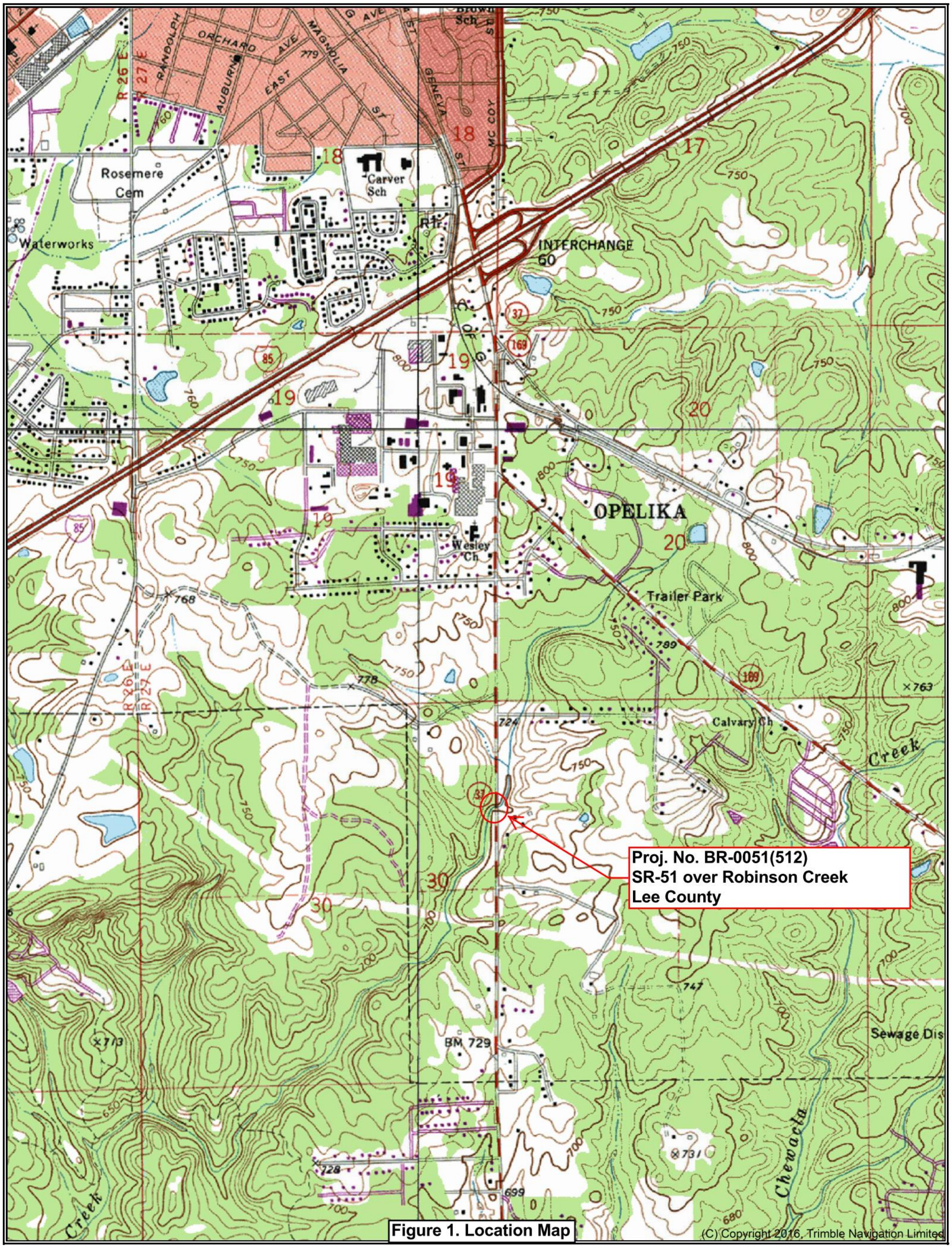


Figure 1. Location Map

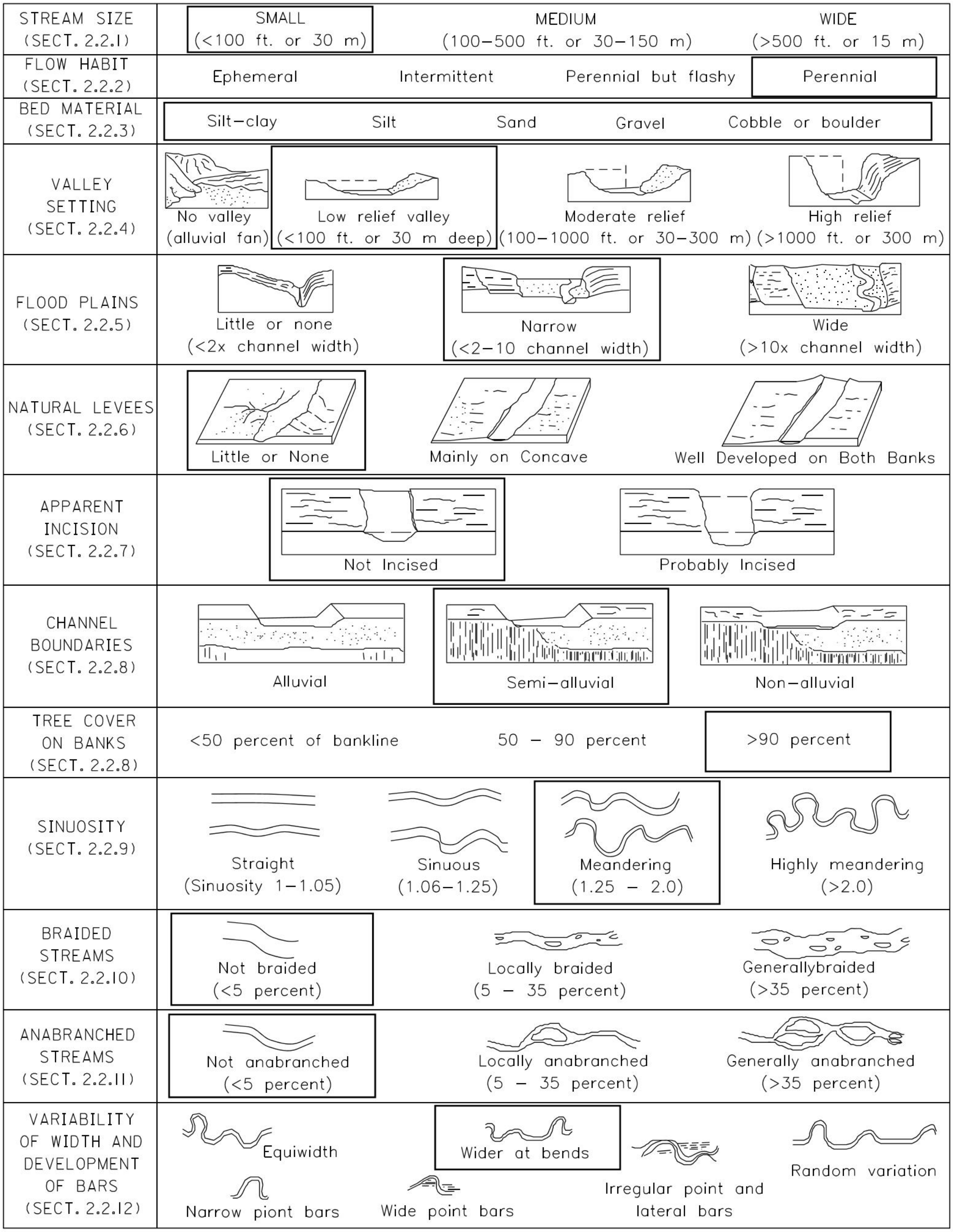


FIGURE 2. Geomorphic factors that affect stream stability (Adapted from [1]).

Peak discharges for selected recurrence intervals are given in Table 2. A stage-discharge relationship was developed just downstream of the site using the computer model WSPRO, a model for water surface profile computations. The model will analyze flow through bridges and over roadways. The Southeast Region provided the downstream valley section used in the analysis. The roughness values were determined during the site inspection. Stages or water surface elevations for the selected recurrence intervals are also given in Table 2.

Recurrence Interval (year)	Discharge (ft^3/s)	Water Surface Elevation (ft)
50	1215	698.9
100	1385	699.3
200	1560	699.6
500	1805	700.1

Table 2. Peak discharges and downstream stages for Robinson Creek at SR-51, Lee County.

Because of the small drainage area and the presence of rock at this site, a bridge culvert is suggested. Recommendations for the structure are as follows:

CT 10x10 Bridge Culvert
Begin Bridge Sta: (+/-) 64+50.00
End Bridge Sta: (+/-) 65+12.33
Culvert Inlet Elev.: (+/-) 690.80 ft
Outlet Elev.: (+/-) 690.00 ft
Culvert Length: (+/-) 220 ft
Skew: 60° LT BK
Five foot toewalls or key into rock.

In order to accommodate the stage construction and onsite diversion of traffic, the culvert was designed to be longer than typically needed. The computer program HY-8 was used to analyze the bridge culvert. Table 3 contains some of the hydraulic variables for the proposed culvert. Class II riprap should be placed between the wing on the inlet end of the culvert and from the outlet end to the ROW. The channel banks should be plated with Class II riprap if disturbed during construction.

Recurrence Interval (year)	Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
50	699.7	1215	4.4
100	700.2	1385	4.8

Table 3. Hydraulic variables for the proposed bridge culvert at Robinson Creek at SR-51, Lee County.

This reach of the Robinson Creek is not within a flood hazard area and is designated as “Zone X” on the Flood Insurance Rate Map (FIRM) for Lee County dated November 2, 2011 (fig 3). Therefore, no coordination with the Federal Emergency Management Agency (FEMA) or the community floodplain administrator regarding the road and bridge construction in the floodplain will be required.

The hydraulic analysis for this site was done for headwater flooding only and does not take into consideration the effects of backwater caused by inflow of tributaries or other encroachments downstream of the project.

Date

John Doe, PE
Hydraulic Engineer

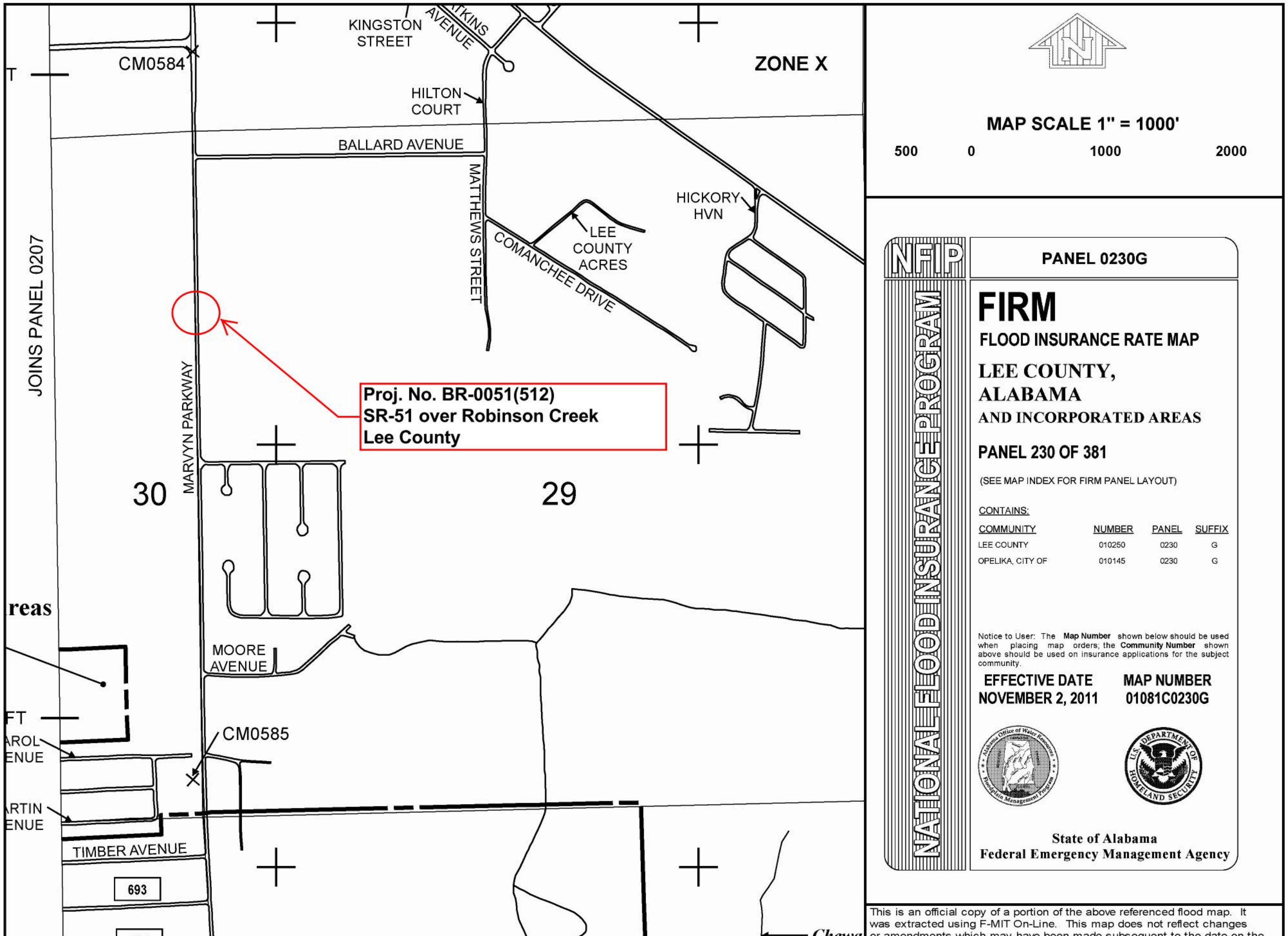


Figure 3. FEMA FIRMette

88°20'0"W
88°16'0"W
88°12'0"W
88°8'0"W
88°4'0"W
88°0'0"W
87°56'0"W
87°52'0"W
87°48'0"W
87°44'0"W
87°40'0"W
87°36'0"W
31°2'0"N
31°0'0"N
30°58'0"N
30°56'0"N
30°54'0"N
30°52'0"N
30°50'0"N
30°48'0"N
30°46'0"N
30°44'0"N
30°42'0"N
30°40'0"N
30°38'0"N
30°36'0"N
30°34'0"N
30°32'0"N
30°30'0"N
30°28'0"N
30°26'0"N
30°24'0"N
30°22'0"N
30°20'0"N
30°18'0"N
30°16'0"N
30°14'0"N
30°12'0"N
30°10'0"N
30°8'0"N
2500+00
2000+00
1500+00
1000+00
500+00
Mobile Bay
Bon Secour Bay
Mobile
Saraland
Satsuma
Chickasaw
Prichard
Spanish Fort
Daphne
Montrose
Fairhope
Point Clear
Loxley
Stapleton
Bay Minette
Stockton
Silverhill
Robertsdale
Summerdale
Foley
Magnolia Springs
Bon Secour
Gulf Shores
Perdido
Wilmer
Semmes
Theodore
Tillmans Corner
Grand Bay
Bayou la Batre
Dauphin Island
Bon Secour National Wildlife Refuge
ALABAMA
FLORIDA
Esri, HERE, DeLorme, MapmyIndia, © OpenStreetMap contributors, and the GIS user community, Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Appendix I: Post-Development Stormwater

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1. Computation of Travel Time and Time of Concentration

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a given outlet point. T_c is sum of T_t values for the various consecutive flow segments. These segments can be sheet flow, shallow concentrated flow, open channel flow, or a combination of these.

Sheet Flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. Manning's kinematic solution can be used to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

Where: T_t is travel time (hr)
n is Manning's roughness coefficient (Table I.1)
L is flow length (ft)
P_2 is 2 year, 24-hour rainfall (in)
S is slope (ft/ft)

Table I.1 Manning's n for sheet flow (USDA 2010)

Surface description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils: Residue cover ≤20% Residue cover >20%	 0.06 0.17
Grass: Short grass prairie Dense grasses [2] Bermuda grass	 0.15 0.24 0.41
Range (natural)	0.13
Woods:[3] Light underbrush Dense underbrush	 0.40 0.80

[1] The n values are a composite of information compiled by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Shallow concentrated flow

Sheet flow becomes shallow concentrated flow after approximately 100 feet. The average velocity is a function of the watercourse slope and the type of channel and can be determined from Figure

I.1. After determining the velocity, travel time for the shallow concentrated flow can be estimated as follows:

$$T_t = \frac{L}{60\ v}$$

Where: T_t is travel time (min)
L is flow length (ft)
v is average velocity (ft/s)

Open channel flow

Shallow concentrated flow occurs at shallow depths of 0.1 to 0.5 feet. Beyond that channel flow is assumed to occur. Manning's equation can be used to estimate average flow velocity for open channel flow:

$$v = \frac{1.49(R)^{\frac{2}{3}}(S)^{\frac{1}{2}}}{n}$$

Where: v is average velocity (ft/s)
R is hydraulic radius (ft)
S is channel slope (ft/ft)
n is Manning's n value for open channel flow

Manning's n value can be obtained from Chow (1959) and other references.

2. **Graphical Peak Discharge Method**

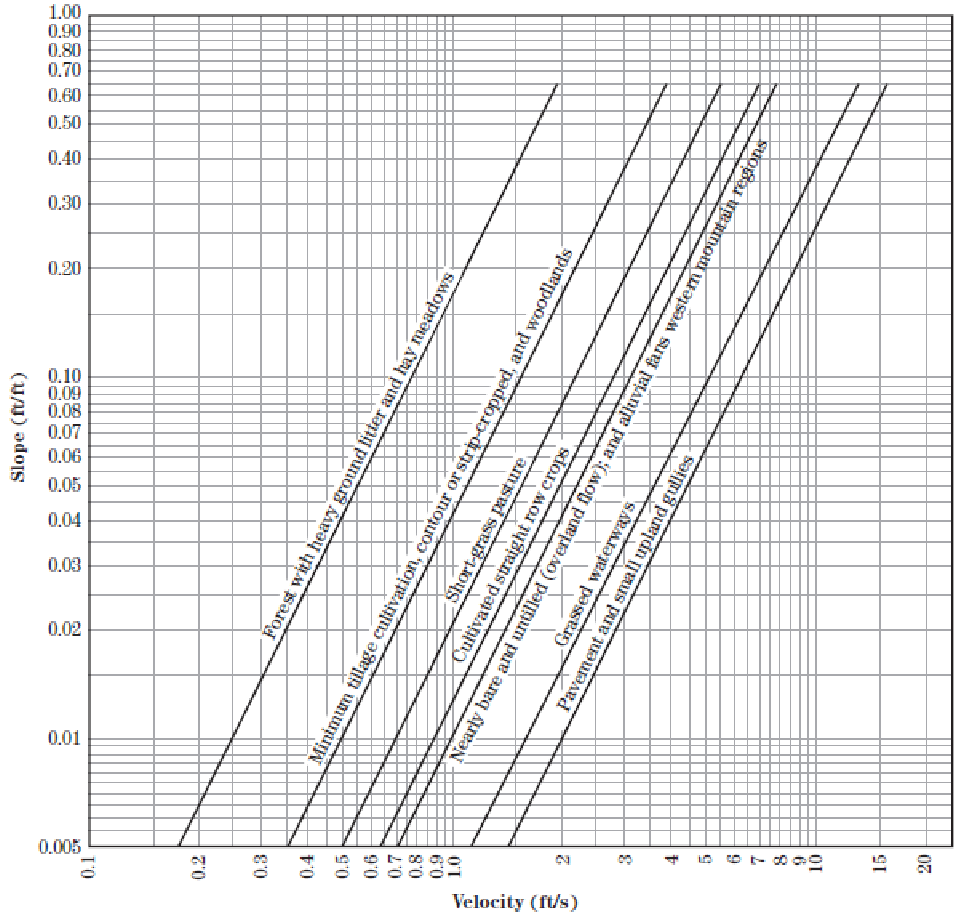
This method was developed from hydrograph analyses using TR-20, "Computer Program for Project Formulation - Hydrology" (SCS 1983). The peak discharge equation used is:

$$Q_p = q_u\ A\ Q\ F_p$$

Where: Q_p is peak dischage (ft^3/s),
q_u is unit peak discharge (csm/in),
A is drainage area (mi^2),
Q is runoff volume (in), and

F_p is pond and swamp factor (Table I.2)

After the modified CN and T_c are computed, peak discharge per square mile per inch of runoff (q_u) is obtained from Figure I.2 or I.3 by using rainfall distribution type and I_a/P ratio.



Flow type	Depth (ft)	Manning's *n*	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V = 20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V=16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V=9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V=8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V=6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V=5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V=2.516(s)^{0.5}$

Figure I.1 Average velocities for estimating travel time for shallow concentrated flow (USDA 2010)

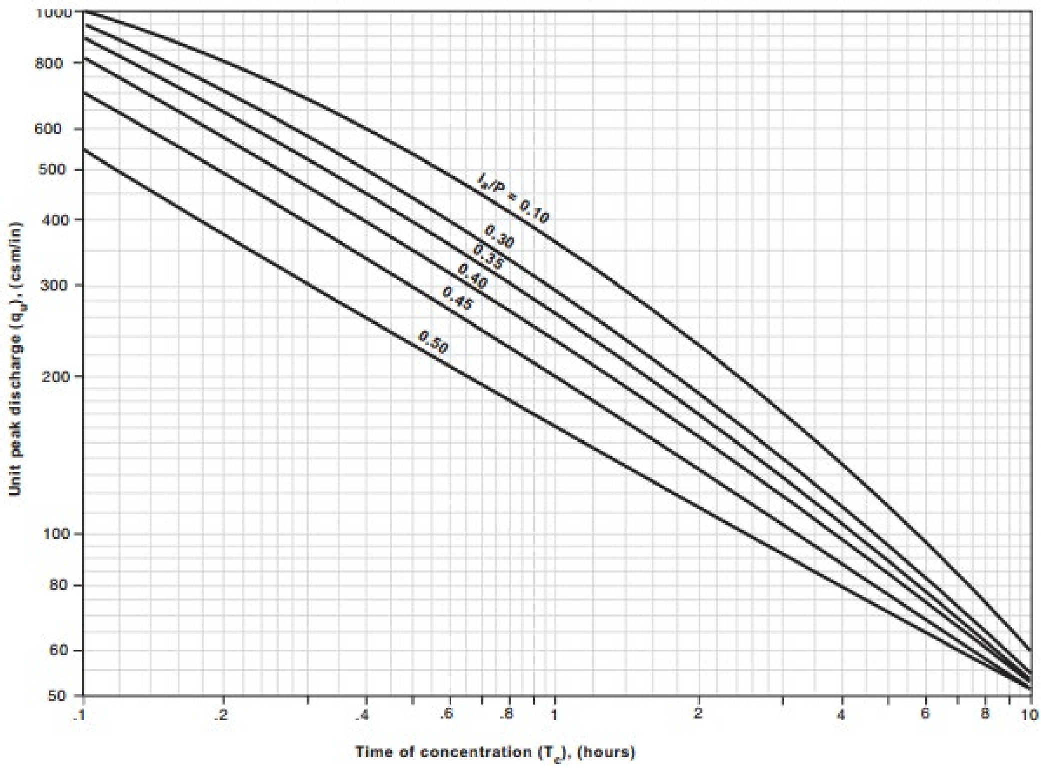


Figure I.2 Unit peak discharge (q_u) for Type II rainfall distribution (USDA 1986)

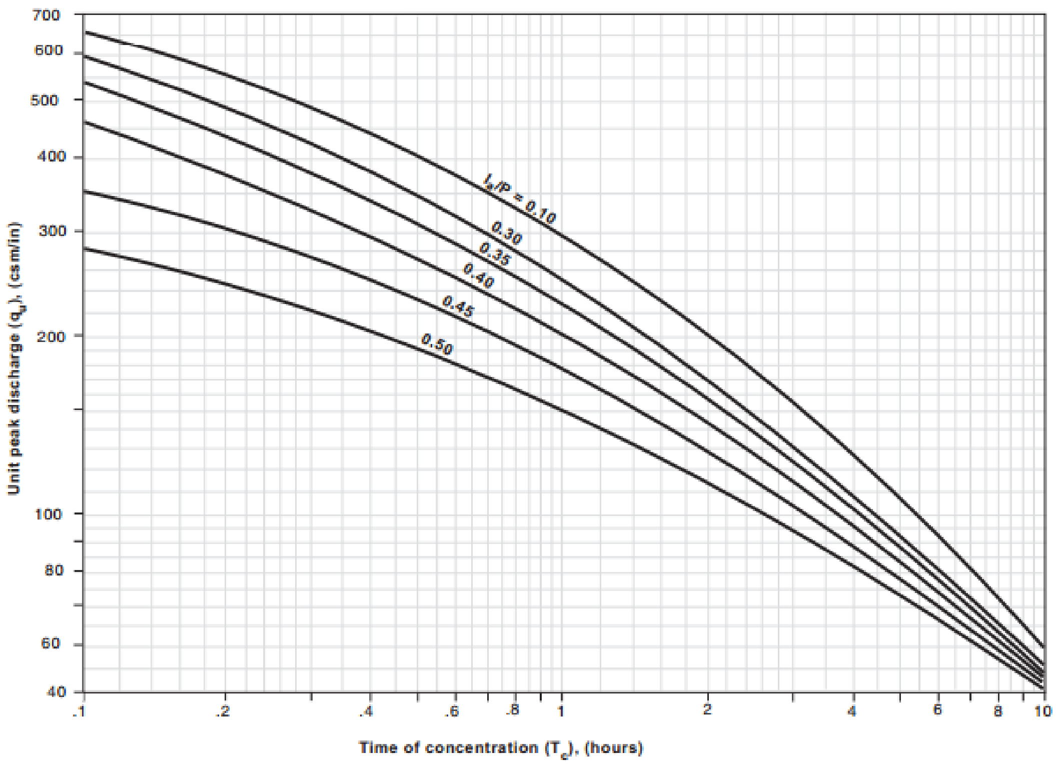


Figure I.3 Unit peak discharge (q_u) for Type III rainfall distribution (USDA 1986)

Table I.2 Factor for Pond and Swamp Areas (USDA, 1986)

Pond and Swamp Areas (%1)	Fp
0	1.00
0.2	0.97
1	0.87
3	0.75
5 or greater	0.72

[1] Percent of entire drainage basin

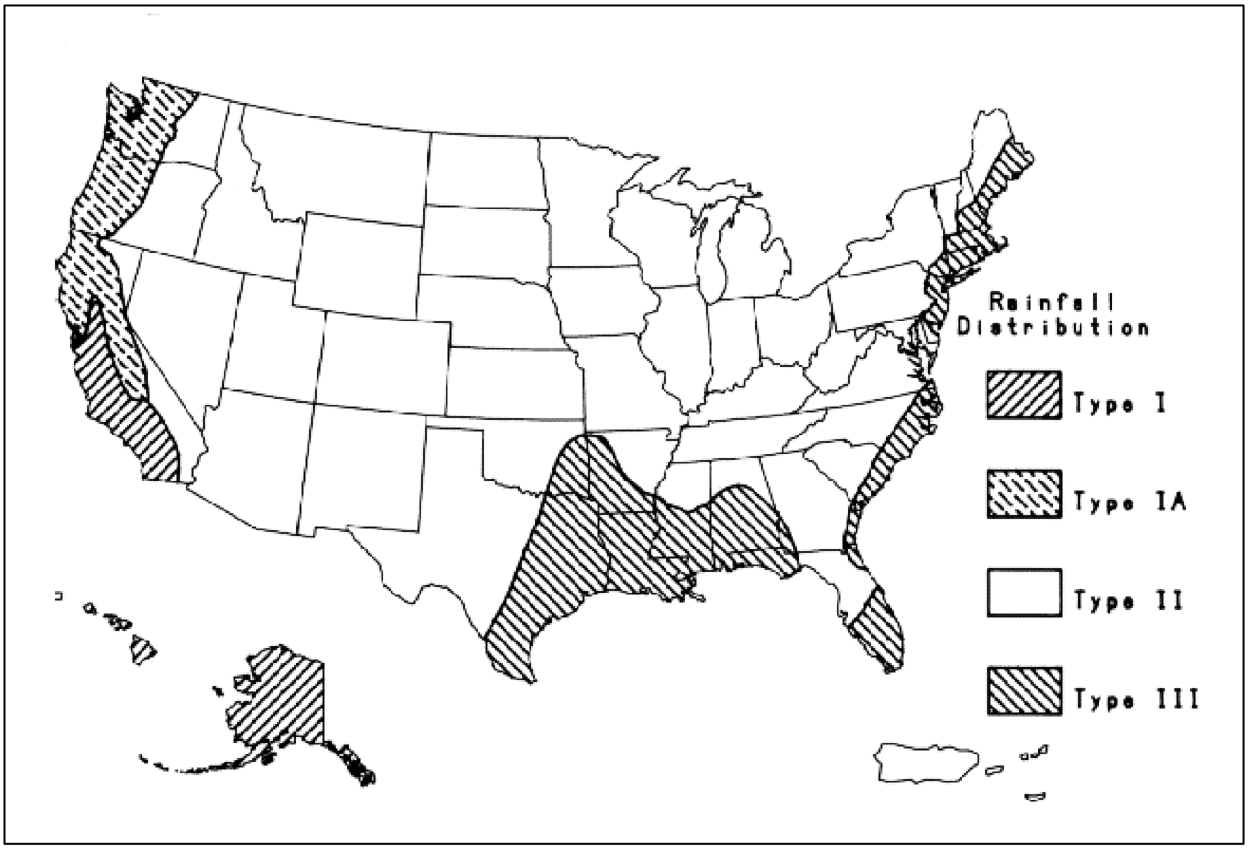


Figure I.4 Approximate geographic boundaries for rainfall distribution Types

POST-DEVELOPMENT STORMWATER RISK ASSESSMENT

This document provides the rationale and sequential procedures for assessing risk of impacts from post-development stormwater discharge.

Pursuant to the GFO 3-73, and working within the constraints of the project, designers must provide features and practices that cause post-development hydrology to mimic pre-development baseline hydrology of the site to the maximum extent practicable for small, frequent rain events up to and including a 95th percentile rain event at all locations of discharge. The risk assessment for post-development changes in stormwater discharges will focus on two categories of possible impacts: impacts to structures near or downstream from the site, and impacts to any streams, ponds or lakes that may receive the stormwater discharges. Although the risk assessment analysis is focused on impacts from the small, frequent rainfall events up to and including a 95th percentile rainfall, these small storm events can predict possible impacts of larger storm events from a 2-year storm up to a 100-year storm. Stormwater discharges may affect downstream structures such as buildings, culverts, bridges, levees, dams, etc. by flooding. Such damage could occur as a result of the direct flow of stormwater or by increasing the flow of downstream receiving waters. Evidence of pre-development flood damage and/or evidence of potential post-development damage after small rain events will provide guidance for selection and installation of appropriate stormwater controls that can reduce risk of more significant damage from larger storm events.

Post-development increase in stormwater discharge may also affect the stability and function of existing streams that receive the stormwater discharge. Increased stream flow above the baseline caused by stormwater discharge could incise the streambed and/or banks of receiving waters, resulting in post-development changes such as widening or deepening of the streambed, downstream deposition of sediment, impacts to aquatic biological organisms, or other problems. Thus, the potential damage or impairment of the streambeds of receiving waters from increased stormwater discharges should be assessed.

The following procedure serves as guidance for assessing post-development impacts, including scour and erosion, associated with site topographic modification, installation of facilities, and related infrastructure, including increased impervious areas, which could result in increased volume and force of stormwater discharges and potential flooding. A flow chart illustrating the procedure is included as Figure I.4.

Perform Hydrologic Analysis for the 95th Percentile Event

- Run hydrologic models for all discharge points leaving the right-of-way to determine if there will be increases in discharge for the 95th percentile storm event. If increased discharges are predicted, provide BMPs to mimic precondition hydrology to the maximum extent practicable and perform hydrologic analysis for larger storm events.

Perform Hydrologic Analysis for Larger Storm Events

- Run hydrologic models for all discharge points leaving the right-of-way to determine if larger events will increase discharge. If a possible increase in discharge is indicated, perform storage routing using the proposed culvert. If increased discharge will be present after storage routing, begin risk assessment.

Perform Risk Assessment

Desktop Review

- Complete Section A of Form HYD-100
  - Determine drainage area to outlet location
  - Review current aerials with drainage areas located
  - Note if there are buildings, ponds, or other structures downstream within the drainage area
  - If ponds exist, determine date of construction if possible

- Complete Section B of Form HYD-100
  - Review current flood studies
  - View floodplain and/or floodway boundary on the most current aerials
  - Identify other structures downstream that may be located in or near the floodplain or floodway
  - Identify and interview National Floodplain Insurance Program (NFIP) coordinator regarding community policies
  - Consult city engineer, county engineer, NFIP coordinator, or other public or knowledgeable private personnel regarding information including previous studies, surveys, or other available materials that may identify sensitive features or areas that would require additional attention to avoid or minimize future claims and impacts

- Complete Section C of Form HYD-100
  - Determine environmental impacts that could affect hydraulic design
  - Determine if the receiving waters are ephemeral, intermittent, or perennial
  - Using soil survey or core borings, identify the types of soil and/or other geological features in or near the site (sand, silt, or clay)

- Complete Section D of Form HYD-100
  - Determine average daily traffic for present year and design year
  - Determine what routes may be affected (school, mail, emergency etc.)
  - Determine if detours are available if route is closed
  - Determine if the available detour route(s) is an interstate, freeway, arterial, collector, or local
  - Describe the existing roadway including the pavement type, shoulder type, number of lanes, median type, and width of each (N/A for new alignment)

Site Visit

- Complete Section A of Form HYD-101
  - Determine the stream slope and if there are any drops greater than 2 feet
  - Determine the material in the stream bottom

  - Determine the material in the stream banks
  - Determine if the stream material is cohesive or non-cohesive
  - Determine if the stream shows evidence of degradation such as bank scour
  - Determine the material in the floodplain
  - Determine the kind and amount of vegetation in and along the channel
  - Determine the kind and amount of vegetation in the floodplain
  - Estimate Manning's n-values for the stream channel and floodplain
  - Determine other features that might affect water surface elevations

- Complete Section B of Form HYD-101
  - Note if scour is present around or near the structure
  - Describe the alignment and size of structure
  - Provide elevations for elements of structure such as low bridge superstructure, pipe or culvert inverts, low point of road, etc.
  - Provide road width, either shoulder-shoulder or curb-curb
  - Describe the condition of the existing structure

- Complete Section C of Form HYD-101
  - Estimate the flood damage potential
  - Note any buildings in and around the floodplain
  - Determine finished floor elevations of buildings
  - Describe the land use upstream and downstream

- Complete Section D of Form HYD-101
  - Determine if there is any historical highwater information
  - List the source and the location of the information
  - If information exists, note the date and elevation of the highwater
  - Estimate allowable highwater
  - Note any informal or available record(s) of damage from previous floods

- Complete Section E of Form HYD-101
  - Photograph pertinent features such as existing drainage structures, stream channel, floodplain, and any other key features
  - Provide an identification number or description for recording photos

- Complete Section F of Form HYD-101
  - Collect cross-section information and stream slope at any proposed crossing if it cannot be effectively obtained from a digital terrain model (dtm)

Risk Factor Assessment Form

Complete the Risk Factor Assessment form to identify any high risk factors that may be present. If any questions are answered "Yes," further hydrologic and/or hydraulic analysis should be performed to determine the extent of the possible impact.

Structures / Property

- During the desktop review, identify and note buildings or structures of any kind, including ponds, dams, levees, etc., within the boundaries of the FEMA mapped floodplain or special flood hazard area

- During the site visit, identify and note houses or structures of any kind, including ponds, dams, levees, etc., built near a stream that does not have a FEMA mapped floodplain
- Determine if there is personal property, including but not limited to vehicles or other movable property that could be impacted by flooding
- Determine from the property owner, city engineer, floodplain manager, etc. if there have been previous issues with flooding

Streams

- Determine if the streambed and stream banks consist mostly of a non-cohesive sand or silt. This can be determined during the site inspection or from soil borings
- Determine if there is pre-development evidence of scouring or incision of the streambed and/or stream banks, and/or if there is little to no stream bank vegetation
- Determine if the flood flow would likely break over the stream banks into the floodplain during a 2-year flood event
- Determine if any endangered or threatened species are present within the stream
- Determine if there will be outlets without energy dissipation that could accelerate channel degradation

RISK FACTOR ASSESSMENT FORM

Project Name/No: ______________________________ Date: ______________

County: ______________________________ Site No: ______________

Stream: ______________________________ By: ______________

High Risk Factors - Structures and Property

		Yes*	No
1	Is there a structure in the mapped FEMA Special Flood Hazard Area?		
2	Is there a structure built near the stream in an unmapped floodplain area?		
3	Is there a threat of property damage (other than a structure)?		
4	Is there history of previous flooding?		
5	Is there a privately owned pond, levee, etc. that will be impacted?		
6	Other? Describe if Yes.		

* If any of these items were answered Yes, then perform a hydrologic and hydraulic analysis for the 2-year 24-hour event through the 100-year 24-hour storm event

High Risk Factors - Streams

		Yes*	No
1	Does the stream mainly consist of a non-cohesive silt or sand?		
2	Is the stream already degrading and have little to no bank vegetation?		
3	Is the stream unable to utilize the floodplain on a 2-year event?		
4	Are there endangered species that are impacted?		
5	Will proposed outlet flow be concentrated without energy dissipation?		
6	Other? Describe if Yes.		

* If any of these items were answered Yes, then perform a hydrologic and hydraulic analysis for the 2-year 24-hour storm event

Criteria of recurrence intervals for hydrologic and hydraulic analysis

- If any items on the Risk Factor Assessment Form were answered "Yes," further hydrologic and hydraulic analysis shall be performed
- If any items on the 'Structures and Property' Form were answered "Yes," analyze the 2-year 24-hour storm and all other events up to and including the 100-year 24-hour storm event
- If any items on the 'Stream' Form were answered "Yes," analyze the 2-year 24-hour storm only
- Interchanges, support facilities, and rest areas shall meet the local stormwater ordinance criteria

In some instances there may be specific sites that require greater management of stormwater due to the conditions of the location. In these cases, a context sensitive design approach will be used.

Figure I.4 Risk Assessment Flow Chart