Hydraulic Design Manual

Revised September 2019

Manual Notice 2019-1

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Manual: *Hydraulic Design Manual*

Effective Date: September 12, 2019

Purpose

To implement new research and best practices.

Contents

The following updates were made to the *Hydraulic Design Manual*:

Chapter 4 – Hydrology

- Section 2 – Added brief discussion on significant digits.
- Section 9 – Updated Statistical Analysis of Stream Gage Data with new release of USGS Bulletin 17C.
- Section 10 – Updated mean annual precipitation map for use in Regression equations.
- Section 11 – Minor edits to time of concentration (Tc) guidance.
- Section 12 &13 – Updated to NOAA Atlas 14 rainfall data.
- Section 13 – Update on Rainfall Temporal Distribution based on NRCS guidance.
- Section 13 – Added additional Peak Rate Factor (PRF) guidance.

Chapter 15 – Coastal Hydraulic Design

- Added a new chapter providing guidance for designing or evaluating coastal hydraulic transportation infrastructure projects.

Supersedes

The revised manual supersedes prior versions of the *Hydraulic Design Manual.*

Contact

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Archives

Past manual notices are available in a PDF archive.

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Chapter 1 — Manual Introduction

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Section 1 — About this Manual

Purpose

Hydraulic facilities include open channels, bridges, culverts, storm drains, pump stations, and storm-water quantity and quality control systems. Each can be part of a larger facility that drains water. In analyzing or designing drainage facilities, your investment of time, expense, concentration, and task completeness should be influenced by the relative importance of the facility. This manual provides procedures recommended by the Texas Department of Transportation (TxDOT) for analyzing and designing effective highway drainage facilities.

Conventions and Assumptions

This manual assumes that hydraulic designers have access to programmable calculators, computer spreadsheets, and specific hydraulic computer programs.

Organization

This manual is organized as follows:

- Chapter 1: Manual Introduction
- Chapter 2: Hydraulic Policy and Governing Law
- Chapter 3: Processes and Procedures in TxDOT Hydrologic and Hydraulic Activities
- Chapter 4: Hydrology
- Chapter 5: Federal Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP) Compliant Design of Floodplain Encroachments and Minor Structures
- Chapter 6: Hydraulic Principles
- Chapter 7: Channels
- Chapter 8: Culverts
- Chapter 9: Bridges
- Chapter 10: Storm Drains
- Chapter 11: Pump Stations
- Chapter 12: Reservoirs
- Chapter 13: Storm Water Management

Feedback

Direct any questions or comments on the content of the manual to the Director of the Design Division, Texas Department of Transportation.

Section 2 — Introduction to Hydraulic Analysis and Design

The involvement of hydraulic engineers from the Design Division or at the district level should ideally begin in the project initiation phase of a project. In some cases such early involvement may not be justified or feasible. In all projects requiring any significant input from hydraulics, input should start no later than the beginning of planning phase. Hydraulic engineering input at the earliest stages of the project can help the project manager to anticipate important project elements that could impact the project cost or schedule. Examples of such elements include but are not limited to the following:

- Regulatory elements, such as National Flood Insurance Program (NFIP) floodways, that could impose significant constraints,
- Existing drainage structures (such as culverts) that are hydraulically inadequate and which may require complete replacement rather than mere extension as part of a widening project,
- Opportunities to avoid complete replacement of drainage structures through various types of rehabilitation,
- Fundamental hydraulic or stream stability problems at a proposed new stream crossing location,
- Upcoming or ongoing flood control projects by other parties that could improve or alter the drainage situation at a given location.

Hydraulic engineering expertise can be applied to a broad range of aspects of a TxDOT project including environmental documentation and mitigation, cross-drainage design, pavement drainage and storm drain design, detention facilities, storm water quality best management practices, and regulatory compliance. The types of projects requiring or benefiting from hydraulics input include:

- Highway widening or reconstruction
- Urban street reconstruction
- Intersection improvements
- Interchange addition or modifications
- Bridge replacements
- Constructing routes on new alignments
- Safety improvement projects
- Chronic maintenance problem remediation
- Pump stations
- Storm water quantity and quality control systems.

The hydraulic design or analysis of highway drainage facilities usually involves a general procedure, the specific components of which vary for each project. Some of the basic components inherent in the design or analysis of any highway drainage facility include data, surveys of existing characteristics, estimates of future characteristics, engineering design criteria, discharge estimates, structure requirements and constraints, and receiving facilities.

Time, expense, focus, and completeness of the design or analysis process should all be commensurate with the relative importance of the facility, that is, its cost, level of use, public safety, impact to adjacent lands, and similar factors. These aspects of the design process are often subjective. The funding or time constraints associated with any engineered project often are determining factors in the designer’s involvement.

Chapter 2 — Hydraulic Practices and Governing Law

Contents:

Section 1 — Overview

This chapter briefly describes the laws and related policies that affect hydraulic design for TxDOT projects. Federal and state regulations and rules have the force of law, and compliance is not at the discretion of TxDOT.

Federal Highway Administration (FHWA) sets forth policy and guidance in the Federal Aid Policy Guide (FAPG). The primary policy for drainage is 23 Code of Federal Regulations (CFR) 650, which is described later in this chapter.

Section 2 — Federal Laws, Regulations, and Agencies Governing Hydraulic Design

This section provides an overview of the federal regulatory environment as it relates to hydraulic considerations for TxDOT projects. It is not, however, an exhaustive list of all federal regulations that may pertain to highway drainage.

The following subsections discuss:

- National Flood Insurance Act
- Executive Order 11988
  - U.S. Department of Transportation Order 5650.2
- National Environmental Policy Act
- Rivers and Harbors Act
- Clean Water Act
  - Section 402 National Pollutant Discharge Elimination System
  - Section 404 Regulatory Program
  - Section 401 Water Quality Certification
- 23 CFR Part 650 Subpart A
- 23 CFR Part 650 Subparts C and H
- Memoranda of Understanding

It is possible to comply with the Federal requirements regarding the encroachment of a highway on a floodplain and still risk future legal liabilities because of the impact of the highway on the floodplain and the stream. Hydraulic engineers should review these potential liabilities and ensure that their evaluation is considered in design of highway projects.

National Flood Insurance Program

The National Flood Insurance Program (NFIP) was established under the National Flood Insurance Act (NFIA) in 1968 to reduce future flood losses through local floodplain management. NFIP requires participating cities, counties, or states, to adopt floodplain management ordinances containing certain minimum requirements intended to reduce future flood losses.

Special Flood Hazard Areas (SFHAs) are depicted on Flood Insurance Rate Maps (FIRMs) or Flood Hazard Boundary Maps (FHBMs) that have been prepared by Federal Emergency Management Agency (FEMA) for each participating community. The participating community is responsible for informing FEMA of any alterations or changes to the floodplain. TxDOT requires that designers inform the participating community through its Floodplain Administrator (FPA) of any changes to the floodplain or its parts via FPA notification.

The following list identifies some typical conditions that must be checked for consistency with the requirements:

- Replacement of existing bridge with smaller opening area, e.g., shorter length, deeper deck, higher or less hydraulically efficient railing.
- Replacement of bridge and approach roadway with an increase in the roadway profile.
- Safety project involving addition of safety barrier.
- Rehabilitation or maintenance of roadway resulting in a higher profile.
- Highway crossing at a new location.
- Longitudinal encroachment of highway on floodplain (with or without crossing).

For more information on the NFIP, see Chapter 5 of this manual.

Executive Order 11988

Executive Order 11988, May 24, 1977, requires each federal agency, in carrying out its activities, to take action (1) to reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare, and to restore and preserve the natural and beneficial values served by floodplains; (2) to evaluate the potential effects of any actions it may take in a floodplain, to ensure its planning programs reflect consideration of flood hazards and floodplain management; and (3) to submit a report to the Council of Environmental Quality (CEQ) and the National Water Resources Council (WRC) on the status of procedures and the impact of the Order on the agency's operations. This executive order applies mostly to state buildings in the floodplain, but also requires TxDOT to consider alternatives that will not impact the floodplain. U.S. Department of Transportation Order 5650.2 contains DOT policies and procedures for implementing E.O. 11988.

National Environmental Policy Act

The National Environmental Policy Act (NEPA) was passed in 1969, 42 United States Code (USC) 4321-4347, to establish a national policy to protect the environment.

For more information on NEPA, see the TxDOT *Environmental Management System Manual*.

Rivers and Harbors Act

The U.S. Army Corps of Engineers (USACE) began regulating activities in navigable waters with the Rivers and Harbors Act of 1899.

For more information on the Rivers and Harbors Act, see the TxDOT *Environmental Management System Manual* and the TxDOT Bridge Project Development Manual.

Clean Water Act

The Clean Water Act (CWA) of 1972, 33 USC 1251-1387, was enacted to maintain and restore the chemical, physical, and biological integrity of the waters of the U.S. The broader jurisdiction under this law includes not only navigable waters, but also most waters of the country and adjacent wetlands. Provisions of the CWA are enforced by the Texas Commission on Environmental Quality (TCEQ) and the USACE.

A water discharge permit or coordination is required whenever a project directly or indirectly impacts water resources. For more information on the CWA, see the TxDOT *Environmental Management System Manual*.

- **Section 402 National Pollutant Discharge Elimination System** In 1990, the EPA published 40 CFR Part 122, which contains regulations for the National Pollutant Discharge Elimination System (NPDES) storm water discharge permits. The purpose of this legislation is to improve the quality of the nation's rivers, lakes, and streams. NPDES regulations are administered by Environmental Protection Agency (EPA) and TCEQ through the Texas Pollutant Discharge Elimination System (TPDES) and the Construction General Permit (CGP).

  The CWA makes it unlawful to discharge storm water from most construction sites in Texas, unless authorized by the TPDES CGP. Unlike an individual permit that authorizes discharge activities for a specific location, the general permits are for a specific activity (i.e. construction). The operator seeking authorization to discharge storm water is required to comply with the terms of the permit.

  For more information on the CGP, see the TxDOT *Environmental Management System Manual*.

- **Section 404 Regulatory Program** Section 404 of the Clean Water Act (CWA) establishes a program to regulate the discharge of dredged and fill material into waters of the U.S., including wetlands. Section 404 makes it unlawful to discharge dredged or fill material into waters of the U.S. without first receiving authorization from the USACE. Activities that typically require authorization include placement of culvert pipes, bridge piers, riprap, or any other alteration to the stream including relocation.

  The Section 404 Program can issue the following permits:

  - nationwide permits
  - individual 404 permit
  - general permit

Some types of permits do not require individual review and approval by the USACE, while others may take several years to process and require extensive mitigation for impacts to Waters of the U.S. The type of permit that will be required depends on the degree of impact. Projects that impact less than 0.10 acre below the ordinary high water mark of the water body, and do not impact any wetlands, can often be authorized without individual review by the USACE.

For more information on the Section 404 Regulatory Program, see the TxDOT *Environmental Management System Manual*.

- **Section 401 Water Quality Certification** The issuance of any of the above permits is contingent on receipt of a water quality certificate or waiver of certification from the State in which the work is to be done. This certification assures that the proposed project will not violate effluent limitations and water quality standards established pursuant to Section 401 of the CWA, 33 USC 1341, as amended. Under Section 401, TCEQ is authorized to certify that federally issued permits will meet the state's water quality standards. TCEQ regulates this section under the USACE permit program and requires the installation of temporary and permanent storm water best management practice devices (BMPs) that have been approved by TCEQ. Environmental documents should include a general description of the measures that will be taken to minimize the potential for impacts to receiving waters under Section 404 and a discussion regarding compliance with Section 401 of the Clean Water Act.

  For more information on Section 401 Water Quality Certification, see the TxDOT *Environmental Management System Manual*.

23 Code of Federal Regulations 650 Subpart A

When a TxDOT project with participation by the FHWA involves an encroachment on the 1% Annual Exceedance Probability (AEP) (100-yr event) floodplain, the location and design of the project must comply with FHWA Policy 23 CFR 650, Subpart A. Compliance with this regulation is required when a proposed project includes a new or expanded encroachment on a floodplain regulated by FEMA, or contains the potential for adversely impacting private property or insurable buildings on or near a floodplain.

The FHWA has prepared a non-regulatory supplement, 23 CFR 650, Subpart A, Attachment 2, which explains the requirements for coordination with FEMA and the local community responsible for administering the NFIP under different floodplain encroachment scenarios. Chapter 5 of this manual explains TxDOT procedures for compliance with these requirements.

23 Code of Federal Regulations 650 Subparts C and H

The January 2005 updated regulation, 23 CFR 650, Subpart C, underscores FHWA guidance regarding Plans of Action (POA) for scour critical bridges. TxDOT scour issues and countermeasure designs are handled by the Bridge Division, Geotechnical Section. Refer to the TxDOT Geotechnical Manual for more information. The regulation 23 CFR 650, Subpart H requires coordination with the United States Coast Guard (USCG) and USACE in providing adequate vertical and horizontal clearance for navigation on navigable waterways and is covered in the TxDOT Bridge Project Development Manual.

Memoranda of Understanding (Federal)

Some projects may be governed or affected by a Memorandum of Understanding (MOU). An MOU is an executed understanding between TxDOT and other state or federal agencies. The purpose of an MOU is to guide both parties concerning their roles and responsibilities necessary to achieve effective coordination of project activities. MOUs are used to expedite the review process and minimize the required documentation for such items as:

- Funding
- Design criteria
- Construction
- Maintenance.

TxDOT has not negotiated MOUs for hydraulic design with any federal agencies.

Section 3 — State Statutes and Rules Governing Hydraulic Design

As with Federal laws, TxDOT must comply with State regulations and statutes. This chapter explains some of the relevant state regulations. It is not an exhaustive discussion of state regulations that could affect TxDOT hydraulic design.

The following subsections discuss:

- Texas Water Code Chapter 11
- Texas Water Code Chapter 16 Subchapter I
- Title 30 Texas Administrative Code Chapter 299
- Title 43 Texas Administrative Code Rule 15.54(e)
- Memoranda of Understanding, State
- Texas Executive Order D.B. No. 34

Texas Water Code Chapter 11

Section 11.021

The Texas Water Code Section 11.021 states that the water of the ordinary flow, underflow, and tides of every flowing river, natural stream, and lake, and of every bay or arm of the Gulf of Mexico, and the storm water, floodwater, and rainwater of every river, natural stream, canyon, ravine, depression, and watershed in the state is the property of the state. Water imported from any source outside the boundaries of the state for use in the state and which is transported through the beds and banks of any navigable stream within the state or by utilizing any facilities owned or operated by the state is the property of the state.

Section 11.086

The Texas Water Code Sections 11.086 states that no person may divert or impound the natural flow of surface waters in Texas, or permit a diversion or impoundment to continue, in a manner that damages the property of another by the overflow of the water diverted or impounded. A person whose property is injured by an overflow of water caused by an unlawful diversion or impoundment has remedies at law and in equity and may recover damages occasioned by the overflow.

Texas Water Code Chapter 16 Subchapter I

Texas Water Code Chapter 16, Subchapter I establishes the positive interest of the State of Texas in the NFIP. TxDOT is an entity of the state and is prohibited from obtaining permits from subordinate jurisdictions. Also, the State of Texas (and therefore TxDOT) is not a participating community in the NFIP. TxDOT will, however, work with communities to prevent flood damage and minimize impacts, as obligated by this statute. See Chapter 5 for more information.

Title 30 Texas Administrative Code Chapter 299

Regulation of the Texas Dam Safety Program was established by the TCEQ and is contained in 30 TAC Chapter 299, which provides for the safe construction, maintenance, repair and removal of dams in Texas.

Title 43 Texas Administrative Code Rule 15.54(e)

This section of the TAC describes the conditions under which state, federal and local financing of drainage construction costs are to be shared. In general TxDOT's responsibility includes:

- Constructing drainage systems, including outfalls, within the state right of way
- Adjusting or relocating existing drainage channels when necessary
- Adjusting structures and channels to accommodate any approved drainage plan.

Although TxDOT can adjust a facility to accommodate public improvement works that directly benefit the operation of the highway, it is not required to make changes to highway facilities just to accommodate development in the drainage area.

Parties wishing to discharge drainage onto or across the state highway right of way, where there is no existing drainage system, must obtain approval from TxDOT and provide design, construction, and maintenance costs. Local governments wanting to connect to a TxDOT drainage system must first have approval from TxDOT, and then must bear the cost of collecting and carrying its water to the TxDOT system as well as contribute a share of the TxDOT system costs.

Memoranda of Understanding (State)

The Texas Transportation Code, 201.607 requires TxDOT to adopt a MOU with each state agency that has responsibility for protection of the natural environment or for preservation of historical or archeological resources. Environmental documents that meet MOU criteria are sent to these agencies for review and comment.

One MOU significant to TxDOT Hydraulic Design is the agreement between TxDOT and the Texas Natural Resources Conservation Commission (TNRCC), now the TCEQ, which acknowledges that TxDOT is complying with minimum NFIP regulations in the work that it conducts in flood hazard areas.

Texas Executive Order D.B. No. 34

This 1977 Executive Order, Evaluation of Flood Hazard in Locating State Owned or Financed Buildings, Roads, and Other Facilities, was signed to bring the State of Texas into compliance with Presidentail Executive Order 11988.

Section 4 — Policies, Standard Practices, Requirements, and Guidance

This manual identifies those policies, standard practices, criteria, guidance and references approved for use in carrying out the hydraulic design responsibilities in TxDOT. In this regard, the following definitions will be used:

- A **policy** is a statement of position, reflecting the preferred philosophy of the agency. Policy comprises a set of self-imposed boundaries on decisions in the course of business under ordinary or anticipated conditions.
- A **standard** is a fixed reference to guide the outcome and content (product) of the work. Standards are established where there is a consistent level of risk or there is a consistent technical or performance expectation for a specific product to work well in most cases. In this manual standards frequently refer to the expected design Annual Exceedance Probability (AEP) for a particular type of drainage structure on a particular class of highway. (See Chapter 4 for an explanation of AEP). Variances to TxDOT standards are not uncommon, but they need always be justified in writing. Attention to variances and guidance on how to request and justify them are included in this manual. Exceptions and waivers to the standards are handled in each section as applicable.
- **Criteria** are tests or indicators, in addition to standards, used to measure/judge achievement of applicable policy or standard objectives. Criteria may vary from project to project. An example of a criterion in this manual is the constraint on headwater depth at a culvert.
- TxDOT **standard practices** are methods and procedures that have a history of use within the department for addressing situations characteristic and commonly encountered. The only justification needed for the use of standard practices is evidence that they are appropriate for the situation at hand. Deviation from standard practices may be required in any situation where evidence that standard practices are applicable cannot be readily demonstrated.
- **Guidance** refers to recommended, but not necessarily required, actions to meet policies and standards, and expectations for applying discretion.
- **Discretion** in this context refers to engineering judgment applied by the practitioner to an appropriate technique or solution that is within an acceptable range of values.

In this manual, considerations for identifying appropriate standards, design criteria, and standard practices are included at the beginning of each chapter. Guidance is provided where appropriate to assist the user in formulating an approach to meet the standards and criteria.

Ignoring the appropriate standards and design criteria may result in a projeict delayed from the scheduled letting. If an exception is needed to a standard or design criterion, the TxDOT Project Development Process Manual (PDP) should be consulted for directions on applying for exceptions and waivers. Design Division Hydraulics Branch (DES-HYD) should be consulted as early as possible to work out an acceptable alternative.

Section 5 — Roles and Responsibilities for Hydraulic Analysis and Design

This section presents information on the roles and responsibilities of design engineers, managers and consultants in the hydraulic analysis and design process of TxDOT projects.

Area/ Design Office Engineers

Design Engineers. The responsibilities of the Design Engineers as they relate to hydraulic analysis and design include the following:

- Develop and maintain proficiency and competency in all aspects of hydrologic and hydraulic analysis and design. Educate, train and develop junior engineers and designers.
- Prepare and oversee preparation of drainage analyses and designs including hydrology, open channel hydraulics, bridge hydraulics, culvert hydraulics, storm drains, and pump stations.
- Ensure appropriate coordination and communication between drainage design functions and all other project design functions, including roadway, geotechnical, bridge, and signage.
- Ensure compliance with applicable laws, regulations, policies, and District/Area Office preferences.
- Provide and maintain documentation of all drainage analyses and designs in accordance with District/Area Office preferences and this manual.
- Develop and maintain systematic documentation files of facility experiences either at the district or at the local level
- Provide quality control techniques for work performed by subordinates and perform quality assurance on the work.
- Promptly report drainage complaints to the District Hydraulics Engineer (DHE).
- Promptly notify the DHE of any significant runoff and flood events and collect appropriate data (photographs, survey data, etc) for documentation.

Consultant Engineers. The responsibilities of Consultant Engineers as they relate to hydraulic analysis and design of TxDOT projects include the following:

- Develop and maintain proficiency and competency in all aspects of hydrologic and hydraulic analysis and design. Educate, train and develop junior engineers and designers.
- Prepare and oversee preparation of drainage analyses and designs, including hydrology, open channel hydraulics, bridge hydraulics, culvert hydraulics, storm drains, and pump stations.
- Ensure compliance with applicable laws, regulations, policies, and District/Area Office preferences.

- Provide and maintain documentation of all drainage analyses and designs in accordance with District/Area Office preferences and this manual.
- Provide quality control for work performed by subordinates.
- Comply with contract scope and requirements.

TxDOT Contract Managers

- Oversee consultant work authorizations in accordance with Design Division – Consultant Contract Office (DES-CCO) "Roles and Responsibilities" table.
- Review consultant work to ensure compliance with contract scope, applicable laws, regulations, policies, District/Area Office preferences, and adherence to TxDOT comments and directives.

TxDOT Project Managers

- Direct and coordinate preparation of drainage analyses and designs, including hydrology, open channel hydraulics, bridge hydraulics, culvert hydraulics, storm drains, and pump stations.
- Verify compliance with applicable laws, regulations, policies, and District/Area Office preferences.
- Ensure and oversee documentation of all drainage analyses and designs in accordance with District/Area Office preferences.
- Provide quality assurance for work performed by subordinates.
- Promptly report drainage complaints to the District Hydraulics Engineer (DHE).
- Promptly notify the DHE of any significant rainfall/runoff and flood events and collect appropriate data (photographs, survey data, etc) for documentation.
- Oversee consultant work authorizations in accordance with DES-CCO "Roles and Responsibilities" table.
- Review consultant work (Work Authorizations) to ensure compliance with contract scope, applicable laws, regulations, policies, District/Area Office preferences, and adherence to TxDOT comments and directives.
- Coordinate with the DHE to identify training needs and pursue training opportunities.

District Hydraulics Engineers

- Develop and maintain proficiency and competency in all aspects of hydrologic and hydraulic analysis and design. Educate, train and develop junior engineers and designers.
- Serve as point of contact for DES-HYD for dissemination of statewide policy, guidance, research, training, software and other related hydraulics and hydrologic issues or needs

- Serve as point of contact for district and area office staff for fundamental expertise on hydraulics and hydrology including consistent application of statewide and district design policy
- Serve as the liaison with the DES-HYD on complex hydraulics design and hydraulics-related design or policy issues
- Provide design support and approval for hydraulic and hydrologic methods for use by district, area office, and consultant designers
- Oversee and approve or perform routine hydraulic and hydrologic studies, designs, and analyses for district projects and consultant contracts
- Prepare, review, and comment on or approve hydrologic/hydraulic reports.
- Provide hydraulic, hydrologic and related regulatory review of projects prior to preliminary layout or Plans, Specifications, and Estimates (PS&E) submission for letting
- Provide mentoring to other district personnel in hydraulics and hydrology and for district staff in hydraulic and hydrology rotation
- Support the district environmental quality coordinator and/or environmental coordinator on hydraulics for environmental and water quality issues
- Investigate and help resolve drainage related issues with the public
- Support district maintenance staff with performance and maintenance of drainage structures, including data collection on chronic problem areas or extreme events
- Recommend research and volunteer as research project directors and advisors as approved by the district engineer
- Coordinate with local governments, developers, and property owners concerning hydraulics-related issues, including the monitoring of local studies and activities that may impact TxDOT drainage facilities or operations
- Coordinate with TxDOT project managers, DES-HYD staff, and District Human Resources staff to provide hydrologic and hydraulic training for TxDOT employees.

Design Division Hydraulics Branch (DES-HYD)

- Hydraulic Report review - DES-HYD is tasked to review and comment on hydraulic reports with regard to applicability, methodology, detail, documentation, accuracy, and composition. The sealing engineer (District or Consultant) is expected to address all comments and return the report in a timely manner.
- Preliminary review – DES-HYD is tasked to review preliminary designs. Typical designs reviewed include bridges and bridge class culverts. DES-HYD review covers methodology, detail, documentation, and accuracy to TxDOT standards. Preliminary submittals are preferred because they are early enough in the process that design corrections usually can be implemented.

- PS&E review – DES-HYD is tasked to review plans submitted for letting with regard to methodology, detail, documentation, and accuracy to TxDOT standards. The District or its Consultant is expected to make all reasonable changes to the plans prior to letting.
- Expert Guidance – DES-HYD is tasked to provide guidance, help, or advice on any hydrology and hydraulics subject as needed and called upon by the District or Area Offices.

Section 6 — Local Agency Ordinances and Requirements

Local agencies may be involved in numerous phases in TxDOT projects for various reasons. For instance, coordination with local agencies may be needed in order to ensure a project complements the surrounding community, or when facilities not owned or maintained by TxDOT are to be constructed, modified or affected by a TxDOT project. Local agencies may be required to participate in project development by providing funding.

TxDOT is not generally obligated to design or meet local agency requirements that may differ from or be more stringent than state or federal requirements. Certain situations may lead to TxDOT's acceptance of local requirements. For example, the Record of Decision (ROD) associated with an Environmental Impact Statement or Environmental Assessment may require adherence to certain local requirements as a condition of project approval. Another example may be a case in which the local agency is to assume ownership and maintenance of a drainage facility once the construction by TxDOT is complete. Any costs beyond what TxDOT deems necessary and proper associated with meeting local requirements will be the full responsibility of the local agency.

At the discretion of the District Engineer or other designated District personnel, TxDOT may choose to accommodate criteria different or more restrictive than those customary for TxDOT. Each District has reasonable latitude to act in the spirit of cooperation with other agencies, when to do so is deemed by District staff to be in the best interest of the public. Such accommodation should be on a case-by-case basis; prior accommodation should not be viewed as assurance of future accommodation.

Section 7 — Responding to Drainage Complaints

Drainage complaints should be dealt with promptly and in an unbiased manner. The following steps are presented to help TxDOT to obtain a thorough understanding of the basis of an individual complaint and assure the appropriate action is taken.

1. Office receiving the complaint must acknowledge the complaint within one day if possible. Individuals do not register complaints with only a casual interest in their outcome. Timely acknowledgement is indispensable.
2. Office receiving the complaint must notify or forward the complaint to the District Hydraulics Engineer within 1 day of receipt.
3. The DHE or District must notify DES-HYD of the complaint.
4. District must notify Office of General Counsel (OGC) if litigation is filed or threatened.
5. District must investigate the facts. Clearly determine the basis for the complaint, including the extent of flooding, complainant's opinion of what caused the flooding, description of alleged damages, and dates, times, and duration of flooding. Relate the history of other grievances at the site. DES-HYD should be called for technical assistance if necessary.
   a. Visit the site as soon as possible after receiving complaint
   b. Talk to the complainants
   c. Take photographs
   d. Take measurements
   e. Prepare notes from the site visit and investigation
   f. Locate and obtain as-built plans from latest and all applicable projects
   g. Obtain accurate GIS files if available
   h. Locate and obtain applicable hydrology and hydraulics reports
6. District must determine an appropriate course of action. Analyze the facts and decide what action to take to relieve the problem, regardless of who has responsibility for the remedy. Make conclusions and recommendations, describe the contributing factors leading to the alleged flood damage, and specify feasible remedies. Keep DES-HYD informed of the progress and developments.
7. District must prepare and file documentation. Ensure a file documenting the complaint, response and resolution is maintained.

Section 8 — Developments Connecting into TxDOT Hydraulic Structures

Drainage related issues are covered in the 43 TAC, Part 1, Chapter 15, Subchapter E. See Section 3 for more details.

TxDOT does not allow private or municipal connection to TxDOT storm drainage facilities without approval. Requests are submitted to the DHE and must be supported by full hydrologic and hydraulic analyses comparing existing and proposed conditions.

When a request is received by TxDOT, the DHE or DHE's representative will verify the applicant's existing conditions runoff computations and consider available increased flow, if any, for which the facility was designed. The DHE will use this information to determine how much if any additional flow may be received, at what design Annual Exceedance Probability (AEP), and whether by overland flow or direct connection. Once an acceptable design AEP flow rate is determined, the requestor is required to prepare plan sheets conforming to the TxDOT requirements and containing the required detail. TxDOT will review the engineering drawings for completeness and compliance, and approves the request after comments are addressed.

Section 9 — Dams

Under 30 TAC Chapter 299 a dam is defined as any barrier, including one for flood detention, designed to impound liquid volumes and which has a height of dam greater than six feet. This does not include highway, railroad, or other roadway embankments, including low water crossings that may temporarily detain floodwater, levees designed to prevent inundation by floodwater, closed dikes designed to temporarily impound liquids in the event of emergencies, or off-channel impoundments authorized by the TCEQ under Texas Water Code Chapter 26.

Dam Safety rules do not apply to roadway embankments, even though they may temporarily impound water, unless the embankment was also intended to function as a detention dam. TxDOT practice is to avoid using a highway embankment as a detention dam unless the embankment has been specifically designed to TCEQ dam specifications. TxDOT practice is to comply with 30 TAC Chapter 299 and avoid building roads on or near dams. Any questions regarding a roadway on or near a dam should be directed to the DHE or DES-HYD.

Chapter 3 — Processes and Procedures in TxDOT Hydrologic and Hydraulic Activities

Contents:

Section 1 — Overview

The nature and scope of hydraulic analysis and design work varies depending on the type of project being undertaken and on the hydrologic/hydraulic (H&H) setting of the project. Projects that consist of repair or minor alteration without change to the roadway profile may only require cursory examination for hydraulic effects. An overlay project may or may not require a full H&H analysis, depending on surrounding factors. A project with a bridge or culvert replacement may require different H&H inputs than a culvert extension or a bridge modification. All of the above project types differ from an urban street project with curb, gutter and storm drain. This chapter provides a general discussion on establishing the appropriate scope of hydraulic activities for the overall project and for various phases within a project. It also describes the required deliverables for a variety of project types.

Section 2 — Scope of Hydrologic and Hydraulic Activities

Scoping and reconnaissance are the investigative processes aimed at determining which issues are to be addressed by the project. Scoping initially identifies the major needs, issues, constraints, and feasibility of proposed improvements from which the more comprehensive, interdisciplinary preliminary engineering activities, surveys, investigations, environmental studies, and analysis can be effectively planned and budgeted. This includes the major elements of hydrologic and hydraulic work necessary to develop the project.

Reconnaissance is the collection of information that would generally be sought, collected, and used, as standard practices for the design whenever available and applicable. The following list includes broad categories for the H&H portion of the work:

- Previous hydrology/hydraulic studies and reports
- Hydrological data (rainfall, gage data, flood history, etc.)
- Site visit and reconnaissance
- Aerial/site photography
- Survey and mapping
- Land use, ground cover, soils information
- Fluvial geomorphic data (plan forms, bed and bank sediment characteristics, etc.)
- As-built plans
- Bridge inspection reports
- Maintenance reports

The nature and extent of work proposed for drainage structures will affect the level of H&H analysis and the applicability of the standards and criteria presented in this manual. The scoping and reconnaissance effort should always include an appropriate assessment of the existing physical condition and the hydraulic performance of all drainage structures. A site visit is usually required for proper reconnaissance. The findings of the assessment will lead to recommendations as to whether existing structures should be replaced, rehabilitated, modified, abandoned, or left undisturbed.

Hydraulic Considerations for Rehabilitated Structures

This chapter defines rehabilitated structures as existing structures that are not to be replaced, but may be substantially repaired, modified, or extended as part of the project. Common examples of rehabilitated structures include, but are not limited to:

- A culvert that is to be extended to accommodate roadway widening
- A culvert needing repair due to heavy corrosion

- A bridge deck to be reconstructed or widened
- A cross drainage structure beneath a road that is to be reconstructed
- A structure being retrofitted for fish passage
- An existing storm drain receiving additional or improved curb inlets
- A storm drain outfall requiring mitigation of scour problems

If a structure is to be rehabilitated, the standards and criteria presented in this manual may not be feasible, applicable or appropriate because of constraints imposed by project budget, right-of-way, or schedule. However, the impacts of a rehabilitation project, whether safety or maintenance projects, must be considered and evaluated. Such projects, for instance may cause changes to the flood surface profile, stream stability, or increase flood risk to neighboring properties. In determining whether a variance from standards and criteria is appropriate, an assessment of the risk involved should be undertaken, as discussed in the next section. The complete replacement of an existing structure that has exhibited no history of past problems must be justified by a compelling reason; simply that it does not meet current hydraulic criteria for new design is not sufficient.

Hydraulic Considerations for New Structures

The standards and criteria presented in this manual should be regarded as the minimum acceptable for projects involving new drainage structures or replacements of existing structures. Exceptions or variances may be justified by a risk assessment or detailed risk analysis. New and replacement structures should be, to the extent feasible, located, oriented, and sized so as to minimize the potential for hydraulic problems such as excessive scour or adverse impact on flood profiles.

Section 3 — Evaluation of Risk

As with other natural phenomena, occurrence of flooding appears to be governed by chance. The chance of flooding is described by statistical analysis of flooding history in the subject watershed or in similar watersheds. Because it is not economically feasible to design a structure for the maximum possible runoff from watershed, the designer must choose a design frequency, or inversely the Annual Exceedance Probability (AEP) of a flood appropriate for the structure. (See Chapter 4 for an explanation of AEP). Once a design AEP is selected, the structure should be designed to provide adequate capacity to appropriately convey the discharge associated with that probability. In this process the designer sets the level of conservatism by the selection of the design AEP. This is in contrast to the conservatism associated with structural design elements, which is typically based on safety factors in loading and structural capacity.

The design AEP can be established by standards or limited by factors such as economic considerations. Numerous methods have been developed to assist the engineer in assessing the risk involved in choosing the design flood and the check flood. For the purposes of this manual, risk is defined as the consequences associated with the probability of flooding attributable to the project, including the potential for property loss and hazard to life during the service life of the highway. A project can be fully compliant with policy and standards yet still incur an inappropriate level of risk. Consequently, all sources of potential risk should be considered as part of the H&H investigation for hydraulic structures in order to determine whether modified site-specific standards or criteria are appropriate.

If the consideration of risks appears to warrant a design criteria more or less stringent than the standard, a risk assessment should be conducted. As described below, the assessment of risk can be either qualitative or quantitative in nature. If the results of the assessment show that lower standards are warranted, the assessment will be documented through the design exception process and coordinated with the District Hydraulics Engineer (DHE) and the Hydraulics Branch of the Design Division (DES-HYD).

Most projects will require only a qualitative risk assessment. A qualitative risk assessment may be determined appropriate or inappropriate based on such considerations as the presence or absence of structures that could be impacted by the project, the perceived economic impact of temporary road closures, the environmental impact, or the cost of the roadway facility itself.

Highly complex, expensive projects or those with particularly high levels of risk may justify detailed and quantitative risk analyses. A quantitative risk analysis provides a detailed economic comparison of design alternatives using expected total costs (construction costs and maintenance costs plus risk costs such as the economic cost of an extensive and long-duration detour in the event of a failure, the cost of repair, etc.) to determine the alternative with the least total expected cost to the public. A quantitative risk analysis supports the appropriate design discharge and criteria based on the economic comparison of alternatives rather than a set of predetermined design AEPs and criteria such as those presented in this manual.

HEC-17, FHWA 1981 is a publication that provides procedures for the design of encroachments on floodplains using risk analysis. HEC-17 describes a quantitative assessment method called Least Total Expected Cost (LTEC). Least total expected cost refers to the result of a detailed economic analysis that attempts to account for all viable costs associated with a project. The analysis is ideally based on actual cost data.

Risk Assessment Forms

The TxDOT form titled "Economic and Risk Assessment for Bridge Class Structures" is a helpful resource in developing a qualitative risk assessment. The form has an associated worksheet to assist in developing a simplified estimation of the annual risk cost and annual capital cost. The form provides guidance on when a more detailed analysis following the HEC-17 LTEC approach is justified.

Section 4 — Design Activities by Project Phase

The following section describes the hydraulic design activities typically expected to occur in each phase of a project. The descriptions are largely derived from the TxDOT Project Development Process Manual.

Planning and Programming

One of the objectives of the planning and programming phase is to develop a planning-level cost estimate. Certain projects involving significant drainage-related challenges may require some initial hydrologic and hydraulic investigation in order to appropriately estimate the nature and approximate size of required drainage structures for estimating purposes. The DHE should be consulted during the planning and programming phase to assess whether drainage issues will pose significant challenges to the project.

Preliminary Design

In the preliminary design phase, the DHE should expect to participate in the Design Concept Conference to provide general background information on hydrology and hydraulics, and to identify major drainage features and regulatory constraints.

Drainage-related agreements and contracts that TxDOT has with other agencies need to be identified and taken into consideration during the preliminary design phase. Identification of existing agreements also helps determine the possible need for additional agreements. Some agreements may need to be amended and the appropriate division can assist. The Right of Way (ROW) and DES are involved with coordination of existing drainage agreements and in determining the need for additional agreements.

The locations and sizes of proposed cross-drainage structures (bridges and culverts) must be determined early in the preliminary design phase because of their potential to affect the roadway profile and other elements of the preliminary design of the project. Preliminary hydraulics analyses for bridges will enable the determination of the bridge limits, span/girder type, span lengths, bent locations, and bent orientation. An important aspect of the hydraulic analysis at this stage is consideration of the NFIP and whether the project will cross an NFIP designated Special Flood Hazard Area (SFHA).

Since many of the design parameters for drainage structures are to be established during the preliminary design phase, it is necessary to conduct the bulk of the hydrologic and hydraulic analysis during this phase. These analyses will usually include, but may not be limited to:

- Field reconnaissance
- Collection of relevant data on the stream and watershed
- Gathering of relevant previous hydrologic studies by TxDOT and other entities
- Conducting required hydraulic surveys of existing structures and streams
- Obtaining available topographic mapping of the streams and floodplains
- Establishing the relationship between flood discharge and AEP through hydrologic analysis or by adopting previous hydrologic analyses
- Determining stream flood profiles for existing conditions through hydraulic modeling
- Determining required sizes of drainage structures to meet design criteria
  - For bridges this includes establishing preliminary opening size, span lengths, pier locations and girder elevation
  - For culverts preliminary design of opening size and profile is performed
  - For storm drains this includes preliminary design of trunk alignment, size and profile
- Estimating stream flood profiles under proposed project conditions (potentially for multiple design alternatives) to determine project impacts
- Adjusting proposed structure designs as necessary to mitigate project impacts

All projects affecting a waterway used for navigation require coordination with the USCG and the USACE. Hydraulic investigations or design may also be required for ensuring compliance with the USCG and USACE regulations.

See the TxDOT Project Development Process Manual, Chapter 2 for further discussion of the preliminary design phase.

Environmental

Preliminary hydraulic studies are needed in the preparation of environmental documentation to evaluate the impacts of the proposed project on waterways and floodplains. Changes in water surface elevation, construction in channels, bridge construction methods, etc. commonly impact water resources. The identification of appropriate temporary and permanent stormwater quality best management practices may require input from the DHE and the District Environmental Quality Coordinator during the environmental documentation phase.

See the TxDOT Project Development Process Manual, Chapter 3 for further discussion of the environmental permitting and documentation phase.

PS&E Development

The Design Concept Conference marks the beginning of Plans, Specifications, and Estimates (PS&E) preparation and occurs after most of the background data is gathered and the preliminary hydrologic/hydraulic analysis and design is complete.

As part of the detailed design process, stream crossing hydrology and hydraulics should be refined and finalized. Refinement is usually needed to reflect detailed field survey data, changes in basic design conditions or assumptions, or to reflect revised methodology if there has been a significant delay between schematic development and PS&E development.

The FHWA requires a bridge scour evaluation during the hydraulic design process for all bridges. The results of a scour evaluation may highlight the need for design adjustments such as increasing the opening size, deepening the foundations, adding pier or abutment protection, or incorporating other mitigation measures. Scour countermeasure design is to be performed or directed and approved by the Geotechnical Section of the Bridge Division (BRG).

In addition to bridge hydraulic design and scour evaluations, a number of other H&H tasks are required as a project design is being finalized. These tasks include, but are not limited to:

- Refining the hydraulic design of culverts to finalize sizes, invert profile, end treatment and outlet protection;
- Preparing final storm drain details including design of appropriate sized inlets at the proper spacing and lateral sizing;
- Preparing pump station details for projects involving pump facility construction: and
- Preparing or contributing to the development of Stormwater Pollution Prevention Plans (SW3Ps) after the roadway drainage design is completed

Finalized hydraulic calculation sheets and hydraulic reports should be reviewed by the district and then submitted to DES-HYD for review and approval before PS&E submittal.

See the TxDOT Project Development Process Manual, for further discussion of the PS&E development phase.

Section 5 — Documentation and Deliverables

This section provides a general summary of the required documentation for hydraulic analysis and design. The specific documentation requirements for particular types of drainage structures are explained in the chapters dealing with those structures.

Key Elements of Hydraulic Documentation

The type and nature of documentation and deliverables required varies depending upon the project or effort being undertaken. Whatever the context may be and whatever format of documentation may be used, certain key elements should typically be documented:

- Parameter and criteria considerations -- Documentation of parameter and criteria considerations includes data source identification, evaluation of data, assessments of the reliability of data, what decisions were made and why, qualifying statements such as limitations and disclaimers, and design values comprising the set of parameters and criteria that govern the design. Design parameters define the limits of the facility design. For example, in sizing a structure, design parameters include economically available shapes, environmentally suitable materials, and physical geometric limitations. Examples of criteria include allowable headwater (for a culvert), allowable through-bridge velocity (for a bridge), and maximum allowable water discharge rate from a pump station. Both design parameters and criteria are established from the unique characteristics of the design site and situation. The parameter and criteria considerations should be fully documented for the design of TxDOT drainage facilities.
- Federal and state regulatory criteria (see Chapter 2).
- TxDOT procedures and practices (see Chapter 2).
- Past performance of existing facilities at the subject location. Such experience may include operation during flood events, erosion activity, structural response to flood events, failures, maintenance required (and for what reason), and description and cost of maintenance. District offices should develop and maintain systematic documentation files of facility experiences either at the district or at the local level.
- Judgments, assumptions and decisions incorporated in the decision process or design.
- Plan, profile and detail drawings explaining the design.
- Special Provisions, Special Specifications, or General Notes governing material and construction requirements for any element of the drainage design not addressed by standard specifications.

Special Documentation Requirements for Projects crossing NFIP designated SFHA

A detailed report is required for any bridge replacement or rehabilitation project, or any roadway reconstruction project impacting NFIP floodplains mapped as Zone A (approximate) or Zone AE (detailed study with base flood elevations determined). The TxDOT document "Recommended Format for Drainage Reports," describes the expected content of a report in this context. The report typically consists of an introduction, a hydrology description, description of hydraulic analysis and a summary of conclusions and recommendations.

The introduction describes the purpose of the project, the specific impacts on the stream crossing, and the purpose of the study. The hydrology section describes the watershed, including climate, soils, and other pertinent data, and identifies the methodology used to compute flow. The hydraulics section includes analysis of the existing structure or conditions and design alternatives as well as an overview of the hydraulic modeling process. The hydraulics section also discusses design alternatives and provides a preferred alternative.

Permanent Retention of Documentation

Hydraulic reports and H&H calculations should be retained in the District/Area Office project file for permanent reference. The need for such records may not arise until years after the project has been completed. Retaining these records will provide many benefits, including:

- Ease of reference for future alteration or rehabilitation of the subject drainage structure
- Justification of design decisions in case of future challenges or litigation
- Valuable reference information for the design of other structures crossing the same stream or in the same watershed
- Proof of intended compliance with regulations such as NFIP rules

Documentation Reference Tables

The following tables indicate the required documentation of various facility types for preliminary review, PS&E review, and field change requests. The tables also indicate whether the information should reside in construction plans. The construction plans constitute part of the permanent file, but not all project information resides in the construction plans.

The following table shows the data documentation requirements:

Table 3-1: Data Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Data	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
Field survey data	X	X	X		X	
Vertical Datum	X	X	X	X	X	X
Historical data	X	X	X	X	X	X
FEMA FIS summary data and maps (where applicable)	X	X	X		X	X
Soil maps	X	X	X		X	
Land use maps (when applicable)	X	X	X	X	X	
Stream gauge data (where applicable)	X	X	X		X	X

The following table shows the hydrology documentation requirements:

Table 3-2: Hydrology Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Hydrology	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
Drainage area map(s) showing boundaries, outfalls, flow paths, etc.	X	X	X	X	X	X
Relevant watershed parameters (e.g. areas, runoff coefficients, slopes, etc.)	X	X	X	X	X	X
Assumptions and limitations	X	X	X		X	X
Hydrologic method(s) used	X	X	X	X	X	X
Hydrologic calculations	X	X	X	X	X	X
Peak discharges for design and check floods	X	X	X	X	X	X
Runoff hydrographs for design and check floods (where applicable)	X	X	X	X	X	X

The following table shows the channel documentation requirements:

Table 3-3: Channel Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Channels	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
See Hydrology for runoff determination	X	X	X	X	X	X
Channel cross sections and thalweg profile	X	X	X	X	X	X
Plan showing location of sections	X	X	X		X	X
Cross section subdivisions and "n"-values	X	X	X	X	X	X
Assumptions and limitations	X	X	X		X	X
Hydraulic method or program used	X	X	X	X	X	X
Water surface elevations and average velocities for design and check floods	X	X	X	X	X	X
Analysis of existing channel for comparison (if improvements proposed)	X	X	X	X	X	X

The following table shows the culvert documentation requirements:

Table 3-4: Culvert Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Culverts	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
See Hydrology for discharge data	X	X	X	X	X	X
See Channels for tailwater data	X	X	X	X	X	X
Design criteria (Allowable headwater, outlet velocities, FEMA etc.)	X	X	X	X	X	X
Culvert hydraulic computations	X	X	X	X	X	X
Unconstricted and through-culvert velocities for design and check floods	X	X	X	X	X	X
Calculated headwater for design and check floods	X	X	X	X	X	X
Estimated distance upstream of backwater effect	X	X	X	X	X	X
Magnitude and frequency of overtopping flood	X	X	X	X	X	X

The following table shows the bridge documentation requirements:

Table 3-5: Bridge Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Bridges	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
See Hydrology for discharge data	X	X	X	X	X	X
See Channels for highwater data	X	X	X	X	X	X
Design criteria/parameters/ assumptions (velocities, back-water, FEMA, etc.)	X	X	X	X	X	X
Plan showing location of HEC-RAS cross sections	X	X	X	X	X	X
Bridge hydraulic computations (cross-section output)	X	X	X			X
Unconstricted and through-bridge velocities for design and check floods	X	X	X	X	X	X
Calculated maximum backwater for design and check floods	X	X	X	X	X	X
Estimated distance upstream of backwater effect	X	X	X	X	X	X
Magnitude and frequency of overtopping flood	X	X	X	X	X	X
Scour calculations*	X					*
Estimated scour envelope*	X	X	X	X	X	*
* Can be combined with the scour computations report required by Bridge Division, Geotechnical Section.						

The following table shows the pump station documentation requirements:

Table 3-6: Pump Station Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
	Preliminary Review	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
Pump Stations						
See Hydrology for discharge data	X	X	X	X	X	X
See Channels for tailwater data	X	X	X	X	X	X
See Storm Drains for inlet and outlet conduit data	X	X	X	X	X	X
Stage/storage curve	X	X	X	X		X
Pump capacity and performance computations	X	X	X		X	X
Pump hydraulic performance curves	X	X	X	X		X
Design peak and attenuated peak discharges	X	X	X	X	X	X
Maximum allowable headwater elevation	X	X	X	X	X	X
Switch-on and cut-off elevations	X	X	X	X		X
Sump dimensions	X	X	X	X		
Head loss calculations and total dynamic head	X	X	X		X	X
Pump sizes	X	X	X	X		X
Pump station details		X	X	X		

The following table shows the storm drain documentation requirements:

Table 3-7: Storm Drain Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Storm Drains	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
See Hydrology for discharge data	X	X	X	X	X	
See Channels for tailwater data	X	X	X	X	X	
Storm drain schematic/layout showing trunklines, laterals, inlets, outfall etc.	X	X	X	X		
Storm drain hydraulic computations including all allowables	X	X	X	X	X	
Storm drain plan/profile sheets w/ hydraulic grade line		X	X	X	X	
Outfall considerations and information					X	
Flow direction arrows	X	X	X	X		
Evaluation of existing facility (if present)	X	X			X	

The following table shows the facility documentation requirements:

Table 3-8: Other Facility Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information		
Other Facilities	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
Drainage area maps	X	X	X	X	X	X
Design criteria/parameters/ assumptions	X	X	X		X	X
Hydrologic computations	X	X	X	X	X	X
Hydraulic computations	X	X	X	X	X	X
Plan/profile and details	X	X	X	X		X
Design and check flood before and after conditions (highwater, velocities, etc.)	X	X	X	X	X	X

The following table shows the SW3P Layout requirements for projects requiring authorization under the Construction General Permit (TXR150000):

Table 3-9: SW3P Layout Requirements

Documentation Item (by facility type)	Stage			Location of Information		
SW3P Layouts	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
Drainage patterns	X	X	X	X	X	X
Approximate slopes anticipated after major grading activities	X	X	X	X	X	X
Areas that will and will not be disturbed	X	X	X	X	X	X
Locations where storm water will discharge from the project (i.e. discharge points)	X	X	X	X	X	X
Acres of disturbed area that will drain to each discharge point[1]	X	X	X	X	X	X
Locations of all major BMPs (e.g. silt fence, rock berm, sediment traps, etc.) [2]	X	X	X	X	X	X
Areas that will receive temporary or permanent stabilization (e.g. temporary seeding, soil retention blankets, slope texturing, etc.) [3]	X	X	X			X
Locations of surface waters and wetlands on or adjacent to the site (if known)	X	X	X	X	X	X
Names of surface waters that will receive discharge from the project[1]	X	X	X	X	X	X

[1]Alternatively, this could be included in the SW3P Summary Sheet

[2]Structural controls are required at all down slope boundaries, and side slope boundaries as appropriate. Velocity dissipation devices at discharge locations are required if necessary to provide a non-erosive flow velocity from the structure to a watercourse. If it will be necessary to pump or channel standing water from the site, controls to remove sediment from this water are required. BMPs may also be used to divert storm water around disturbed areas.

[3]Temporary stabilization is required when work in a disturbed area will cease for more than 21 days.

The following table shows the SW3P Summary Sheet requirements for projects requiring authorization under the Construction General Permit (TXR150000):

Table 3-10: SW3P Summary Sheet Requirements

Documentation Item (by facility type)	Stage			Location of Information		
SW3P Summary Sheet	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
If there are more than 10 disturbed acres that drain to a single discharge point, and if it is not possible to install a sediment basin that provides storage for the runoff from a 50%AEP, 24-hour storm per acre drained, a reason why such a basin is not feasible[1]	X	X	X	X	X	X
A description of the nature of the construction activity	X	X	X	X	X	X
Description of controls to reduce off site tracking of sediment	X	X	X	X	X	X
Description of construction and waste materials expected to be stored on-site and a description of controls to minimize pollutants from these materials.	X	X	X	X	X	X
A list of other potential pollutants and their sources, and description of controls to minimize pollutants from these sources[2]	X	X	X	X	X	X
Total acreage of the project area	X	X	X	X	X	X
Total acreage of the project area that will be disturbed	X	X	X			X
Description of the soil or quality of the existing discharge from the site[3]	X	X	X	X	X	X
A description of the intended schedule or sequence of activities that will disturb soils for major portions of the site. [4]	X	X	X	X	X	X

[1]Acceptable reasons include soils type, slope, available area, public safety, precipitation patterns, site geometry, site vegetation, infiltration capacity, geotechnical factors, depth to groundwater, and other similar considerations.

[2]Consider if the following may be a potential pollutant: sediment, oil and grease, coolant, pathogens, concrete truck washout, nutrients, etc. Effective controls may include requiring the contractor to maintain equipment free of leaks, develop a spill response plan, cover stored material/chemicals, prohibit concrete wash out in the rain or within a certain distance of waterways, prohibit the storage of materials/chemicals on a paved surface, prohibit application of fertilizer when rain is forecast or in excess of required amounts, etc.

[3]Consider whether or not preexisting conditions indicate the discharges from the area already contain excessive sediment, or if the site will be unusually vulnerable to erosion during construction.

[4]The sequences of disturbance and BMP implementation should correspond. For example, “Silt fence and rock berm will be installed prior to initial clearing and grading…after final grading, permanent seeding will be employed.”

Table 3-10: SW3P Summary Sheet Requirements

Documentation Item (by facility type)	**Stage**			**Location of Information**		
SW3P Summary Sheet	**Preliminary Review**	**PS&E Review**	**Field Changes**	**Construction Plans**	**Permanent File**	**Report**
A description of the intended sequence of erosion and sediment control BMP implementation[4]	X	X	X	X	X	X
A note that the contractor will be responsible for compliance with all applicable environmental laws, rules and regulations for any work not described in the plans.	X	X	X	X	X	X
A note that the contractor is responsible for installing and maintaining BMPs as described in the plans and as directed by TxDOT personnel.	X					

[1]Acceptable reasons include soils type, slope, available area, public safety, precipitation patterns, site geometry, site vegetation, infiltration capacity, geotechnical factors, depth to groundwater, and other similar considerations.

[2]Consider if the following may be a potential pollutant: sediment, oil and grease, coolant, pathogens, concrete truck washout, nutrients, etc. Effective controls may include requiring the contractor to maintain equipment free of leaks, develop a spill response plan, cover stored material/chemicals, prohibit concrete wash out in the rain or within a certain distance of waterways, prohibit the storage of materials/chemicals on a paved surface, prohibit application of fertilizer when rain is forecast or in excess of required amounts, etc.

[3]Consider whether or not preexisting conditions indicate the discharges from the area already contain excessive sediment, or if the site will be unusually vulnerable to erosion during construction.

[4]The sequences of disturbance and BMP implementation should correspond. For example, “Silt fence and rock berm will be installed prior to initial clearing and grading…after final grading, permanent seeding will be employed.”

Chapter 4 — Hydrology

Contents:

Section 1 — Hydrology's Role in Hydraulic Design

In the context of hydraulic design, hydrologic analysis provides estimates of flood magnitudes as a result of precipitation. These estimates consider processes in a watershed that transform precipitation to runoff and that transport water through the system to a project's location.

The design of drainage facilities requires the designer to:

- Select the level of protection desired, specified in terms of probability of capacity exceedance.
- Find the corresponding flow rate and/or volume, computing in many cases the corresponding water surface elevation.
- Use that as a basis for design.

In the design of facilities such as storm drain systems, culverts, and bridges, floods are usually considered in terms of peak runoff or discharge in cubic feet per second or cubic meters per second. For systems that are designed to control the volume of runoff, such as detention storage facilities, or where flood routing through culverts is used, the discharge per time will be of interest. Thus, depending on the needs of a particular project, the hydrology study may provide:

- A flow rate for which the probability of exceedance is specified.
- A volume of water expected with a specified storm duration, for which the probability of exceedance is specified.
- A hydrograph—flow rate as a function of time—for a specified probability of exceedance. This provides information about peak, volume, and timing of runoff level of protection desired.

These results may be obtained through statistical analysis of historical observations or through empirical or conceptual models of the relevant watershed and channel processes.

Section 2 — Probability of Exceedance

The probability of exceedance describes the likelihood of a specified flow rate (or volume of water with specified duration) being exceeded in a given year. The probability of capacity exceedance describes the likelihood of the design flow rate (or volume of water with specified duration) of a hydraulic structure being exceeded in a given year.

Annual Exceedance Probability (AEP)

In this manual, the preferred terminology for describing the probability of exceedance is annual exceedance probability (AEP).

There are several ways to express AEP. The TxDOT preferred unit for expressing AEP is percent. An event having a 1 in 100 chance of occurring in any single year will be described in this manual as the 1% AEP event. Annual recurrence interval (ARI), or return period, is also used by designers to express probability of exceedance. A 5-year return interval is the average number of years between years containing one or more events exceeding the specified AEP. Lastly, AEP can also be expressed as probability (a number between 0 and 1), such as p = 0.01. Examples of equivalent expressions for exceedance probability for a range of AEPs are provided in Table 4-1.

Table 4-1: Three Ways to Describe Probability of Exceedance

AEP (as percent)	AEP (as probability)	Annual Recurrence Interval (ARI)
50%	0.50	2-year
20%	0.20	5-year
10%	0.10	10-year
4%	0.04	25-year
2%	0.02	50-year
1%	0.01	100-year

While AEP, expressed as a percent, is the preferred method for expressing probability of exceedance, there are instances in this manual where other terms, such as those in Table 4-1, are used. These instances include equation subscripts based on return period (e.g. Q_{10}), plot axes generated by statistical software, and text and tables where readability was improved as a result.

Design AEP

The designer will determine the required level of protection to be provided by a hydraulic structure. The level of protection is expressed as the design AEP. The designer will apply principles of hydrology to determine flows and volumes corresponding to the design AEP. The purpose of most structures will be to provide protection against, or prevent, high stages; resulting from the design AEP event.

If stage is primarily dependent on flow rate, as is the case in a free-flowing channel, then the designer will estimate the peak flow value corresponding to the design AEP. If stage is primarily dependent on accumulated volume, as is the case with a storage facility, then the designer will seek to estimate the flow volume and duration corresponding to the design AEP.

Flows with computed AEP values can be plotted as a flood frequency curve as illustrated in Figure 4-1. In this example, the discharge is plotted on a logarithmic scale and AEP is plotted on a probability scale. As would be expected the curve indicates that flow increases as AEP decreases.

The AEP scale ranges from 100% to 0% (shown in Figure 4-1 as 1 to 0).

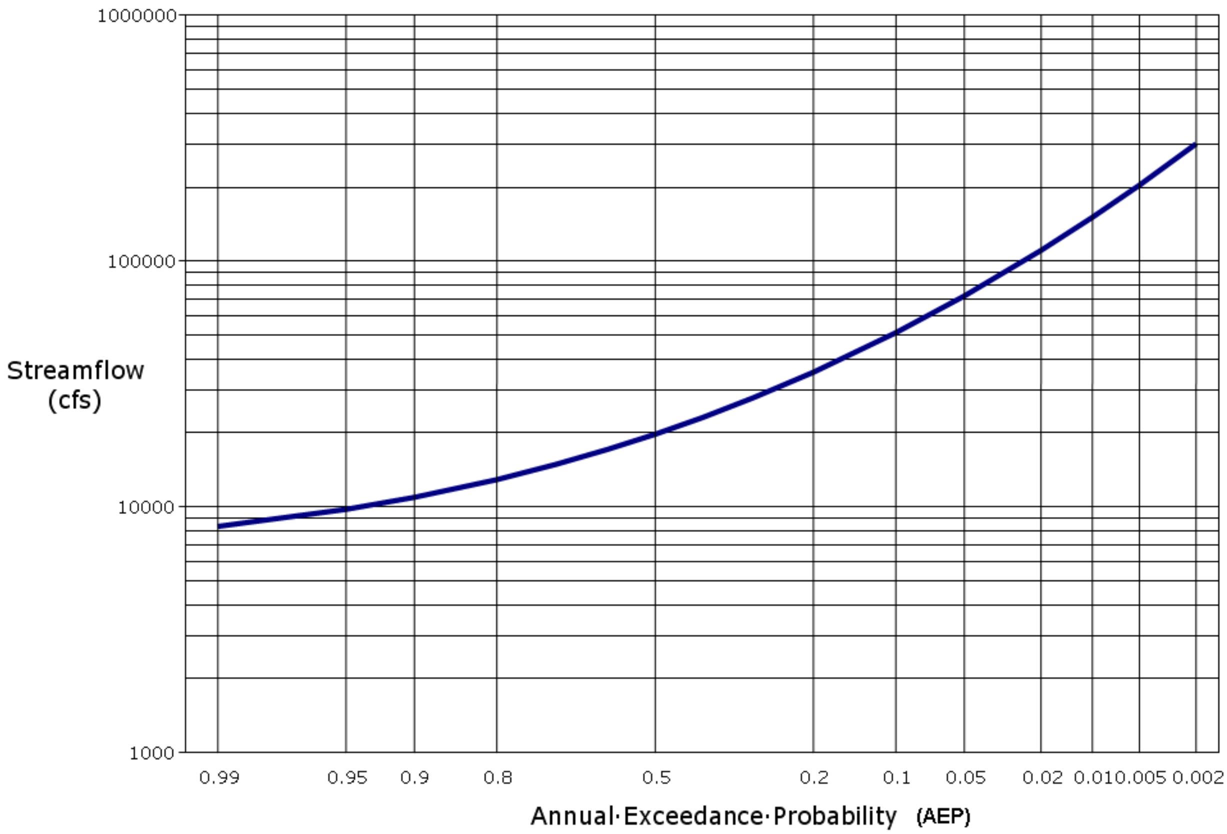

Figure 4-1. Typical flood frequency curve

Accuracy

The peak discharges determined by analytical methods are approximations. The drainage system will rarely operate at the design discharge. Flow will always be more or less in actual practice, merely passing through the design flow as it rises and falls. Thus, the design engineer should not overemphasize the accuracy of the computed discharges. The design engineer should consider a reasonable number of significant digits for each result based on the level of detail of each analysis. For example, flows computed for small areas like inlets should typically be reported to whole numbers for cfs values or at most tenths (e.g. Q_{10}=14 cfs or 8.3 cfs rather than 14.39 cfs and 8.34 cfs). Whereas, flows for larger areas like streams may be reported by rounding off values produced in models (e.g. Q_{50}=3,200 cfs rather than 3,217 cfs). Care should be taken to not allow rounding to create exaggerated results. For example, 1049 cfs for existing conditions and 1052 cfs for proposed conditions, should not translate to 1000 cfs and 1100 cfs respectively, which would then imply more difference than expected. Nor should both these values be rounded to 1050 cfs to imply parity in the results. In these cases, reporting more significant digits to show minimal change may be preferred. FEMA or other agencies may require reporting more significant digits than the accuracy of the computational method. When reporting to those agencies, to avoid minor disagreements, it is acceptable to follow their reporting preferences. The design engineer should emphasize the design of a practical and hydraulically balanced system based on sound logic and engineering.

Section 3 — Hydrology Policies and Standards

TxDOT uses the drainage practices and design standards described in this manual for designing drainage facilities and flood control works associated with transportation projects. They are also used for evaluating the design, construction, and performance of projects in TxDOT's right-of-way.

General Guidelines

The designer should keep the following in mind when using this manual:

- The standards and methods in this manual are aimed at sound planning and design; they are guidelines rather than steadfast rules.
- The design standards in this chapter represent minimums. Alternatives that meet a higher standard than presented herein must be approved by the TxDOT District Hydraulics Engineer (DHE).
- Exceptions to these design criteria may be allowed by TxDOT when they are in the best interest of the public and the alternative will be equivalent to the normally accepted method.
- Errors in peak flow estimates, whether due to computational errors or errors in judgment, may result in a drainage structure that is either undersized, which could cause drainage problems, or oversized, which costs more than necessary. On the other hand, any hydrologic analysis is only an approximation. Although some hydrologic analysis is necessary for all highway drainage facilities, the extent of such studies should be commensurate with the hazards associated with the facilities and with other concerns, including engineering, economic, social, and environmental factors.
- Design details are the responsibility of the design engineer and will be determined by good engineering practice.

Third Party Studies

If third party studies (previous studies by others) are available and appropriate for the project area, TxDOT may, but is not required to, use these studies. In some circumstances, TxDOT may be required to use third party studies, such as for compliance with the Federal Emergency Management Agency's (FEMA) National Flood Insurance Program (NFIP.) The use of third party studies should be cleared with the DHE.

Hydraulic Design for Existing Land Use Conditions

All drainage facilities shall be designed for existing land use conditions. Drainage design must include capacity to convey runoff from all existing adjacent properties. The district office may consider runoff from future land use conditions at its discretion after consulting with and receiving approval from the DHE.

Effect on Existing Facilities

TxDOT drainage projects must be designed so that they do not have a negative impact on existing facilities either upstream, neighboring or downstream of the project. A channel's hydraulic grade line must be determined to evaluate the risks and potential impacts of flooding. The designer must define the upstream and downstream water surfaces until the effects of the new drainage facility match the pre-project hydraulic grade line. The proposed project must not induce flooding during the event for which it was designed.

Section 4 — Hydrology Study Requirements

The design engineer must coordinate with the district office at the beginning of a hydrology study to establish study requirements. Typically, TxDOT requires the designer to:

- Select an appropriate frequency for the event that will be the basis for design or evaluation, considering the risk of capacity exceedance. The event selected may range from one with a 50% AEP (2-year event) to one with a 1% AEP (100-year event). (Design Flood and Check Flood Standards provides guidance on selection of the risk-based flood event.)
- Choose an appropriate hydrologic analysis method, following guidance given herein, and use that method to compute the flow for the selected frequency.
- Explain and justify all assumptions. This includes explaining and justifying choice of the analysis method and choice of parameters and other inputs used with the methods.
- Verify and support results. This may be accomplished, in part, by demonstrating that the method used for design flow computation also can reproduce observed streamflow. Or it may be accomplished by comparing results of the selected method with those of valid alternative methods or other studies for nearby locations to establish confidence in results. (Driveway culverts and storm drains may be exempted from this requirement.)
- Provide, in reports and on plan sheets, information to enable a reviewer to understand and to reproduce the results. When using computer software for the computations, this will require the designer to identify the program used and version, to display all relevant input values, and to specify options and methods used in the software.
- Calculate flows for flood frequencies larger than and smaller than the selected design level, repeating the tasks described above. This will provide the designer with information for testing the resiliency and robustness of designs.

Section 5 — Hydrology Study Data Requirements

Strictly speaking, the term data refers to measurements or observations, and the term information refers to results of analysis or synthesis of data. Both data and information are needed for hydrologic studies, and the terms are used interchangeably here. To determine what data are needed, the designer must determine which hydrologic analysis method(s) will be used.

The major task of a hydrology study is to compute design flow. There are conceptual methods and empirical methods for computation of design flow.

Hydrology Analysis Methods

Conceptual methods in this category simulate, with a mathematical model, channel flow and watershed runoff processes. Movement and storage of water through the watershed are simulated at varying time and space scales, with varying degrees of complexity, omitting, including, or combining elements, depending on the model used and the requirements of the study.

Conceptual methods that TxDOT designers may use include the Rational method (loosely classified as a conceptual method here) and the hydrograph method.

Like conceptual methods, empirical methods also use a mathematical relation that predicts the design flow, given properties of the watershed, channels, rainfall, or streamflow. However, the relationship does not represent explicitly the physical processes. Instead, the relationships are derived with statistical analyses. (Some analysts even refer to empirical methods as black box methods because the presentation of the process is not visible and obvious.)

Empirical methods that TxDOT designers may use include flood frequency analysis of streamflow observations and regression equations. With flood frequency analysis, the empirical relationship predicts the design flow from statistical properties of the historical streamflow in the watershed. With regression equations, the design flow is predicted with an equation that has been developed by correlating flows observed with watershed, channel, and rainfall properties.

Data Requirements Vary with Method Used

Data and information required for hydrologic analysis vary from method to method. The conceptual methods require somewhat detailed information about the watershed and channel properties, whereas the empirical methods require streamflow data to establish the relationships and only limited data on watershed and channel properties to use the derived relationship.

Specific requirements for the different methods are called out in later sections of this Chapter, but broad categories of data required include the following:

- Geographic and geometric properties of the watershed.

- Land use, natural storage, vegetative cover, and soil property information.
- Description of the drainage features of the watershed.
- Rainfall observations and statistics of the precipitation.
- Streamflow observations and statistics of the streamflow.

Geographic and Geometric Properties of the Watershed

All hydrologic analyses for TxDOT studies require collection of data about the geographic and geometric properties of the watershed. These data include, but are not limited to, the following:

- Geographic location of the point at which design flow must be computed.
- Location of the boundaries of the watershed from which runoff contributes to flow at the point of interest. This information will, for example, govern selection of design rainfall intensities that will be used with the rational method if that is selected for design flow computation.
- Properties of the watershed within those boundaries. These properties include area, slope, shape, and topographic information. This information is needed, for example, to develop a model with which to simulate overland flow, as shown in Figure 4-10 whereby water ponded on the surface moves across the watershed into channels.

Land Use, Natural Storage, Vegetative Cover, and Soil Property Information

Data that describe the watershed properties are needed for the conceptual models, and to a limited extent, by certain empirical models.

A conceptual model of watershed runoff, with components as illustrated in Figure 4-10, represents processes of infiltration and overland flow. To do so, the model must be configured and calibrated with knowledge of the properties of the watershed that will affect infiltration and overland flow. Those include:

- Land use in the watershed. Especially important in this is gathering information about the distribution of impervious and pervious cover in the watershed. Rain that falls on impervious surfaces, such as parking lots and rooftops, will run off as overland flow. Rain that falls on a pervious surface may infiltrate, entering the soil layers, and not running off immediately or at all. The rate of this infiltration is related with land use, as well.
- Natural storage in the watershed. Water that ponds in natural depressions, lakes, and similar features in a watershed will not run off or may runoff with some delay and with reduced rates. The location of, capacities of, and behavior of storage must be identified if this is to be represented in computations of design flows.
- Vegetative cover and soil property information. Rates of infiltration depend on properties of soils in the watershed and upon the presence of vegetation. For example, water ponded on sandy soils may infiltrate at four or five times the rate of water ponded on clay soils. And crops

planted on clay soils will increase the rate of infiltration there. Thus, the designer must gather information on the cover and soils. That information should define the spatial variations across the watershed.

These data are needed with conceptual models that do not seek to represent in great detail the physical processes. For example, with the rational method, a runoff coefficient relates runoff rate and rainfall rate. That coefficient is related to land use within the watershed. And knowledge of land use, particularly knowledge of presence or absence of impervious area, is critical for assessing the applicability of regression equations.

Description of the Drainage Features of the Watershed

Channels, ponds, reservoirs, culverts, and other natural or constructed drainage features in a watershed affect the runoff from the watershed. Thus data that describe those must be collected.

For a conceptual model, data about the features are needed to make a decision about which model to use and configure the model appropriately. For example, with a hydrograph method, data describing channels are needed to select, calibrate, and use a routing method that accounts for the impact of a channel on the design flood peak.

For an empirical model, data on drainage features is needed first to enable wise decisions about which model(s) to use, and second, to estimate model parameters. For example, flood frequency (stream gauge) analysis procedures require that the streamflow records be without significant regulation. To determine if this is so, the designer must have information on regulation in the watershed, including descriptions of ponds, reservoirs, detention structures, and diversions in the watershed.

Rainfall Observations and Statistics of the Precipitation

Conceptual models simulate conversion of rainfall to runoff by simulating some or all of the processes illustrated in Figure 4-10. Thus, to use a conceptual model, rainfall data are required. These data include both observations of rainfall at gauges in the watershed and statistics on rainfall from which design storms are developed.

With observations of rainfall at gauges, models can be calibrated and tested to ensure that they truly represent the behavior of the watershed.

With statistics of rainfall depths, a design storm can be developed, and the required design flow can be computed following the design storm assumption. This assumption is that “if median or average values of all other parameters are used, the frequency of the derived flood should be approximately equal to the frequency of the design rainfall” (Pilgrim and Cordery 1975).

Streamflow Observations and Statistics of the Streamflow

Streamflow observations at or near to the location of interest are the designer's best index of how a watershed will behave under conditions existing in the watershed at the time of observation of the flow. These data serve the following purposes:

- Calibration of statistical model. If available, long records of annual maximum streamflow permit flood frequency analysis and design flow determination.
- Calibration and verification of conceptual model. Shorter records of runoff from individual floods permit calibration and verification of conceptual models of the rainfall to runoff transformation, if corresponding records of rainfall are available. In this process, model parameters are estimated, runoff from observed rainfall is computed, and the computed flows are compared to the observed. Parameters are adjusted if the fit is not acceptable.
- Assessment of reasonableness of results. Records of annual maximum flows at a site for limited periods permit assessment of reasonableness of predicted design flows. For example, if a record of annual maximum flows for 12 years at a site includes six peaks that exceed the predicted 10% chance design flow, a designer can apply the binomial statistical distribution to determine that the probability is only 0.0005 that this could happen. This is so unlikely that it raises doubt about the estimated 10% chance design flow.

Section 6 — Design Flood and Check Flood Standards

TxDOT's approach to selecting the design standard for a drainage facility is to use a reference table that specifies a range of design AEPs for different types of facilities. Table 4-2 provides the design frequencies for TxDOT projects. For most types of facilities a range of design frequencies is presented. For those types of facilities with a range of possible design frequencies, usually one design frequency in the range is recommended (indicated by an X with square brackets in Table 4-2). Structures and roadways should be serviceable (not inundated) up to the design standard.

Table 4-2: Recommended Design Standards for Various Drainage Facilities

Functional classification and structure type	Design AEP (Design ARI)				
	50% (2-yr)	20% (5-yr)	10% (10-yr)	4% (25-yr)	2% (50-yr)
Freeways (main lanes):					
Culverts					X
Bridges+					X
Principal arterials:					
Culverts			X	[X]	X
Small bridges+			X	[X]	X
Major river crossings+					[X]
Minor arterials and collectors (including frontage roads):					
Culverts		X	[X]	X	
Small bridges+			X	[X]	X
Major river crossings+				X	[X]
Local roads and streets:					
Culverts	X	X	X		
Small bridges+	X	X	X		
Off-system projects:					
Culverts	FHWA policy is "same or slightly better" than existing.				
Small bridges+					
Storm drain systems on interstates and controlled access highways (main lanes):					
Inlets, drain pipe, and roadside ditches			X		
Inlets for depressed roadways*					X

Table 4-2: Recommended Design Standards for Various Drainage Facilities

Functional classification and structure type	Design AEP (Design ARI) 50% (2-yr)	20% (5-yr)	10% (10-yr)	4% (25-yr)	2% (50-yr)
Storm drain systems on other highways and frontage roads:					
Inlets, drain pipe, and roadside ditches	X	[X]	X		
Inlets for depressed roadways*				[X]	X

Table 4-2 notes:
* A depressed roadway provides nowhere for water to drain even when the curb height is exceeded.
[ ] Brackets indicate recommended AEP. Federal directives require interstate highways, bridges, and culverts be designed for the 2% AEP flood event. Storm drains on facilities such as underpasses, depressed roadways, etc., where no overflow relief is available should be designed for the 2% AEP event.
+ It may be necessary to calculate the 4% (25-yr), 2% (50-yr), 0.5% (200-yr), or 0.2% (500-yr) AEP for scour computations. See the TxDOT *Geotechnical Manual*.

All facilities must be evaluated to the 1% AEP flood event.

Selecting a design flood is a matter of judgment; it requires balancing the flood risk with budgetary constraints. When considering the standard for a drainage facility, the designer should follow these guidelines:

- Decide on the design standard by considering the importance of the highway, the level of service, potential hazard to adjacent property, future development, and budgetary constraints.
- Develop alternative solutions that satisfy design considerations to varying degrees.
- After evaluating each alternative, select the design that best satisfies the requirements of the structure.
- Consider additional factors such as the design standards of other structures along the same highway corridor to ensure that the new structure is compatible with the rest of the roadway. Also assess the probability of any part of a link of roadway being cut off due to flooding.

The designer should design a facility that will operate:

- Efficiently for floods smaller than the design flood.
- Adequately for the design flood.
- Acceptably for greater floods.

In addition, for all drainage facilities, including storm drain systems, the designer must evaluate the performance for the check flood (1% AEP event). The purpose of the check flood standard is to ensure the safety of the drainage structure and downstream development by identifying significant risk to life or property in the event of capacity exceedance.

The intent of the check flood is not to force the 1% AEP through the storm drain, but to examine where the overflow would travel when this major storm does occur. For example, the water may travel down the gutter to the same creek as the outfall, travel down a driveway and directly into a home, inundate the mainlanes, erode a new drainage path to the outfall, or other problems.

The examination of the check flood should also include assessment of the tailwater. There may be locations on the project that are lower than the 100 year water surface elevation (or tailwater) of the creek. This situation may increase the hydraulic grade line through the storm drain system, or may even cause negative flow through the system. This may cause blowouts which may in turn cause any of the same problems as above.

For scour design and check flood frequencies, please refer to the TxDOT Geotech Manual.

Section 7 — Selection of the Appropriate Method for Calculating Runoff

The designer is expected to select an appropriate hydrologic analysis method for each project, seeking assistance from the DHE or DES-HYD and other resources, as needed. TxDOT has no standard method, realizing that methods used must satisfy the requirements of individual studies.

To select the appropriate method, the TxDOT designer should consider, at a minimum, the following:

- Information required for design or evaluation and where that information is needed. For example, if the TxDOT project requires designing a culvert, the rational method, which computes peak only, may be adequate. However, if the TxDOT project is affected by or will affect behavior of a detention or retention pond, a runoff hydrograph will be required for the evaluation.
- Data available to develop the required hydrologic information. For example, the designer must determine if flow records are available from a stream gauge at or near the location of interest. If not, frequency analysis to find the design flow is not possible, nor is proper calibration of a conceptual model that will compute a hydrograph.
- Conditions in the watershed that may limit applicability of alternative models. For example, regression equations for Texas were estimated for watersheds with less than 10 percent impervious cover. If the watershed upstream of the point of interest has more impervious cover, the equations are not applicable. Similarly, if ponds, lakes, and depressions in the watershed will affect runoff by storing water, the rational equation will not be appropriate, as it does not simulate behavior of these features.

Methods acceptable for estimating peak discharges and runoff hydrographs for TxDOT design and evaluation include, but are not limited to the following:

- **Statistical Analysis of Stream Gauge Data**. This empirical method calibrates a probability model with peak annual discharge observations. The probability model relates design flow magnitude to frequency directly, without explicit consideration of rainfall or watershed properties or processes. The method is particularly useful where records in excess of 20 years of stream gauge data are available at or on the same stream near the drainage facility site.
- **Omega EM Regression Equations**. This empirical method relies on application of equations, previously developed through extensive statistical analysis, to predict the peak discharge for a specified frequency (TxDOT 0-5521-1). The equations relate the peak to watershed properties, including watershed area, mean annual precipitation, and main channel slope. This method is useful if streamflow data are not available at or near the project site, or other methods are judged inappropriate. TxDOT designers may use Omega EM regression equations for validation and verification of results from other methods, or for computation of flows for limited detail evaluation of impacts of TxDOT designs on off-system facilities. Omega EM regression equations are reliable beyond about 10 sq. mi. drainage area. A comparison method should be used for drainage areas below 10 sq. mi., and must be used for drainage areas below about 5 sq.

mi. This method should not be used for drainage areas less than 1 sq. mi.

Discretion may be used on off-system bridges and culverts. As the design of these crossings is typically "hydraulically same or slightly better," the importance of having an exact flowrate is of lesser importance than on-system crossings. At the engineer's discretion, the use of a comparison method may be disregarded.

- **Omega EM Regression Equations**. This empirical method relies on application of equations, previously developed through extensive statistical analysis, to predict the peak discharge for a specified frequency (TxDOT 0-5521-1). The equations relate the peak to watershed properties, including watershed area, mean annual precipitation, and main channel slope. This method is useful if streamflow data are not available at or near the project site, or other methods are judged inappropriate. TxDOT designers may use Omega EM regression equations for validation and verification of results from other methods, or for computation of flows for limited detail evaluation of impacts of TxDOT designs on off-system facilities. Omega EM regression equations are reliable beyond about 10 sq. mi. drainage area. A comparison method should be used for drainage areas below 10 sq. mi., and must be used for drainage areas below about 5 sq. mi. This method should not be used for drainage areas less than 1 sq. mi.

  Discretion may be used on off-system bridges and culverts. As the design of these crossings is typically "hydraulically same or slightly better," the importance of having an exact flowrate is of lesser importance than on-system crossings. At the engineer's discretion, the use of a comparison method may be disregarded.

- **Rational Method**. This simple conceptual method estimates peak runoff rate for a selected frequency. It is appropriate for urban and rural watersheds less than 200 acres (80 hectares) in which natural or man-made storage is minor. It relies on an assumption that the design flow of a specified frequency is caused by rainfall of the same frequency. This method is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes.

- **Hydrograph Method**. This conceptual method (actually, a set of methods and models) relies on a mathematical representation of the critical processes by which rainfall on a watershed is transformed to runoff from the watershed. The method is used with a design rainfall hyetograph, which specifies the time distribution of rainfall over a watershed. The method computes a runoff hydrograph, which shows how runoff varies with time; from that, the peak flow, time of peak, and corresponding volume can be found.

Figure 4-2 is a flowchart to aid the designer in selecting an appropriate hydrologic method from among these. The designer must ensure that the conditions in the watershed conform to the limitations of the selected hydrologic method, as described in detail in the sections that follow.

The TxDOT designer is not limited to using only the methods shown here. If none of the methods is judged appropriate, the designer may use an alternative method, with the approval of the DHE or

DES-HYD. In every case, the rationale for selecting the method must be presented as a component of the design report.

The TxDOT designer should:

- Identify and apply alternative methods, recognizing that these will yield different results.
- Compare the results from several methods and the historical performance of the site.
- Use the discharge that best reflects local project conditions. Averaging of results of several methods is **not** recommended.
- Document the reasons for selection of the methods and the historical performance of the site.

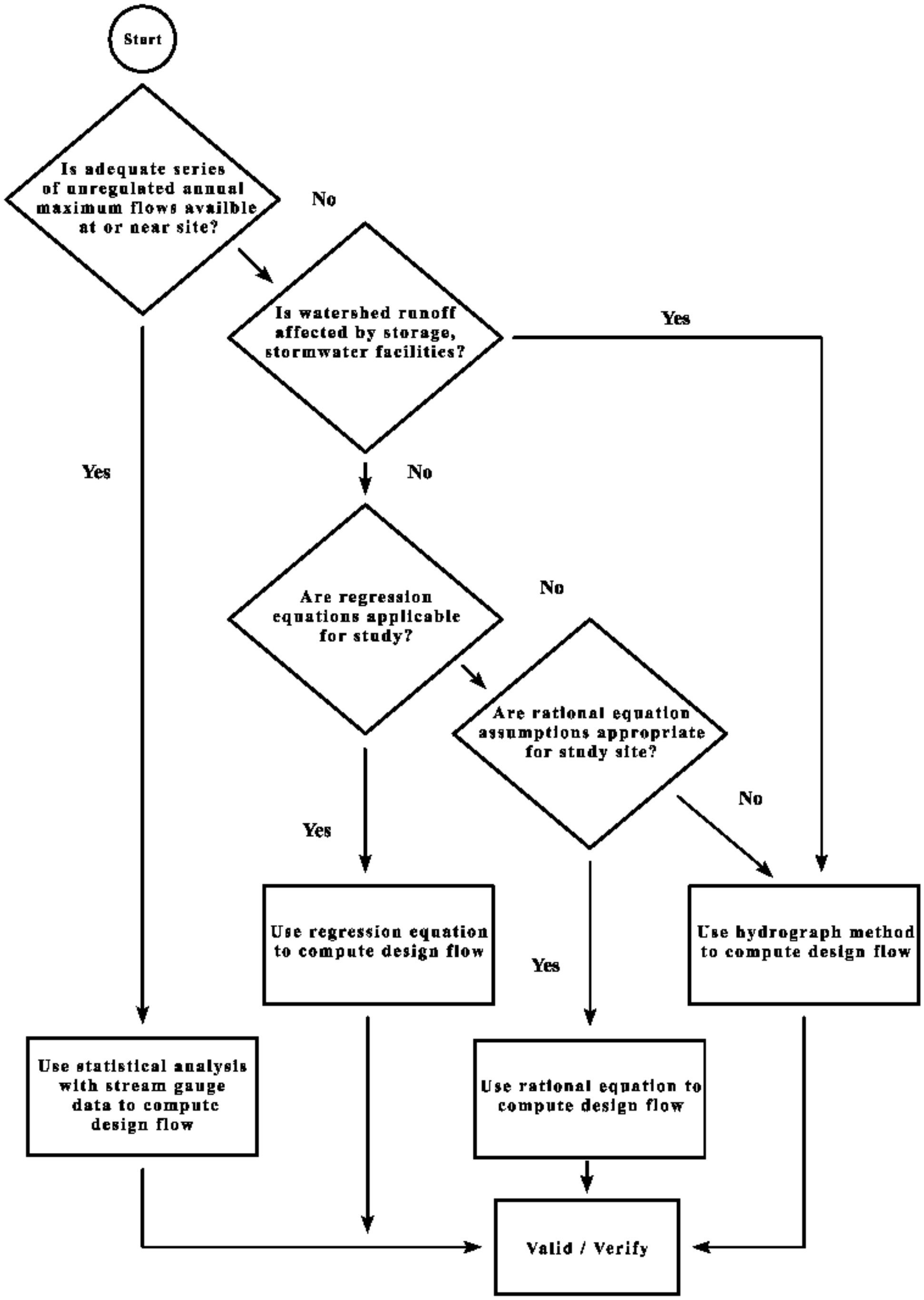

Figure 4-2. Hydrologic method selection chart

Section 8 — Validation of Results from the Chosen Method

Design flows estimated with any method used should be confirmed and validated. This may be achieved by:

1. Comparing the predicted design flow for the selected frequency with observed flows to assess reasonableness. The binomial distribution, which is available as a function in most spreadsheet software, is helpful for this assessment. It computes the probability of y exceedances in a given period of n years of a design event that has an AEP of p as:

$$P = \frac{n!}{y!(n-y)!}p^y(1-p)^{n-y}$$

Equation 4-1.

Where:

P = probability (0 to 1) of y exceedances in n years for design level p

y = number of design exceedances

n = number of years

p = Design AEP (0 to 1)

Note that p in this equation is AEP and ranges from 0 to 1. So for this equation, p is the selected design frequency divided by 100.

Suppose, for example, that in a 20-year long record of observed flows, the computed 1% AEP flow was found to have been exceeded in three years. With the binomial distribution, the probability of this is computed as 0.001 or about one chance in 1000. This is so unlikely that it raises doubt about the estimate of the 1% chance flow, suggesting that the computed design flow is too low. Fewer exceedances would be reasonable.

Similarly, suppose that the 10% AEP design flow was not exceeded at all in a 30-year-long record. The binomial distribution shows that the probability of no exceedances of the 10% AEP (10-year) flow in 30 years is 0.04—again an unlikely scenario. This suggests that the 10% flow predicted is too high; more exceedances would be reasonable.

2. Comparing the design flow computed with the selected method with those computed for the same AEP for watersheds with similar properties in other studies in the region. A “flow per unit area” comparison is useful. Significant differences should be investigated and explained.

3. Comparing the results of other methods, if those are appropriate. For example, in some cases, for different areas, either the rational method or the regression equations will be acceptable for design flow computation. Or if a frequency function is fitted with statistical methods, the design flows can be transposed from the gauged site to the location of interest, using methods described later in this manual.

4. Comparing the design flow computed with the selected method to design flow for the same frequency computed by other agencies with different methods. These may include local public

works agencies, the United States Army Corps of Engineers (USACE), the FEMA, and the Natural Resource Conservation Service (NRCS). Significant differences should be investigated and explained.

The results of these alternative methods can be compared. Again, significant differences should be investigated and explained.

Section 9 — Statistical Analysis of Stream Gauge Data

If the gauging record covers a sufficient period of time, it is possible to develop a flow-frequency relation by statistical analysis of the series of recorded annual maximum flows. The designer can then use the flow-frequency relation in one of two ways:

- If the facility site is near the gauging station on the same stream and watershed, the designer can directly use the discharge obtained from the flow-frequency relation for the design AEP.
- If the facility site is on the same stream, but not proximate to the gauging station, it may be possible to transpose gauge analysis results.

Widely accepted and applied guidelines for statistical analyses of stream gauge data are published in Guidelines for Determining Flood Flow Frequency, Bulletin #17C (USGS 2018). Procedures from Bulletin #17C, with some Texas-specific refinements, as outlined in this manual, are recommended. They include:

- Obtaining a sufficiently large sample of streamflow data for statistical analysis,
- Using the log-Pearson type III distribution fitting procedure,
- Using a weighted skew value,
- Accommodating outliers,
- Transposing gauge analysis results, if necessary and appropriate.

Software most commonly used to perform these analyses in Texas are PeakFQ by USGS and HEC-SSP by USACE.

Data Requirements for Statistical Analysis

The greatest challenge in applying the statistical analysis of stream gauge data is obtaining a sufficiently large sample of streamflow measurements (or estimates) so that the sample is representative of the entire population of flows. Three types of data may be considered (USGS 2018), systematic data, historical data, and paleoflood and botanical information.

Systematic data are flow records generated from a defined set of rules and recorded on a regular basis. For example, the United States Geological Survey (USGS) annual maximum flow record for a gauge consists of the maximum instantaneous flow value for each year, recorded every year over a given time period. If annual maximum flow values were recorded only for years in which large events occurred, then the record would no longer be systematic. Gaps (missing years) in the systematic record do not preclude use of such data so long as the gaps are the result of missing data, and not the result of filtering the data based on flow magnitude.

Historical data are flow estimates for events not included in the systematic record. These data typically consist of historically significant events, and thus are a sample of extreme events observed by locals. Historical data should be included in the analysis when possible. In cases where only a short systematic record is available, historical data are particularly valuable. Use of historical data also ensures that the results of the analysis will be consistent with the experience of the local community (USGS 2018). Bulletin 17C incorporates new procedures on how to better include historical data in the analysis.

Paleoflood and botanical information can also be part of a statistical stream gauge analysis. Paleofloods are different from historical floods in that they are determined by geologic and physical evidence of past floods rather than human records or referenced from built infrastructure. Geomorphic surfaces, like terraces adjacent to rivers, can be used to place limits on flood discharges to estimate nonexceedance bounds. Paleoflood data are treated similarly to historical flood data for flood frequency analysis. Botanical information consists of vegetation that records evidence of flood(s) or stability of a geomorphic surface over time. Examples include corrasion scars, adventitious sprouts, tree age, and tree ring anomalies. For flood frequency analysis, it is common to describe botanical information as binomial-censored observations. Bulletin 17C includes guidance on how to incorporate this information.

For highway drainage design purposes, a statistical analysis of stream gauge data is typically applied only when adequate data from stream gauging stations are available. The definition of adequate data comes from USGS practice and is provided in Table 4-3.

Table 4-3: Recommended Minimum Stream Gauge Record Lengths (Dalrymple and Benson 1960)

Desired percent chance exceedance (ARI)	Minimum record length (years)
10-year	8
25-year	10
50-year	15
100-year	20

For TxDOT application, sources for annual peak flow data include:

- USGS National Water Information System (NWIS).
- US Department of the Interior, USGS Water Resources Data - Texas, Surface Water. These are prepared annually and contain records for 1 water year per publication. As a result, abstracting annual peaks for a long record is time consuming.

- International Boundary and Water Commission water bulletins.
- River authority and municipal sources such as Lower Colorado River Authority (LCRA) Hydromet.

If the available data sources allow the designer to construct a sufficiently large sample of annual peak streamflow values, then the following conditions must also be satisfied or accounted for before undertaking the statistical analysis:

- The data must be representative of the design condition of the watershed.
- The data must not be significantly affected by upstream regulation (such as, dams, reservoirs, and diversions).
- The systematic record must be stationary, with no general trend of increasing or decreasing flows resulting from changes to the watershed.
- The data must be homogeneous, with flow values resulting from the same types of events. If annual peak flows can result from either rainfall or snowmelt, then a mixed population analysis may be required.
- Errors in flow measurements must not be significant relative to other uncertainties in the analysis.

Log-Pearson Type III Distribution Fitting Procedure

The log-Pearson type III (LPIII) statistical distribution method is recommended in Bulletin #17C and is the standard of practice for estimating annual probability of exceedance of peak flows. An outline of this method follows. However, the designer is not limited to using this method, especially if the resulting flow-frequency relation does not seem to fit the data.

The following general procedure is used for LPIII analyses. The LPIII, Skew, and Accommodation of Outliers procedures described below are still based on information from Bulletin #17B. The latest HDM update occurred soon after the release of #17C and during ongoing TxDOT research on updating skew procedures. Recent edits in this section simply introduce Bulletin #17C. A future HDM version will update these sections to reflect latest #17C procedures. Meanwhile, refer to Bulletin #17C for further information.

1. Acquire and assess the annual peak discharge record.
2. Compute the base 10 logarithm of each discharge value.
3. Compute the mean, standard deviation, and (station) skew of the log flow values.
4. Compute the weighted skew coefficient from the station skew and regional skew.
5. Identify high and low outliers from the sample set.

6. Recompute the mean, standard deviation, and station skew of the log flow values with outliers removed from the sample set.
7. Compute flow values for desired AEPs.

With the LPIII method, the logarithm of the discharge for any AEP is calculated as:

$$\log Q_p = \overline{Q_L} + KS_L$$

Equation 4-2.

Where:

$\overline{Q_L}$ = *mean of the logarithms of the annual peak discharges*

Q_p = *flood magnitude (cfs or m³/s) of AEP p*

K = *frequency factor for AEP p and coefficient of skew appropriate for site*

S_L = standard of deviation of logarithms of the annual peak discharges

See the spreadsheet freqfrac.xls for values of K, based on station skew coefficient.

The three statistical moments used to describe the LPIII distribution are the mean, standard deviation, and skew. Estimates of these moments for the distribution of the entire population of flows are computed for the available sample of flows with the equations below.

The mean is given by:

$$\overline{Q_L} = \frac{\sum X}{N}$$

Equation 4-3.

Where:

$\overline{Q_L}$ = mean of the (base 10) logarithms of the annual peak discharges

X = logarithm of the annual peak discharge

N = number of observations

The standard deviation is given by:

$$S_L = \left\{\frac{\Sigma x^2 - \frac{(\Sigma x)^2}{N}}{N-1}\right\}^{\frac{1}{2}}$$

Equation 4-4.

Where:

S_L = standard deviation of the logarithms of the annual peak discharge; N and X are defined as above

The coefficient of skew (station skew) is given by:

$$G = \frac{N^2(\sum X^3) - 3N(\sum X)(\sum X^2) + 2(\sum X)^3}{N(N-1)(N-2)S_L^3}$$

Equation 4-5.

Where:

G = coefficient of skew of log values; N, X, and S_L are defined as above

Skew represents the degree of curvature to the flow-frequency curve as shown in Figure 4-3. In Figure 4-3 the X-axis scale is probability (symmetric at about AEP = 0.5) and the Y-axis scale is base 10 logarithmic flow. A skew of zero results in a straight-line flow frequency curve. A negative skew value produces a flow-frequency curve with lesser flows than the zero skew line, and a positive skew produces a flow-frequency curve with greater flows than the zero skew line.

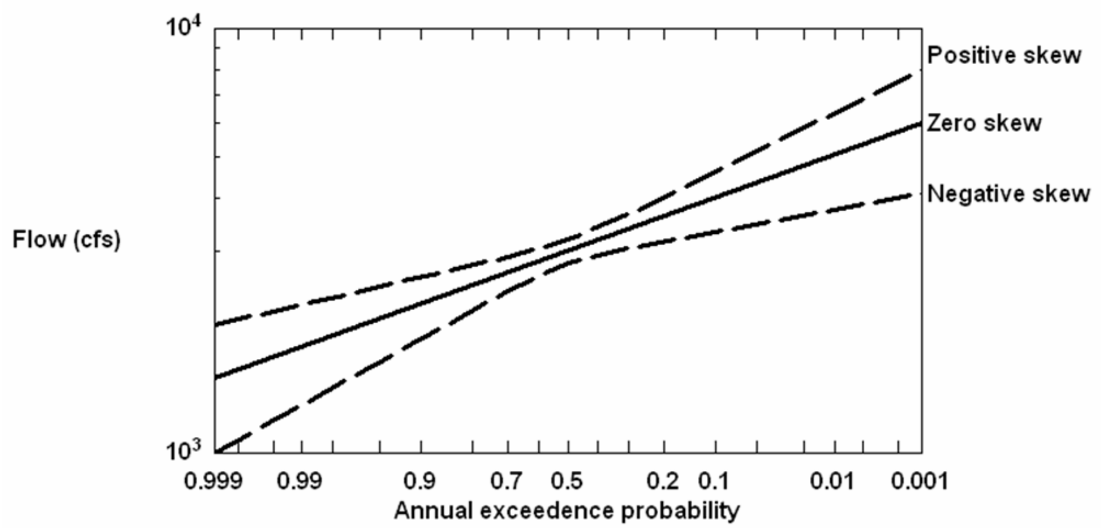

Figure 4-3. Skew of discharge versus frequency plots

The following cases require special consideration. Bulletin #17C provides further guidance:

- Record is incomplete—flows missing from record because they were too small or too large to measure (flows filtered from record based on flow magnitude).
- Record contains zero flow values—stream was dry all year.
- Record contains historical flows not recorded in a systematic fashion. Examples are extreme events recorded prior to or after installation of a stream gauge. These are indicated by code in USGS annual peak discharge data.

- Flows are the result of two distinct types (a mixed population) of hydrologic events such as snowmelt and rainstorms.

Skew

Bulletin #17B recommends using the weighted skew value, G_W, to determine frequency factor values in Equation 4-2.

To calculate weighted skew, use this equation, and follow the steps provided in Bulletin #17B:

$$G_W = \frac{(MSE_{\overline{G}})(G) + (MSE_G)(\overline{G})}{MSE_{\overline{G}} + MSE_G}$$

Equation 4-6.

Where:

G_W = weighted skew value

$\overline{G}$ = regional skew from Figure 4-4

G = station skew from Equation 4-5

$MSE_{\overline{G}}$ = mean square error of $\overline{G}$ for Texas, is = 0.123 (RMSE = 0.35) (Judd et al. 1996), which replaces the value of 0.302 (RMSE = 0.55) presented in Bulletin #17B.

MSE_G = mean square error of G. MSE_G is a function of G and period of record

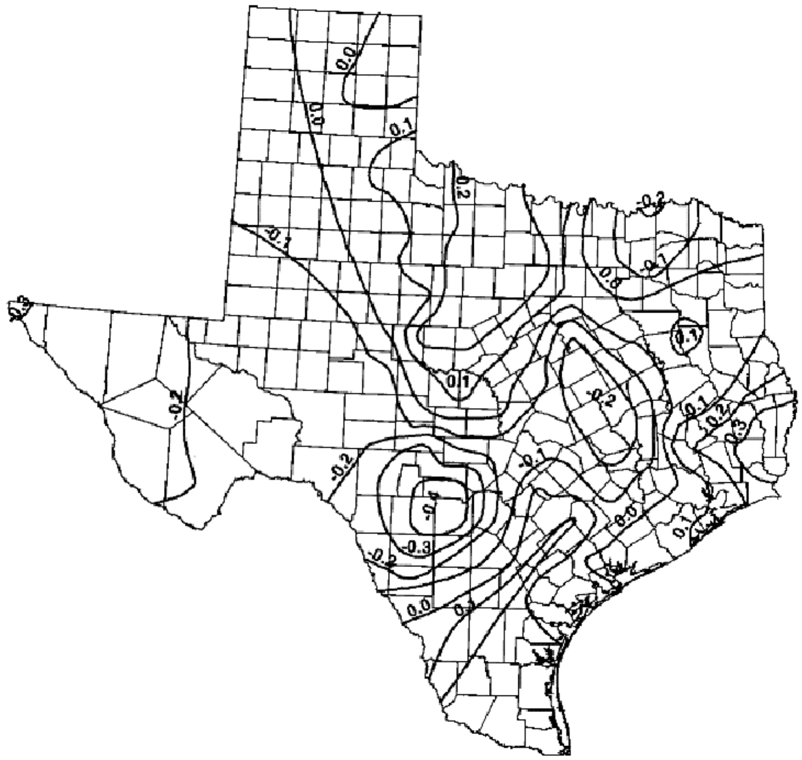

Figure 4-4. Generalized skew coefficients for Texas (Judd 1996) ($\overline{G}$)

$$MSE_G \approx 10^{\left[A - B\left[\log\left(\frac{N}{10}\right)\right]\right]}$$

Equation 4-7.

Where N is the record of length and

$A = -0.33 + 0.08\ |G|$ for $|G| \leq 0.90$

$A = -0.52 + 0.30\ |G|$ for $|G| > 0.90$

And

$B = 0.94 - 0.26\ |G|$ for $|G| \leq 1.50$

$B = 0.55$ for $|G| > 1.50$

Accommodation of Outliers

The distribution of all the annual and historical peak discharges determines the shape of the flow-frequency curve and thus the design-peak discharges. The shape of the frequency curve generated by a log-Pearson type III analysis is symmetrical about the center of the curve. Therefore, the distribution of the higher peak discharges affects the shape of the curve, as does the distribution of the lower peak discharges.

Flooding is erratic in Texas, so a series of observed floods may include annual peak discharge rates that do not seem to belong to the population of the series. The values may be extremely large or extremely small with respect to the rest of the series of observations. Such values may be outliers that should be excluded from the set of data to be analyzed or treated as historical data. Bulletin #17C calls for identification of these outliers.

Design flows are typically infrequent large flows. Therefore, it is desirable to base the frequency curve on the distribution of the larger peaks. This is accomplished by eliminating from the analyses peak discharges lower than a low-outlier threshold. The value for the low-outlier threshold, therefore, should exclude those peaks not indicative of the distribution for the higher peaks. This value is chosen by reviewing the sequentially ranked values for all peak discharges used in the analysis.

Equation 4-8 provides a means of identifying the low outlier threshold (Asquith et. al 1995):

$$LOT = 10^{\left(a\overline{Q_L} + bS_L + cG + d\right)}$$

Equation 4-8.

Where:

LOT = estimated low-outlier threshold (cfs)

$\overline{Q_L}$ = mean of the logarithms of the annual peak discharge (see Equation 4-3)

S_L = standard deviation of the logarithms of the annual peak discharge (see Equation 4-4)

G = coefficient of skew of log values (station skew, see Equation 4-5)

a = 1.09

b = -0.584

c = 0.140

d = -0.799

This equation was developed for English units only and does not currently have a metric equivalent.

High outlier thresholds permit identification of extremely high peak discharges with probability smaller than indicated by the period of record for a station. For example, if a true 1% percent chance exceedance (100-year) peak discharge were gauged during a 10-year period of record, the frequency curve computed from the 10 years of record would be unduly shaped by the 1% percent chance exceedance peak.

The USGS has made efforts to identify high outliers, referred to as historical peaks, by identifying and interviewing residents living proximate to the gauging stations. In many cases, residents have identified a particular flood peak as being the highest since a previous higher peak. These peaks are identified as the highest since a specific date.

In other cases, residents have identified a specific peak as the highest since they have lived proximate to the gauging station. Those peaks are identified as the highest since at least a specific date. The historical peaks may precede or be within the period of gauged record for the station.

Equation 4-9 provides a means of identifying the high outlier threshold (Bulletin #17B):

$$HOT = \overline{X} + K_N S_L$$

Equation 4-9.

Where:

HOT = estimated high-outlier threshold (logarithm of flow)

N = number of systematic peaks remaining in sample after previously detected outliers have been removed

$\overline{X}$ = mean of the logarithms of the systematic annual peak discharges, with previously detected outliers removed

S_L = standard of deviation of the logarithms of the annual peak discharges

K_N = frequency factor for sample size N from Appendix 4 of Bulletin #17B

All known historical peak discharges and their associated gauge heights and dates appear on the USGS Texas Water Science web site.

To incorporate high outlier information when fitting the LPIII distribution according to Bulletin #17B procedures, the designer will:

- Use Equation 4-9 to define the high-outlier threshold.
- Collect supporting information about the identified high outlying flows.
- Retain as part of the systematic record any high outlying flows found not to be the maximum flow of record.
- Extend the period of record for the analysis to include the flow if the flow's value is found to be the maximum flow of record and lies outside the systematic record. If the value does lie within the systematic record, the period of record is not extended. In both cases, the designer shall recompute the LPIII parameters following the procedure described in Section V.A.9 and Appendix 6 of Bulletin #17B.
- Thoroughly document data, interviews, decisions, and assumptions used to justify the identification of high outliers and recomputation of LPIII parameters.

TxDOT recommends the use of hydrologic statistical analysis computer programs that can detect outlying values and recomputed LPIII parameters consistent with `Bulletin #17C` procedures.

Transposition of Gauge Analysis Results

If gauge data are not available at the design location, discharge values can be estimated by transposition if a peak flow-frequency curve is available at a nearby gauged location. This method is appropriate for hydrologically similar watersheds that differ in area by less than 50 percent, with outlet locations less than 100 miles apart.

From the research of Asquith and Thompson 2008, an estimate of the desired AEP peak flow at the ungauged site is provided by Equation 4-10:

$$Q_1 = Q_2\sqrt{\frac{A_1}{A_2}}$$

Equation 4-10.

Where:

Q_1 = Estimated AEP discharge at ungauged watershed 1

Q_2 = Known AEP discharge at gauged watershed 2

A_1 = Area of watershed 1

A_2 = Area of watershed 2

Transposition of peak flow is demonstrated with the following example. A designer requires an estimate of the 1% AEP streamflow at an ungauged location with drainage area of 200 square miles. A nearby (within 100 miles) stream gauge has a hydrologically similar drainage area of 450 square miles. The 1% AEP peak streamflow at the gauged location is 420 cfs based on the peak flow-frequency curve developed for that location. Substituting into Equation 4-10 results in 280 cfs as an estimate of the 1% AEP peak discharge at the ungauged location:

$$Q_1 = Q_2\sqrt{\frac{A_1}{A_2}} = 420\sqrt{\frac{200}{450}} = 280$$

If flow-frequency curves are available at multiple gauged sites, Equation 4-10 can be used to estimate the desired peak AEP flow from each site. Then, with judgment and knowledge of the watersheds, those estimates could be weighted to provide an estimate of the desired AEP flow at the ungauged location. This process should be well documented.

Design of a storage facility, such as a detention pond, may require estimates of AEP flows for longer durations. If a flow-frequency curve for longer flow duration is available at a nearby gauged location, then Equation 4-11, based on an analysis of mean-daily flows (Asquith et al. 2006), may be used for transposition:

$$Q_1 = Q_2\left(\frac{A_1}{A_2}\right)^{0.9}$$

Equation 4-11.

Section 10 — Regression Equations Method

Regression equations are recommended as the primary hydrologic method for off-system (non-TxDOT) projects; for on-system projects, they are recommended as a check on other methods. Omega EM regression equations are reliable beyond 10 sq. mi. drainage area. A comparison method should be used for drainage areas below 10 sq. mi. and must be used for drainage areas below about 5 sq. mi. This method should not be used for drainage areas less than 1 sq. mi.

Discretion may be used on off-system bridges and culverts. As the design of these crossings is typically "hydraulically same or slightly better," the importance of having an exact flowrate is of lesser importance than on-system crossings. At the engineer's discretion, the use of a comparison method may be disregarded.

If an adequate record of streamflow is not available at or near the project site, an LPIII distribution cannot be developed with `Bulletin #17C` procedures. An alternative for estimating the needed design flow is to use a regression equation.

Regression equations are used to transfer flood characteristics from gauged to ungauged sites through the use of watershed and climatic characteristics as explanatory or predictor variables. USGS has developed such regression equations for natural basins throughout the State of Texas.

Procedure for Using Omega EM Regression Equations for Natural Basins

Equations have been developed for natural basins in 1-degree latitude and longitude quadrangles in Texas. Figure 4-5 shows the geographic extents of each quadrangle. The approach used to develop the regional equations is referred to as the “Regression Equations for Estimation of Annual Peak-Streamflow Frequency for Undeveloped Watersheds in Texas Using an L-moment-Based, PRESS-Minimized, Residual-Adjusted Approach.” (USGS 2009) For development and use of regression equations a natural basin is defined as having less than 10 percent impervious cover, less than 10 percent of its drainage area controlled by reservoirs, and no other human-related factors affecting streamflow (USGS 2001). The equations are therefore not applicable to urban watersheds.

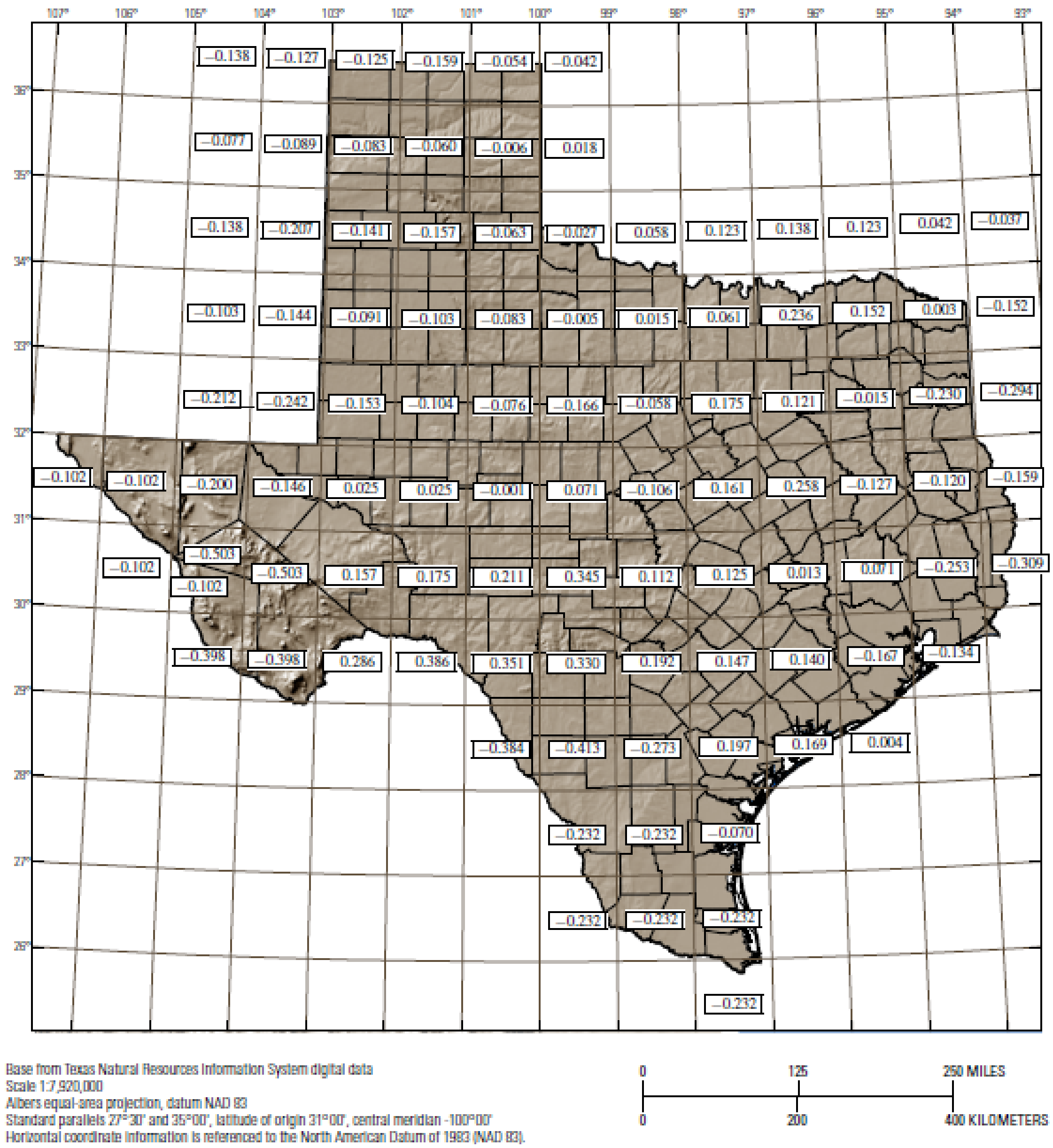

Figure 4-5. OmegaEM (Ω) quadrangles for Texas regression equations. To view a .pdf of this image, click here.

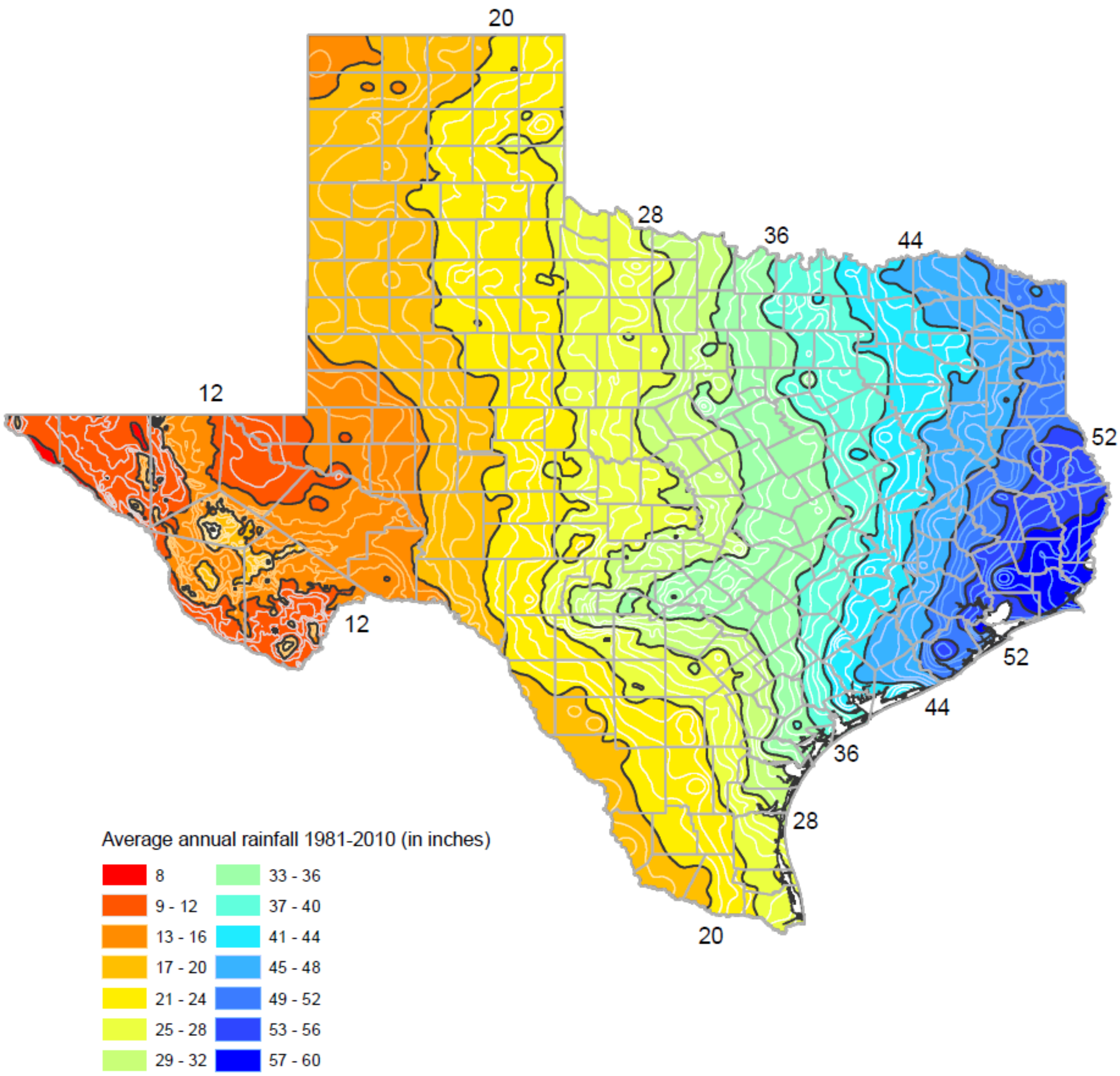

Figure 4-6. Mean annual precipitation, in inches (Source: Texas Water Development Board 2017)

Equation 4-12 is the general form of the Omega EM regression equation for Texas:

$$Q_T = P^c S^d \times 10^{[e\Omega + a + bA^{\lambda}]}$$

Equation 4-12.

Where:

Q_T = peak discharge of recurrence interval *T* years (cfs)

P = mean annual precipitation in inches from Figure 4-6

S = dimensionless main channel slope

Ω = OmegaEM from Figure 4-5

A = contributing drainage area (mi^2)

l = a power determined by iterative PRESS-minimization for the recurrence interval

a, b, c, d, e = regression coefficients specific for the recurrence interval

Mean annual precipitation is the arithmetic mean of a suitably long period of time of total annual precipitation in inches. The mean annual precipitation was assigned based on the approximate center of the watersheds. Asquith and Roussel (2009 TxDOT 0-5521-1) considers that any general and authoritative source of mean annual precipitation for any suitably long period (perhaps 30 years) is sufficient for substitution into the regression equations.

Main channel length is defined as the length in stream-course miles of the longest defined channel from the approximate watershed headwaters to the outlet. Main channel slope is defined as the change in elevation, in feet, between the two end points of the main channel divided by the main channel length in feet.

OmegaEM (Ω) parameter represents a generalized terrain and climate index that expresses relative differences in peak-streamflow potential not represented in the watershed characteristics of drainage area, slope, and mean annual precipitation.

Since the gauges used to develop the equations are points in space, and that the variables used (contributing area, slope, precipitation) are actually attributes of that specific point, the OmegaEM should also pertain to the point in question. As such, if the contributing drainage area overlaps more than one quadrant on Figure 4-5, the OmegaEM must not be weighted or averaged. The OmegaEM specific for the quadrant of the site must be selected.

The summary of weighted-least-squares, PRESS-minimized, regional regression equations using drainage area, dimensionless main-channel slope, mean annual precipitation, and OmegaEM are provided in Table 4-4.

RSE, residual standard error in $\log_{10}$- units of cubic feet per second; Adj., adjusted; AIC, Akaike Information Criterion; PRESS, Prediction Error Sum of Squares.

Table 4-4: Regression Equations

Regression Equations	RSE	Adj. R-squared	AIC statistic	PRESS statistic
$Q_2 = P^{1.398} S^{0.270} \times 10^{[0.776\Omega + 50.98 - 50.30A^{-0.0058}]}$	0.29	0.84	273	64.6
$Q_5 = P^{1.308} S^{0.372} \times 10^{[0.885\Omega + 16.62 - 15.32A^{-0.0215}]}$	0.26	0.88	122	49.1
$Q_{10} = P^{1.203} S^{0.403} \times 10^{[0.918\Omega + 13.62 - 11.97A^{-0.0289}]}$	0.25	0.89	86.5	46.6

Table 4-4: Regression Equations

Regression Equations	RSE	Adj. R-squared	AIC statistic	PRESS statistic
$Q_{25} = P^{1.140} S^{0.446} \times 10^{[0.945\Omega + 11.79 - 9.819A^{-0.0374}]}$	0.26	0.89	140	49.5
$Q_{50} = P^{1.105} S^{0.476} \times 10^{[0.961\Omega + 11.17 - 8.997A^{-0.0424}]}$	0.28	0.87	220	55.6
$Q_{100} = P^{1.071} S^{0.507} \times 10^{[0.969\Omega + 10.82 - 8.448A^{-0.0467}]}$	0.30	0.86	320	64.8
$Q_{500} = P^{0.988} S^{0.569} \times 10^{[0.976\Omega + 10.40 - 7.605A^{-0.0554}]}$	0.37	0.81	591	98.7

Section 11 — Time of Concentration

Time of concentration (t_c) is the time required for an entire watershed to contribute to runoff at the point of interest for hydraulic design; this time is calculated as the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. Travel time and t_c are functions of length and velocity for a particular watercourse. A long but steep flow path with a high velocity may actually have a shorter travel time than a short but relatively flat flow path. There may be multiple paths to consider in determining the longest travel time. The designer must identify the flow path along which the longest travel time is likely to occur.

In watersheds with low (flat) topographic slope, the calculation of t_c using commonly accepted equations with slope in the denominator often results in unreasonably large values. That is, as the slope approaches zero, the travel time approaches infinity. In addition, since intensity is a function of depth divided by t_c, a long t_c produces a very small intensity and thus small flowrate. Cleveland et al. 2012 recommends an adjustment of 0.0005 to the slope in both the Kerby and Kirpich methods to allow more realistic results for low topographic slope watersheds.

- The adjusted slope becomes S_{low} slope = S_0 + 0.0005 (dimensionless)
- If the slope is less than 0.002 ft/ft (0.2%), a low slope condition exists and the adjusted slope should be used.
- If the slope is between 0.002 ft/ft (0.2%) and 0.003 ft/ft (0.3%), the situation is transitional and the user must use judgment on whether or not to use the low slope adjustment.

When runoff is computed using the rational method, t_c is the appropriate storm duration and in turn determines the appropriate precipitation intensity.

When peak discharge and streamflow timing are computed using the hydrograph method, t_c is used to compute certain rainfall-runoff parameters for the watershed. The value of t_c is used as an input to define the appropriate storm duration and appropriate precipitation depth.

When applicable, the Kerby-Kirpich method (Roussel et al. 2005) can be used for estimating t_c. The National Resources Conservation Service (1986) method is also commonly used and acceptable. Both of these methods estimate t_c as the sum of travel times for discrete flow regimes. One good practice is to run both methods concurrently and compare results. Another good practice is to compare t_c values against either watershed length or area for multiple basins across each project to assess reasonableness of results.

Kerby-Kirpich Method

Roussel et al. 2005 conclude that, in general, Kirpich-inclusive approaches, [and particularly] the Kerby-Kirpich approach, for estimating watershed time of concentration are preferable. The

Kerby-Kirpich approach requires comparatively few input parameters, is straightforward to apply, and produces readily interpretable results. The Kerby-Kirpich approach produces time of concentration estimates consistent with watershed time values independently derived from real-world storms and runoff hydrographs. Similar to other methods for calculation of t_c, the total time of concentration is obtained by adding the overland flow time (Kerby) and the channel flow time (Kirpich):

$$t_c = t_{ov} + t_{ch}$$

Equation 4-13.

Where:

t_{ov} = overland flow time

t_{ch} = channel flow time

The Kerby-Kirpich method for estimating t_c is applicable to watersheds ranging from 0.25 square miles to 150 square miles, main channel lengths between 1 and 50 miles, and main channel slopes between 0.002 and 0.02 (ft/ft) (Roussel et al. 2005).

Main channel slope is computed as the change in elevation from the watershed divide to the watershed outlet divided by the curvilinear distance of the main channel (primary flow path) between the watershed divide and the outlet.

No watersheds with low topographic slopes are available in the underlying database. Therefore, the Kerby and Kerpich methods are not usually applicable to watersheds with limited topographic slope. However, Cleveland et al. 2012 makes recommendations for adjustments to the method to allow more realistic results for low topographic slope watersheds. See Time of Concentration.

The Kerby Method

For small watersheds where overland flow is an important component of overall travel time, the Kerby method can be used. The Kerby equation is

$$t_{ov} = K(L \times N)^{0.467} S^{-0.235}$$

Equation 4-14.

Where:

t_{ov} = overland flow time of concentration, in minutes

K = a units conversion coefficient, in which K = 0.828 for traditional units and K = 1.44 for SI units

L = the overland-flow length, in feet or meters as dictated by K

N = a dimensionless retardance coefficient

S = the dimensionless slope of terrain conveying the overland flow

In the development of the Kerby equation, the length of overland flow was as much as 1,200 feet (366 meters). Hence, this length is considered an upper limit and shorter values in practice generally are expected. The dimensionless retardance coefficient used is similar in concept to the well-known Manning's roughness coefficient; however, for a given type of surface, the retardance coefficient for overland flow will be considerably larger than for open-channel flow. Typical values for the retardance coefficient are listed in Table 4-5. Roussel et al. 2005 recommends that the user should not interpolate the retardance coefficients in Table 4-5. If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the time of concentration. See Time of Concentration.

Table 4-5: Kerby Equation Retardance Coefficient Values

Generalized terrain description	Dimensionless retardance coefficient (N)
Pavement	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

The Kirpich Method

For channel-flow component of runoff, the Kirpich equation is:

$$t_{ch} = KL^{0.770}S^{-0.385}$$

Equation 4-15.

Where:

t_{ch} = the time of concentration, in minutes

K = a units conversion coefficient, in which K = 0.0078 for traditional units and K = 0.0195 for SI units

L = the channel flow length, in feet or meters as dictated by K

S = the dimensionless main-channel slope

If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the time of concentration. See Time of Concentration.

Application of the Kerby-Kirpich Method

An example (shown below) illustrating application of the Kerby-Kirpich method is informative. For example, suppose a hydraulic design is needed to convey runoff from a small watershed with a drainage area of 0.5 square miles. On the basis of field examination and topographic maps, the length of the main channel from the watershed outlet (the design point) to the watershed divide is 5,280 feet. Elevation of the watershed at the outlet is 700 feet. From a topographic map, elevation along the main channel at the watershed divide is estimated to be 750 feet. The analyst assumes that overland flow will have an appreciable contribution to the time of concentration for the watershed. The analyst estimates that the length of overland flow is about 500 feet and that the slope for the overland-flow component is 2 percent (S = 0.02). The area representing overland flow is average grass (N = 0.40). For the overland-flow t_c, the analyst applies the Kerby equation,

$$t_{ov} = 0.828(500 \times 0.40)^{0.467}(0.02)^{-0.235}$$

from which t_{ov} is about 25 minutes. For the channel t_{ch}, the analyst applies the Kirpich equation, but first dimensionless main-channel slope is required,

$$S = \frac{750 - 700}{5{,}280} = 0.0095$$

or about 1 percent. The value for slope and the channel length are used in the Kirpich equation,

$$t_{ch} = 0.0078(5,280 - 500)^{0.770}(0.0095)^{-0.385}$$

from which t_{ch} is about 32 minutes. Because the overland flow t_{ov} is used for this watershed, the subtraction of the overland flow length from the overall main-channel length (watershed divide to outlet) is necessary and reflected in the calculation. Adding the overland flow and channel flow components gives `total time of concentration for` a watershed of about 57 minutes. Finally, as a quick check, the analyst can evaluate the t_c by using an ad hoc method representing t_c, in hours, as the square root of drainage area, in square miles. For the example, the square root of the drainage area yields a t_c estimate of about 0.71 hours or about 42 minutes, which is reasonably close to 57 minutes. However, 57 minutes is preferable. This example is shown in Figure 4-7.

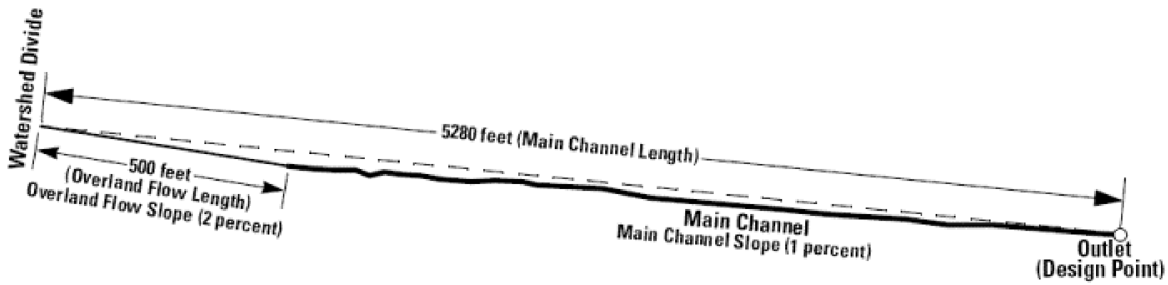

Figure 4-7. Example application of Kerby-Kirpich method

Natural Resources Conservation Service (NRCS) Method for Estimating t_c

The NRCS method for estimating t_c is applicable for small watersheds, in which the majority of flow is overland flow such that timing of the peak flow is not significantly affected by the contribution flow routed through underground storm drain systems. With the NRCS method:

$$t_c = t_{sh} + t_{sc} + t_{ch}$$

Equation 4-16.

***Where**:*

t_{sh} = sheet flow travel time

t_{sc} = shallow concentrated flow travel time

t_{ch} = channel flow travel time

NRCS 1986 provides the following descriptions of these flow components:

Sheet flow is flow over plane surfaces, usually occurring in the headwater of streams. With sheet flow, the friction value is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment.

Sheet flow usually becomes shallow concentrated flow after around 100 feet.

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on USGS quadrangle sheets.

For open channel flow, consider the uniform flow velocity based on bank-full flow conditions. That is, the main channel is flowing full without flow in the overbanks. This assumption avoids the significant iteration associated with rainfall intensity or discharges (because rainfall intensity and discharge are dependent on time of concentration).

For conduit flow, in a proposed storm drain system, compute the velocity at uniform depth based on the computed discharge at the upstream. Otherwise, if the conduit is in existence, determine full capacity flow in the conduit, and determine the velocity at capacity flow. You may need to compare this velocity later with the velocity calculated during conduit analysis. If there is a significant difference and the conduit is a relatively large component of the total travel path, recompute the time of concentration using the latter velocity estimate.

If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the time of concentration. See Time of Concentration.

Sheet Flow Time Calculation

Sheet flow travel time is computed as:

$$t_{sh} = \frac{0.007(n_{ol}L_{sh})^{0.8}}{(P_2)^{0.5}S_{sh}^{0.4}}$$

Equation 4-17.

Where*:*

t_{sh} = sheet flow travel time (hr.)

n_{ol} = overland flow roughness coefficient (provided in Table 4-6)

L_{sh} = sheet flow length (ft) (100 ft. maximum)

P_2 = 2-year, 24-h rainfall depth (in.) (provided in - NOAA's Precipitation Frequency Data Server for Atlas 14)

S_{sh} = sheet flow slope (ft/ft)

Table 4-6: Overland Flow Roughness Coefficients for Use in NRCS Method in Calculating Sheet Flow Travel Time (NRCS 1986)

Surface description		n_{ol}
Smooth surfaces (concrete, asphalt, gravel, or bare soil)		0.011
Fallow (no residue)		0.05
Cultivated soils:	Residue $cover \leq 20\%$	0.06
	Residue cover > 20%	0.17
Grass:	Short grass prairie	0.15
	Dense grasses	0.24
	Bermuda	0.41

Table 4-6: Overland Flow Roughness Coefficients for Use in NRCS Method in Calculating Sheet Flow Travel Time (NRCS 1986)

Surface description		n_{ol}
Range (natural):		0.13
Woods:	Light underbrush	0.40
	Dense underbrush	0.80

NOTE: 'n' values for overland flows (nol) are not to be used in other channel or floodplain applications.

Shallow Concentrated Flow

Shallow concentrated flow travel time is computed as:

$$t_{sc} = \frac{L_{sc}}{3600KS_{sc}^{0.5}}$$

Equation 4-18.

***Where**:*

t_{sc} = shallow concentrated flow time (hr.)

L_{sc} = shallow concentrated flow length (ft)

K = 16.13 for unpaved surface, 20.32 for paved surface

S_{sc} = shallow concentrated flow slope (ft/ft)

Channel Flow

Channel flow travel time is computed by dividing the channel distance by the flow rate obtained from Manning’s equation. This can be written as:

$$t_{ch} = L_{ch} \Big/ \left((3600 \frac{1.49}{n} R^{\frac{2}{3}} S_{ch}^{\frac{1}{2}}) \right)$$

Equation 4-19.

***Where**:*

t_{ch} = channel flow time (hr.)

L_{ch} = channel flow length (ft)

S_{ch} = channel flow slope (ft/ft)

n = Manning’s roughness coefficient

R = channel hydraulic radius (ft), and is equal to $\frac{a}{p_w}$, where: a = cross sectional area (ft^2) and p_w = wetted perimeter (ft), consider the uniform flow velocity based on bank-full flow conditions. That is, the main channel is flowing full without flow in the overbanks. This assumption avoids the significant iteration associated with other methods that employ rainfall intensity or discharges (because rainfall intensity and discharge are dependent on time of concentration).

Manning's Roughness Coefficient Values

Manning's roughness coefficients are used to calculate flows using Manning's equation. Values from American Society of Civil Engineers (ASCE) 1992, FHWA 2001, and Chow 1959 are reproduced in Table 4-7, Table 4-8, and Table 4-9.

Table 4-7: Manning's Roughness Coefficients for Open Channels

Type of channel	Manning's n
A. Natural streams	
1. Minor streams (top width at flood stage < 100 ft)	
a. Clean, straight, full, no rifts or deep pools	0.025-0.033
b. Same as a, but more stones and weeds	0.030-0.040
c. Clean, winding, some pools and shoals	0.033-0.045
d. Same as c, but some weeds and stones	0.035-0.050
e. Same as d, lower stages, more ineffective	0.040-0.055
f. Same as d, more stones	0.045-0.060
g. Sluggish reaches, weedy, deep pools	0.050-0.080
h. Very weedy, heavy stand of timber and underbrush	0.075-0.150
i. Mountain streams with gravel and cobbles, few boulders on bottom	0.030-0.050
j. Mountain streams with cobbles and large boulders on bottom	0.040-0.070
2. Floodplains	
a. Pasture, no brush, short grass	0.025-0.035
b. Pasture, no brush, high grass	0.030-0.050
c. Cultivated areas, no crop	0.020-0.040
d. Cultivated areas, mature row crops	0.025-0.045
e. Cultivated areas, mature field crops	0.030-0.050
f. Scattered brush, heavy weeds	0.035-0.070
g. Light brush and trees in winter	0.035-0.060
h. Light brush and trees in summer	0.040-0.080

Table 4-7: Manning's Roughness Coefficients for Open Channels

Type of channel	Manning's n
i. Medium to dense brush in winter	0.045-0.110
j. Medium to dense brush in summer	0.070-0.160
k. Trees, dense willows summer, straight	0.110-0.200
l. Trees, cleared land with tree stumps, no sprouts	0.030-0.050
m. Trees, cleared land with tree stumps, with sprouts	0.050-0.080
n. Trees, heavy stand of timber, few down trees, flood stage below branches	0.080-0.120
o. Trees, heavy stand of timber, few down trees, flood stage reaching branches	0.100-0.160
3. Major streams (top width at flood stage > 100 ft)	
a. Regular section with no boulders or brush	0.025-0.060
b. Irregular rough section	0.035-0.100
B. Excavated or dredged channels	
1. Earth, straight and uniform	
a. Clean, recently completed	0.016-0.020
b. Clean, after weathering	0.018-0.025
c. Gravel, uniform section, clean	0.022-0.030
d. With short grass, few weeds	0.022-0.033
2. Earth, winding and sluggish	
a. No vegetation	0.023-0.030
b. Grass, some weeds	0.025-0.033
c. Deep weeds or aquatic plants in deep channels	0.030-0.040
d. Earth bottom and rubble sides	0.028-0.035
e. Stony bottom and weedy banks	0.025-0.040
f. Cobble bottom and clean sides	0.030-0.050
g. Winding, sluggish, stony bottom, weedy banks	0.025-0.040
h. Dense weeds as high as flow depth	0.050-0.120
3. Dragline-excavated or dredged	
a. No vegetation	0.025-0.033
b. Light brush on banks	0.035-0.060
4. Rock cuts	

Table 4-7: Manning's Roughness Coefficients for Open Channels

Type of channel	Manning's n
a. Smooth and uniform	0.025-0.040
b. Jagged and irregular	0.035-0.050
5. Unmaintained channels	
a. Dense weeds, high as flow depth	0.050-0.120
b. Clean bottom, brush on sides	0.040-0.080
c. Clean bottom, brush on sides, highest stage	0.045-0.110
d. Dense brush, high stage	0.080-0.140
C. Lined channels	
1. Asphalt	0.013-0.016
2. Brick (in cement mortar)	0.012-0.018
3. Concrete	
a. Trowel finish	0.011-0.015
b. Float finish	0.013-0.016
c. Unfinished	0.014-0.020
d. Gunite, regular	0.016-0.023
e. Gunite, wavy	0.018-0.025
4. Riprap (n-value depends on rock size)	0.020-0.035
5. Vegetal lining	0.030-0.500

Table 4-8: Manning's Coefficients for Streets and Gutters

Type of gutter or pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement: smooth texture	0.013
Asphalt pavement: rough texture	0.016
Concrete gutter with asphalt pavement: smooth texture	0.013
Concrete gutter with asphalt pavement: rough texture	0.015
Concrete pavement: float finish	0.014
Concrete pavement: broom finish	0.016
Table 4-8 note: For gutters with small slope or where sediment may accumulate, increase n values by 0.02 (USDOT, FHWA 2001).	

Table 4-9: Manning's Roughness Coefficients for Closed Conduits (ASCE 1982, FHWA 2001)

Material		Manning's n
Asbestos-cement pipe		0.011-0.015
Brick		0.013-0.017
Cast iron pipe		
	Cement-lined & seal coated	0.011-0.015
Concrete (monolithic)		
	Smooth forms	0.012-0.014
	Rough forms	0.015-0.017
	Concrete pipe	0.011-0.015
	Box (smooth)	0.012-0.015
Corrugated-metal pipe -- (2-1/2 in. x 1/2 in. corrugations)		
	Plain	0.022-0.026
	Paved invert	0.018-0.022
	Spun asphalt lined	0.011-0.015
	Plastic pipe (smooth)	0.011-0.015
Corrugated-metal pipe -- (2-2/3 in. by 1/2 in. annular)		0.022-0.027
Corrugated-metal pipe -- (2-2/3 in. by 1/2 in. helical)		0.011-0.023
Corrugated-metal pipe -- (6 in. by 1 in. helical)		0.022-0.025
Corrugated-metal pipe -- (5 in. by 1 in. helical)		0.025–0.026
Corrugated-metal pipe -- (3 in. by 1 in. helical)		0.027–0.028
Corrugated-metal pipe -- (6 in. by 2 in. structural plate)		0.033–0.035
Corrugated-metal pipe -- (9 in. by 2-1/2 in. structural plate)		0.033–0.037
Corrugated polyethylene		0.010–0.013
	Smooth	0.009-0.015
	Corrugated	0.018–0.025
Spiral rib metal pipe (smooth)		0.012-0.013
Vitrified clay		
	Pipes	0.011-0.015
	Liner plates	0.013-0.017
Polyvinyl chloride (PVC) (smooth)		0.009-0.011
Table 4-9 note: Manning's n for corrugated pipes is a function of the corrugation size, pipe size, and whether the corrugations are annular or helical (see USGS 1993).		

Section 12 — Rational Method

The Rational method is appropriate for estimating peak discharges for small drainage areas of up to about 200 acres (80 hectares) with no significant flood storage. The method provides the designer with a peak discharge value, but does not provide a time series of flow nor flow volume.

Assumptions and Limitations

Use of the rational method includes the following assumptions and limitations:

- The method is applicable if t_c for the drainage area is less than the duration of peak rainfall intensity.
- The calculated runoff is directly proportional to the rainfall intensity.
- Rainfall intensity is uniform throughout the duration of the storm.
- The frequency of occurrence for the peak discharge is the same as the frequency of the rainfall producing that event.
- Rainfall is distributed uniformly over the drainage area.
- The minimum duration to be used for computation of rainfall intensity is 10 minutes. If the time of concentration computed for the drainage area is less than 10 minutes, then 10 minutes should be adopted for rainfall intensity computations.
- The rational method does not account for storage in the drainage area. Available storage is assumed to be filled.

The above assumptions and limitations are the reason the rational method is limited to watersheds 200 acres or smaller. If any one of these conditions is not true for the watershed of interest, the designer should use an alternative method.

The rational method represents a steady inflow-outflow condition of the watershed during the peak intensity of the design storm. Any storage features having sufficient volume that they do not completely fill and reach a steady inflow-outflow condition during the duration of the design storm cannot be properly represented with the rational method. Such features include detention ponds, channels with significant volume, and floodplain storage. When these features are present, an alternate rainfall-runoff method is required that accounts for the time-varying nature of the design storm and/or filling/emptying of floodplain storage. In these cases, the hydrograph method is recommended.

The steps in developing and applying the rational method are illustrated in Figure 4-8.

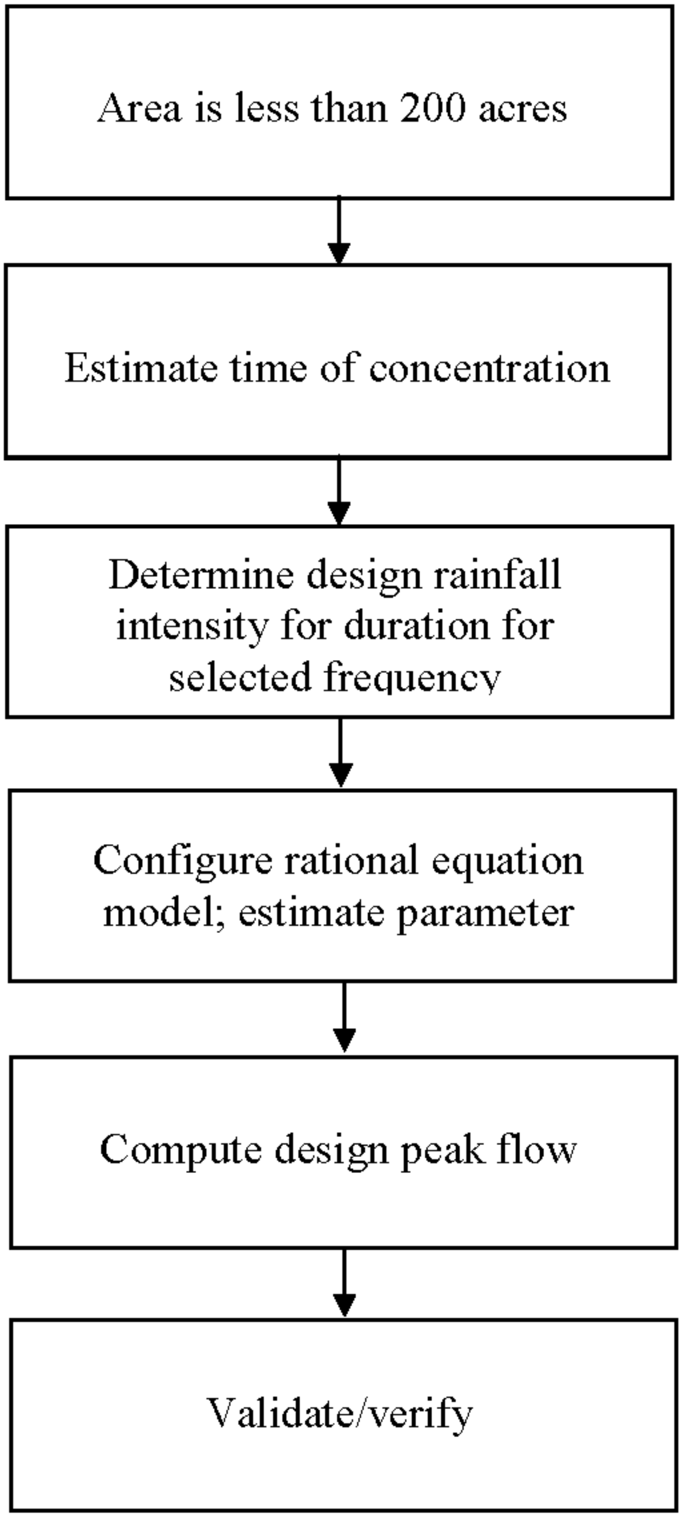

Figure 4-8. Steps in developing and applying the rational method

Procedure for using the Rational Method

The rational formula estimates the peak rate of runoff at a specific location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration. The rational formula is:

$$Q = \frac{CIA}{Z}$$

Equation 4-20.

Where:

Q = maximum rate of runoff (cfs or m^3/sec.)

C = runoff coefficient

I = average rainfall intensity (in./hr. or mm/hr.)

A = drainage area (ac or ha)

Z = conversion factor, 1 for English, 360 for metric

Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in in./hr. for a specific rainfall duration and a selected frequency. The duration is assumed to be equal to the time of concentration. For drainage areas in Texas, you may compute the rainfall intensity using Equation 4-21, which is known as a rainfall intensity-duration-frequency (IDF) relationship (power-law model).

$$I = \frac{b}{(t_c + d)^e}$$

Equation 4-21.

Where:

I = design rainfall intensity (in./hr.)

t_c = time of concentration (min) as discussed in Section 11

e, b, d = coefficients based on rainfall IDF data.

In September 2018, the National Oceanic and Atmospheric Administration (NOAA) released updated precipitation frequency estimates for Texas. These estimates are available through NOAA's Precipitation Frequency Data Server (PFDS) website and the report documenting the approach is also available at the same website - NOAA Atlas 14, Volume 11: Precipitation-Frequency Atlas of the United States. This new rainfall data is considered best available data and should be used for all projects. Tabular IDF data are available from the PFDS, but linear interpolation or curve generation is needed to obtain intensity values between tabular durations. Ongoing TxDOT research will produce future e, b, d coefficients to better automate intensity calculations. However, barring significant project implementation concerns, Atlas 14 IDF data should be used. Exceptions must be approved by the DHE or DES HYD and noted on the plans or drainage report.

Currently, the coefficients in Equation 4-21 can be found in the EBDLKUP-2015v2.1.xlsx spreadsheet lookup tool (developed by Cleveland et al. 2015) for specific frequencies listed by county (See video/tutorial on the use of the EBDLKUP-2015v2.1.xlsx spreadsheet tool). This spreadsheet is based on prior rainfall frequency-duration data contained in the Atlas of Depth-Duration Frequency (DDF) of Precipitation of Annual Maxima for Texas (TxDOT 5-1301-01-1).

If a project is approved to use the older values from the EBDLKUP-2015v2.1.xlsx spreadsheet lookup tool or from existing functionality in design software like GEOPAK, they should still evaluate the new NOAA rainfall changes for their project area and, if there are increases for the design frequency, estimate an appropriate level of freeboard for use. The freeboard amount and a description of how it was generated should be noted in both the plans and the drainage report. Software that facilitates Rational Method calculations often has IDF curves from rainfall data embedded into the software. Location-specific IDF from the new NOAA rainfall data can be imported for each project into the software.

TxDOT is currently working with Texas Transportation Institute (TTI) staff, as part of research project 0-6980, to update the IDF curve relationships for the state of Texas based on the 2018 NOAA rainfall data. This work will include an update of the EBDLKUP-2015v2.1.xlsx file linked above and planned for inclusion in the next HDM update.

The general shape of a rainfall IDF curve is shown in Figure 4-9. As rainfall duration approaches zero, the rainfall intensity tends towards infinity. Because the rainfall intensity/duration relationship is assessed by assuming that the duration is equal to the time of concentration, small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. To minimize this likelihood, use a minimum time of concentration of 10 minutes. As the duration tends to infinity, the design rainfall tends towards zero. Usually, the area limitation of 200 acres for Rational Method calculations should result in rainfall intensities that are not unrealistically low. However, if the estimated time of concentration is extremely long, such as may occur in extremely flat areas, it may be necessary to consider an upper threshold of time or use a different hydrologic method.

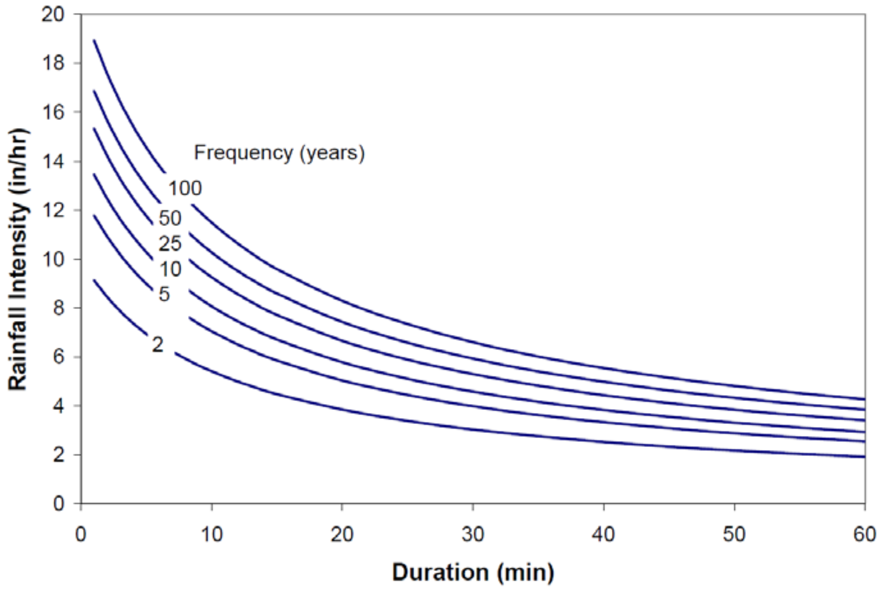

Figure 4-9. Typical Rainfall Intensity Duration Frequency Curve

In some instances alternate methods of determining rainfall intensity may be desired, especially for coordination with other agencies. Ensure that any alternate methods are applicable and documented.

Runoff Coefficients

Urban Watersheds

Table 4-10 suggests ranges of C values for urban watersheds for various combinations of land use and soil/surface type. This table is typical of design guides found in civil engineering texts dealing with hydrology.

Table 4-10: Runoff Coefficients for Urban Watersheds

Type of drainage area	Runoff coefficient
Business:	
Downtown areas	0.70-0.95
Neighborhood areas	0.30-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.35-0.40
Apartment dwelling areas	0.30-0.70
Industrial:	
Light areas	0.30-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.30-0.40
Railroad yards	0.30-0.40
Unimproved areas:	
Sand or sandy loam soil, 0-3%	0.15-0.20
Sand or sandy loam soil, 3-5%	0.20-0.25
Black or loessial soil, 0-3%	0.18-0.25
Black or loessial soil, 3-5%	0.25-0.30
Black or loessial soil, > 5%	0.70-0.80
Deep sand area	0.05-0.15
Steep grassed slopes	0.70
Lawns:	
Sandy soil, flat 2%	0.05-0.10
Sandy soil, average 2-7%	0.10-0.15
Sandy soil, steep 7%	0.15-0.20
Heavy soil, flat 2%	0.13-0.17
Heavy soil, average 2-7%	0.18-0.22

Table 4-10: Runoff Coefficients for Urban Watersheds

Type of drainage area	Runoff coefficient
Heavy soil, steep 7%	0.25-0.35
Streets:	
Asphaltic	0.85-0.95
Concrete	0.90-0.95
Brick	0.70-0.85
Drives and walks	0.75-0.95
Roofs	0.75-0.95

Rural and Mixed-Use Watershed

Table 4-11 shows an alternate, systematic approach for developing the runoff coefficient. This table applies to rural watersheds only, addressing the watershed as a series of aspects. For each of four aspects, the designer makes a systematic assignment of a runoff coefficient "component." Using Equation 4-22, the four assigned components are added to form an overall runoff coefficient for the specific watershed segment.

The runoff coefficient for rural watersheds is given by:

$$C = C_r + C_i + C_v + C_s$$

Equation 4-22.

Where:

C = runoff coefficient for rural watershed

C_r = component of coefficient accounting for watershed relief

C_i = component of coefficient accounting for soil infiltration

C_v = component of coefficient accounting for vegetal cover

C_s = component of coefficient accounting for surface type

The designer selects the most appropriate values for C_r, C_i, C_v, and C_s from Table 4-11.

Table 4-11: Runoff Coefficients for Rural Watersheds

Watershed characteristic	Extreme	High	Normal	Low
Relief - C_r	0.28-0.35 Steep, rugged terrain with average slopes above 30%	0.20-0.28 Hilly, with average slopes of 10-30%	0.14-0.20 Rolling, with average slopes of 5-10%	0.08-0.14 Relatively flat land, with average slopes of 0-5%
Soil infiltration - C_i	0.12-0.16 No effective soil cover; either rock or thin soil mantle of negligible infiltration capacity	0.08-0.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity or poorly drained	0.06-0.08 Normal; well drained light or medium textured soils, sandy loams	0.04-0.06 Deep sand or other soil that takes up water readily; very light, well-drained soils
Vegetal cover - C_v	0.12-0.16 No effective plant cover, bare or very sparse cover	0.08-0.12 Poor to fair; clean cultivation, crops or poor natural cover, less than 20% of drainage area has good cover	0.06-0.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	0.04-0.06 Good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover
Surface Storage - C_s	0.10-0.12 Negligible; surface depressions few and shallow, drainageways steep and small, no marshes	0.08-0.10 Well-defined system of small drainageways, no ponds or marshes	0.06-0.08 Normal; considerable surface depression, e.g., storage lakes and ponds and marshes	0.04-0.06 Much surface storage, drainage system not sharply defined; large floodplain storage, large number of ponds or marshes
Table 4-11 note: The total runoff coefficient based on the 4 runoff components is $C = C_r + C_i + C_v + C_s$				

While this approach was developed for application to rural watersheds, it can be used as a check against mixed-use runoff coefficients computed using other methods. In so doing, the designer would use judgment, primarily in specifying C_s, to account for partially developed conditions within the watershed.

Mixed Land Use

For areas with a mixture of land uses, a composite runoff coefficient should be used. The composite runoff coefficient is weighted based on the area of each respective land use and can be calculated as:

$$C_w = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=1}^{n} A_j}$$

Equation 4-23.

Where:

C_W = weighted runoff coefficient

C_j = runoff coefficient for area j

A_j = area for land cover j (ft^2)

n = number of distinct land uses

Section 13 — Hydrograph Method

A hydrograph represents runoff as it varies over time at a particular location within the watershed. The area integrated under the hydrograph represents the volume of runoff.

Estimation of a runoff hydrograph, as opposed to the peak rate of runoff, is necessary for watersheds with complex runoff characteristics. The hydrograph method also should be used when storage must be evaluated, as it accounts explicitly for volume and timing of runoff. The hydrograph method has no drainage area size limitation.

Figure 4-2 shows that in cases for which a statistical distribution cannot be fitted and a regression equation will not predict adequately the design flow, some sort of empirical or conceptual rainfall-runoff model can be used to predict the design flow. Such application is founded on the principle that the AEP of the computed runoff peak or volume is the same as the AEP of the rainfall used as input to (the boundary condition for) the model.

The hydrograph method is applicable for watersheds in which t_c is longer than the duration of peak rainfall intensity of the design storm. Precipitation applied to the watershed model is uniform spatially, but varies with time. The hydrograph method accounts for losses (soil infiltration for example) and transforms the remaining (excess) rainfall into a runoff hydrograph at the outlet of the watershed. Figure 4-10 shows the different components that must be represented to simulate the complete response of a watershed.

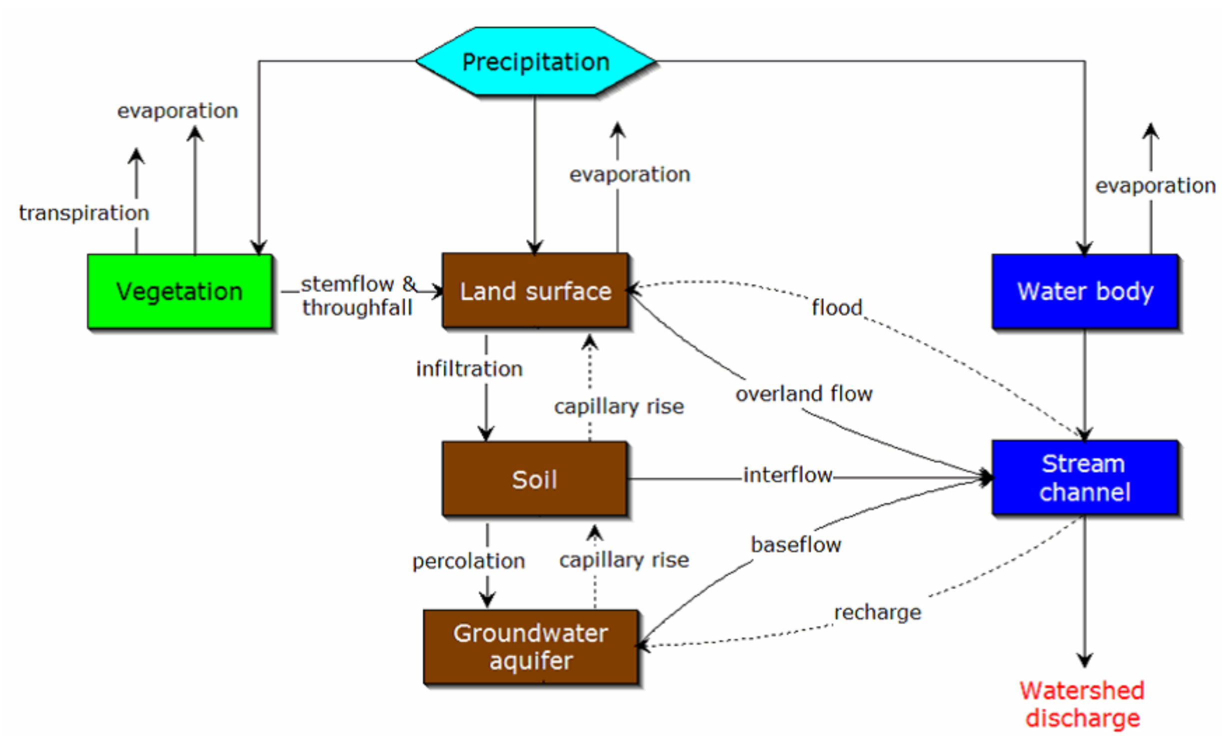

Figure 4-10. Components of the hydrograph method

Because the resulting runoff hydrograph is a time series of flow values, the method provides a peak flow value as well as volume of runoff. This makes the method suitable for design problems requiring runoff volume as a design parameter.

Successful application of the hydrograph method requires the designer to:

- Define the temporal and spatial distribution of the desired AEP design storm.
- Specify appropriate loss model parameters to compute the amount of precipitation lost to other processes, such as infiltration, and does not run off the watershed.
- Specify appropriate parameters to compute runoff hydrograph resulting from excess (not lost) precipitation.
- If necessary for the application, specify appropriate parameters to compute the lagged and attenuated hydrograph at downstream locations.

Basic steps to developing and applying a rainfall-runoff model for predicting the required design flow are illustrated in Figure 4-11. These steps are described in more detail below.

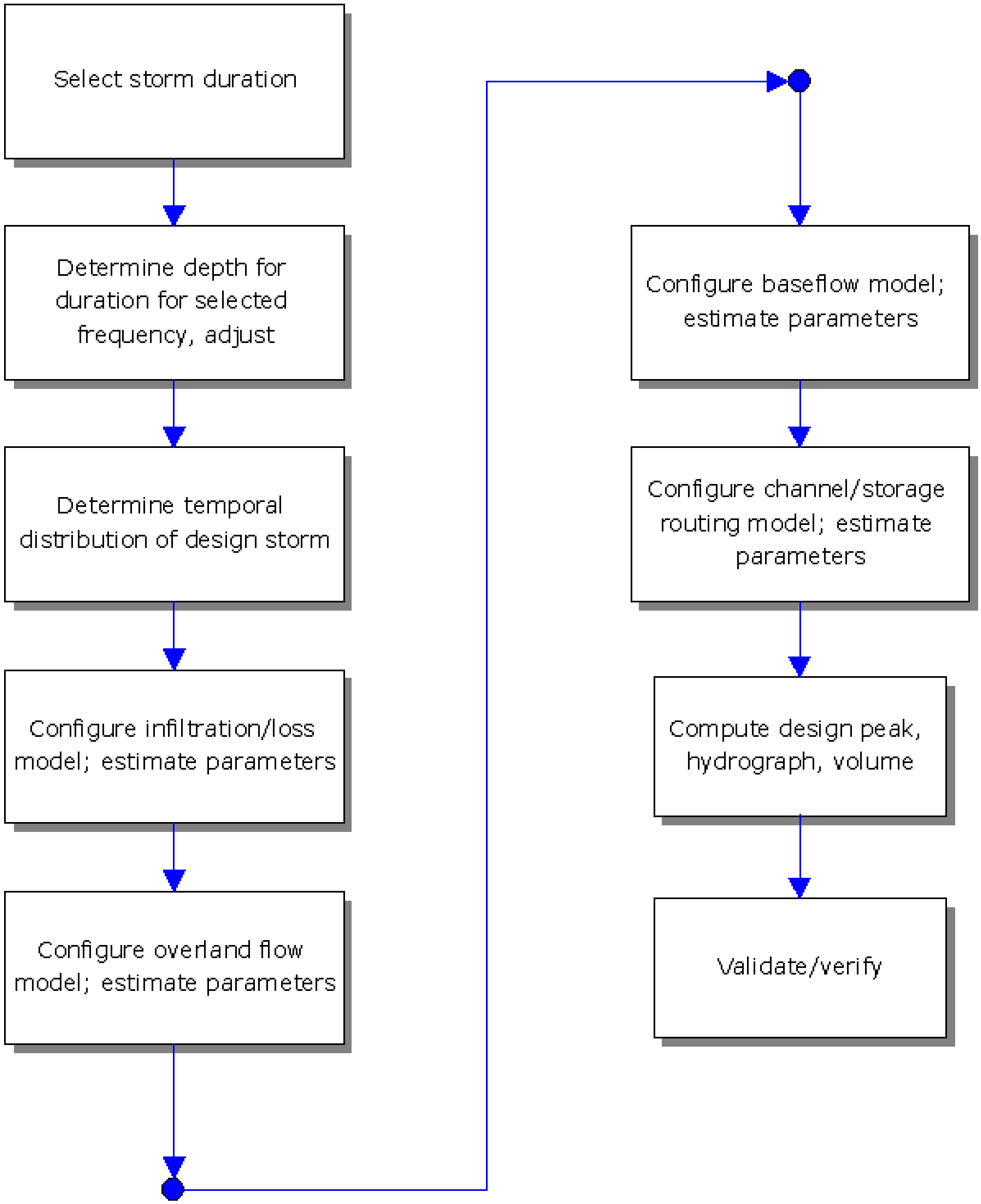

Figure 4-11. Steps in developing and applying the hydrograph method

Watershed Subdivision

The method is also applicable to complex watersheds, in which runoff hydrographs for multiple subbasins are computed, then routed to a common point and combined to yield the total runoff hydrograph at that location.

TxDOT research on undeveloped watersheds (0-5822-01-2) has indicated that there is little justification for subdividing a watershed for the purpose of improving model accuracy. In general, subdivision had little or no impact on runoff volume for the following reasons:

1. In general, subdivision of watersheds for modeling results in no more than modest improvements in prediction of peak discharge. Improvements generally are not observed with more than about five to seven subdivisions;
2. Watershed subdivision multiplies the number of sub-process model parameters required to model watershed response and introduces the requirement to route flows through the watershed drainage network. Discrimination of parameters between sub-watersheds is difficult to justify from a technical perspective;
3. The introduction of watershed subdivisions requires hydrologic (or hydraulic) routing for movement of sub-watershed discharges toward the watershed outlet. The routing sub-process model requires estimates of additional parameters that are subject to uncertainty;
4. The dependence of computed hydrographs on internal routing became more apparent as the number of subdivisions increased; and
5. Application of distributed modeling, as currently implemented in HEC-HMS, was difficult and time consuming. It is unclear what technical advantage is gained by application of this modeling approach in an uncalibrated mode, given the level of effort required to develop the models.

There are circumstances in which watershed subdivision is appropriate. If one of the sub-watersheds is distinctly different than the other components of the watershed, and if the drainage of that sub-watershed is a significant fraction of the whole (20-50%), then a subdivision might be appropriate. Specific examples of an appropriate application of watershed subdivision would be:

- the presence of a reservoir on a tributary stream,
- a significant difference in the level of urbanization of one component of a watershed, or
- a substantial difference in physical characteristics (main channel slope, overland flow slope, loss characteristics, and so forth).
- unique storm depths are appropriate for the different subbasin areas.
- computed hydrographs are needed at more than one location.

Design Storm Development

A design storm is a precipitation pattern or intensity value defined for design of drainage facilities. Design storms are either based on historical precipitation data or rainfall characteristics in the project area or region. Application of design storms ranges from point precipitation for calculation of peak flows using the rational method to storm hyetographs as input for rainfall-runoff analysis in the hydrograph method. The fundamental assumption using design storms is that precipitation of an AEP yields runoff of the same AEP.

Selection of Storm Duration

Selecting storm duration is the first step in design storm modeling. The appropriate storm duration for stormwater runoff calculations is dependent on the drainage area's hydrologic response. The selected storm duration should be sufficiently long that the entire drainage area contributes to discharge at the point of interest. Storm duration is defined in terms of time of concentration (t_c), which is the time it takes for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed.

For complete drainage of the area, time for overland flow, channel flow, and storage must be considered. Typically for hydrograph computations the storm duration should be four or five times the time of concentration. Longer duration of storm will not increase the peak discharge substantially, but will contribute greater volume of runoff.

Commonly, a storm duration of 24 hours is used. However the 24-hour storm duration should not be used blindly. Runoff from longer and shorter storms should be computed to demonstrate the sensitivity of the design choices.

For TxDOT, the 24-hour storm should be used as a starting point for analysis. However, if the analysis results appear inconsistent with expectations, site performance, or experience, an alternative storm duration should be considered. In that case, the designer should consult the Design Division Hydraulics Branch for advice.

Storm Depth: Depth-Duration-Frequency (DDF) Relationships

Once the storm duration is selected, the next step is to determine the rainfall depth or intensity of that duration for the selected AEP. Depth-Duration-Frequency (DDF) data at each project location is available from the 2018 NOAA Atlas 14 data and accessible through NOAA's Precipitation Frequency Data Server (PFDS). The PFDS is a point-and-click interface developed to deliver NOAA Atlas 14 precipitation frequency estimates and associated information. The PFDS values have been developed on a 300m grid system. For larger watersheds, engineers will need to use judgment in selecting a reasonable point to establish depth values. Consideration should be taken with respect to varying depths across the watershed. The location of the selected depth values and a brief explanation should be reported in the drainage report or plans. Estimates and their confidence intervals can be displayed directly as tables or graphs. From drop-down options on the website, appropriate data type (depth or intensity) and timeseries type (partial duration or annual maximum) should be selected prior to selecting the location. Annual maximum should be selected as the time series for most analyses. Certain municipalities may have a stated preference for use of partial duration and if so, to minimize model differences, that time series type may be used.

The AEPs represented are 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500, 1/1000 (2-, 5-, 10-, 25-, 50-, 100-, 200-, 500-, and 1000-years). The storm durations represented are 5, 10, 15,30 and 60 minutes; 2, 3, 6, and 12 hours; and 1, 2, 3, 4, 7, 10, 20, 30, 45 and 60 days. The depth or intensity for the strom with an Average Recurrence Interval (ARI) of one year is provided with partial duration series.

The prior data source for rainfall was the Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas (TxDOT 5-1301-01-1, 2004). This was an extension of a 1998 USGS study and an update of Technical Paper No. 40: Rainfall Frequency Atlas of the United States (Hershfield 1961), Technical Paper No. 49: 2- to 10-Day Precipitation for Return Periods of 2 to 100 Year in the Contiguous United States (Miller 1964), and NOAA NWS Hydro-35: 5 to 60 Minute Precipitation Frequency for the Eastern and Central United States (Frederick et al. 1977). All these prior precipitation reports are considered superseded for Texas with the 2018 NOAA Atlas 14 data.

Intensity-Duration-Frequency Relationships

While hydrograph methods require both rainfall depth and temporal distribution, the rational method requires only intensity. The rainfall intensity (I) is the average rainfall rate in inches/hour for a specific rainfall duration and a selected frequency. For drainage areas in Texas, rainfall intensity may be computed by:

1. Using Depth-Duration-Frequency (DDF) tabular data/graphs for Texas from the NOAA Precipitation Frequency Data Server (PFDS) to obtain the precipitation depth for a given frequency.
2. Converting the precipitation depth to a precipitation intensity by dividing the depth by the storm duration. The precipitation intensity is measured in inches/hour.

For example, if the 100-year, 6-hour depth is 3.2 inches, the average precipitation intensity over those 6 hours equals 3.2 inches/6 hours = 0.53 inches/hour. However, the IDF `relationships are available from the PFDS server and may be obtained directly without performing this conversion.`

Areal Depth Adjustment

When estimating runoff due to a rainfall event, a uniform areal distribution of rainfall over the watershed is assumed. However, for intense storms, uniform rainfall is unlikely. Rather, rainfall varies across the drainage area. To account for this variation, an areal adjustment is made to convert point depths to an average areal depth. For drainage areas smaller than 10 square miles, the areal adjustment is negligible. For larger areas, point rainfall depths and intensities must be adjusted. Two methods are presented here for use in design of drainage facilities: the first is by the US Weather Bureau and the second is by USGS.

US Weather Bureau Areal Depth Adjustment

The US Weather Bureau (1958) developed Figure 4-11 from an annual series of rain gauge networks. It shows the percentage of point depths that should be used to yield average areal depths.

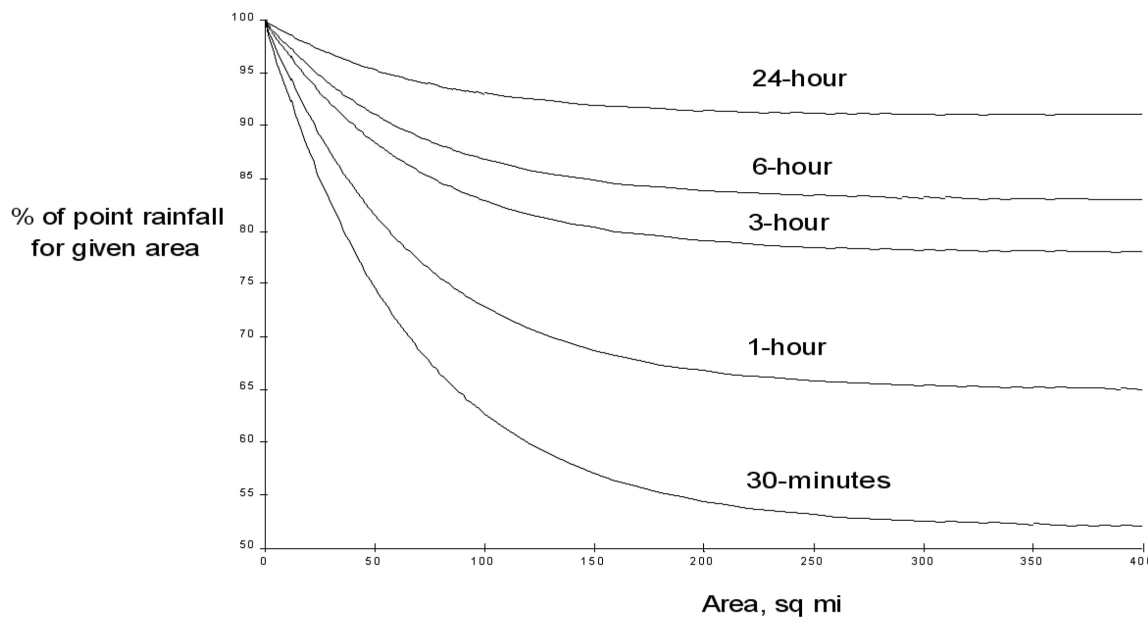

Figure 4-12. Depth area adjustment (US Weather Bureau 1958)

USGS Areal-Reduction Factors for the Precipitation of the 1-Day Design Storm in Texas

Areal reduction factors (ARFs) specific for Texas for a 1-day design storm were developed by Asquith (1999). Asquith's method uses an areal reduction factor that ranges from 0 to 1. The method is a function of watershed characteristics such as size and shape, geographic location, and time of year that the design storm is presumed to occur. The study was based on precipitation monitoring networks in the Austin, Dallas, and Houston areas. If using a 1-day design storm, this is the appropriate method of areal reduction to use for design of highway drainage facilities in Texas.

However, the applicability of this method diminishes the farther away from the Austin, Dallas, or Houston areas the study area is and as the duration of the design storm increasingly differs from that of 1 day. For further information and example problems on calculating the ARF, refer to Asquith (1999).

A relationship exists between the point of an annual precipitation maxima and the distance between both the centroid of the watershed and every location radiating out from the centroid. This is assuming the watershed is nearly circular. ST(r) is the expected value of the ratio between the depth at some location a distance r from the point of the design storm. T refers to the frequency of the design storm. Equations for ST(r) for the 50% (2-year) or smaller AEP vary by proximity to Austin, Dallas, and Houston. For an approximately circular watershed, the ARF is calculated with the following equation:

$$ARF = \frac{\int_0^R 2rS_2(r)dr}{R^2}$$

Equation 4-24.

Where:

r = variable of integration ranging from 0 to R

R = radius of the watershed (mi)

$S_2(r)$ = estimated 2-year or greater depth-distance relation (mi)

The site-specific equations for $S_2(r)$ for differing watershed radii are in Table 4-12 at the end of this section.

Once the ARF is calculated, the effective depth of the design storm is found by multiplying the ARF by the point precipitation depth found using NOAA Precipitation Frequency Data Server.

For example, an approximately circular watershed in the Dallas area is 50.3 square miles (R = 4 miles). From Table 4-12:

$S_2 = 1.0000 - 0.06(r)$ for $0 \le r \le 2$

$S_2 = 0.9670 - 0.0435(r)$ for $2 \le r \le 4$

Substituting the above expressions into Equation 4-24 gives:

$$ARF = \frac{2}{(4)^2}\left\{\int_0^2 r[1 - 0.06r]dr + \int_2^4 r[0.9670 - 0.0435r]dr\right\}$$

$$ARF = 0.125\left\{\left[\frac{r^2}{2} - \frac{0.06r^3}{3}\right]_0^2 + \left[\frac{0.9670r^2}{2} - \frac{0.0435r^3}{3}\right]_2^4\right\}$$

$$ARF = 0.125\left\{\left[\frac{2^2}{2} - \frac{0.06(2^3)}{3}\right] + \left[\frac{0.9670(4^2 - 2^2)}{2} - \frac{0.0435(4^3 - 2^3)}{3}\right]\right\}$$

$$ARF = 0.125(1.84 + 4.99)$$

ARF = 0.85

An easier way to determine ARF for circular watersheds is to use the equation from Table 4-12 in column “ARF for circular watersheds having radius r” for the city and radius of interest. For the previous example (City of Dallas, R = 4 miles), the equation would be:

ARF = 0.9670-0.0290(r) + (0.0440/r^2)

ARF = 0.85

From NOAA's Precipitation Frequency Data Server, the 1% (100-year) 1-day depth is 9.55 inches. Multiply this depth by 0.85 to obtain the 24-hour 1% AEP areally reduced storm depth of 8.12 inches.

If the designer finds that a circular approximation of the watershed is inappropriate for the watershed of interest, the following procedure for non-circular watersheds should be used. The procedure for non-circular watersheds is as follows:

1. Represent the watershed as discrete cells; the cells do not have to be the same area.
2. Locate the cell containing the centroid of the watershed.
3. For each cell, calculate the distance to the centroid (r).
4. Using the distances from Step 3, solve the appropriate equations from Table 4-12 for S_2(r) for each cell.
5. Multiply S_2(r) by the corresponding cell area to compute ARF; the area multiplication simply acts as a weight for a weighted mean.
6. Compute the sum of the cell areas.
7. Compute the sum of the product of S_2(r) and cell area from Step 5.

8. Divide the result of Step 7 by Step 6.

Table 4-12: Equations That Define the Estimated 2-Year or Greater Depth-Distance Relation and the Areal-Reduction Factor for Circular Watersheds

City	Estimated 2-yr or greater depth-distance relation for distance r (mi)	ARF for circular watersheds having radius r (mi)	Equation limits
Austin	$S_2(r) = 1.000 - 0.1400\,(r)$	$ARF = 1.000 - 0.0933\,(r)$	$0 \le r \le 1$
	$S_2(r) = 0.9490 - 0.0890\,(r)$	$ARF = 0.9490 - 0.0593\,(r) + \left(0.0170/r^2\right)$	$1 \le r \le 2$
	$S_2(r) = 0.8410 - 0.0350\,(r)$	$ARF = 0.8410 - 0.0233\,(r) + \left(0.1610/r^2\right)$	$2 \le r \le 3$
	$S_2(r) = 0.8080 - 0.0240\,(r)$	$ARF = 0.8080 - 0.0160\,(r) + \left(0.2600/r^2\right)$	$3 \le r \le 4.5$
	$S_2(r) = 0.7750 - 0.0167\,(r)$	$ARF = 0.7750 - 0.0111\,(r) + \left(0.4828/r^2\right)$	$4.5 \le r \le 9$
	$S_2(r) = 0.7420 - 0.0130\,(r)$	$ARF = 0.7420 - 0.0087\,(r) + \left(1.3737/r^2\right)$	$9 \le r \le 13$
	$S_2(r) = 0.7203 - 0.0113\,(r)$	$ARF = 0.7203 - 0.0076\,(r) + \left(2.5943/r^2\right)$	$13 \le r \le 19$
	$S_2(r) = 0.6950 - 0.0100\,(r)$	$ARF = 0.6950 - 0.0067\,(r) + \left(5.6427/r^2\right)$	$19 \le r \le 28$
	$S_2(r) = 0.6502 - 0.0084\,(r)$	$ARF = 0.6502 - 0.0056\,(r) + \left(17.3505/r^2\right)$	$28 \le r \le 33$
	$S_2(r) = 0.6040 - 0.0070\,(r)$	$ARF = 0.6040 - 0.0047\,(r) + \left(34.1211/r^2\right)$	$33 \le r \le 41$
	$S_2(r) = 0.3717 - 0.0013\,(r)$	$ARF = 0.3717 - 0.0009\,(r) + \left(164.3052/r^2\right)$	$41 \le r \le 50$
Dallas	$S_2(r) = 1.000 - 0.0600\,(r)$	$ARF = 1.000 - 0.0400\,(r)$	$0 \le r \le 2$
	$S_2(r) = 0.9670 - 0.0435\,(r)$	$ARF = 0.9670 - 0.0290\,(r) + \left(0.0440/r^2\right)$	$2 \le r \le 4$
	$S_2(r) = 0.8910 - 0.0245\,(r)$	$ARF = 0.8910 - 0.0163\,(r) + \left(0.4493/r^2\right)$	$4 \le r \le 6$
	$S_2(r) = 0.8760 - 0.0220\,(r)$	$ARF = 0.8760 - 0.0147\,(r) + \left(0.6293/r^2\right)$	$6 \le r \le 8$
	$S_2(r) = 0.8460 - 0.0183\,(r)$	$ARF = 0.8460 - 0.0122\,(r) + \left(1.2693/r^2\right)$	$8 \le r \le 12$
	$S_2(r) = 0.8130 - 0.0155\,(r)$	$ARF = 0.8130 - 0.0103\,(r) + \left(2.8533/r^2\right)$	$12 \le r \le 16$

Table 4-12: Equations That Define the Estimated 2-Year or Greater Depth-Distance Relation and the Areal-Reduction Factor for Circular Watersheds

City	Estimated 2-yr or greater depth-distance relation for distance r (mi)	ARF for circular watersheds having radius r (mi)	Equation limits
	$S_2(r) = 0.7650 - 0.0125\,(r)$	$ARF = 0.7650 - 0.0083\,(r) + \left(6.9493 / r^2\right)$	$16 \le r \le 18$
	$S_2(r) = 0.7200 - 0.0100\,(r)$	$ARF = 0.7200 - 0.0067\,(r) + \left(11.8093 / r^2\right)$	$18 \le r \le 24$
	$S_2(r) = 0.6880 - 0.0087\,(r)$	$ARF = 0.6800 - 0.0058\,(r) + \left(17.9533 / r^2\right)$	$24 \le r \le 27$
	$S_2(r) = 0.6228 - 0.0063\,(r)$	$ARF = 0.6228 - 0.0042\,(r) + \left(33.8091 / r^2\right)$	$27 \le r \le 31$
	$S_2(r) = 0.5563 - 0.0041\,(r)$	$ARF = 0.5563 - 0.0027\,(r) + \left(55.1070 / r^2\right)$	$31 \le r \le 50$
Houston	$S_2(r) = 1.000 - 0.1200\,(r)$	$ARF = 1.000 - 0.0800\,(r)$	$0 \le r \le 1$
	$S_2(r) = 0.9400 - 0.0600\,(r)$	$ARF = 0.9400 - 0.0400\,(r) + \left(0.0200 / r^2\right)$	$1 \le r \le 2$
	$S_2(r) = 0.8800 - 0.0300\,(r)$	$ARF = 0.8800 - 0.0200\,(r) + \left(0.1000 / r^2\right)$	$2 \le r \le 4$
	$S_2(r) = 0.8667 - 0.0267\,(r)$	$ARF = 0.8667 - 0.0178\,(r) + \left(0.1711 / r^2\right)$	$4 \le r \le 7$
	$S_2(r) = 0.8078 - 0.0183\,(r)$	$ARF = 0.8708 - 0.0122\,(r) + \left(1.1334 / r^2\right)$	$7 \le r \le 11$
	$S_2(r) = 0.7363 - 0.0118\,(r)$	$ARF = 0.7363 - 0.0078\,(r) + \left(4.0173 / r^2\right)$	$11 \le r \le 15$
	$S_2(r) = 0.6800 - 0.0080\,(r)$	$ARF = 0.6800 - 0.00053\,(r) + \left(8.2360 / r^2\right)$	$15 \le r \le 20$
	$S_2(r) = 0.6187 - 0.0049\,(r)$	$ARF = 0.6187 - 0.0033\,(r) + \left(16.4138 / r^2\right)$	$20 \le r \le 50$

Rainfall Temporal Distribution

The temporal rainfall distribution is how rainfall intensity varies over time for a single event. The mass rainfall curve, illustrated in Figure 4-13, is the cumulative precipitation up to a specific time. In drainage design, the storm is divided into time increments, and the average depth during each time increment is estimated, resulting in a rainfall hyetograph as shown in Figure 4-14.

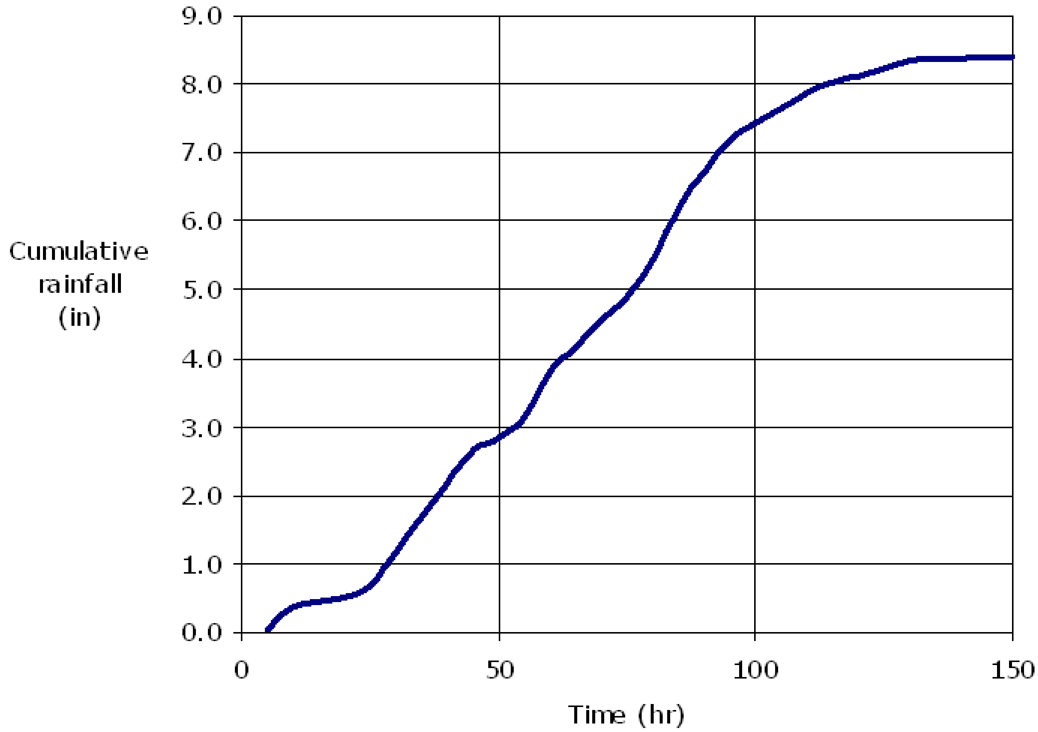

Figure 4-13. Example mass rainfall curve from historical storm

Hyetograph Development Procedure

In the rational method, the intensity is considered to be uniform over the storm period. Hydrograph techniques, however, account for variability of the intensity throughout a storm. Therefore, when using hydrograph techniques, the designer must determine a rainfall hyetograph: a temporal distribution of the watershed rainfall, as shown in Figure 4-14.

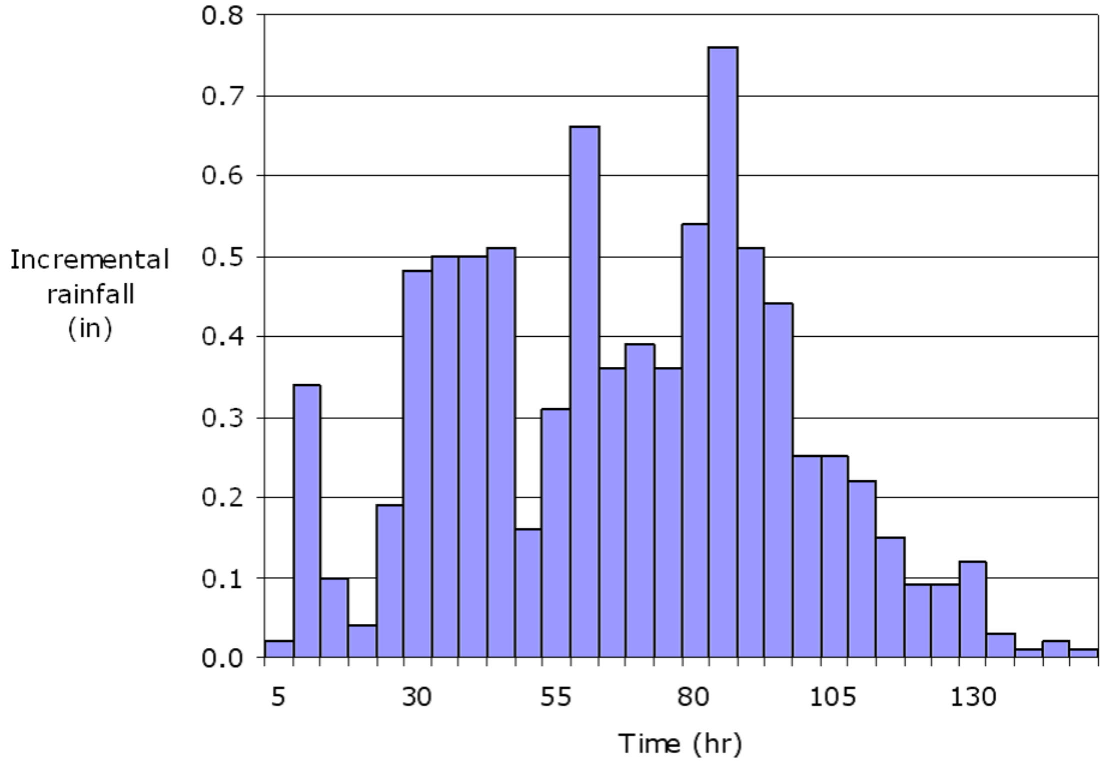

Figure 4-14. Rainfall hyetograph

Methods acceptable for developing a rainfall hyetograph for a design storm include the NRCS method, the balanced storm method, and the Texas storm method.

NRCS Hyetograph Development Procedure

The NRCS design storm hyetographs were derived by averaging storm patterns for regions of the U.S. The storms thus represent a pattern distribution of rainfall over a 24-hour period to which a design rainfall depth can be applied. The distribution itself is arranged in a critical pattern with the maximum precipitation period occurring just before the midpoint of the storm.

The Natural Resources Conservation Service (NRCS) National Engineering Handbook (NEH), Part 630, Chapter 4 includes a detailed discussion on updating the temporal distribution of rainfall based on the new NOAA Atlas 14 rainfall data from the older NRCS Type II-III distributions commonly used in Texas, through 2019. TxDOT is evaluating whether to develop certain statewide temporal distribution zones similar to what Type II-III provided and further guidance may be forthcoming. Meanwhile, continued use of Type II and III temporal distribution of rainfall for Texas is no longer recommended, and the balanced storm method (also known as Frequency storm in HEC-HMS) is now preferred based on NRCS recommendations (NEH, Part 630.04).

Balanced Storm Hyetograph Development Procedure

The temporal distribution, with the peak of the storm located at the center of the hyetograph, is also called balanced storm. It uses DDF values that are based on a statistical analysis of historical data. HEC-HMS software can derive hyetographs with the balanced method when the center of the storm is specified to be at 50% of the total storm duration. HEC-HMS also provides the ability to shift the peak from 50% of the total storm duration to 25%, 33%, 67%, or 75% as well, while maintaining the "nested" effect of the balanced storm.

The procedure for deriving a hyetograph with this method is as follows:

1. For the selected AEP, tabulate rainfall amounts for a storm of a given return period for all durations up to a specified limit (for 24-hour, 15-minute, 30-minute, 1-hour, 2-hour, 3-hour, 6-hour, 12-hour, 24-hour, etc.). Use NOAA's Precipitation Frequency Data Server for the duration and AEP selected for design.

2. Select an appropriate time interval. An appropriate time interval is related to the time of concentration of the watershed. To calculate the time interval, use:

$$\Delta t = \frac{1}{5} or \frac{1}{6} t_c$$

Equation 4-25.

Where:

Dt = time interval

t_c = time of concentration

For example, if the time of concentration is 1 hour, $\Delta t = 1/5 t_c$ = 1/5 of 1 hour = 12 minutes, or 1/6 of 1 hour = 10 minutes. Choosing 1/5 or 1/6 will not make a significant difference in the distribution of the rainfall; use one fraction or the other to determine a convenient time interval.

3. For successive times of interval Δt, find the cumulative rainfall depths from the DDF values. For depths at time intervals not included in the DDF tables, interpolate depths for intermediate durations using a log-log interpolation. (Durations from the table are usually given in hours, but in minutes on the plot.) For example, given a study area in the northern part of Bexar County, the log-log plot in Figure 4-15 shows the 10% depths for the 15-, 30-, 60-, 120-, 180-, 360-, 720-, and 1440-minute durations included in Asquith and Roussel 2004. The precipitation depth at 500 minutes is interpolated as 5.0 inches.

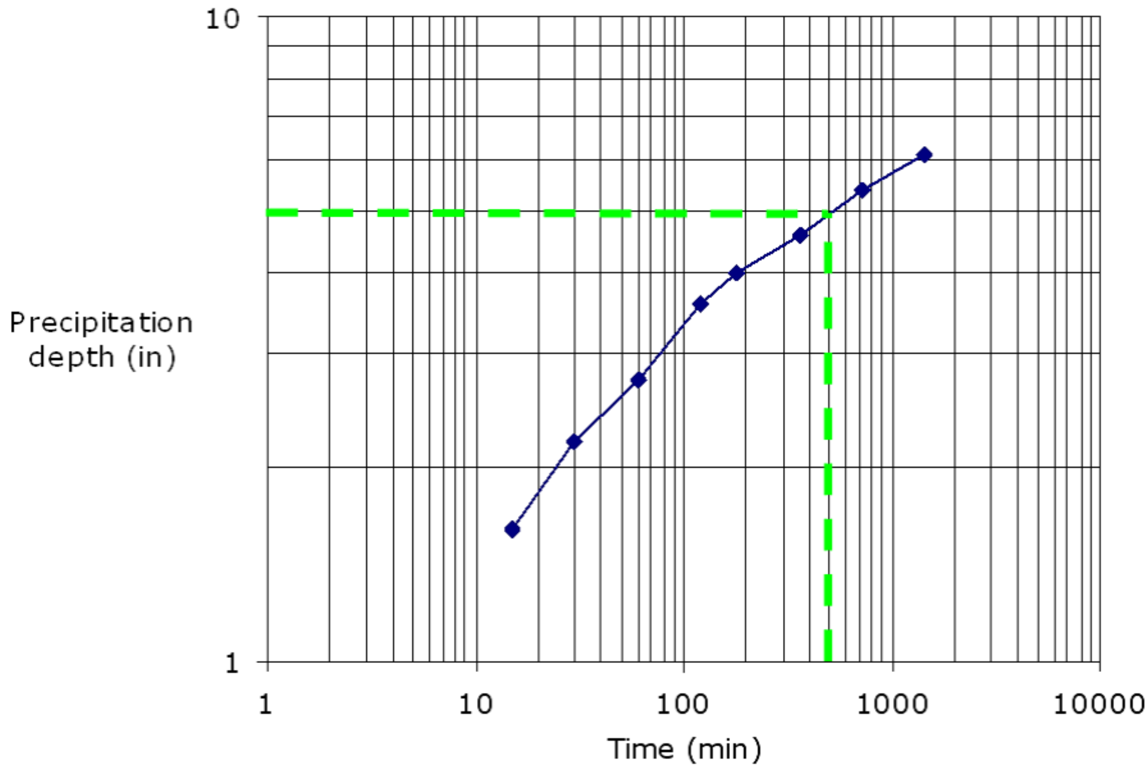

Figure 4-15. Log time versus log precipitation depth

4. Find the incremental depths by subtracting the cumulative depth at a particular time interval from the depth at the previous time interval.
5. Rearrange the incremental depths so that the peak depth is at the center of the storm and the remaining incremental depths alternate (to left and right of peak) in descending order.

For example, in Figure 4-16, the largest incremental depth for a 24-hour storm (1,440 minutes) is placed at the 720-minute time interval and the remaining incremental depths are placed about the 720-minute interval in alternating decreasing order.

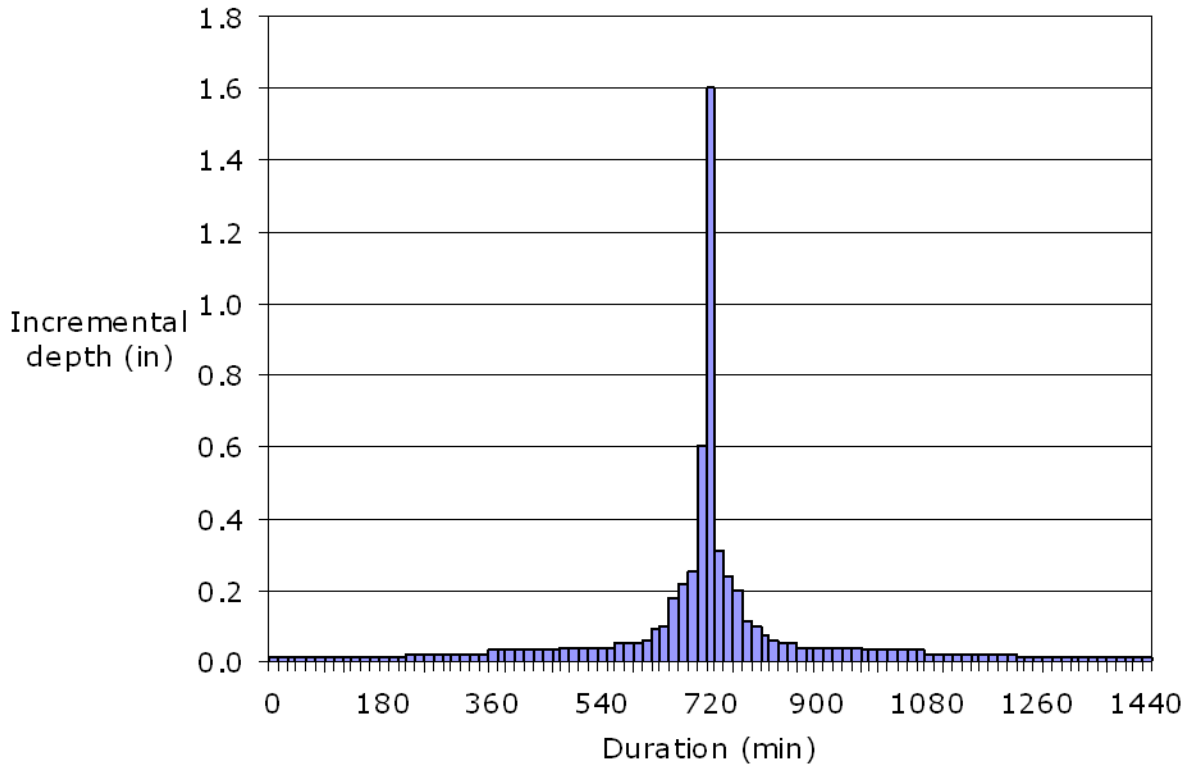

Figure 4-16. Balanced storm hyetograph

Texas Storm Hyetograph Development Procedure

Texas specific dimensionless hyetographs were developed by researchers at USGS, Texas Tech University, University of Houston, and Lamar University (Williams-Sether et al. 2004, Asquith et al. 2005). Two databases were used to estimate the hyetographs: 1) rainfall recorded for more than 1,600 storms over mostly small watersheds as part of historical USGS studies, and 2) hourly rainfall data collection network from the NWS over eastern New Mexico, Oklahoma, and Texas.

Three methods of developing dimensionless hyetographs are presented: 1) triangular dimensionless hyetograph; 2) L-gamma dimensionless hyetograph; and 3) empirical dimensionless hyetograph. Any of these hyetographs can be used for TxDOT design. Brief descriptions of the three methods are presented here. For further information and example problems on the Texas hyetographs, refer to Asquith et al. 2005.

Triangular Dimensionless Hyetograph

A triangular dimensionless hyetograph is presented in Figure 4-17. The vertical axis represents relative rainfall intensity. The rainfall intensity increases linearly until the time of peak intensity, then decreases linearly until the end of the storm. The triangular hyetograph, in terms of relative cumulative storm depth, is defined by Equations 4-26 and 4-27, with values for parameters a and b provided in Table 4-13.

$$p_1(0 \le F \le a) = \frac{1}{a}F^2$$

Equation 4-26.

$$p_2(a < F \le 1) = -\frac{1}{b}F^2 + \left(\frac{2a}{b} + 2\right)F - \left(\frac{a^2}{b} + a\right)$$

Equation 4-27.

***Where**:*

p_1 = normalized cumulative rainfall depth, (ranging from 0 to 1) for F ranging from 0 to a

p_2 = normalized cumulative rainfall depth, (ranging from 0 to 1) for F ranging from a to 1

F = elapsed time, relative to storm duration, ranging from 0 to 1

a = relative storm duration prior to peak intensity, from Table 4-13

b = relative storm duration prior to peak intensity, from Table 4-13

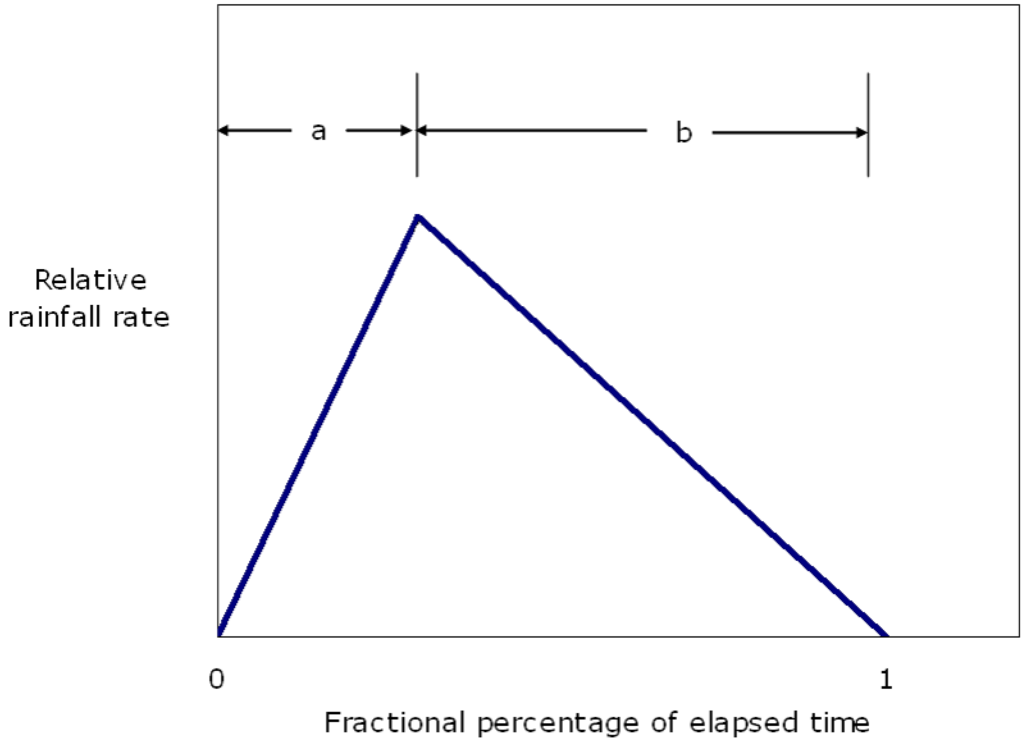

Figure 4-17. Triangular dimensionless Texas hyetograph

Table 4-13: Triangular Model Parameters a and b

Triangular hyetograph model parameters	Storm duration		
	5-12 hours	**13-24 hours**	**25-72 hours**
a	0.02197	0.28936	0.38959
b	0.97803	0.71064	0.61041

Based on the storm duration, the designer selects the appropriate parameters a and b for use in Equations 4-26 and 4-27. The ordinates of cumulative storm depth, normalized to total storm depth, are thus obtained. Values of rainfall intensity are obtained by computing total storm depth for durations of interest, and dividing by the duration.

Triangular Dimensionless Hyetograph Procedure

The following is an example computation using the triangular dimensionless hyetograph procedure for a 12-hour storm with cumulative depth of 8 inches:

1. Express F in Equations 4-26 and 4-27 in terms of time t and total storm duration T: F = t / T.
2. Express p in terms of cumulative rainfall depth d and total storm depth D: p = d / D.
3. Substituting into Equations 4-26 and 4-27 gives:

$$p_1 = \frac{d}{D}\left(0 \le \frac{t}{T} \le a\right) = \frac{1}{a}\left(\frac{t}{T}\right)^2$$

$$p_2 = \frac{d}{D}\left(a < \frac{t}{T} \le 1\right) = -\frac{1}{b}F^2 + \left(\frac{2a}{b} + 2\right)F - \left(\frac{a^2}{b} + a\right)$$

4. From Table 4-13, a = 0.02197 and b = 0.97803.

$$p_1 = \frac{d}{D}\left(0 \le \frac{t}{T} \le 0.02197\right) = \frac{1}{0.02197}\left(\frac{t}{T}\right)^2$$

$$p_2 = \frac{d}{D}\left(0.02197 < \frac{t}{T} \le 1\right) = -\frac{1}{0.97803}\left(\frac{t}{T}\right)^2 + \left(\frac{2(0.02197)}{0.97803} + 2\right)\left(\frac{t}{T}\right) - \left(\frac{0.02197^2}{0.97803} + 0.02197\right)$$

5. Substituting 12 (hours) for T and 8 (inches) for D gives:

$$p_1 = \frac{d}{8}\left(0 \le \frac{t}{12} \le 0.02197\right) = \frac{1}{0.02197}\left(\frac{t}{12}\right)^2$$

$$p_2 = \frac{d}{8}\left((0.02197) < \frac{t}{12} \le 1\right) = -\frac{1}{0.97803}\left(\frac{t}{12}\right)^2 + \left(\frac{2(0.02197)}{0.97803} + 2\right)\left(\frac{t}{12}\right) - \left(\frac{0.02197^2}{0.97803} + 0.02197\right)$$

6. Simplifying:

$$p_1 = d(0 \le t \le 0.26364) = \frac{8}{0.02197}\left(\frac{t}{12}\right)^2$$

$$p_2 = d(0.26364 < t \le 12) = \frac{8}{0.97803}\left(\frac{t}{12}\right)^2 + 8\left(\frac{2(0.02197)}{0.97803} + 2\right)\left(\frac{t}{12}\right) - 8\left(\frac{0.02197^2}{0.97803} + 0.02197\right)$$

These resulting equations provide cumulative depth in inches as a function of elapsed time in hours, as shown in Table 4-14.

Table 4-14: Example Dimensionless Hyetograph Ordinates

Time, t (hr.)	Precipitation Depth, d (in.)	Precipitation Intensity, I (in./hr.)
0	0	0
0.13	0.04	0.33
0.26	0.17	0.99
0.50	0.49	1.32
0.75	0.81	1.29

Table 4-14: Example Dimensionless Hyetograph Ordinates

Time, t (hr.)	Precipitation Depth, d (in.)	Precipitation Intensity, I (in./hr.)
1.00	1.13	1.26
2.00	2.32	1.19
3.00	3.40	1.08
4.00	4.36	0.97
5.00	5.22	0.85
6.00	5.96	0.74
7.00	6.58	0.62
8.00	7.09	0.51
9.00	7.49	0.40
10.00	7.77	0.28
11.00	7.94	0.17
12.00	8.00	0.06

L-gamma Dimensionless Hyetograph

Asquith (2003) and Asquith et al. (2005) computed sample L-moments of 1,659 dimensionless hyetographs for runoff-producing storms. Storms were divided by duration into 3 categories, 0 to 12 hours, 12 to 24 hours, and 24 to 72 hours. Dimensionless hyetographs based on the L-gamma distribution were developed and are defined by:

$$p(F) = F^{b} e^{c(1-F)}$$

Equation 4-28.

Where:

e = 2.718282

p = normalized cumulative rainfall depth, ranging from 0 to 1

F = elapsed time, relative to storm duration, ranging from 0 to 1

b = distribution parameter from Table 4-15

c = distribution parameter from Table 4-15

Parameters b and c of the L-gamma distribution for the corresponding storm durations are shown in Table 4-15. Until specific guidance is developed for selecting parameters for storms of exactly 12 hours and 24 hours, the designer should adopt distribution parameters for the duration range resulting in the more severe runoff condition.

Table 4-15: Estimated L-Gamma Distribution Parameters b and c

Storm duration	L-gamma distribution parameters	
	b	c
0 – 12 hours	1.262	1.227
12 – 24 hours	0.783	0.4368
24 - 72 hours	0.3388	-0.8152

L-gamma Dimensionless Hyetograph Procedure

Use the following steps to develop an L-gamma dimensionless Texas hyetograph for storm duration of 24 hours and a storm depth of 15 inches:

1. Enter the L-gamma distribution parameters for the selected storm duration into the following equation:

$$p(F) = F^{0.783} e^{0.4368(1-F)}$$

2. Express F in terms of time t and total storm duration T: F = t / T. Express p in terms of cumulative rainfall depth d and total storm depth D: p = d / D. Substituting gives:

$$d = D\left(\frac{t}{T}\right)^{0.783} e^{0.4368(1-t/T)}$$

3. Substitute 24 (hours) for T and 15 (inches) for D:

$$d = 15\left(\frac{t}{24}\right)^{0.783} e^{0.4368(1-t/24)}$$

This equation defines the storm hyetograph. d is the cumulative depth in inches, and t is the elapsed time in hours.

Empirical Dimensionless Hyetograph

Empirical dimensionless hyetographs (Williams-Sether et al. 2004, Asquith et al. 2005) have been developed for application to small drainage areas (less than approximately 160 square miles) in urban and rural areas in Texas. The cumulative hyetographs are dimensionless in both duration and depth, and are applicable for storm durations ranging from 0 to 72 hours. The hyetograph shapes are not given by a mathematical expression but are provided graphically for 1^{st}, 2^{nd}, 3^{rd}, and 4^{th} quartile storms as well as for a combined (1^{st} through 4^{th} quartile) storm.

To use the hyetographs, the designer determines the appropriate storm depth and duration for the annual exceedance probability (AEP) of interest. The quartile defines in which temporal quarter of the storm the majority of the precipitation occurs – the graphs for individual quartiles as well as corresponding tabulations are available in Williams-Sether et al., (2004).

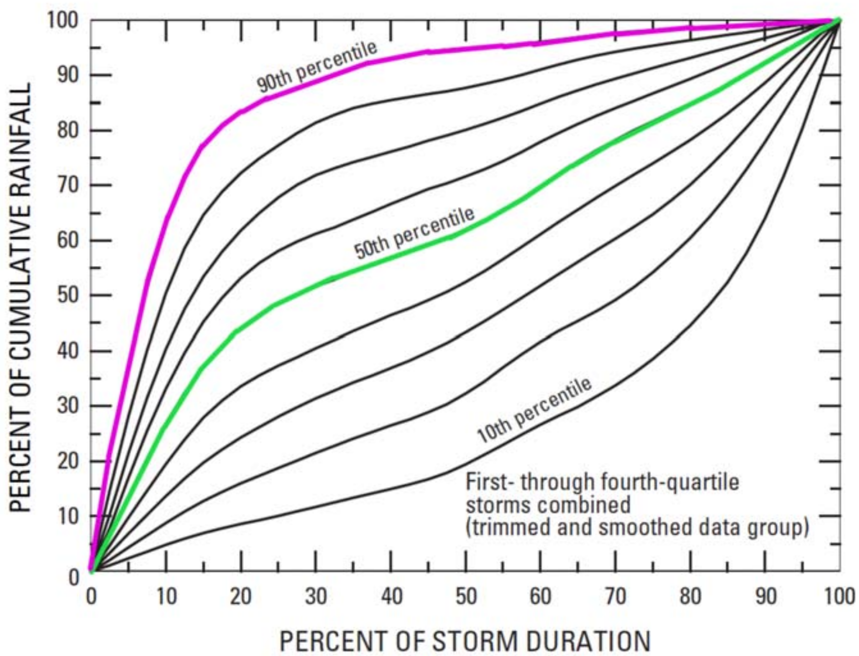

Figure 4-18. Dimensionless hyetographs for 0 to 72 hours storm duration (from Williams-Sether et al. 2004)

Table 4-16: Median (50th) and 90th Percentile, 1st- through 4th- Quartile, Empirical Dimensionless Hyetographs (from Williams-Sether et al. 2004)

Storm duration (%)	50th Percentile Depth (%)	90th Percentile Depth (%)
0.00	0.00	0.00
2.50	8.70	21.60
5.00	13.58	37.57
7.50	20.49	51.55
10.0	26.83	63.04
12.5	32.42	71.66
15.0	37.21	77.38
17.5	41.00	80.89
20.0	44.11	83.32
22.5	46.55	85.01

Table 4-16: Median (50th) and 90th Percentile, 1st- through 4th- Quartile, Empirical Dimensionless Hyetographs (from Williams-Sether et al. 2004)

Storm duration (%)	50th Percentile Depth (%)	90th Percentile Depth (%)
25.0	48.54	86.35
27.5	50.23	87.66
30.0	51.68	88.96
32.5	52.9	90.18
35.0	54.27	91.29
37.5	55.49	92.25
40.0	56.80	93.05
42.5	58.03	93.72
45.0	59.31	94.24
47.5	60.49	94.64
50.0	61.97	94.92
52.5	63.51	95.18
55.0	65.39	95.40
57.5	67.56	95.70
60.0	69.85	96.06
62.5	72.11	96.47
65.0	74.32	96.9
67.5	76.38	97.32
70.0	78.21	97.68
72.5	80.00	97.97
75.0	81.61	98.19
77.5	83.25	98.38
80.0	84.84	98.56
82.5	86.54	98.72
85.0	88.30	98.90
87.5	90.21	99.09
90.0	92.18	99.29
92.5	94.22	99.49

Table 4-16: Median (50th) and 90th Percentile, 1st- through 4th- Quartile, Empirical Dimensionless Hyetographs (from Williams-Sether et al. 2004)

Storm duration (%)	50th Percentile Depth (%)	90th Percentile Depth (%)
95.0	96.21	99.70
97.5	98.21	99.92
100.0	100.00	100.00

Figure 4-18 is a graphical representation of the combined storm with the 50th percentile (green) and 90th percentile (magenta) storm hyetograph highlighted, and Table 4-17 is the corresponding tabulation for a 50th percentile (median) storm and a 90th percentile storm. The recommended 50th percentile curve represents a median combined (1st through 4th quartile) storm. The 90th percentile curve represents an upper support combined (1st through 4th quartile) storm where 90 percent of hyetographs would be anticipated to track either on or below the curve.

Confidence limits for the empirical dimensionless hydrographs have been computed for each of the four quartile hyetographs and are reported in Williams-Sether et al., (2004). Because the hyetographs are dimensionless, all of the percentile hyetographs have the same dimensionless storm depth but represent variations in the temporal distribution of rainfall during the storm duration.

A spreadsheet tool, TXHYETO-2015.xlsx (developed by Cleveland et al., (2015)) is available to facilitate the use of the dimensionless hyetograph. It will assist the designer in producing elapsed time in minutes (or hours) and cumulative depth in inches (or millimeters) for the 50th or 90th percentile hyetograph. A video tutorial for use of the tool is included in Cleveland et al.,(2015). The tool can also be used in conjunction with the EBDLKUP-2015v2.1 spread sheet.

Models for Estimating Losses

Losses refer to the volume of rain falling on a watershed that does not run off. With each model, precipitation loss is found for each computation time interval, and is subtracted from the precipitation depth for that interval. The remaining depth is referred to as precipitation excess. This depth is considered uniformly distributed over a watershed area, so it represents a volume of runoff.

Loss models available to the TxDOT designer include:

- Initial and constant-rate loss model.
- Texas initial and constant-rate loss model.
- NRCS curve number loss model.
- Green and Ampt loss model.

Initial and Constant-Rate Loss Model Basic Concepts and Equations

For the initial loss and constant–rate loss model, no runoff occurs in the watershed until an initial loss capacity has been satisfied, regardless of the rainfall rate. Once the initial loss has been satisfied, a constant potential loss rate occurs for the duration of the storm. This method is a simple approximation of a typical infiltration curve, where the initial loss decays over the storm duration to a final near-constant loss rate. In the example in Figure 4-19, the initial loss is satisfied in the first time increment, and the constant rate accounts for losses thereafter.

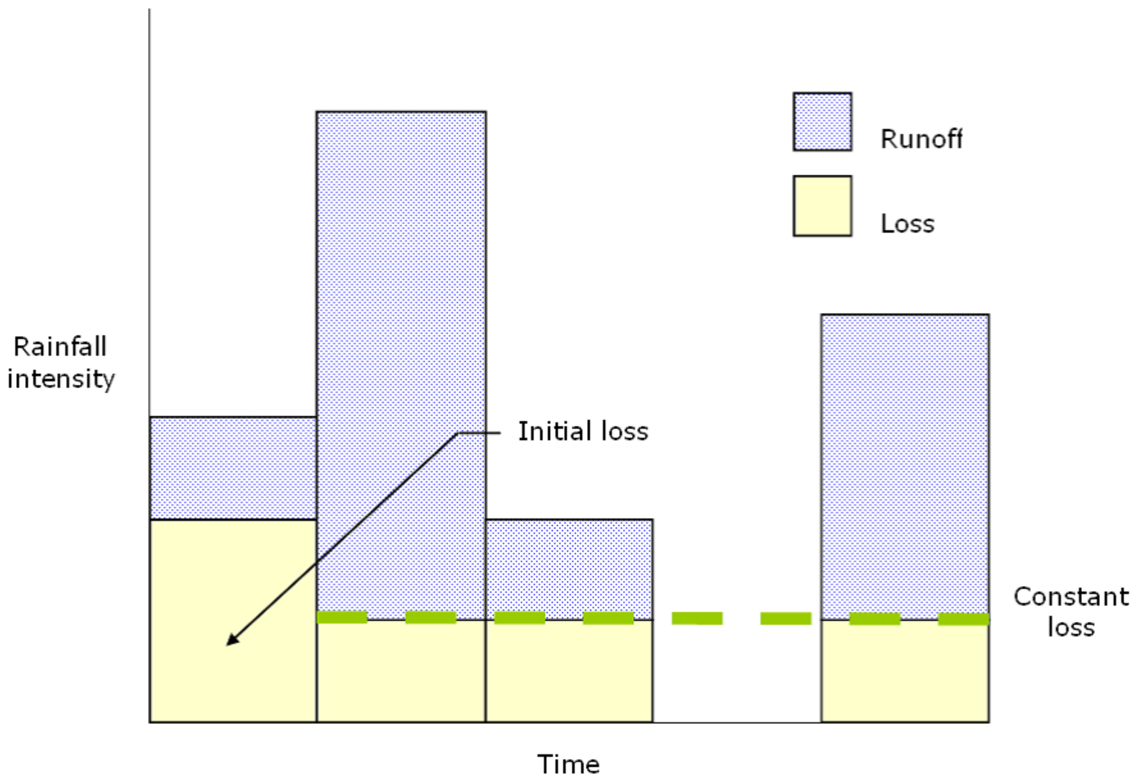

Figure 4-19. Initial and constant-loss rate model

The initial and constant loss-rate model is described mathematically as:

$$f(t) = I(t) \qquad for \qquad P(t) < I_a$$

Equation 4-29.

$$f(t) = I(t) - L \qquad for \qquad I(t) > L, P(t) \geq I_a$$

Equation 4-30.

$$f(t) = I(t) \qquad for \qquad I(t) \leq L$$

Equation 4-31.

Where:

I(t) = rainfall intensity (in./hr.)

f(t) = loss rate (in./hr.)

$P(t)$ = cumulative rainfall depth (in.) at time t

I_a = initial loss (in.)

L = constant loss rate (in./hr.)

I_a accounts for interception and depression storage, and the initial rate of infiltration at the beginning of the storm event. Interception refers to the capture of rainfall on the leaves and stems of vegetation before it reaches the ground surface. Depression storage is where the ponded rainfall fills small depressions and irregularities in the ground surface. Depression storage eventually infiltrates or evaporates during dry-weather periods. Until the accumulated precipitation on the pervious area exceeds the initial loss volume, no runoff occurs.

Estimating Initial Loss and Constant Rate

The initial and constant-rate loss model includes one parameter (the constant rate) and one initial condition (the initial loss). Respectively, these represent physical properties of the watershed soils and land use and the antecedent condition.

If the watershed is in a saturated state, I_a will approach 0. If the watershed is dry, then I_a will increase to represent the maximum precipitation depth that can fall on the watershed with no runoff; this will depend on the watershed terrain, land use, soil types, and soil treatment.

The constant loss rate can be viewed as the ultimate infiltration capacity of the soils. The NRCS classified soils on the basis of this infiltration capacity as presented in Table 4-17; values in Column 4 represent reasonable estimates of the rates.

Texas Initial and Constant-Rate Loss Model

Recent research (TxDOT 0-4193-7) developed four computational approaches for estimating initial abstraction (I_A) and constant loss (C_L) values for watersheds in Texas. The approaches are all based on the analysis of rainfall and runoff data of 92 gauged watersheds in Texas. One of those methods, presented here, allows the designer to compute I_A and C_L from regression equations:

$$I_A = 2.045 - 0.5497(L)^{-0.9041} - 0.1943(D) + 0.2414(R) - 0.01354(CN)$$

Equation 4-32.

$$C_L = 2.535 - 0.4820(L)^{0.2312} + 0.2271(R) - 0.01676(CN)$$

Equation 4-33.

Where:

I_A = initial abstraction (in.)

C_L = constant loss rate (in./hr.)

L = main channel length (mi.)

D = 0 for undeveloped watersheds, 1 for developed watersheds

R = 0 for non-rocky watersheds, 1 for rocky watersheds

CN = NRCS curve number

In the above equations, L is defined as “the length in stream-course miles of the longest defined channel shown in a 30-meter digital elevation model from the approximate watershed headwaters to the outlet” (TxDOT 0-4193-7).

NRCS Curve Number Loss Model

NRCS has developed a procedure to divide total depth of rainfall into soil retention, initial abstractions, and effective rainfall. This parameter is referred to as a curve number (CN). The CN is based on soil type, land use, and vegetative cover of the watershed. The maximum possible soil retention is estimated using a parameter that represents the impermeability of the land in a watershed. Theoretically, CN can range from 0 (100% rainfall infiltration) to 100 (impervious). In practice, based on values tabulated in NRCS 1986, the lowest CN the designer will likely encounter is 30, and the maximum CN is 98.

The CN may also be adjusted to account for wet or dry antecedent moisture conditions. Dry soil conditions are referred to as CN I, average conditions (those calculated using Estimating the CN) are referred to as CN II, and wet soils are referred to as CN III. Antecedent moisture conditions should be estimated considering a minimum of a five-day period. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

$$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)}$$

Equation 4-34.

$$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)}$$

Equation 4-35.

Hydrologic Soil Groups

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. NRCS divides soils into four hydrologic soil groups based on infiltration rates (Groups A-D). Urbanization has an effect on soil groups, as well. See Table 4-17 for more information.

Table 4-17: Hydrologic Soil Groups

Soil group	Description	Soil type	Range of loss rates (in./hr.)	Range of loss rates (mm/hr.)
A	Low runoff potential due to high infiltration rates even when saturated	Deep sand, deep loess, aggregated silts	0.30-0.45	7.6-11.4
B	Moderately low runoff potential due to moderate infiltration rates when saturated	Shallow loess, sandy loam	0.15-0.30	3.8-7.6
C	Moderately high runoff potential due to slow infiltration rates Soils in which a layer near the surface impedes the downward movement of water or soils with moderately fine to fine texture	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05-0.15	1.3-3.8
D	High runoff potential due to very slow infiltration rates	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00-0.05	1.3

Estimating the CN

Rainfall infiltration losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff CNs. Suggested runoff curve numbers are provided in Table 4-18, Table 4-19, Table 4-20, and Table 4-21. Note that CNs are whole numbers.

For a watershed that has variability in land cover and soil type, a composite CN is calculated and weighted by area.

Table 4-18: Runoff Curve Numbers For Urban Areas

Cover type and hydrologic condition	Average percent impervious area	A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.):					
Notes: Values are for average runoff condition, and $I_a = 0.2S$. The average percent impervious area shown was used to develop the composite CNs. Other assumptions are: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.					

Table 4-18: Runoff Curve Numbers For Urban Areas

Cover type and hydrologic condition	Average percent impervious area	A	B	C	D
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-in. sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (townhouses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas: Newly graded areas (pervious area only, no vegetation)		77	86	91	94
Notes: Values are for average runoff condition, and $I_a = 0.2S$. The average percent impervious area shown was used to develop the composite CNs. Other assumptions are: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.					

Table 4-19: Runoff Curve Numbers For Cultivated Agricultural Land

Cover type	Treatment	Hydrologic condition	A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

Notes: Values are for average runoff condition, and $I_a = 0.2S$. Crop residue cover applies only if residue is on at least 5% of the surface throughout the year. Hydrologic condition is based on a combination of factors affecting infiltration and runoff: density and canopy of vegetative areas, amount of year-round cover, amount of grass or closed-seeded legumes in rotations, percent of residue cover on land surface (good > 20%), and degree of roughness. Poor = Factors impair infiltration and tend to increase runoff. Good = Factors encourage average and better infiltration and tend to decrease runoff.

Table 4-20: Runoff Curve Numbers For Other Agricultural Lands

Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow – continuous grass, protected from grazing and generally mowed for hay	-	30	58	71	78
Brush – brush-weed-grass mixture, with brush the major element	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods – grass combination (orchard or tree farm)	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads – buildings, lanes, driveways, and surrounding lots	-	59	74	82	86
Notes: Values are for average runoff condition, and $I_a = 0.2S$. Pasture: Poor is < 50% ground cover or heavily grazed with no mulch, Fair is 50% to 75% ground cover and not heavily grazed, and Good is > 75% ground cover and lightly or only occasionally grazed. Meadow: Poor is < 50% ground cover, Fair is 50% to 75% ground cover, Good is > 75% ground cover. Woods/grass: CNs shown were computed for areas with 50 percent grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture. Woods: Poor = forest litter, small trees, and brush destroyed by heavy grazing or regular burning. Fair = woods grazed but not burned and with some forest litter covering the soil. Good = woods protected from grazing and with litter and brush adequately covering soil.					

Table 4-21: Runoff Curve Numbers For Arid And Semi-arid Rangelands

Cover type	Hydrologic condition	A	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Saltbush, greasewood, creosote-bush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84
Notes: Values are for average runoff condition, and $I_a = 0.2S$. Hydrologic Condition: Poor = < 30% ground cover (litter, grass, and brush overstory), Fair = 30% to 70% ground cover, Good = > 70% ground cover. Curve numbers for Group A have been developed only for desert shrub.					

Soil Retention

The potential maximum retention (S) is calculated as:

$$S = z\left(\frac{100}{CN} - 1\right)$$

Equation 4-36.

Where:

z = 10 for English measurement units, or 254 for metric

CN = runoff curve number

Equation 4-36 is valid if S is less than the rainfall excess, defined as precipitation (P) minus runoff (R) or S < (P-R). This equation was developed mainly for small watersheds from recorded storm data that included total rainfall amount in a calendar day but not its distribution with respect to time. Therefore, this method is appropriate for estimating direct runoff from 24-hour or 1-day storm rainfall.

Initial Abstraction

The initial abstraction consists of interception by vegetation, infiltration during early parts of the storm, and surface depression storage.

Generally, I_a is estimated as:

$$I_a = 0.2S$$

Equation 4-37.

Effective Rainfall Runoff Volume

The effective rainfall (or the total rainfall minus the initial abstractions and retention) used for run-off hydrograph computations can be estimated using:

$$P_e = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Equation 4-38.

Where:

P_e = accumulated excess rainfall (in.)

I_a = initial abstraction before ponding (in.)

P = total depth of rainfall (in.)

S = potential maximum depth of water retained in the watershed (in.)

Substituting Equation 4-37, Equation 4-38 becomes:

$$P_e = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Equation 4-39.

P_e and P have units of depth, P_e and P reflect volumes and are often referred to as volumes because it is usually assumed that the same depths occurred over the entire watershed. Therefore P_e is considered the volume of direct runoff per unit area, i.e., the rainfall that is neither retained on the surface nor infiltrated into the soil. P_e also can be applied sequentially during a storm to compute incremental precipitation for selected time interval Δt.

Climatic Adjustment of CN

NRCS curve numbers, estimated (predicted) using the procedure described in **Estimating the CN**, may be adjusted to account for the variation of climate within Texas. The adjustment is applied as follows:

$$CN_{obs} = CN_{pred} + CN_{dev}$$

Equation 4-40.

Where:

CN_{obs} = CN adjusted for climate

CN_{pred} = Estimated CN from NRCS procedures described in **Estimating the CN**

CN_{dev} = Deviation of CN_{obs} from CN_{pred} = climatic adjustment factor

In two studies (Hailey and McGill 1983, Thompson et al. 2003) CN_{dev} was computed for gauged watersheds in Texas as CN_{obs} - CN_{pred} based on historical rainfall and runoff volumes. These studies show that CN_{dev} varies by location within the state.

The following excerpt (Thompson et al. 2003) guides the designer in selection and application of the appropriate climatic adjustment to the predicted CN.

Given the differences between CN_{obs} and CN_{pred}, it is possible to construct a general adjustment to CN_{pred} such that an approximation of CN_{obs} can be obtained. The large amount of variation in CN_{obs} does not lend to smooth contours or function fits. There is simply an insufficient amount of information for these types of approaches. However, a general adjustment can be implemented using regions with a general adjustment factor. Such an approach was taken and is presented in Figure 4-20.

The bulk of rainfall and runoff data available for study were measured near the I-35 corridor. Therefore, estimates for this region are the most reliable. The greater the distance from the majority of the watershed that were part of this study, then the more uncertainty must be implied about the results. For the south high plains, that area south of the Balcones escarpment, and the coastal plain, there was insufficient data to make any general conclusions.

Application of the tool is straightforward. For areas where adjustment factors are defined (see Figure 4-20) the analyst should:

- Determine CN_{pred} using the normal NRCS procedure.
- Find the location of the watershed on the design aid (Figure 4-21). Determine an adjustment factor from the design aid and adjust the curve number.
- Examine Figure 4-21 and find the location of the watershed. Use the location of the watershed to determine nearby study watersheds. Then refer to Figure 4-20 and Table 4-22, Table 4-23, Table 4-24, Table 4-25, and Table 4-26 and determine CN_{pred} and CN_{obs} for study watersheds near the site in question, if any are near the watershed in question.
- Compare the adjusted curve number with local values of CN_{obs}.

The result should be a range of values that are reasonable for the particular site.

As a comparison, the adjusted curve number from Hailey and McGill (Figure 4-22) can be used.

A lower bound equivalent to the curve number for AMC I (dry antecedent conditions), or a curve number of 60, whichever is greater, should be considered.

Note that CN values are whole numbers. Rounding of values of CN_{pred} in the tables may be required.

Judgment is required for application of any hydrologic tool. The adjustments presented on Figure 4-20 are no exception. A lower limit of AMC I may be used to prevent an overadjustment downward. For areas that have few study watersheds, the Hailey and McGill approach should provide some guidance on the amount of reduction to CN_{pred} is appropriate, if any.

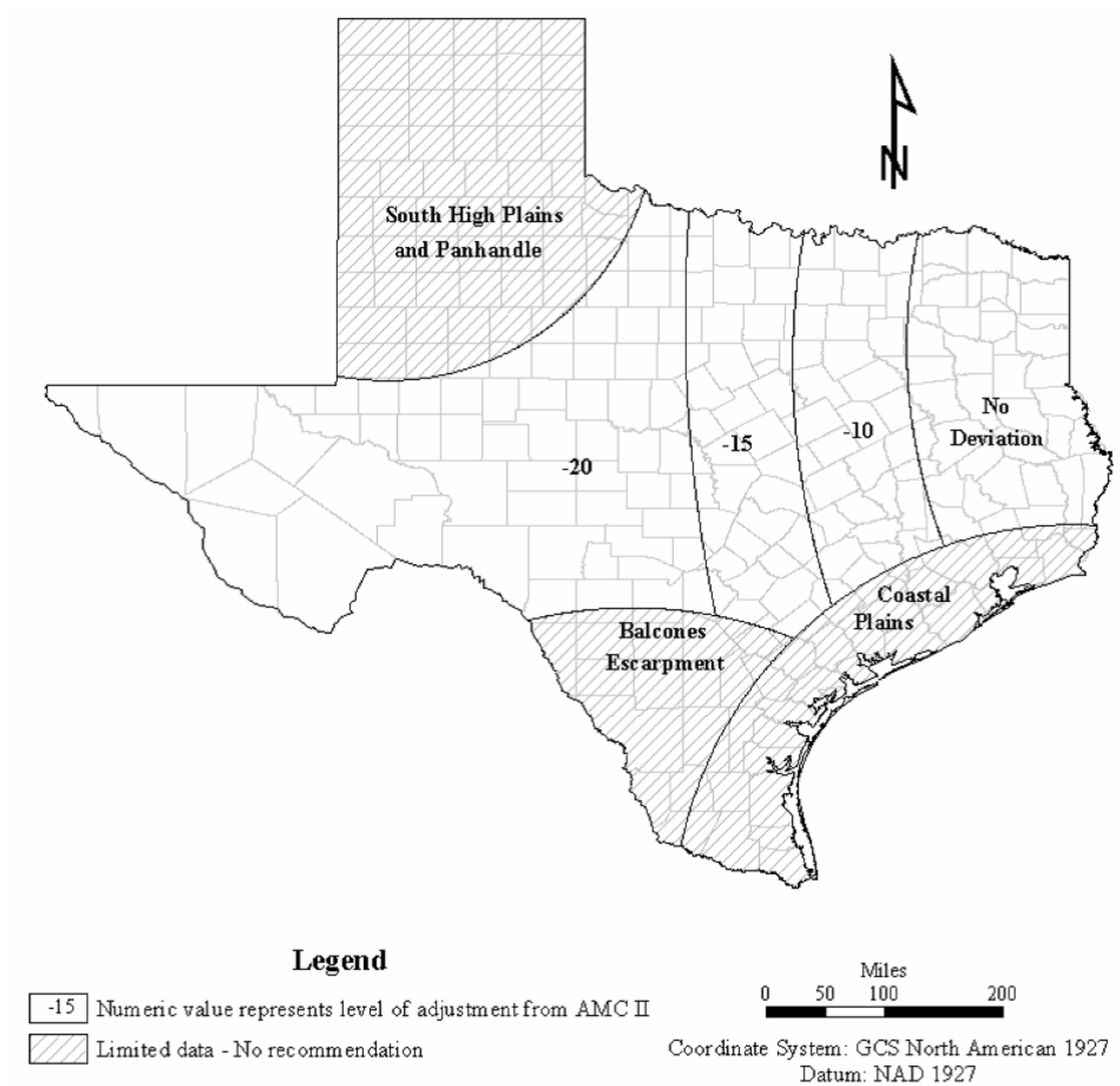

Figure 4-20. Climatic adjustment factor CN_{dev}

Figure 4-21. Location of CNdev watersheds

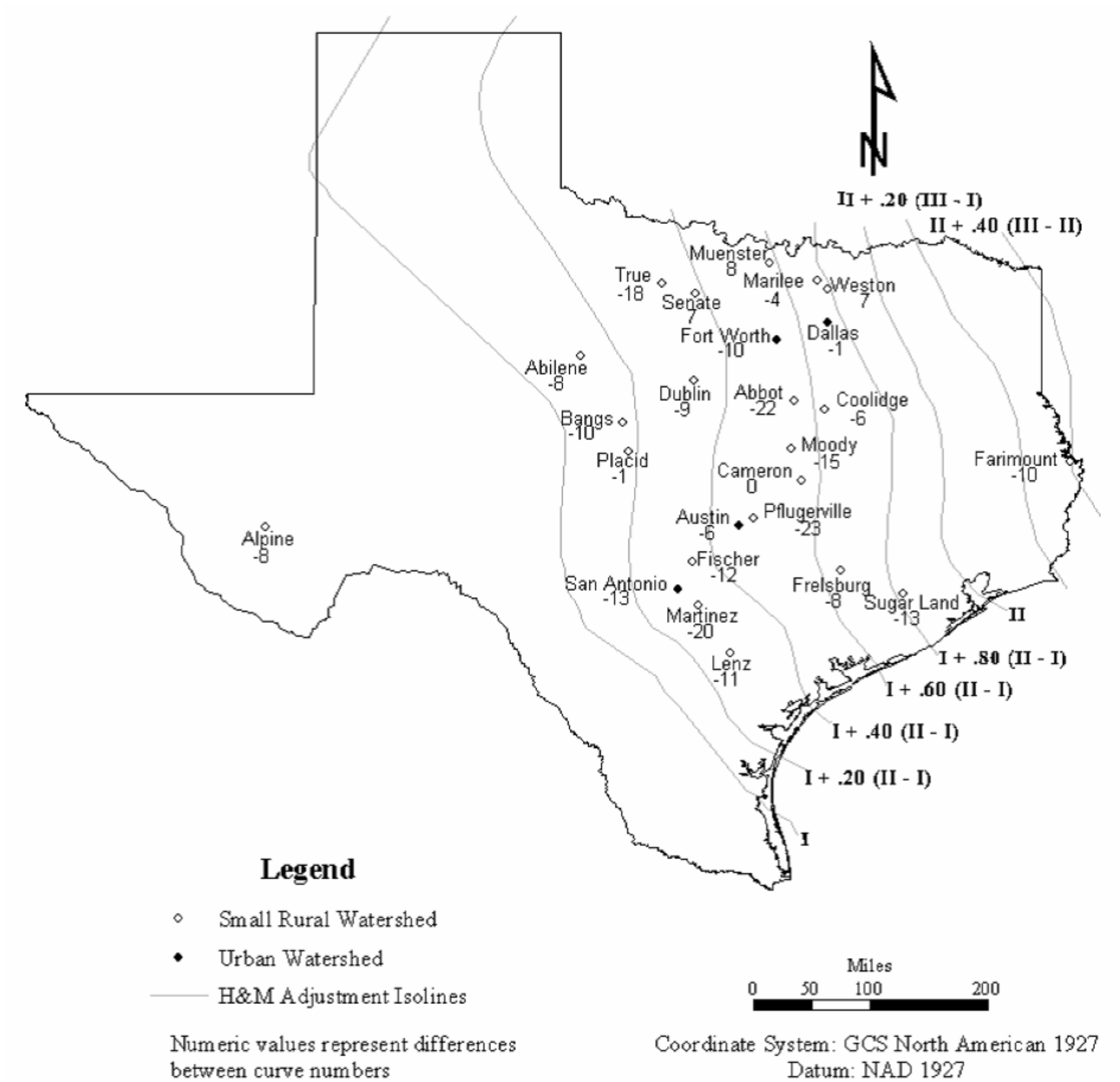

Figure 4-22. Climatic adjustment of CN - comparison of Hailey and McGill adjusted curve numbers, CNH&M, with CNobs. Negative differences indicate that CNH&M is larger than CNobs. Also shown are the lines of equal adjustment to curve number from Hailey and McGill's (1983) Figure 4.

Table 4-22: CN_{obs}, CN_{pred}, and CN_{dev} for the Austin region

USGS Gauge ID	Quad Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8154700	Austin West	59	68.9	-9.9
8155200	Bee Cave	65	70.7	-5.7
8155300	Oak Hill	64	69.8	-5.8
8155550	Austin West	50	87.3	-37.3

Table 4-22: CN_{obs}, CN_{pred}, and CN_{dev} for the Austin region

USGS Gauge ID	Quad Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8156650	Austin East	60	83.6	-23.6
8156700	Austin East	78	86.6	-8.6
8156750	Austin East	66	86.8	-20.8
8156800	Austin East	66	87	-21
8157000	Austin East	68	88.3	-20.3
8157500	Austin East	67	89.1	-22.1
8158050	Austin East	71	83.9	-12.9
8158100	Pflugerville West	60	72.6	-12.6
8158200	Austin East	62	75.6	-13.6
8158400	Austin East	79	88.9	-9.9
8158500	Austin East	71	85.6	-14.6
8158600	Austin East	73	76.7	-3.7
8158700	Driftwood	69	74.5	-5.5
8158800	Buda	64	73.3	-9.3
8158810	Signal Hill	64	69.8	-5.8
8158820	Oak Hill	60	67.9	-7.9
8158825	Oak Hill	49	67.2	-18.2
8158840	Signal Hill	74	69.8	4.2
8158860	Oak Hill	60	68	-8
8158880	Oak Hill	67	79.4	-12.4
8158920	Oak Hill	71	77.5	-6.5
8158930	Oak Hill	56	75.2	-19.2
8158970	Montopolis	56	77.7	-21.7
8159150	Pflugerville East	63	78.8	-15.8

Table 4-23: CN_{obs}, CN_{pred}, and CN_{dev} for the Dallas Region

USGS Gauge ID	Quad Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8055580	Garland	85	85.2	-0.2
8055600	Dallas	82	86.1	-4.1

Table 4-23: CN_{obs}, CN_{pred}, and CN_{dev} for the Dallas Region

USGS Gauge ID	Quad Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8055700	Dallas	73	85.5	-12.5
8056500	Dallas	85	85.8	-0.8
8057020	Dallas	75	85.5	-10.5
8057050	Oak Cliff	75	85.7	-10.7
8057120	Addison	77	80.2	-3.2
8057130	Addison	89	82.9	6.1
8057140	Addison	78	86.8	-8.8
8057160	Addison	80	90.3	-10.3
8057320	White Rock Lake	85	85.7	-0.7
8057415	Hutchins	73	87.8	-14.8
8057418	Oak Cliff	85	79.1	5.9
8057420	Oak Cliff	80	81	-1
8057425	Oak Cliff	90	82.9	7.1
8057435	Oak Cliff	82	81.1	0.9
8057440	Hutchins	67	79.1	-12.1
8057445	Hutchins	60	86.5	-26.5
8061620	Garland	82	85	-3
8061920	Mesquite	85	86	-1
8061950	Seagoville	82	85.3	-3.3

Table 4-24: CN_{obs}, CN_{pred}, and CN_{dev} for the Fort Worth Region

Gauge ID	Quad Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8048520	Fort Worth	72	82.3	-10.3
8048530	Fort Worth	69	86.7	-17.7
8048540	Covington	73	88	-15
8048550	Haltom City	74	91.2	-17.2
8048600	Haltom City	65	84.3	-19.3
8048820	Haltom City	67	83.4	-16.4
8048850	Haltom City	72	83	-11

Table 4-25: CN_{obs}, CN_{pred}, and CN_{dev} for the San Antonio Region

USGS Gauge ID	Quad Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8177600	Castle Hills	70	84.8	-14.8
8178300	San Antonio West	72	85.7	-13.7
8178555	Southton	75	84.2	-9.2
8178600	Camp Bullis	60	79.7	-19.7
8178640	Longhorn	56	78.4	-22.4
8178645	Longhorn	59	78.2	-19.2
8178690	Longhorn	78	84.4	-6.4
8178736	San Antonio East	74	92.3	-18.3
8181000	Helotes	50	79.2	-29.2
8181400	Helotes	56	79.8	-23.8
8181450	San Antonio West	60	87.3	-27.3

Table 4-26: CN_{obs}, CN_{pred}, and CN_{dev} for the Small Rural Watersheds

USGS Gauge ID	Quadrangle Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8025307	Fairmount	53	55.4	-2.4
8083420	Abilene East	65	84.7	-19.7
8088100	True	60	85.9	-25.9
8093400	Abbott	61	88.1	-27.1
8116400	Sugarland	70	82.9	-12.9
8159150	Pflugerville East	55	83.7	-28.7
8160800	Freisburg	56	67.8	-11.8
8167600	Fischer	51	74.3	-23.3
8436520	Alpine South	64	86.4	-22.4
8435660	Alpine South	48	86.7	-38.7
8098300	Rosebud	88	80.5	7.5
8108200	Yarrelton	77	79.9	-2.9
8096800	Bruceville	62	80	-18
8094000	Bunyan	60	78.4	-18.4
8136900	Bangs West	51	75.8	-24.8

Table 4-26: CN_{obs}, CN_{pred}, and CN_{dev} for the Small Rural Watersheds

USGS Gauge ID	Quadrangle Sheet Name	CN_{obs}	CN_{pred}	CN_{dev}
8137000	Bangs West	52	74.5	-22.5
8137500	Trickham	53	76.5	-23.5
8139000	Placid	53	74.6	-21.6
8140000	Mercury	63	74.4	-11.4
8182400	Martinez	52	80	-28
8187000	Lenz	53	83.8	-30.8
8187900	Kenedy	63	73.3	-10.3
8050200	Freemound	80	79.6	0.4
8057500	Weston	80	78.2	1.8
8058000	Weston	86	80.1	5.9
8052630	Marilee	80	85.4	-5.4
8052700	Aubrey	74	84.1	-10.1
8042650	Senate	59	63.4	-4.4
8042700	Lynn Creek	50	62.5	-12.5
8042700	Senate	56	62	-6
8042700	Senate	65	55.9	9.1
8063200	Coolidge	70	79.4	-9.4

Green and Ampt Loss Model

Basic Concepts and Equations

The Green and Ampt loss model is based on a theoretical application of Darcy's law. The model, first developed in 1911, has the form:

$$f = K_s + \frac{K_s S_w (\theta_s - \theta_i)}{F}$$

Equation 4-41.

Where:

f = infiltration capacity (in./hr.)

K_s = saturated hydraulic conductivity (permeability) (in./hr.)

S_w = soil water suction (in.)

θ_s = volumetric water content (water volume per unit soil volume) under saturated conditions

θ_i = volumetric moisture content under initial conditions

F = total accumulated infiltration (in.)

The parameters can be related to soil properties.

Assumptions underlying the Green and Ampt model are the following:

- As rain continues to fall and water infiltrates, the wetting front advances at the same rate throughout the groundwater system, which produces a well-defined wetting front.
- The volumetric water contents, θ_s and θ_i, remain constant above and below the wetting front as it advances.
- The soil-water suction immediately below the wetting front remains constant with both time and location as the wetting front advances.

To calculate the infiltration rate at a given time, the cumulative infiltration is calculated using Equation 4-42 and differences computed in successive cumulative values:

$$F = K_s t + S_w(\theta_s - \theta_i)\ln\left(1 + \frac{F}{S_w(\theta_s - \theta_i)}\right)$$

Equation 4-42.

Where:

t = time (hr.)

Equation 4-42 cannot be solved explicitly. Instead, solution by numerical methods is required. Once F is solved for, the infiltration rate, f, can be solved using Equation 4-41. These computations are typically performed by hydrologic computer programs equipped with Green-Ampt computational routines. With these programs, the designer is required to specify θ_s, S_w, and K_s.

Estimating Green-Ampt Parameters

To apply the Green and Ampt loss model, the designer must estimate the volumetric moisture content, θ_s, the wetting front suction head, S_w, and the saturated hydraulic conductivity, K_s. Rawls et al. (1993) provide Green-Ampt parameters for several USDA soil textures as shown in Table 4-27. A range is given for volumetric moisture content in parentheses with typical values for each also listed.

Table 4-27: Green-Ampt Parameters

Soil texture class	Volumetric moisture content under saturated conditions θ_s	Volumetric moisture content under initial conditions θ_i	Wetting front suction head S_w	Saturated hydraulic conductivity K_s
Sand	0.437 (0.374-0.500)	0.417 (0.354-0.480)	1.95	9.28
Loamy sand	0.437 (0.363-0.506)	0.401 (0.329-0.473)	2.41	2.35
Sandy loam	0.453 (0.351-0.555)	0.412 (0.283-0.541)	4.33	0.86
Loam	0.463 (0.375-0.551)	0.434 (0.334-0.534)	3.50	0.52
Silt loam	0.501 (0.420-0.582)	0.486 (0.394-0.578)	6.57	0.27
Sandy clay loam	0.398 (0.332-0.464)	0.330 (0.235-0.425)	8.60	0.12
Clay loam	0.464 (0.409-0.519)	0.309 (0.279-0.501)	8.22	0.08
Silty clay loam	0.471 (0.418-0.524)	0.432 (0.347-0.517)	10.75	0.08
Sandy clay	0.430 (0.370-0.490)	0.321 (0.207-0.435)	9.41	0.05
Silty clay	0.479 (0.425-0.533)	0.423 (0.334-0.512)	11.50	0.04
Clay	0.475 (0.427-0.523)	0.385 (0.269-0.501)	12.45	0.02

Capabilities and Limitations of Loss Models

Selecting a loss model and estimating the model parameters are critical steps in estimating runoff. Some pros and cons of the different alternatives are shown in Table 4-28. These are guidelines and should be used as such. The designer should be familiar with the models and the watershed where applied to determine which loss model is most appropriate.

Table 4-28: Comparison of Different Loss Models, Based on USACE 2000

Model	Pros	Cons
Initial and constant-loss rate	Has been successfully applied in many studies throughout the US. Easy to set up and use. Model only requires a few parameters to explain the variation of runoff parameters.	Difficult to apply to ungauged areas due to lack of direct physical relationship of parameters and watershed properties. Model may be too simple to predict losses within event, even if it does predict total losses well.
Texas initial and constant-loss rate	Developed specifically from Texas watershed data for application to sites in Texas. Method is product of recent and extensive research. Simple to apply.	Method is dependent on NRCS CN. Relatively new method, and not yet widely used.

Table 4-28: Comparison of Different Loss Models, Based on USACE 2000

Model	Pros	Cons
NRCS CN	Simple, predictable, and stable. Relies on only one parameter, which varies as a function of soil group, land use, surface condition, and antecedent moisture condition. Widely accepted and applied throughout the U.S.	Predicted values not in accordance with classical unsaturated flow theory. Infiltration rate will approach zero during a storm of long duration, rather than constant rate as expected. Developed with data from small agricultural watersheds in midwestern US, so applicability elsewhere is uncertain. Default initial abstraction (0.2S) does not depend upon storm characteristics or timing. Thus, if used with design storm, abstraction will be same with 0.5 AEP storm and 0.01 AEP storm. Rainfall intensity not considered.
Green and Ampt	Parameters can be estimated for ungauged watersheds from information about soils.	Not widely used, less experience in professional community.

Rainfall to Runoff Transform

After the design storm hyetograph is defined, and losses are computed and subtracted from rainfall to compute runoff volume, the time distribution and magnitude of runoff is computed with a rainfall to runoff transform.

Two options are described herein for these direct runoff hydrograph computations:

- Unit hydrograph (UH) model. This is an empirical model that relies on scaling a pattern of watershed runoff.
- Kinematic wave model. This is a conceptual model that computes the overland flow hydrograph method with channel routing methods to convert rainfall to runoff and route it to the point of interest.

Unit Hydrograph Method

A unit hydrograph for a watershed is defined as the discharge hydrograph that results from one unit depth of excess rainfall distributed uniformly, spatially and temporally, over a watershed for a duration of one unit of time. The unit depth of excess precipitation is one inch for English units. The unit of time becomes the time step of the analysis, and is selected as short enough to capture the detail of the storm temporal distribution and rising limb of the unit hydrograph.

The unit hydrograph assumes that the rainfall over a given area does not vary in intensity. If rainfall does vary, the watershed must be divided into smaller subbasins and varying rainfall applied with multiple unit hydrographs. The runoff can then be routed from subbasin to subbasin.

For each time step of the analysis, the unit hydrograph ordinates are multiplied by the excess rainfall depth. The resulting time-coincident ordinates from each resulting hydrograph are summed to produce the total runoff hydrograph for the watershed. This process is shown graphically in Figure 4-23. Hydrographs a, b, c, and d are 1-hour unit hydrographs multiplied by the depth of excess rainfall in the individual 1-hour time steps. The total runoff hydrograph resulting from 4 hours of rainfall is the sum of hydrographs a, b, c, and d.

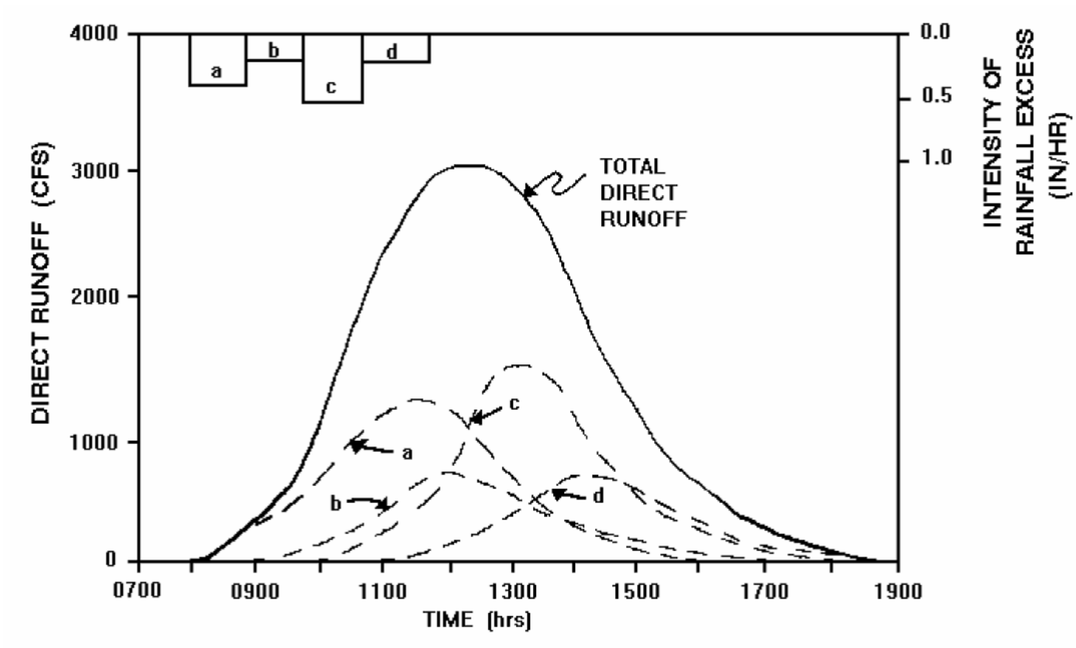

Figure 4-23. Unit hydrograph superposition (USACE 1994)

Mathematically, the computation of the runoff hydrograph is given by:

$$Q_n = \sum_{m=1}^{n \le M} P_m Q_{u(n-m+1)}$$

Equation 4-43.

Where:

n = number of time steps

Q_n = the runoff hydrograph ordinate n (at time nΔt)

P_m = effective rainfall ordinate m (in time interval mΔt)

Δt = computation time interval

$Q_{u\,(n-m+1)}$ = unit hydrograph ordinate (n-m+1) (at time (n-m+1)Δt)

m = number of periods of effective rainfall (of duration Δt)

M = total number of discrete rainfall pulses

Equation 4-43 simplified becomes $Q_1 = P_1U_1$, $Q_2 = P_1U_2+P_2U_1$, $Q_3 = P_1U_3+P_2U_2+P_3U_1$, etc.

Several different unit hydrograph methods are available to the designer. Each defines a temporal flow distribution. The time to peak flow and general shape of the distribution are defined by parameters specific to each method. The choice of unit hydrograph method will depend on available options within the hydrologic software being used, and also the availability of information from which to estimate the unit hydrograph parameters.

Two unit hydrograph methods commonly used by TxDOT designers are Snyder's unit hydrograph and the NRCS unit hydrograph. These methods are supported by many rainfall-runoff software programs, which require the designer only to specify the parameters of the method. These two methods are discussed in the following sections.

Snyder's Unit Hydrograph

Snyder developed a parametric unit hydrograph in 1938, based on research in the Appalachian Highlands using basins 10 to 10,000 square miles. Snyder's unit hydrograph is described with two parameters: C_t, which is a storage or timing coefficient; and C_p, which is a peaking coefficient. As C_t increases, the peak of the unit hydrograph is delayed. As C_p increases, the magnitude of the unit hydrograph peak increases. Both C_t and C_p must be estimated for the watershed of interest. Values for C_p range from 0.4 to 0.8 and generally indicate retention or storage capacity of the watershed.

The peak discharge of the unit hydrograph is given by:

$$Q_p = \frac{640AC_p}{t_L}$$

Equation 4-44.

Where*:*

Q_p = peak discharge (cfs/in.)

A = drainage area (mi^2)

C_p = second coefficient of the Snyder method accounting for flood wave and storage conditions

t_L = time lag (hr.) from the centroid of rainfall excess to peak of hydrograph

t_L is given by:

$$t_L = C_t(LL_{ca})^{0.3}$$

Equation 4-45.

Where*:*

C_t = storage coefficient, usually ranging from 1.8 to 2.2

L = length of main channel (mi)

L_{ca} = length along the main channel from watershed outlet to the watershed centroid (mi)

The duration of excess rainfall (t_d) can be computed using:

$$t_d = \frac{t_L}{5.5}$$

Equation 4-46.

Equation 4-46 implies that the relationship between lag time and the duration of excess rainfall is constant. To adjust values of lag time for other values of rainfall excess duration, the following equation should be used:

$$t_{La} = t_L + 0.25(t_{da} - t_d)$$

Equation 4-47.

***Where**:*

t_{La} = adjusted time lag (hr.)

t_{da} = alternative unit hydrograph duration (hr.)

The time base of the unit hydrograph is a function of the lag time:

$$t_b = 3 + \frac{t_L}{8}$$

Equation 4-48.

***Where**:*

t_b = time base (days)

The time to peak of the unit hydrograph is calculated by:

$$t_p = t_L + \frac{t_d}{2}$$

Equation 4-49.

Empirical relations of Snyder's unit hydrograph were later found to aid the designer in constructing the unit hydrograph (McCuen 1989). The USACE relations, shown in Figure 4-24, are used to construct the Snyder unit hydrograph using the time to peak (t_p), the peak discharge (Q_p), the time base (t_b), and 2 time parameters, W_{50} and W_{75}. W_{50} and W_{75} are the widths of the unit hydrograph at discharges of 50 percent and 75 percent of the peak discharge. The widths are distributed 1/3 before the peak discharge and 2/3 after.

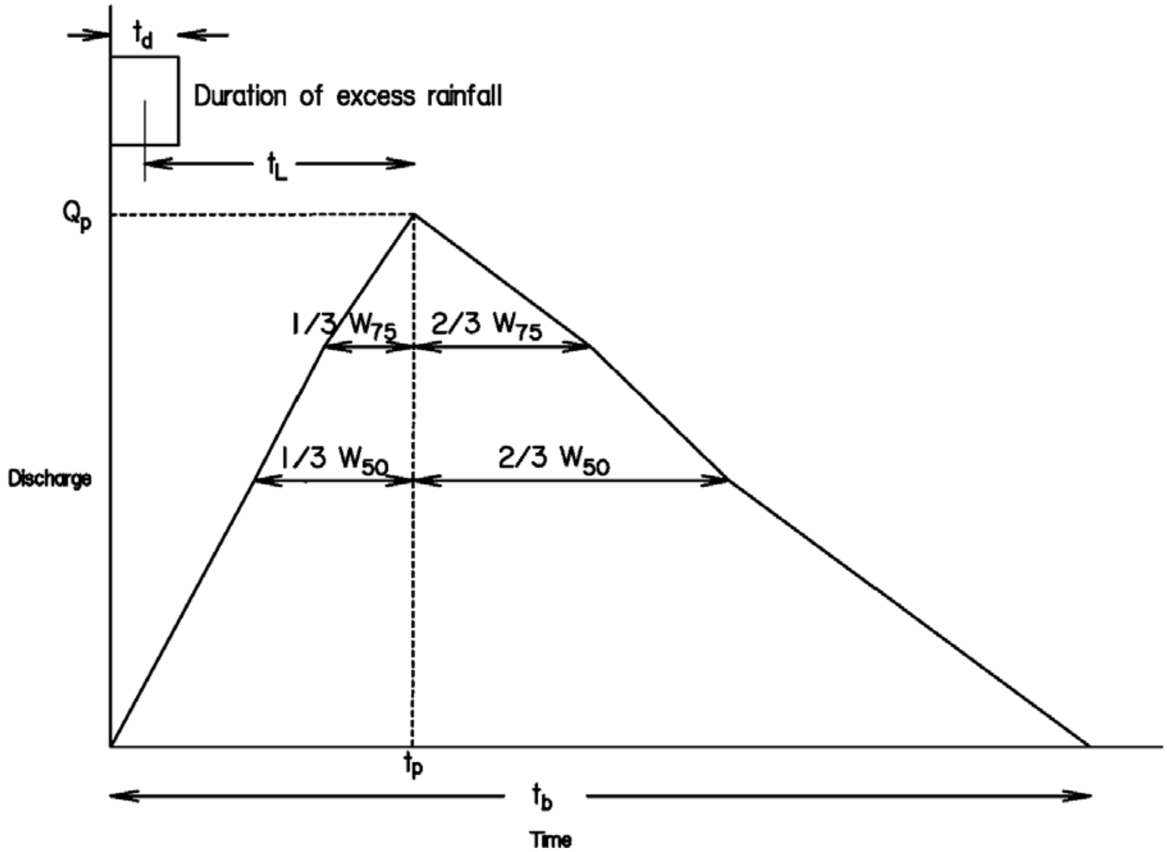

Figure 4-24. Snyder's unit hydrograph

Values for W_{50} and W_{75} are computed using these equations (McCuen 1989):

$$W_{75} = 450 q_a^{-1.081}$$

Equation 4-50.

$$W_{50} = 756 q_a^{-1.081}$$

Equation 4-51.

Where:

q_a = peak discharge per square mile (i.e., Q_p/A, $ft^3/sec/mi^2$)

NRCS Dimensionless Unit Hydrograph

The NRCS unit hydrograph model is based upon an analysis and averaging of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. For convenience, the hydrograph is dimensionless, with discharge ordinates (Q_u) divided by the peak discharge (Q_p) and the time values (t) divided by the time to peak (t_p).

The time-base of the dimensionless unit hydrograph is approximately five times the time to peak, and approximately 3/8 of the total volume occurs before the time to peak. The inflection point on the recession limb occurs at 1.67 times the time to peak, and the hydrograph has a curvilinear shape. The curvilinear hydrograph can be approximated by a triangular hydrograph with similar characteristics.

The curvilinear dimensionless NRCS unit hydrograph is shown in Figure 4-25.

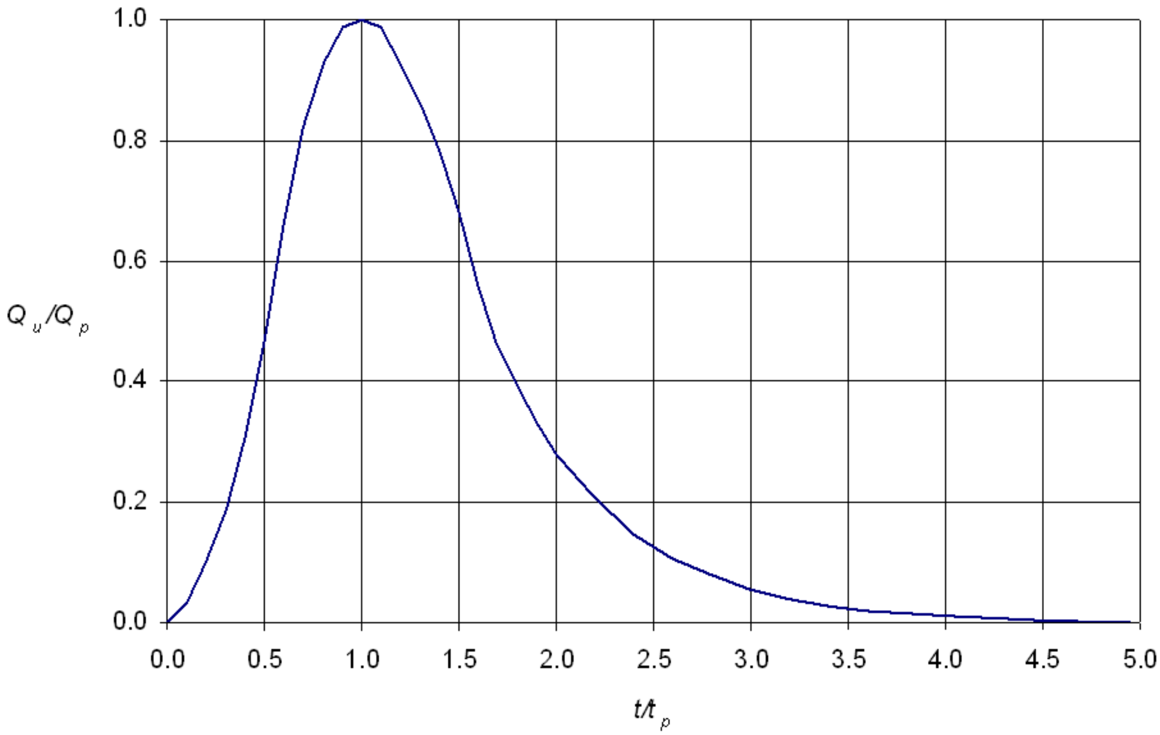

Figure 4-25. NRCS dimensionless unit hydrograph

The ordinates of the dimensionless unit hydrograph are provided in Table 4-29.

Table 4-29: NRCS Dimensionless Unit Hydrograph Ordinates

t/t_p	Q/Q_p
0.0	0.00
0.1	0.03
0.2	0.10
0.3	0.19
0.4	0.31
0.5	0.47
0.6	0.66
0.7	0.82
Notes: Variables are defined as follows: t = time (min.); t_p = time to peak of unit hydrograph (min.); Q = discharge (cfs); and Q_p = peak discharge of unit hydrograph (cfs).	

Table 4-29: NRCS Dimensionless Unit Hydrograph Ordinates

t/t_p	Q/Q_p
0.8	0.93
0.9	0.99
1.0	1.00
1.1	0.99
1.2	0.93
1.3	0.86
1.4	0.78
1.5	0.68
1.6	0.56
1.7	0.46
1.8	0.39
1.9	0.33
2.0	0.28
2.2	0.207
2.4	0.147
2.6	0.107
2.8	0.077
3.0	0.055
3.2	0.04
3.4	0.029
3.6	0.021
3.8	0.015
4.0	0.011
4.5	0.005
5.0	0.00
Notes: Variables are defined as follows: t = time (min.); t_p = time to peak of unit hydrograph (min.); Q = discharge (cfs); and Q_p = peak discharge of unit hydrograph (cfs).	

The following procedure assumes the area or subarea is reasonably homogeneous. That is, the watershed is subdivided into homogeneous areas. The procedure results in a hydrograph only from the direct uncontrolled area. If the watershed has been subdivided, it might be necessary to perform hydrograph channel routing, storage routing, and hydrograph superposition to determine the hydrograph at the outlet of the watershed.

Application of the NRCS dimensionless unit hydrograph to a watershed produces a site-specific unit hydrograph model with which storm runoff can be computed. To do this, the basin lag time must be estimated. The time to peak of the unit hydrograph is related to the lag time by:

$$t_p = \frac{\Delta t}{2} + t_L$$

Equation 4-52.

Where:

t_p = time to peak of unit hydrograph (min.)

t_L = basin lag time (min.)

Dt = the time interval of the unit hydrograph (min.)

This time interval must be the same as the Δt chosen for the design storm.

The time interval may be calculated by:

$$\Delta t = 0.133 t_c$$

Equation 4-53.

And the lag time is calculated by:

$$t_L = 0.6 t_c$$

Equation 4-54.

The peak discharge of the unit hydrograph is calculated by:

$$Q_p = \frac{C_f KA}{t_p}$$

Equation 4-55.

Where:

Q_p = peak discharge (cfs)

C_f = conversion factor (645.33)

K = 0.75 (constant based on geometric shape of dimensionless unit hydrograph)

A = drainage area (mi^2); and

t_p = time to peak (hr.)

Equation 4-55 can be simplified to:

$$Q_p = \frac{484A}{t_p}$$

Equation 4-56.

The constant 484, or peak rate `factor (PRF)`, defines a unit hydrograph with 3/8 of its area under the rising limb. As the watershed slope becomes very steep (mountainous), the constant in Equation 4-56 can approach a value of approximately 600. For flat, swampy areas, the constant may decrease to a value 100 or lower (210-VI-NEH, March 2007). For applications in Texas, the use of the constant 484 is recommended as a starting point, but adjustments to a different value may be warranted. Limited sources are available for guidance on PRF adjustments. One source, while developed for the southeast US, provides practical guidance based on watershed size and slope (Sheridan, 2002). Any adjustments to the PRF must be well documented in the drainage report and model notes.

After t_p and Q_p are estimated using Equations 4-52 and 4-56, the site specific unit hydrograph may be developed by scaling the dimensionless unit hydrograph.

For each time step of the analysis, the site specific unit hydrograph ordinates are multiplied by the excess rainfall depth. The resulting hydrograph are summed to produce the total runoff hydrograph for the watershed. This process is shown graphically in Figure 4-23. While the computations can be completed using a spreadsheet model, a manual convolution can be somewhat time-consuming. These computations are typically performed by hydrologic computer programs.

For example, assume an area of 240 acres (0.375 sq. mi.) with T_c of 1.12 hours and CN of 80. For 1 inch of excess rainfall, Δt = 9 min, t_p = 45 min, and Q_p = 243 cfs, using Equations 4-53, 4-52 and 4-56 respectively.

Column 1 of Table 4-30 shows the time interval of 9 minutes. Column 2 is calculated by dividing the time interval by t_p, in this case 45 minutes. Values in Column 3 are found by using the t/t_p value in Column 2 to find the associated Q_u/Q_p value from the dimensionless unit hydrograph shown in Figure 4-25, interpolating if necessary. Column 4 is calculated by multiplying Column 3 by Q_p, in this case 243 cfs.

Table 4-30: Example Site-specific Unit Hydrograph

t (min.)	t/t_p	Q_u/Q_p	Q_u (cfs)
0	0.00	0.000	0
9	0.20	0.100	24
18	0.40	0.310	75
27	0.60	0.660	160
36	0.80	0.930	226
45	1.00	1.000	243
54	1.20	0.930	226
63	1.40	0.780	190
72	1.60	0.560	136
81	1.80	0.390	95
90	2.00	0.280	68
99	2.20	0.207	50
108	2.40	0.147	36
117	2.60	0.107	26
126	2.80	0.077	19

Table 4-30: Example Site-specific Unit Hydrograph

t (min.)	t/t_p	Q_u/Q_p	Q_u (cfs)
135	3.00	0.055	13
144	3.20	0.040	10
153	3.40	0.023	6
162	3.60	0.021	5
171	3.80	0.015	4
180	4.00	0.011	3
189	4.20	0.009	2
198	4.40	0.006	2
207	4.60	0.004	1
216	4.80	0.002	0
225	5.00	0	0

The example site-specific unit hydrograph is shown in Figure 4-26.

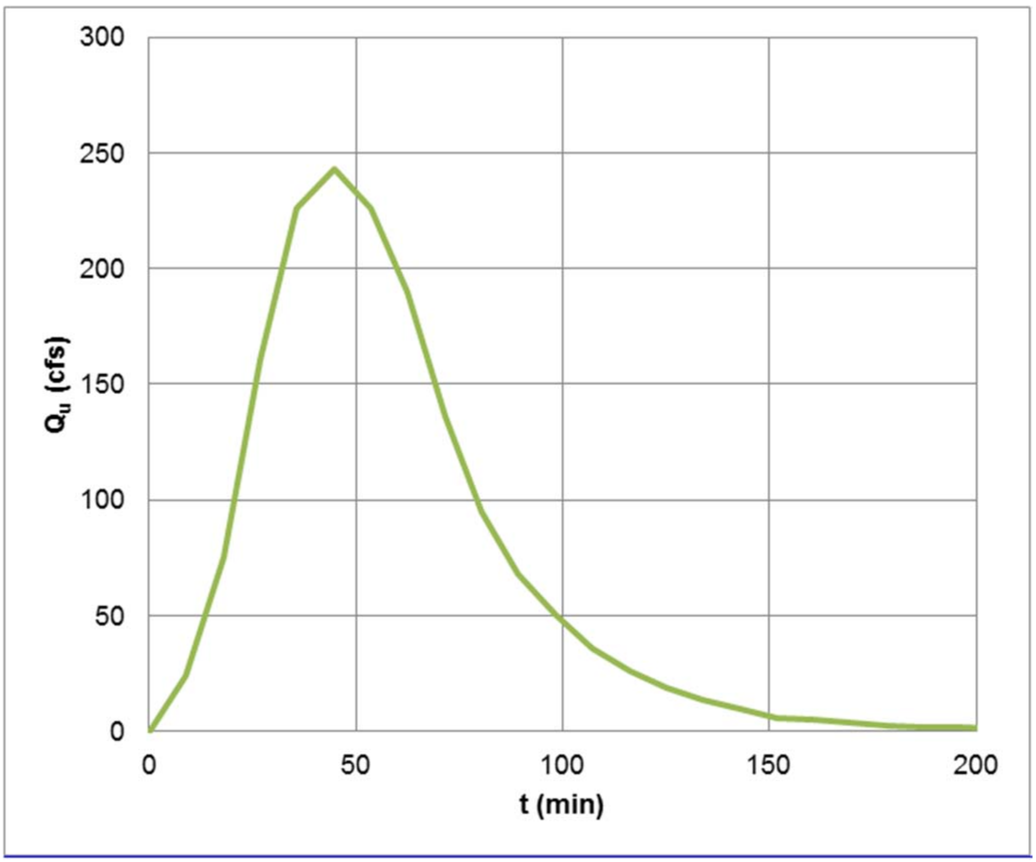

Figure 4-26. Example site-specific unit hydrograph

Remember that the site-specific hydrograph developed in Figure 4-26 was based on 1 inch of excess rainfall. For each time step of the analysis, the unit hydrograph ordinates are multiplied by the excess rainfall depth. Excess rainfall is obtained from a rainfall hyetograph such as a distribution developed from locally observed rainfall or the NRCS 24-hour, Type II or Type III rainfall distributions. The resulting hydrograph are summed to produce the total runoff hydrograph for the watershed. This process is shown graphically in Figure 4-24.

The capabilities and limitations of the NRCS unit hydrograph model include the following:

- This method does not account for variation in rainfall intensity or duration over the watershed.
- Baseflow is accounted for separately.

Kinematic Wave Overland Flow Model

A kinematic wave model is a conceptual model of watershed response that uses laws of conservation of mass and momentum to simulate overland and channelized flows. The model represents the watershed as a wide open channel, with inflow equal to the excess precipitation. Then it simulates unsteady channel flow over the surface to compute the watershed runoff hydrograph. The watershed is represented as a set of overland flow planes and collector channels.

In kinematic wave modeling, the watershed shown in Figure 4-27(a) is represented in Figure 4-27(b) as series of overland flow planes (gray areas) and a collector channel (dashed line). The collector channel conveys flow to the watershed outlet.

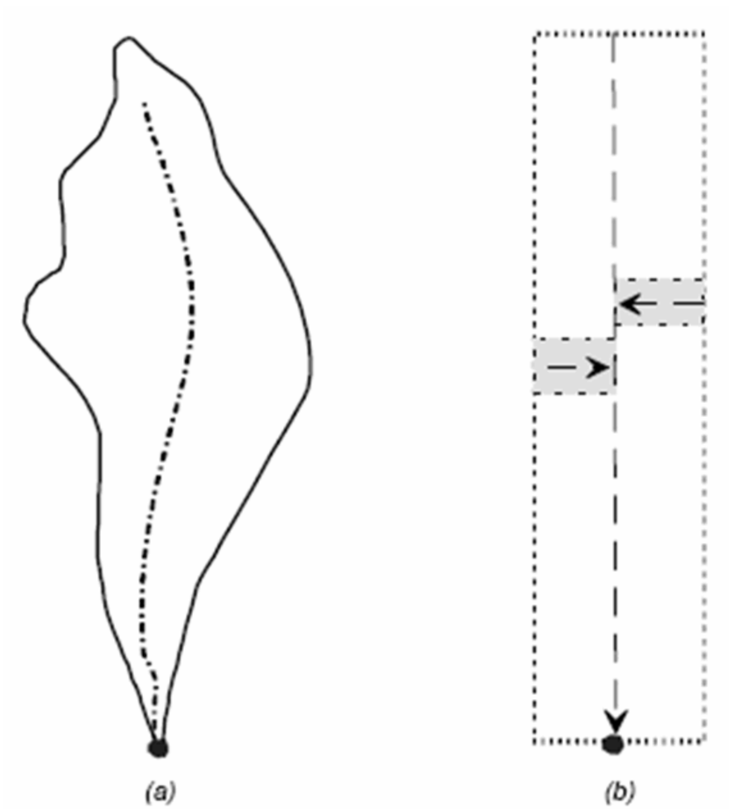

Figure 4-27. Kinematic wave model representation of a watershed (USACE 2000)

The equations used to define conservation of mass and momentum are the Saint Venant equations. The conservation of mass equation is:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_o$$

Equation 4-57.

Where:

A = cross sectional area of flow (ft^2, m^2)

T = time (sec.)

Q = flow rate (cfs, m^3/sec.)

x = distance along the flow path (ft, m)

q_o = lateral discharge added to the flow path per unit length of the flow path (cfs/ft, m^3/sec./m)

The momentum equation energy gradient is approximated by:

$$A = \alpha Q^{\beta}$$

Equation 4-58.

Where:

a and b = coefficients related to the physical properties of the watershed.

Substituting Equation 4-56 into Equation 4-55 yields a single partial differential equation in Q:

$$\alpha Q^{\beta} \frac{\beta}{Q} \frac{\partial Q}{\partial t} + \frac{\partial Q}{\partial x} = q_L$$

Equation 4-59.

Where:

q_L = lateral inflow (cfs/ft, m^3/s/m)

Equation 4-54 can be expressed in terms of Manning's n, wetted perimeter, and bed slope by substituting the following expression for αQ^{β} into Equation 4-56:

$$\alpha Q^{\beta} = \left(\frac{QnP^{2/3}}{1.49 S_o^{1/2}}\right)^{3/5}$$

Equation 4-60.

Where:

n = Manning's roughness coefficient

P = wetted perimeter (ft, m)

S_o = flow plane slope (ft/ft, m/m)

The solution to the resulting equation, its terms, and basic concepts are detailed in Chow (1959) and other texts.

Hydrograph Routing

In some cases, the watershed of interest will be divided into subbasins. This is necessary when ground conditions vary significantly between subbasin areas, or when the total watershed area is sufficiently large that variations in precipitation depth within the watershed must be modeled. A rainfall-runoff method (unit hydrograph or kinematic wave) will produce a flow hydrograph at the outlet of each subbasin. Before these hydrographs can be summed to represent flow at the watershed outlet, the effects of travel time and channel/floodplain storage between the subbasin outlets and watershed outlet must be accounted for. The process of starting with a hydrograph at a location and recomputing the hydrograph at a downstream location is called hydrograph routing.

Figure 4-28 shows an example of a hydrograph at upstream location A, and the routed hydrograph at downstream location B. The resulting delay in flood peak is the travel time of the flood hydrograph. The resulting decrease in magnitude of the flood peak is the attenuation of the flood hydrograph.

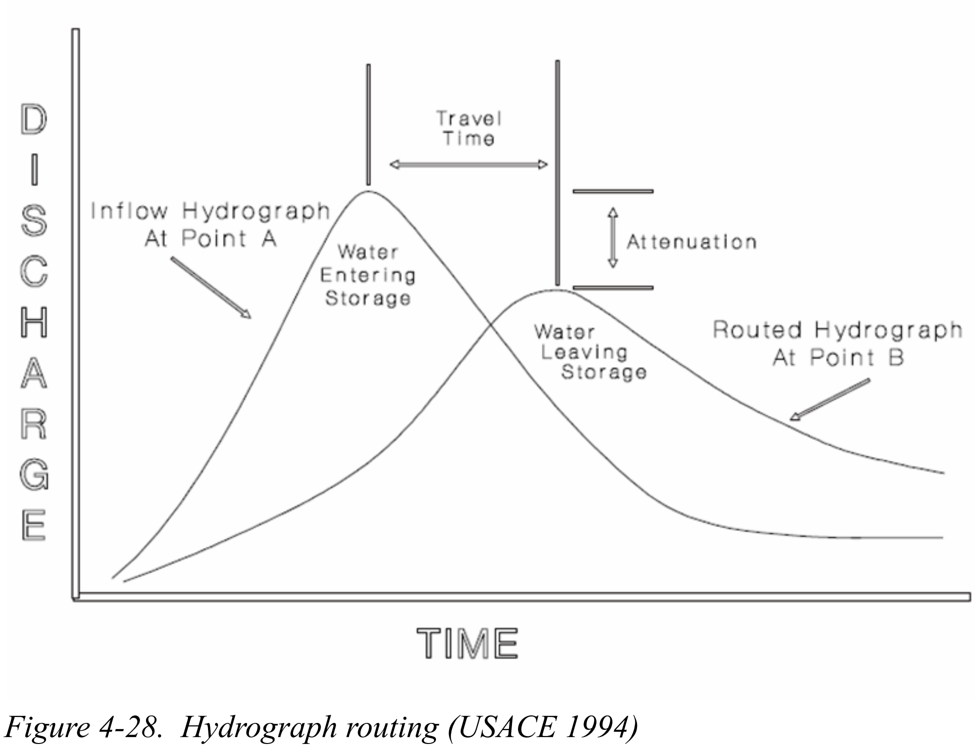

Figure 4-28. Hydrograph routing (USACE 1994)

There are two general methods for routing hydrographs: hydrologic and hydraulic. The methods are distinguished by which equations are solved to compute the routed hydrograph.

Hydrologic methods solve the equation of continuity (conservation of mass), and typically rely on a second relationship (such as relation of storage to outflow) to complete the solution. The equation of continuity can be written as:

$$I - O = \frac{\Delta S}{\Delta t}$$

Equation 4-61.

Where:

I = average inflow to reach or storage area during Dt

O = average outflow to reach or storage area during Dt

S = storage in reach or storage area

Dt = time step

Hydrologic methods are generally most appropriate for steep slope conditions with no significant backwater effects. Hydrologic routing methods include (USACE 1994):

- Modified Puls—for a single reservoir or channel modeled as series of level-pool reservoirs.
- Muskingum—channel modeled as a series of sloped-pool reservoirs.
- Muskingum-Cunge—enhanced version of Muskingum method incorporating channel geometry and roughness information.

Most hydrologic software applications capable of multi-basin analysis offer a selection of hydrologic routing methods.

Hydraulic routing methods solve the Saint Venant equations. These are the one-dimensional equations of continuity (Equation 4-60) and conservation of momentum (Equation 4-61) written for open-channel flow. The equations are valid for gradually varied unsteady flow.

The one-dimensional equation of continuity is:

$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q$$

Equation 4-62.

Where:

A = cross-sectional flow area

V = average velocity of water

x = distance along channel

B = water surface width

y = depth of water

t = time

q = lateral inflow per unit length of channel

The one-dimensional equation of conservation of momentum is:

$$S_f - S_o = \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial y}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t}$$

Equation 4-63.

Where:

S_f = friction slope

S_o = channel bed slope

g = acceleration due to gravity

Hydraulic routing methods are computationally more intensive than hydrologic methods and are distinguished by which terms in the momentum equation (Equation 4-61) are included (not neglected) in the solution algorithm. Hydraulic routing methods include (USACE 1994):

- Dynamic wave (all terms of St. Venant equations)
- Diffusion wave
- Kinematic wave

One-dimensional unsteady open-channel flow software applications implicitly route hydrographs from one location to another by solving for depth and velocity at all locations (cross sections) in a stream reach (or network of reaches) for every time step. The hydraulic routing method employed is defined by the solution algorithm of the software application. Some applications allow the user to select which hydraulic routing method is used, while other applications support only one method.

The most robust routing method (in terms of steep/mild stream slope and with/without backwater effects) is dynamic wave routing.

Selection of Routing Method

Selection of an appropriate routing method depends on several factors. The application of any method will be improved if observed data are available for calibration/verification of routing parameters. Generally, hydrologic methods are most suitable for steeper reaches having little or no backwater effects resulting from high stages downstream of the routing reach. Hydraulic methods are generally more appropriate for a wider range of channel slopes, including gradual slopes, and can accommodate backwater effects. The exception to this is the Muskingum-Cunge method, which does not perform well with steep-rising hydrographs in gradual slopes, or backwater conditions. Of all methods, only the dynamic wave routing method is appropriate for steep and gradual slopes, as well as with or without backwater conditions.

As a baseline approach, the designer may consider using the Muskingum-Cunge method in cases having steep slope (greater than 10 feet per mile) and no backwater effects. This method, which is described in Chow (1988) and Fread (1993), has the advantage that it will incorporate the shape of the cross section into computations. In some cases, cross section data may be obtained from existing hydraulic models of the reach. If channel geometry data are unavailable, then the Muskingum or modified Puls methods, which are described below, may be used. However, these two methods should be avoided for channel routing applications unless observed data are available for calibration/verification of routing parameters.

In cases having backwater that significantly affect the storage-outflow relationship of the routing reach, and thereby significantly affect the routed hydrograph, the dynamic wave, diffusion wave, and modified Puls methods are appropriate.

All methods, except for kinematic wave, are appropriate in cases having a channel slope between 2 to 10 feet per mile, no backwater effects, and satisfying the condition given by Equation 4-62 (USACE 1994):

$$\frac{TS_o u_o}{d_0} \geq 171$$

Equation 4-64.

Where:

T = hydrograph duration (s)

S_o = average friction or slope (ft/ft)

u_o = mean velocity (ft/s)

d_o = average flow depth (ft)

Only the dynamic wave, diffusion wave, and Muskingum-Cunge methods are appropriate in cases having a channel slope less than 2 feet per mile, no backwater effects, and satisfying the condition given by Equation 4-63 (USACE 1994):

$$TS_o\left(\frac{g}{d_o}\right)^{1/2} \geq 30$$

Equation 4-65.

Where:

g = 32.2 ft/s

In cases having a channel slope less than 2 feet per mile, no backwater effects, and not satisfying the condition given by Equation 4-63, then only the dynamic wave method is appropriate.

It may be tempting for the designer to select the dynamic wave routing method as a general approach for all conditions. However, the designer will find that the substantial amount of information (detailed and closely-spaced cross section geometry data) required to construct a one-dimensional unsteady flow model, and the significant time required to ensure that the model is running properly without numerical instabilities, will provide motivation to identify a suitable hydrologic routing method when appropriate. If hydrologic methods are not appropriate for the case under consideration, then an unsteady flow model may be required to properly route flows.

Reservoir Versus Channel Routing

Inflow hydrographs can be routed through reservoirs using a simple (single reservoir) hydrologic routing method, such as the modified Puls storage method. This is because the relationship between storage and discharge is unique (single-valued). In other words, the storage in the reservoir is fully described by the stage in the reservoir because the surface of the reservoir is the same shape and slope during the rising and falling limbs of the hydrograph.

Hydrologic routing methods may also be used for channel routing. A channel does not have a single-valued storage-outflow curve. Instead, the storage-outflow relation is looped, as shown in Figure 4-29. As a result, a hydrologic routing method employing a single reservoir representation cannot be used.

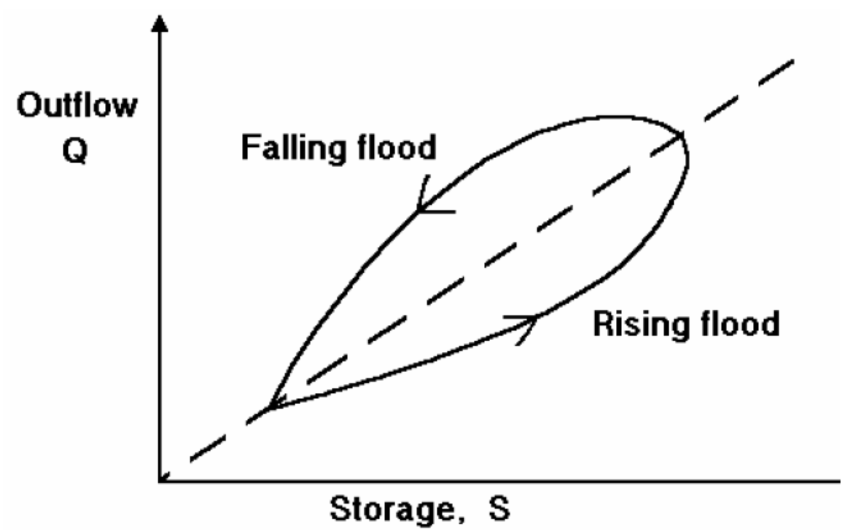

Figure 4-29. Looped storage outflow relation (USACE 1994)

The level-pool limitation of hydrologic routing methods is overcome by representing the channel as a series of reservoirs. These are termed subreaches, or steps, within the routing reach. Another enhancement to the level-pool approach, employed by the Muskingum method, is to represent the storage in each reservoir as a combination of prism storage (similar to level-pool reservoir) and wedge storage (additional sloped water on top of prism).

An estimate of the number of routing steps required for a hydrologic channel routing method is given by (USACE 1994):

$$n = \frac{K}{\Delta t}$$

Equation 4-66.

Where:

n = number of routing steps

K = floodwave travel time through the reach (min.)

Dt = time step (min.)

K in the above equation is given by:

$$K = \frac{L}{V_W}$$

Equation 4-67.

Where:

L = length of routing reach (ft)

V_W = flood wave velocity (ft/s)

V_W may be approximated as equal to the average channel velocity during the flood hydrograph. A better estimate of V_W is given by Seddon's law applied to a cross section representative of the routing reach (USACE 1994):

$$V_W = \frac{1}{B}\frac{dQ}{dy}$$

Equation 4-68.

Where:

B = top with of the channel water surface (ft)

Q = channel discharge (cfs) as function of elevation y

$\frac{dQ}{dy}$ = slope of the discharge rating curve (ft^2/s)

Two hydrologic routing methods and their application are discussed further in the following sections: the modified Puls method for reservoir routing, and the Muskingum method for channel routing.

Modified Puls Method Reservoir Routing

Basic Concepts and Equations

The basic storage routing equation states that mass is conserved and can be expressed as follows: Average inflow - average outflow = Rate of change in storage

In numerical form, this statement of flow continuity can be written as:

$$\frac{I_t + I_{t+1}}{2} - \frac{O_t + O_{t+1}}{2} = \frac{S_{t+1} - S_t}{\Delta t}$$

Equation 4-69.

Where:

I_t = inflow at time step number t

I_{t+1} = inflow at time step number t + 1

O_t = outflow at time step number t

O_{t+1} = outflow at time step number t + 1

S_t = storage in the reservoir at time step number t

S_{t+1} = storage in the reservoir at time step number t + 1

Δt = the time increment

Δt = the time increment *** REMOVE DUPLATE INFO IF DISPLAYS NICELY ***

t = time step number

In Equation 4-64 there are two unknowns: O_{t+1} and S_{t+1}. In order to solve Equation 4-64, either a second equation with O_{t+1} and S_{t+1} is required, or a relationship between O_{t+1} and S_{t+1} is needed. The storage-indication approach is the latter and is presented here. First, it is convenient to rewrite the routing equation as:

$$\frac{2S_{t+1}}{\Delta t} + O_{t+1} = I_t + I_{t+1} + \frac{2S_t}{\Delta t} - O_t$$

Equation 4-70.

In this form, all terms known at time t are on the right hand side of the equation and unknowns are on the left. If a single-valued storage-outflow curve can be determined for the routing reach, then for any value of O_{t+1}, the corresponding value of S_{t+1} will be known. This reduces the number of unknown parameters in Equation 4-65 from two (O_{t+1} and S_{t+1}) to one (O_{t+1}).

Use of the storage routing method requires the designer to determine the relationship between storage and outflow. This is simply the volume of water held by the reservoir, storage facility, or pond as a function of the water surface elevation or depth. For a reservoir or storage facility, this information is often available from the reservoir sponsor or owner.

For a pond or lake or where the stage-storage relation is not available, a relationship between storage and outflow can be derived from considerations of physical properties of channel or pond and simple hydraulic models of outlet works or relationship of flow and water surface elevation. These physical properties include:

- Ratings of the primary and/or emergency spillway of a reservoir.
- Pump flow characteristics in a pump station.
- Hydraulic performance curve of a culvert or bridge on a highway.
- Hydraulic performance curve of a weir and orifice outlet of a detention pond.

With the stage-storage relation established, a storage indication curve corresponding to the left side of Equation 4-68 is developed. The relationship is described in the form of O versus (2S/ΔT) + O. An example of a storage indication curve is provided in Figure 4-30.

Figure 4-30. Sample storage-indication relation

The form of Equation 4-68 shown above is useful because the terms on the left side of the equation are known. With the relation between the outflow and storage determined (Figure 4-30), the ordinates on the outflow hydrograph can be determined directly.

Storage Routing Procedure

Use the following steps to route an inflow flood runoff hydrograph through a storage system such as a reservoir or detention pond:

1. Acquire or develop a design flood runoff hydrograph for the project site watershed.
2. Acquire or develop a stage-storage relation.
3. Acquire or develop a stage-outflow relationship.
4. Develop a storage-outflow relationship.
5. Assume an initial value for O_t as equal to I_t. At time step one (t = 1), assume an initial value for O_t as equal to I_t. Usually, at time step one, inflow equals zero, so outflow will be zero and $2S_1/\Delta T - O_1$ equals zero. Note that to start, t + 1 in the next step is 2.
6. Compute $2S_{t+1}/\Delta T + O_{t+1}$ using Equation 4-68.
7. Interpolate to find the value of outflow. From the storage-outflow relation, interpolate to find the value of outflow (O_{t+1}) at $(2S_{t+1})/(\Delta T)+O_{t+1}$ from step 6.
8. Determine the value of $(2S_{t+1})/(\Delta T)-O_{t+1}$. Use the relation $(2S_{t+1})/(\Delta T)-O_{t+1} = (2S_{t+1})/(\Delta T)+O_{t+1} - 2O_{t+1}$.
9. Assign the next time step to the value of t, e.g., for the first run through set t = 2.
10. Repeat steps 6 through 9 until the outflow value (O_{t+1}) approaches zero.
11. Plot the inflow and outflow hydrographs. The peak outflow value should always coincide with a point on the receding limb of the inflow hydrograph.
12. Check conservation of mass to help verify success of the process. Use Equation 4-69 to compare the inflow volume to the sum of retained and outflow volumes:

$$\Delta T \cdot \sum I_t = S_r + \Delta T \cdot \sum O_t$$

Equation 4-71.

Where:

S_r = difference in starting and ending storage (ft^3 or m^3)

SI_t = sum of inflow hydrograph ordinates (cfs or m^3/s)

SO_t = sum of outflow hydrograph ordinates (cfs or m^3/s)

Muskingum Method Channel Routing

Routing of flood hydrographs by means of channel routing procedures is useful in instances where computed hydrographs are at points other than the points of interest. This is also true in those instances where the channel profile or plan is changed in such a way as to alter the natural velocity or channel storage characteristics. Routing estimates the effect of a channel reach on an inflow hydrograph. This section describes the Muskingum method equations, a lumped flow routing technique that approximates storage effects in the form of a prism and wedge component (Chow 1988).

The Muskingum method also solves the equation of continuity. With the Muskingum method, the storage in the channel is considered the sum of two components: prism storage and wedge storage (Figure 4-31).

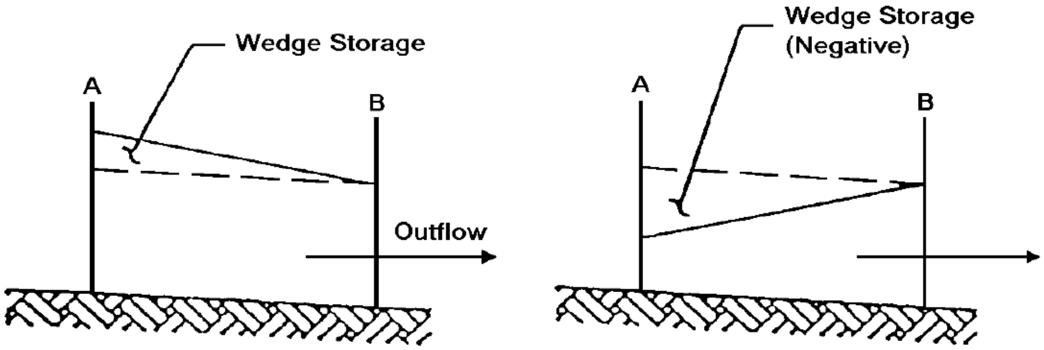

Figure 4-31. Muskingum prism and wedge storage

The constants K and X are used to relate the prism component, KO, and wedge component, KX(I-O), to the inflow and outflow of the reach:

$$S = K[XI + (1 - X)O]$$

Equation 4-72.

***Where**:*

S = total storage (ft^3 or m^3)

K = a proportionality constant representing the time of travel of a flood wave to traverse the reach (s). Often, this is set to the average travel time through the reach.

X = a weighting factor describing the backwater storage effects approximated as a wedge

I = inflow (cfs or m^3/s)

O = outflow (cfs or m^3/s)

The value of X depends on the amount of wedge storage; when X = 0, there is no backwater (reservoir type storage), and when X = 0.5, the storage is described as a full wedge. The weighting factor, X, ranges from 0 to 0.3 in natural streams. A value of 0.2 is typical.

Equation 4-68 represents the time rate of change of storage as the following:

$$\frac{S_{t+1} - S_t}{\Delta T} = \frac{K\{[XI_{t+1} + (1 - X)O_{t+1}] - [XI_t + (1 - X)O_t]\}}{\Delta T}$$

Equation 4-73.

***Where**:*

DT = time interval usually ranging from 0.3K to K

t = time step number

Combining Equation 4-70 with Equation 4-71 yields the Muskingum flow routing equation:

$$O_{t+1} = C_1 I_{t+1} + C_2 I_t + C_3 O$$

Equation 4-74.

***Where**:*

$$C_1 = \frac{\Delta T - 2KX}{2K(1-X) + \Delta T}$$

Equation 4-75.

$$C_2 = \frac{\Delta T + 2KX}{2K(1-X) + \Delta T}$$

Equation 4-76.

$$C_3 = \frac{2K(1-X) - \Delta T}{2K(1-X) + \Delta T}$$

Equation 4-77.

By definition, the sum of C_1, C_2, and C_3 is 1. If measured inflow and outflow hydrographs are available, K and X can be estimated using Equation 4-71. Calculate X by plotting the numerator on the vertical axis and the denominator on the horizontal axis, and adjusting X until the loop collapses into a single line. The slope of the line equals K:

$$K = \frac{0.5\Delta T[(I_{t+1} + I_t) - (O_{t+1} + O_t)]}{X(I_{t+1} - I_t) + (1-X)(O_{t+1} - O_t)}$$

Equation 4-78.

The designer may also estimate K and X using the Muskingum-Cunge method described in Chow 1988 or Fread 1993.

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Section 15 — Glossary of Hydrology Terms

Annual Exceedance Probability (AEP)

The probability of exceedance in a given year.

Annual Flood

The maximum peak discharge in a water year.

Annual Flood Series

A list of annual floods.

Antecedent Conditions

Watershed conditions prevailing prior to an event; normally used to characterize basin wetness, e.g., soil moisture. Also referred to as initial conditions or antecedent moisture conditions (AMC).

Area-Capacity Curve

A graph showing the relation between the surface area of the water in a reservoir and the corresponding volume.

Attenuation

The reduction in the peak of a hydrograph resulting in a broad, flat hydrograph.

Backwater

Water backed up or retarded in its course as compared with its normal or natural condition of flow. In stream gauging, a rise in stage produced by a temporary obstruction such as ice or weeds, or by the flooding of the stream below. The difference between the observed stage and that indicated by the stage-discharge relation is reported as backwater.

Bank

The margins of a channel. Banks are called right or left as viewed facing downstream (in the direction of the flow).

Bank Storage

The water absorbed into the banks of a stream channel, when the stages rise above the water table in the bank formations, then returns to the channel as effluent seepage when the stages fall below the water table.

Bankfull Stage

Maximum stage of a stream before it overflows its banks. (see also flood stage.)

Base Discharge (for peak discharge)

In the USGS annual reports on surface-water supply, the discharge above which peak discharge data are published. The base discharge at each station is selected so that an average of about 3 peaks a year will be presented. (See also partial-duration flood series.)

Baseflow

The sustained or fair weather flow in a channel due to subsurface runoff. In most streams, baseflow is composed largely of groundwater effluent. Also known as base runoff.

Basic Hydrologic Data

Includes inventories of features of land and water that vary spatially (topographic and geologic maps are examples), and records of processes that vary with both place and time. (Records of precipitation, streamflow, ground water, and quality-of-water analyses are examples.) Basic hydrologic information is a broad term that includes surveys of the water resources of particular areas and a study of their physical and related economic processes, interrelations, and mechanisms.

Basic-Stage Flood Series

See partial duration flood series.

Bifurcation

The point where a stream channel splits into two distinct channels.

Binomial Statistical Distribution

The frequency distribution of the probability of a specified number of successes in an arbitrary number of repeated independent Bernoulli trials. Also called Bernoulli distribution.

Boundary Condition

Conditions at the boundary of a problem that govern its solution. For example, when solving a routing problem for a given reach, an upstream inflow boundary condition is necessary to solve for the outflow at the downstream end of the reach.

Calibration

Derivation of a set of model parameter values that produces the best fit to observed data.

Canopy-Interception

Precipitation that falls on and is stored in the leaves or trunks of vegetation. The term can refer to either the process or a volume.

Channel (watercourse)

An open conduit either naturally or artificially created which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. River, creek, run, branch, anabranch, and tributary are some of the terms used to describe natural channels. Natural channels may be single or braided. Canal and floodway are terms used to describe artificial channels.

Channel Storage

The volume of water at a given time in the channel or over the flood plain of the streams in a drainage basin or river reach. Channel storage can be large during the progress of a flood event.

Computation Duration

The user-defined time window used in hydrologic modeling.

Computation Interval

The user-defined time step used by a hydrologic model for performing mathematical computations. For example, if the computation interval is 15 min. and the starting time is 1200, hydrograph ordinates will be computed at 1200, 1215, 1230, 1245, and so on.

Concentration Time

See time of concentration.

Confluence

The point at which two distinct stream channels converge

Continuous Model

A model that tracks the periods between precipitation events, as well as the events themselves. See event-based model.

Correlation

The process of establishing a relation between a variable and one or more related variables. Correlation is simple if there is only one independent variable and multiple when there are two or more independent variables. For gauging station records, the usual variables are the short-term gauging-station record and one or more long-term gauging-station records

Dendritic

Channel pattern of streams with tributaries that branch to form a tree-like pattern.

Depression Storage

The volume of water contained in natural depressions in the land surface, such as puddles.

Design Flood

The flood that is chosen as the basis for the design of a hydraulic structure.

Design Storm

Rainfall amount and distribution in time and space used to determine a design flood or design peak discharge

Detention Basin

Storage facility, such as a small unregulated reservoir, which delays the conveyance of water downstream.

Diffusion

Dissipation of the energy associated with a flood wave; results in the attenuation of the flood wave.

Direct Runoff

The runoff entering stream channels promptly after rainfall or snowmelt. Superimposed on base runoff, it forms the bulk of the hydrograph of a flood. See also surface runoff. The terms base runoff and direct runoff are time classifications of runoff. The terms groundwater runoff and surface runoff are classifications according to source.

Discharge

The volume of water that passes through a given cross-section per unit time; commonly measured in cubic feet per second (cfs) or cubic meters per second (m^3/s). Also referred to as flow.

In its simplest concept discharge means outflow; therefore, the use of this term is not restricted as to course or location, and it can be applied to describe the flow of water from a pipe or from a drainage basin. If the discharge occurs in some course or channel, it is correct to speak of the discharge of a canal or of a river. It is also correct to speak of the discharge of a canal or stream into a lake, a stream, or an ocean. (See also streamflow and runoff.)

Discharge data in USGS reports on surface water represent the total fluids measured. Thus, the terms discharge, streamflow, and runoff represent water with sediment and dissolved solids. Of these terms, discharge is the most comprehensive. The discharge of drainage basins is distinguished as follows:

Yield: Total water runout; includes runoff plus underflow.

Runoff: That part of water yield that appears in streams.

Streamflow: The actual flow in streams, whether or not subject to regulation, or underflow.

Each of these terms can be reported in total volumes or time rates. The differentiation between runoff as a volume and streamflow as a rate is not accepted.

Discharge Rating Curve

See stage discharge relation.

Distribution Graph (distribution hydrograph)

A unit hydrograph of direct runoff modified to show the proportions of the volume of runoff that occurs during successive equal units of time.

Diversion

The taking of water from a stream or other body of water into a canal, pipe, or other conduit.

Drainage Area

The drainage area of a stream at a specified location is that area, measured in a horizontal plane, which is enclosed by a drainage divide.

Drainage Divide

The rim of a drainage basin. (See watershed.)

Duration Curve

See flow-duration curve for one type.

ET

See evapotranspiration.

Effective Precipitation (rainfall)

1. That part of the precipitation that produces runoff.

2. A weighted average of current and antecedent precipitation that is "effective" in correlating with runoff.

Evaporation

The process by which water is changed from the liquid or the solid state into the vapor state. In hydrology, evaporation is vaporization and sublimation that takes place at a temperature below the boiling point. In a general sense, evaporation is often used interchangeably with evapotranspiration or ET.

Evaporation Demand

The maximum potential evaporation generally determined using an evaporation pan. For example, if there is sufficient water in the combination of canopy and surface storage, and in the soil profile, the actual evaporation will equal the evaporation demand. A soil-water retention curve describes the relationship between evaporation demand and actual evaporation when the demand is greater than available water. See tension zone.

Evaporation Pan

An open tank used to contain water for measuring the amount of evaporation. The US National Weather Service class A pan is 4 ft in diameter, 10 in. deep, set up on a timber grillage so that the top rim is about 16 in. from the ground. The water level in the pan during the course of observation is maintained between 2 and 3 in. below the rim.

Evaporation, Total

The sum of water lost from a given land area during any specific time by transpiration from vegetation and building of plant tissue; by evaporation from water surfaces, moist soil, and snow; and by interception. It has been variously termed evaporation, evaporation from land areas, evapotranspiration, total loss, water losses, and fly off.

Evapotranspiration

Water withdrawn from a land area by evaporation from water surfaces and moist soils and plant transpiration.

Event-Based Model

A model that simulates some hydrologic response to a precipitation event. See continuous model.

Exceedance Probability

Hydrologically, the probability that an event selected at random will exceed a specified magnitude.

Excess Precipitation

The precipitation in excess of infiltration capacity, evaporation, transpiration, and other losses. Also referred to as effective precipitation.

Excessive Rainfall

See rainfall, excessive.

Falling Limb

The portion of a hydrograph where runoff is decreasing.

Field Capacity

The quantity of water which can be permanently retained in the soil in opposition to the downward pull of gravity. Also known as field-moisture capacity.

Field-Moisture Deficiency

The quantity of water which would be required to restore the soil moisture to field-moisture capacity.

Flood

An overflow or inundation that comes from a river or other body of water, and causes or threatens damage. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a stream. A relatively high flow as measured by either gauge height or discharge quantity. As it relates to highway drainage design for TxDOT, and for the purposes of this manual, any direct run-off from precipitation; not limited to an out-of-banks event.

Flood Crest

See flood peak.

Flood Event

See flood wave.

Flood Peak

The highest value of the stage or discharge attained by a flood; thus, peak stage or peak discharge. Flood crest has nearly the same meaning, but since it connotes the top of the flood wave, it is properly used only in referring to stage—thus, crest stage, but not crest discharge.

Floodplain

A strip of relatively flat land bordering a stream, built of sediment carried by the stream and dropped in the slack water beyond the influence of the swiftest current. It is called a living flood plain if it is overflowed in times of high water; but a fossil flood plain if it is beyond the reach of the highest flood. The lowland that borders a river, usually dry but subject to flooding. That land outside of a stream channel described by the perimeter of the maximum probable flood.

Flood Profile

A graph of elevation of the water surface of a river in flood, plotted as ordinate, against distance, measured in the downstream direction, plotted as abscissa. A flood profile may be drawn to show elevation at a given time, crests during a particular flood, or to show stages of concordant flows.

Flood Routing

The process of progressively determining the timing and shape of a flood wave at successive points along a river.

Flood Stage

The gauge height of the lowest bank of the reach in which the gauge is situated. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates an unimportant and small area. The stage at which overflow of the natural banks of a stream begins to occur. See also bankfull stage.

Flood Wave

A distinct rise in stage culminating in a crest and followed by recession to lower stages.

Flood, Maximum Probable

The flood magnitude that may be expected from the most critical combination of meteorologic and hydrologic conditions reasonably possible for a given watershed.

Flood-Frequency Curve

1. A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are equaled or exceeded.

2. A similar graph but with recurrence intervals of floods plotted as abscissa.

Floodway

A part of the floodplain otherwise leveed, reserved for emergency diversion of water during floods. A part of the floodplain which, to facilitate the passage of floodwater, is kept clear of encumbrances.

The channel of a river or stream and those parts of the floodplains adjoining the channel which are reasonably required to carry and discharge the floodwater or flood flow of any river or stream.

Flow-Duration Curve

A cumulative frequency curve that shows the percentage of time that specified discharges are equaled or exceeded.

Gauging Station

A particular site on a stream, canal, lake, or reservoir where systematic observations of gauge height or discharge are obtained. (See also stream-gauging station.)

Ground Water

Water in the ground that is in the zone of saturation, from which wells, springs, and groundwater runoff are supplied.

Groundwater Outflow

That part of the discharge from a drainage basin that occurs through the ground water. The term "underflow" is often used to describe the groundwater outflow that takes place in valley alluvium (instead of the surface channel) and thus is not measured at a gauging station.

Groundwater Runoff

That part of the runoff that has passed into the ground, has become ground water, and has been discharged into a stream channel as spring or seepage water. See also base runoff and direct runoff.

Hydraulic Radius

The flow area of a channel cross section divided by the wetted perimeter. The wetted perimeter does not include the free surface.

Hydrograph

A graph showing stage, flow, velocity, or other property of water with respect to time.

Hydrologic Budget

An accounting of the inflow to, outflow from, and storage in a hydrologic unit, such as a drainage basin, aquifer, soil zone, lake, reservoir, or irrigation project.

Hydrologic Cycle

The continuous process of water movement between the oceans, atmosphere, and land.

Hydrology

The study of water; generally focuses on the distribution of water and interaction with the land surface and underlying soils and rocks.

Hyetograph

A plot of rainfall intensity versus time; often represented by a bar graph.

Index Precipitation

An index that can be used to adjust for bias in regional precipitation, often quantified as the expected annual precipitation.

Infiltration

The movement of water from the land surface into the soil.

Infiltration Capacity

The maximum rate at which the soil, when in a given condition, can absorb falling rain or melting snow.

Infiltration Index

An average rate of infiltration, in inches per hour, equal to the average rate of rainfall such that the volume of rainfall at greater rates equals the total direct runoff.

Inflection Point

Generally refers the point on a hydrograph separating the falling limb from the recession curve; any point on the hydrograph where the curve changes concavity.

Initial Condition

The conditions prevailing prior to an event. Refer also to antecedent conditions.

Interception

The capture of precipitation above the ground surface (e.g., by vegetation or buildings).

Isohyetal Line

A line drawn on a map or chart joining points that receive the same amount of precipitation.

Lag

Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of a runoff hydrograph.

Lag Time

The time from the center of mass of excess rainfall to the hydrograph peak. Also referred to as basin lag.

Loss

The difference between the volume of rainfall and the volume of runoff. Losses include water absorbed by infiltration, water stored in surface depressions, and water intercepted by vegetation.

Mass Curve

A graph of the cumulative values of a hydrologic quantity (such as precipitation or runoff), generally as ordinate, plotted against time or date as abscissa. (See double-mass curve and residual-mass curve.)

Maximum Probable Flood

See flood, maximum probable.

Meander

The winding of a stream channel.

Model

A physical or mathematical representation of a process that can be used to predict some aspect of the process.

Moisture

Water diffused in the atmosphere or the ground.

Objective Function

A mathematical expression that allows comparison between a calculated result and a specified goal. For model calibration, the objective function compares calculated discharge with observed discharge.

Overland Flow

The shallow flow of water over the land surface before combining with additional flow to become channel flow.

Parameter

A variable, in a general model, whose value is adjusted to make the model specific to a given situation. A numerical measure of the properties of the real-world system.

Parameter Estimation

The selection of a parameter value based on the results of analysis and/or engineering judgment. Analysis techniques include calibration, regional analysis, estimating equations, and physically based methods. Refer also to calibration.

Partial-Duration Flood Series

A list of all flood peaks that exceed a chosen base stage or discharge, regardless of the number of peaks occurring in a year. (Also called basic-stage flood series, or floods above a base.)

Peak Flow

The point of the hydrograph that has the highest flow.

Peak Stage

The highest elevation reached by a flood wave. Also referred to as the crest.

Percolation

The movement, under hydrostatic pressure, of water through the interstices of a rock or soil.

PMF

Probable maximum flood; see flood, probable maximum.

Precipitation

As used in hydrology, precipitation is the discharge of water, in liquid or solid state, out of the atmosphere, generally upon a land or water surface. It is the common process by which atmospheric water becomes surface or subsurface water. The term precipitation is also commonly used to designate the quantity of water that is precipitated. Precipitation includes rainfall, snow, hail, and sleet, and is therefore a more general term than rainfall.

Precipitation, Probable Maximum

Theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year.

Probability of Capacity Exceedance

The likelihood of the design flow rate (or volume of water with specified duration) of a hydraulic structure being exceeded in a given year.

Probability of Exceedance

The likelihood of a specified flow rate (or volume of water with specified duration) being exceeded in a given year.

Rain

Liquid precipitation.

Rainfall

The quantity of water that falls as rain only. Not synonymous with precipitation.

Rainfall Excess

The volume of rainfall available for direct runoff. It is equal to the total rainfall minus interception, depression storage, and absorption.

Rating Curve

The relationship between stage and discharge.

Reach

A segment of a stream channel.

Recession Curve

The portion of the hydrograph where runoff is predominantly produced from basin storage (subsurface and small land depressions); it is separated from the falling limb of the hydrograph by an inflection point.

Recurrence Interval (return period)

The average interval of time within which the given flood will be equaled or exceeded once. When the recurrence interval is expressed in years, it is the reciprocal of the annual exceedance probability (AEP).

Regulation, Regulated

The artificial manipulation of the flow of a stream.

Reservoir

A pond, lake, or basin, either natural or artificial, for the storage, regulation, and control of water.

Residual-Mass Curve

A graph of the cumulative departures from a given reference such as the arithmetic average, generally as ordinate, plotted against time or date, as abscissa. (See mass curve.)

Retention Basin

Similar to detention basin but water in storage is permanently obstructed from flowing downstream.

Rising Limb

Portion of the hydrograph where runoff is increasing.

Runoff

Precipitation on the ground that is not captured by evaporation, infiltration, interception, or surface storage.

Saturation Zone

The portion of the soil profile where available water storage is completely filled. The boundary between the vadose zone and the saturation zone is called the water table. Note that under certain periods of infiltration, the uppermost layers of the soil profile can be saturated. See vadose zone.

NRCS Curve Number

An empirically derived relationship between location, soil-type, land use, antecedent moisture conditions, and runoff. A Natural Resources Conservation Service (NRCS) curve number is used in an event-based model to establish the initial soil moisture condition and the infiltration.

Snow

A form of precipitation composed of ice crystals.

Soil Moisture Accounting (SMA)

A modeling process that accounts for continuous fluxes to and from the soil profile. Models can be event-based or continuous. When using a continuous simulation, a soil moisture accounting method is used to account for changes in soil moisture between precipitation events.

Soil Moisture (soil water)

Water diffused in the soil, the upper part of the zone of aeration from which water is discharged by the transpiration of plants or by soil evaporation. See field-moisture capacity and field-moisture deficiency.

Soil Profile

A description of the uppermost layers of the ground down to bedrock. In a hydrologic context, the portion of the ground subject to infiltration, evaporation, and percolation fluxes.

Stage

The height of a water surface in relation to a datum.

Stage-Capacity Curve

A graph showing the relation between the surface elevation of the water in a reservoir usually plotted as ordinate, against the volume below that elevation plotted as abscissa.

Stage-Discharge Curve (rating curve)

A graph showing the relation between the water height, usually plotted as ordinate, and the amount of water flowing in a channel, expressed as volume per unit of time, plotted as abscissa.

Stage-Discharge Relation

The relation expressed by the stage-discharge curve.

Stemflow

Rainfall or snowmelt led to the ground down the trunks or stems of plants.

Storage

1. Water artificially or naturally impounded in surface or underground reservoirs. The term regulation refers to the action of this storage in modifying downstream streamflow.

2. Water naturally detained in a drainage basin, such as ground water, channel storage, and depression storage. The term drainage basin storage or simply basin storage is sometimes used to refer collectively to the amount of water in natural storage in a drainage basin.

Storm

A disturbance of the ordinary average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

Stream

A general term for a body of flowing water. In hydrology the term is generally applied to the water flowing in a natural channel as distinct from a canal. More generally, as in the term stream gauging, it is applied to the water flowing in any channel, natural or artificial.

Stream Gauging

The process of measuring the depths, areas, velocities, and rates of flow in natural or artificial channels.

Streamflow

The discharge that occurs in a natural channel. Although the term discharge can be applied to the flow of a canal, the word streamflow uniquely describes the discharge in a surface stream course. The term streamflow is more general than runoff, as streamflow may be applied to discharge whether or not it is affected by diversion or regulation.

Stream-Gauging Station

A gauging station where a record of discharge of a stream is obtained. Within the USGS this term is used only for those gauging stations where a continuous record of discharge is obtained.

Sublimation

The process of transformation directly between a solid and a gas.

Surface Runoff

That part of the runoff that travels over the soil surface to the nearest stream channel. It is also defined as that part of the runoff of a drainage basin that has not passed beneath the surface since precipitation. The term is misused when applied in the sense of direct runoff. See also runoff, overland flow, direct runoff, groundwater runoff, and surface water.

Surface Water

Water on the surface of the earth.

Tension Zone

In the context of HEC-HMS, the portion of the soil profile that will lose water only to evapotranspiration. This designation allows modeling water held in the interstices of the soil. See soil profile.

Time of Concentration

The travel time from the hydraulically furthermost point in a watershed to the outlet. Also defined as the time from the end of rainfall excess to the inflection point on the recession curve.

Time of Rise

The time from the start of rainfall excess to the peak of the hydrograph.

Time to Peak

The time from the center of mass of the rainfall excess to the peak of the hydrograph. Refer also to lag time

TR-20

Computer program developed by the NRCS that provides a hydrologic analysis of a watershed under present conditions. Output consists of peaks and/or flood hydrographs. Subarea surface runoff hydrographs are developed from storm rainfall using the dimensionless unit hydrograph, drainage areas, times of concentration, and NRCS runoff curve numbers. Instructions to develop, route, add, store, divert, or divide hydrographs are established to convey floodwater from the headwaters to the watershed outlet.

TR-55

Urban Hydrology for Small Watershed—Technical Release 55 published by the NRCS. Technical Release 55 (TR-55) presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for floodwater reservoirs. These procedures are applicable to small watersheds, especially urbanizing watersheds, in the United States.

Transpiration

The quantity of water absorbed and transpired and used directly in the building of plant tissue, in a specified time. It does not include soil evaporation. The process by which water vapor escapes from the living plant, principally the leaves, and enters the atmosphere.

Underflow

The downstream flow of water through the permeable deposits that underlie a stream and that are more or less limited by rocks of low permeability.

Unit Hydrograph

A direct runoff hydrograph produced by 1 unit of excess precipitation over a specified duration. For example, a 1-hour unit hydrograph is the direct runoff from one unit of excess precipitation occurring uniformly over one hour.

Vadose Zone

The portion of the soil profile above the saturation zone.

Water Year

In USGS reports dealing with surface-water supply, the 12-month period, October 1 through September 30. The water year is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the year ended September 30, 1959, is called the 1959 water year.

Watershed

An area characterized by all direct runoff being conveyed to the same outlet. Similar terms include basin, drainage basin, catchment, and catch basin.

A part of the surface of the earth that is occupied by a drainage system, which consists of a surface stream or a body of impounded surface water together with all tributary surface streams and bodies of impounded surface water.

WinTR-55

A MS Windows-based computer program developed by NRCS. WinTR-55 uses the procedures presented in TR-20 as the driving engine for more accurate analysis of the hydrology of the small watershed system being studied.

Chapter 5 — NFIP Design of Floodplain Encroachments & Cross Drainage Structures

Contents:

Section 1 — The National Flood Insurance Program (NFIP)

The roadway and drainage facility designers, whether TxDOT or consultant, need to consider flood issues, and be familiar with the intent and requirements of the Federal Emergency Management Agency (FEMA) NFIP.

The NFIP was established by the United States Congress in the National Flood Insurance Act of 1968 (42 U.S.C. 4001 et seq.) and is administered by the Federal Insurance Administration and the Mitigation Directorate of FEMA. Flooding may be from marine, lacustrine, or riverine sources.

The purpose and intent of the NFIP is to discourage development within the floodplain that will increase the risk of loss from flood damage, and to encourage suitable use of floodplains. Parks, playfields, roads, bridges, and culverts are consistent with suitable use of floodplains when properly designed and constructed.

Section 2 — Definitions

Floodplain encroachment, as it applies to TxDOT, is any construction, replacement, or extension of a bridge, culvert, low-water crossing, or storm drain outfall in a floodplain whether the structure interferes with flood waters or not. Encroachments can also be bridge widening, pavement overlays, modification or addition of bridge rails and traffic median barriers, and safety end treatments (SETs). Longitudinal encroachments are roads or walkways constructed in the floodplain paralleling a watercourse instead of crossing it. For the purposes of this chapter, the term “encroachment” includes structures over any waters of the U.S., whether in a FEMA mapped floodplain or not.

Minor structures are smaller culverts under driveways in ditches or under the roadway connecting two ditches, but not conveying waters of the U.S.

Mapped Floodplain or Special Flood Hazard Area (SFHA) is an area that FEMA has designated as having a probability of inundation during a 1% AEP (Annual Exceedance Probability) or 100-year flood, usually shown on a Flood Insurance Rate Map (FIRM). Most of the mapped floodplains or SFHAs are riverine designated Zone A, AE, or A1-30; other types are playas (AH), flatlands with standing waters (AO), and coastal floodplains, (V, VE, or V1-30). The accepted definitions of various risk zones, including SFHAs, are listed below (see Types of Flood Zones).

Flood Insurance Rate Map (FIRM) is a graphical representation of SFHAs and floodways, flood hazard risk zones, base flood elevations, 0.2% AEP (500-year) floodplain areas, and other flood-related information.

Effective Map is the latest FIRM issued by FEMA, which is in effect as of the date shown in the title box of the FIRM as “Effective Date,” “Revised,” or “Map Revised.”

Base Flood Elevation (BFE) is the 1% AEP flood water surface elevation. BFEs are usually given in feet above mean sea level. BFEs are determined through a hydrologic and hydraulic study of the area or the waterway as a whole, not a small area or isolated reach of stream.

Flood Insurance Study (FIS) is the final report which summarizes the results of the detailed studies on which a Zone AE is based. The FIS usually includes an appraisal of a community's flooding problems, engineering methodologies, flood discharges, flood profiles, and floodplain/floodway technical data.

Floodway is the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without increasing the water-surface elevation more than a designated height.

Flood fringe is the area within the floodplain but outside of the floodway.

Types of Flood Zones (Risk Flood Insurance Zone Designations)

ZONE A: SFHAs inundated by the 1% AEP (100-year) flood. No BFEs determined.

ZONE AE (formerly A1-30): SFHAs inundated by the 1% AEP (100-year) flood. BFEs deter mined by detailed study.

ZONE AO: SFHAs inundated by 1% AEP (100-year) flood depths of 1 to 3 feet, usually sheet flow on sloping terrain. Depths shown are average. For areas of alluvial fan flooding, velocities are also determined. See 44CFR59, "Area of shallow flooding".

ZONE AH: SFHAs inundated by 1% AEP (100-year) flood depths of 1 to 3 feet, usually areas of ponding. BFEs are determined. In Texas, Zone AH usually applies to playas, low areas with no outflow. The only escapes are infiltration and evaporation.

ZONE AR: SFHAs that result from the decertification of previously accredited flood protection systems that are in the process of being restored to provide a 1% AEP (100-year) or greater level of flood protection.

ZONES AR/A1-30, AR/AE, AR/AH, AR/AO and AR/A (Dual Zones): SFHAs that result from the decertification of previously accredited flood protection systems that are in the process of being restored to provide a 1% AEP (100-year) or greater level of flood protection. After restoration is complete, these areas will still experience residual flooding from other flooding sources.

ZONE A99: SFHAs inundated by the 1% AEP (100-year) flood to be protected from the 1% AEP flood by a Federal flood protection system under construction. No BFEs determined.

ZONE V: SFHAs in coastal areas with velocity hazards (wave action) inundated by the 1% AEP (100-year) flood; no BFEs determined.

ZONE VE (formerly V1-30): SFHAs in coastal areas with velocity hazards (wave action) inundated by the 1% AEP (100-year) flood. Base flood elevations determined by detailed study.

ZONE B and ZONE X (shaded): Areas of 0.2% AEP (500-year) flood; areas subject to the 1% AEP (100-year) flood with average depths of less than 1 foot or with contributing drainage area less than 1 square mile; areas protected by levees from the base flood.

ZONE C and ZONE X (unshaded): Areas determined to be outside the 0.2% AEP (500-year) floodplain.

ZONE D: Areas in which flood hazards are undetermined.

Section 3 — NFIP Roles

Participation

Participation in the NFIP is voluntary and most communities participate, although not all communities choose to. A community must be a participating community in the NFIP in order to have valid FEMA mapped SFHAs within it. A community must agree to regulate new development in the designated floodplain through a floodplain ordinance to participate in the NFIP. For the purposes of the NFIP, 44CFR78.2 defines community as (1) a political subdivision that has zoning and building code jurisdiction over a particular area having special flood hazards or participating in the NFIP, or (2) a political subdivision of a State or other authority that is designated to develop and administer a mitigation plan by political subdivision. In other words, a community is an entity which has authority to adopt and enforce floodplain management regulations for the areas within its jurisdiction.

The floodplain ordinance must require that development in the designated floodplain be consistent with the intent, standards and criteria set by the NFIP. Communities may adopt ordinances and rules that are more stringent than the requirements in 44CFR60.3 and are encouraged by FEMA to do so. In return for community participation in the NFIP, the property owners in the community are eligible to purchase federal flood insurance for buildings and contents, and FEMA will prepare maps showing the SFHAs to be used by the community, insurance agents, and others. FEMA maintains the list of all NFIP communities within the states, both participating and non-participating.

Floodplain Administrator

A participating community is required to appoint a Floodplain Administrator (FPA) whose duty is enforcement of the floodplain ordinance by permitting activities in the SFHAs and maintaining records of all changes to the water surface elevations in the SFHAs.

Texas

FEMA requires each state to appoint a State Coordinating Agency for the NFIP, which in Texas is the Texas Water Development Board (TWDB). The Texas Water Code (TWC 16.3145) requires all cities and counties to adopt ordinances making them eligible to participate in the NFIP. However, actual participation is at the option of the community.

Non-participating Communities

Not all communities are participating in the NFIP. Some communities have never participated while others were mapped but then withdrew from the program. The FIRMs of the withdrawn communities still exist and are available online even though they are not effective maps.

A community must be a participating community in the NFIP in order to have valid FEMA mapped SFHAs within it. Communities not participating in the NFIP do not have any valid FEMA mapped SFHAs by virtue of the fact that they are non-participating, even though FEMA flood maps may exist.

FEMA issues a community identification number (CID) to every community including non-participating ones. However, FEMA has not issued a CID to any State. Without a CID, a State can not be a participating community in the NFIP, therefore a FEMA mapped SFHA cannot exist on state owned lands and public rights of way (ROW), and the State cannot be held to the requirements of 44CFR60.

Section 4 — TxDOT and the NFIP

Texas and the NFIP

Texas as a State is a non-participating community within the NFIP, and TxDOT is an entity agency of the State of Texas. Therefore, the requirements of 44CFR60 do not apply to TxDOT and no SFHAs exist on TxDOT ROW. A TxDOT office may choose to assist an FPA within the office's ability, but a community's floodplain requirements are not binding on TxDOT because TxDOT is an agency of the State of Texas. The FPA may request or advise certain elements in the project design, but has no approval authority over TxDOT project design or placement.

A FIRM may show a TxDOT ROW crossing a community's FEMA mapped SFHA, but in fact the community's SFHA stops at one ROW line and continues after the other ROW line. Even if the ROW is within the limits of a municipality or a county, the TxDOT ROW is actually property of the State of Texas and is therefore “outside” of the municipality or county. The situation is analogous to a stream passing out of a municipality's corporate limits into a non-participating community and back into the participating community. Actions in the non-participating community do not fall under the purview of 44CFR60.

As mentioned in NFIP Roles: Participation, FEMA encourages communities to adopt ordinances and rules that are more stringent than the minimum (i.e. stricter criteria). These more restrictive technical criteria present a problem of equitable funding of projects among all communities. TxDOT cannot favor one community over another simply because one community has stricter criteria than the other community. A TxDOT office may choose to assist a local FPA within the office's ability, but a local community's floodplain requirements are not binding on TxDOT. There are instances where the local stricter criteria do not place a hardship on TxDOT. For example, there are times where TxDOT criteria call for a 2% AEP (50-year) bridge and the 1% AEP water surface coincidentally falls within the available freeboard. This is still considered a 2% AEP bridge, yet it can also be considered to meet a request for a bridge to pass the 1% AEP flow.

TxDOT and Local Floodplain Regulations

Texas Attorneys General have ruled in a series of opinions [JM-117(1983); C-690(1966); JM-1035(1989)] that state agencies are prohibited from applying for permits from subordinate jurisdictions. Although the opinions were written originally to address the issue of electrical utilities and cities attempting to require TxDOT contractors to apply for permits for roadway illumination installations, the opinion applies equally to community floodplain permit requirements. A court decision speaking to this concept is City of Houston v. Houston ISD, 436 S.W. 2ND 568, at 572, “Properties of the State are excluded as a matter of law from the application of City building regulations.”

Permits versus FPA Notification

FPAs normally require permits for any work in the mapped floodplain in order to prevent any improper development and to track all activity and changes which will affect the floodplain. TxDOT recognizes that activities such as bridge and culvert construction or roadway overlays can have an effect on the floodplain, and that the FPA needs the information. Since TxDOT cannot apply for a permit, as discussed in "TxDOT and Local Floodplain Regulations," TxDOT requires the designing office to send a copy of the plan set along with any and all supporting hydrologic and hydraulic computations and program files to the local FPA. (See FPA Notification Details.) The designer should place a note on the hydrologic plan sheet stating, "H&H files were sent to the local Floodplain Administrator ________name_________ on ___date____." The date is the date the documents, plans, and files are sent to the local FPA.

Hydraulic Structures versus Insurable Structures

One source of confusion for the communities is 44CFR 60, Subpart B (section 60.11 - 13) titled Requirements for State Flood Plain Management Regulations, which applies to structures on State owned properties. Structure is defined in 44CFR59.1 as a walled, roofed building. Hydraulic structures such as bridges, culverts, and storm drains are neither included nor considered because for the purpose of the NFIP they are not insurable structures. Subpart B does apply to TxDOT office buildings, maintenance or repair shops, and highway rest area facilities.

Off System Structures

Bridges and culverts that belong to a county or municipality are not on the TxDOT ROW and are therefore within a community's floodplain. The FPA may attempt to require a permit from TxDOT when TxDOT controls or oversees the funds used to repair or replace structures in the community's floodplain. The community is responsible for any permits because the structure belongs to the community. In this situation, TxDOT is acting on the behalf of the community as its consultant. Just as the community would not require a consulting firm it hired to obtain and be responsible for the permits, the community can not hold TxDOT responsible for these same permits.

Liability

TxDOT project engineers and contract managers should understand that compliance with the NFIP and coordination with the FPA does not relieve TxDOT of the requirements of civil law. The Texas Water Code section 11.086 prohibits diversion or impoundment which will harm another property. Roadway and bridge designers must pay attention to surrounding conditions of building placements, property access, and runoff patterns to assure that the project will not cause adverse impacts to adjacent properties. Every designer for TxDOT has an obligation not to put TxDOT in a position of liability due to their design.

The need for documentation that supports and protects TxDOT from lawsuits or liability claims is the reason for the inclusion of the required hydrologic and hydraulic study details in the plan set as engineering documents on sealed sheets.

Section 5 — Hydrologic and Hydraulic Studies

Before starting the design

The roadway designer should visit the site to observe existing conditions and limitations. The designer must avoid designs that may cause increased flooding or increased damage to adjacent, downstream, or upstream properties. TxDOT can be held liable for damages even if a design only increases the frequency of flooding to properties that are already regularly inundated. Overlays in areas where the water overtops the roadway may require milling prior to resurfacing in order to maintain the same elevation and not increase flooding. Problems caused by culverts and bridges can usually be avoided by alternative designs that will not significantly increase the cost. Ultimately, TxDOT must not be perceived as causing flooding to adjacent, downstream, or upstream property owners.

If the project is within a participating community

The designer should contact the local FPA.   The FPA can be a source of valuable data to aid in the hydraulic design, such as existing flooding issues that the project may exacerbate if unaddressed or may alleviate with minor modification to the project. The FPA may have information or a localized study establishing an approximate BFE for Zone A, or may have the complete study data used to establish BFEs for the entire zone. The FPA may also have knowledge of a CLOMR submitted to FEMA by others, or other changes to the area.

If the project is within or crossing an SFHA

The designer should determine the SFHA zone designation with the current, correct FIRM. The FIRMs are available at www.msc.fema.gov. Locate the project site on the FIRM to determine whether it crosses or is in any SFHAs Zoned A, AE (A1-30), AO, AH, V, or VE (V1-30). Actions for the various SFHAs are as follows:

- **Zone A** - No BFE or only an approximate BFE is available.   A full hydrologic and hydraulic analysis and coordination with the local FPA is required regardless of results (rise, no rise or lowering of the BFE). Any rise in the BFE which extends beyond the project ROW requires discussion with the FPA to make sure the rise is acceptable. The rise must not cause damage to properties in order to be acceptable. The FPA may also be able to use the TxDOT hydrologic and hydraulic analysis to establish informal BFEs in that area.

  The designer should compare the flood footprint from the design hydraulic analysis with the FIRM Zone A footprint, and explore any serious discrepancies.

- **Zone AE** - BFE's have been established by formal hydrologic and hydraulic study. The steps in analyzing a Zone AE are as follows:

A.

1. **Existing Model:** The designer is required to obtain from the FPA or FEMA the effective hydraulic model or study data to use for the analysis. Consult DES-HYD for instructions on ordering the model data. If no model is available, go to **B** below. The entire length of the model usually does not need to be used; the designer should select the appropriate reach for the analysis. However, the selected reach shall fit seamlessly into the entire model; that is, water surface elevations and velocities must match exactly at both the downstream and upstream ends of the selected reach.

2. **Updated Model (if necessary):** Older studies that were modeled in HEC-2, WSPRO, or some other program should be converted to HEC-RAS. Corrections to the model must be made because of differences in modeling practices, such as the tendency of piers to be modeled as ground points in HEC-2. Differences in programmed algorithms within the software will cause differences in the water surface elevations. The reasons for the differences are explained in detail in a memorandum from FEMA dated April 30, 2001, titled "Policy for use of HEC-RAS in the NFIP." FEMA requires that the revised and unrevised BFEs match within 0.5 foot at the bounding cross sections between the output of the older model and HEC-RAS model (44 CFR 65.6(a)(2)).

3. **Corrected Existing Model:** The designer should examine the effective (or updated) hydraulic model for errors such as unrealistic or incorrect flows (Q), inaccurate survey data, missing bridges, and bridges where hydraulically inefficient rails were excluded in the model. The model should be labeled "corrected effective" after the corrections have been made. The water surface elevations in the corrected effective model may be higher or lower than in the existing effective model, but the designer has no requirement to file a CLOMR/LOMR other than supplying the FPA with a copy of the corrected model.

4. **Proposed Model:** The designer should then utilize the corrected effective hydraulic model to include the new structure. The model should be labeled "proposed".

B.

Sometimes the effective hydraulic model or its data are not available, or the data are unreadable and therefore unusable. In such situations, a new complete HEC-RAS model to approximate the current model must be developed as follows:

1. The designer must obtain any available information from the FIS and FIRM, such as flowrate(s), cross section topography, floodplain footprint, and BFE's at pertinent cross-sections.

2. The designer must use the available information to develop a hydraulic model as if a Zone AE was not present.

3. In a Zone AE without a floodway, the designer must compare the output from the new HEC-RAS model with the published BFE's from the FIS and the floodplain footprint from the FIRM. The model must match the published BFEs within 0.10 foot. If only the FIRM is avail-

able or if the FIS has no BFEs listed, the model must match the BFEs on the FIRM to 0.5 foot or less.

4. In a Zone AE with a floodway, flow should be confined to the floodway limits and the elevations match the published floodway elevations to within 0.10 foot (23 CFR 650A, Attach 2).

5. Once the new HEC-RAS model meets the allowable differences, the model should be labeled "replacement effective" model and steps A1 through A4 above should be used to progress toward a "proposed" model.

If the FEMA data or model is not used, the situation and process must be fully explained in the hydrologic and hydraulic report and noted on the plan sheet. The argument that the hydraulic data is only available in hard copy and must be manually entered is not an acceptable explanation for not using the FEMA model.

On occasion, a model is obtained which contains numerous errors throughout the entire reach. TxDOT is not responsible for quality control and comprehensive updating of the NFIP models. In these cases, either limit the reach used for the TxDOT study to the shortest length practical, limit the corrections to the cross-sections closest to the bridge, or both. Consult the DES-HYD if a corrected or updated hydraulic model appears to be warranted.

If the flowrates used in the existing model appear to be in error, the designer is encouraged to develop a new hydrologic model and compare the results. If the existing flowrates are not used, the justification must be explained in the hydrologic and hydraulic report and noted on the plan sheet. The hydrologic process must also be included in the report. Designers shall not, in any circumstance, develop a new model without documented justification.

- **Zone AE with Regulatory Floodway** - Once a regulatory floodway, or floodway, has been determined and mapped on a FIRM, FEMA requires a study to prove no rise of the water surface for any work in a floodway. The designer must obtain the floodway model to determine the limits of the floodway and the effect of the proposed structure on the floodway. The floodway model is not to be confused with the current effective floodplain model. The floodway model is almost identical to the floodplain model with the exception that it contains the floodway boundaries. This model should be obtained along with the floodplain model.

  If the entire structure of a bridge, including abutments, bridge superstructure and piers, can be documented to be well outside the limits of a regulatory floodway, and is above the BFE in the floodway, then the current effective hydraulic model may not need to be obtained. The NFIP allows that any work, including fill, in the flood fringe or above the floodway does not require a study because the study establishing the regulatory floodway assumed that the flood fringe was already filled. If the bridge design meets these criteria, the normally required analysis can be replaced by a prominently placed note in the plans. However, if there are insurable structures in the floodplain, TxDOT requires that the roadway designer acquire and modify the floodplain model as outlined in **Zone AE** above.

- **Zone AH** - Places of no outflow, such as playas, in which the BFE has been determined are labeled Zone AH. Structures in playas are equalizers and as such, the bridges and culverts typically need no hydraulic modeling. However, the designer is required to calculate how much the BFE will be raised because the roadway and structure will reduce storage in the playa if any roadway work will be in the Zone AH and below the BFE.
- **Zone AO** - Average flood depths are given instead of a flood elevation. The designer should examine the source of the flooding to make sure the project will not trap flood waters or block drainage. The design may require relief structures in any elevated roadway or extended bridge approaches.
- **Coastal Zones V, VE (V1-30), A, and AE** - Coastal zones are flooded by the Gulf of Mexico storm waters instead of riverine flows. The FEMA modeling for coastal zones is not the same as for riverine modeling. The designer does not need to acquire the model but must consider tidal flows, wave actions, and storm precipitation. The designer should make sure the project will not trap flood waters or block drainage.

High Bridges

For some bridges, the geometry is such that the bridge either spans the entire floodplain or the low chord is well above the BFE. In these situations, or where the proposed work can be documented to be well above the BFE and outside the limits of the floodplain, a hydraulic analysis may not be necessary. The normally required analysis can be replaced by a prominently placed note in the plans.

Figure 5-1. Fred Hartman Bridge facing Baytown from the ship channel, mostly above the BFE.

Section 6 — Hydrologic and Hydraulic Results

Changes to the BFE

Repair, extension, or replacement of any particular bridge or culvert may cause the BFE to be raised, lowered, or not changed at all. Lowering the water surface elevation usually doesn't cause any adverse impacts, but the site should be visited to confirm that there will be no resulting problems. Many bridge replacements result in a lowered water surface elevation because of reduced pier sets, enlarged openings, raised low chords, or improved channels.

Raising the water surface elevation requires examination of the adjacent properties to assure that the change will not cause any adverse impacts. A rise usually can be considered as having no impact if the rise is contained within the TxDOT ROW. A rise which extends outside of the ROW may be considered either insignificant or significant. A severe rise in an uninhabited area, not excessively flooding adjacent properties, not damaging the stream banks, and not blocking access to properties might be considered insignificant. A rise in an urban area that is contained within the banks without damage to the stream banks or back flooding the sewers may also be considered insignificant. However, in each situation, the effects have to be examined at the site to confirm that the rises are insignificant.

Range of Frequencies

The NFIP only considers the 1% AEP. However, the designer should analyze the effect of a proposed structure on the full range of AEP water surface elevations (the 50%, 20%, 10%, 4%, and 2% as well as the 1% AEP). The proposed structure may not cause any problem at the design AEP flow, but may cause a problem at one of the other frequencies. Analyzing only for the 1% and design AEP flows may fail to reveal these problems. See Liability above.

Some FEMA models contain only the 1% AEP. The designer will need to develop the full range of flows (50%, 20%, 10%, 4%, and 2% AEP) using a suitable method.

Conditional Letter Of Map Revision (CLOMR)/Letter Of Map Revision (LOMR)

Changes to the water surface elevation in studied areas are usually reported to FEMA by submission of a CLOMR and LOMR. A CLOMR, if required, is submitted to FEMA prior to initiating work to receive approval for the project design and the impacts on the floodplain; a LOMR is submitted after the project has been completed. The approval of a CLOMR application by FEMA requires significant time (six to 12 months), which needs to be factored into the required design time to prevent delay of the project letting.

A community may attempt to require TxDOT to submit a CLOMR and a LOMR, based on a 44CFR 60.3 requirement for participating communities to notify FEMA of all changes to the BFE. However, the requirement does not apply to TxDOT because the State of Texas is not a participating community in the NFIP, as discussed above. Additionally, Non-regulatory Supplement 23 CFR 650A, Attachment 2 states in the first paragraph, "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." See also Permit versus FPA Notification. TxDOT provides the technical data to the FPA, as required in 44CFR 60.3, through FPA Notification to enable the FPA to submit it to FEMA.

TxDOT will prepare and submit a CLOMR/LOMR for very few situations, as described below. This is more likely to happen on large projects which involve major changes to the floodplain, such as channel realignment or channel restoration. However, the designer must consult the DES-HYD before proceeding with the CLOMR/LOMR process.

ALL CLOMRS MUST BE REVIEWED BY DES-HYD BEFORE SUBMITTAL TO THE FPA.

A CLOMR may be prepared and submitted in the following limited circumstances:

TxDOT will not file a CLOMR to better define the floodplain for projects in a Zone A. TxDOT will not file a CLOMR to redefine a Zone AE where TxDOT improvements drop the water surface elevation of the BFE, or otherwise change the floodplain footprint, so as to encourage additional development. Improvements in the floodplain that may result from TxDOT projects are considered incidental. However, as for any project, TxDOT will still provide its plans and studies in these cases for the community records.

- An encroachment on a floodway of a SFHA results in a rise (not contained within the TxDOT ROW) of the base flood elevation in a Zone AE with Floodway. Alternatively, a larger bridge or culvert may be preferable.
- Increases in water surface elevations (not contained within the TxDOT ROW) exceed the usually available (or remainder of) a cumulative 1-foot rise in a Zone AE. Alternatively, a larger bridge or culvert may be preferable.
- An increase in water surface elevation (not contained within the TxDOT ROW) results in a significant increase of the horizontal extent of the floodplain footprint in unusually flat areas and in a Zone AE. Alternatively, a larger bridge or culvert may be preferable.
- A major channel relocation in a Zone AE that is outside the TxDOT ROW.
- Where a relief structure is outside the SFHA containing the main structure and a risk exists of development immediately downstream of the relief structure that might interfere with the operation of the relief structure. Alternatively, a larger main structure may be preferable.

TxDOT will not file a CLOMR to better define the floodplain for projects in a Zone A. TxDOT will not file a CLOMR to redefine a Zone AE where TxDOT improvements drop the water surface ele-

vation of the BFE, or otherwise change the floodplain footprint, so as to encourage additional development. Improvements in the floodplain that may result from TxDOT projects are considered incidental. However, as for any project, TxDOT will still provide its plans and studies in these cases for the community records.

FPA Notification Details

The TxDOT office where the plans are being developed shall forward a copy of the plan sheets along with all hydrologic and hydraulic analyses, reports, and electronic copies of the models to the local FPA, or FPAs if more than one community is involved. The submitted FPA Notification must be complete enough for the community to apply for a Conditional Letter of Map Revision (CLOMR)/Letter of Map Revision (LOMR) if the local FPA deems it necessary; it is not intended to be the CLOMR/LOMR submittal itself. The purpose of the FPA Notification is to document to the FPA any changes or non-changes to the BFE. FPA Notification is required no later than when the project is submitted for letting but should be accomplished as soon as the hydraulic design is complete.

The transmittal letter to the FPA should state that the attached information is being sent for the FPA's floodplain records. It must not ask for the FPA's concurrence, approval, or consent. The FPA has no authority to approve, disapprove, or change the modeling or design but may offer valuable guidance to the designer.

The designer should be aware that, while the designer's office is coordinating with a community, it does not mean that the office is automatically coordinating with the FPA. The FPA may be in a completely different department such as Health, and not Engineering. This is a source of confusion, and may result in a project having problems if the FPA is not involved until late in the design. The designer should verify that the FPA is involved and recommend to the community engineer that the local FPA be involved early in the design process.

Communities Without an FPA

A community which is not participating in the NFIP does not have an FPA. The FPA Notification documentation should be sent to the county engineer or county judge, if in an unincorporated area, or the municipal engineer if in an incorporated non-participating city, town, village, tribe, or municipal utility district (MUD). By coordinating with the community, TxDOT may prevent public concerns from growing into problems, and may raise the awareness of flood issues within the community.

Chapter 6 — Hydraulic Principles

Contents:

Section 1 — Open Channel Flow

Introduction

This chapter describes concepts and equations that apply to the design or analysis of open channels and conduit for culverts and storm drains. Refer to the relevant chapters for specific procedures.

Continuity and Velocity

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the following form:

$$Q = A_1 v_1 = A_2 v_2$$

Equation 6-1.

where:

Q = discharge (cfs or m^3/s)

A = flow cross-sectional area (sq. ft. or m^2)

v = mean cross-sectional velocity (fps or m/s, perpendicular to the flow area).

The superscripts *1* and *2* refer to successive cross sections along the flow path.

As indicated by the Continuity Equation, the average velocity in a channel cross-section, (v) is the total discharge divided by the cross-sectional area of flow perpendicular to the cross-section. It is only a general indicator and does not reflect the horizontal and vertical variation in velocity.

Velocity varies horizontally and vertically across a section. Velocities near the ground approach zero. Highest velocities typically occur some depth below the water surface near the station where the deepest flow exists. For one-dimensional analysis techniques such as the Slope Conveyance Method and (Standard) Step Backwater Method (see Chapter 7), ignore the vertical distribution, and estimate the horizontal velocity distribution by subdividing the channel cross section and computing average velocities for each subsection. The resulting velocities represent a velocity distribution.

Channel Capacity

Most of the departmental channel analysis procedures use the Manning's Equation for uniform flow (Equation 6-2) as a basis for analysis:

$$v = \frac{z}{n} R^{2/3} S^{1/2}$$

Equation 6-2.

where:

v = Velocity in cfs or m^3/sec

z = 1.486 for English measurement units, and 1.0 for metric

n = Manning's roughness coefficient (a coefficient for quantifying the roughness characteristics of the channel)

R = hydraulic radius (ft. or m) = A / WP

WP = wetted perimeter of flow (the length of the channel boundary in direct contact with the water) (ft. or m)

S = slope of the energy gradeline (ft./ft. or m/m) (For uniform, steady flow, S = channel slope, ft./ft. or m/m).

Combine Manning's Equation with the continuity equation to determine the channel uniform flow capacity as shown in Equation 6-3.

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

Equation 6-3.

where:

Q = discharge (cfs or m^3/s)

z = 1.486 for English measurement units, and 1.0 for metric

A = cross-sectional area of flow (sq. ft. or m^2).

For convenience, Manning's Equation in this manual assumes the form of Equation 6-3. Since Manning's Equation does not allow a direct solution to water depth (given discharge, longitudinal slope, roughness characteristics, and channel dimensions), an indirect solution to channel flow is necessary. This is accomplished by developing a stage-discharge relationship for flow in the stream.

All conventional procedures for developing the stage-discharge relationship include certain basic parameters as follows:

- geometric descriptions of typical cross section
- identification and quantification of stream roughness characteristics
- a longitudinal water surface slope.

You need careful consideration to make an appropriate selection and estimation of these parameters.

Conveyance

In channel analysis, it is often convenient to group the channel cross-sectional properties in a single term called the channel conveyance (K), shown in Equation 6-4.

$$\mathbf{K = \frac{z}{n} A R^{2/3}}$$

Equation 6-4.

Manning's Equation can then be written as:

$$\mathbf{Q = K S^{1/2}}$$

Equation 6-5.

Conveyance is useful when computing the distribution of overbank flood flows in the cross section and the flow distribution through the opening in a proposed stream crossing.

Energy Equations

Assuming channel slopes of less than 10 percent, the total energy head can be shown as Equation 6-6.

$$H = \frac{P}{\gamma_w} + z + \alpha \frac{v^2}{2g}$$

Equation 6-6.

where:

H = total energy head (ft. or m)

P = pressure (lb./sq.ft. or N/m^2)

γ_w = unit weight of water (62.4 lb./cu.ft. or 9810 N/m^3)

z = elevation head (ft. or m)

$\frac{v^2}{2g}$ = average velocity head, h_v (ft. or m)

g = gravitational acceleration (32.2 ft./ s^2 or 9.81 m/s^2)

α = kinetic energy coefficient, as described in Kinetic Energy Coefficient Computation section *v* = mean velocity (fps or m/s).

In open channel computations, it is often useful to define the total energy head as the sum of the specific energy head and the elevation of the channel bottom with respect to some datum.

$$H = z + d + \alpha \frac{v^2}{2g}$$

Equation 6-7.

where:

d = depth of flow (ft. or m).

For some applications, it may be more practical to compute the total energy head as a sum of the water surface elevation (relative to mean sea level) and velocity head.

$$H = WS + \alpha \frac{v^2}{2g}$$

Equation 6-8.

where:

WS = water-surface elevation or stage (ft. or m) = z + d.

Specific Energy Equation. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel, the specific energy, E, becomes the sum of the depth of flow and velocity head.

$$E = d + \alpha \frac{v^2}{2g}$$

Equation 6-9.

Kinetic Energy Coefficient. Some of the numerous factors that cause variations in velocity from point to point in a cross section are channel roughness, non-uniformities in channel geometry, bends, and upstream obstructions.

The velocity head based on average velocity does not give a true measure of the kinetic energy of the flow because the velocity distribution in a river varies from a maximum in the main channel to essentially zero along the banks. Get a weighted average value of the kinetic energy by multiplying average velocity head by the kinetic energy coefficient (α). The kinetic energy coefficient is taken to have a value of 1.0 for turbulent flow in prismatic channels (channels of constant cross section, roughness, and slope) but may be significantly different than 1.0 in natural channels. Compute the kinetic energy coefficient with Equation 6-10:

$$\alpha = \frac{\sum\left(Q_i v_i^2\right)}{Qv^2} = \frac{\sum\left[K_i\left(K_i / A_i\right)^2\right]}{K_t\left(K_t / A_t\right)^2}$$

Equation 6-10.

where:

- v_i = average velocity in subsection (ft./s or m/s) (see Continuity Equation section)
- Q_i = discharge in same subsection (cfs or m^3/s) (see Continuity Equation section)
- Q = total discharge in channel (cfs or m^3/s)
- v = average velocity in river at section or Q/A (ft./s or m/s)
- K_i = conveyance in subsection (cfs or m^3/s) (see Conveyance section)
- A_i = flow area of same subsection (sq. ft. or m^2)
- K_t = total conveyance for cross-section (cfs or m^3/s)
- A_t = total flow area of cross-section (sq. ft. or m^2).

In manual computations, it is possible to account for dead water or ineffective flows in parts of a cross section by assigning values of zero or negative numbers for the subsection conveyances. The kinetic energy coefficient will, therefore, be properly computed. In computer models, however, it is not easy to assign zero or negative values because of the implicit understanding that conveyance and discharge are similarly distributed across a cross section. This understanding is particularly important at bends, embankments, and expansions, and at cross sections downstream from natural and manmade constrictions. The subdivisions should isolate any places where ineffective or upstream flow is suspected. Then, by omitting the subsections or assigning very large roughness coefficients to them, a more realistic kinetic energy coefficient is computed.

In some cases, your calculations may show kinetic energy coefficients in excess of 20, with no satisfactory explanations for the enormous magnitude of the coefficient. If adjacent cross sections have comparable values or if the changes are not sudden between cross sections, such values can be accepted. If the change is sudden, however, make some attempt to attain uniformity, such as using more cross sections to achieve gradual change, or by re-subdividing the cross section.

Energy Balance Equation

The Energy Balance Equation, Equation 6-1, relates the total energy of an upstream section (2) along a channel with the total energy of a downstream section (1). The parameters in the Energy Equation are illustrated in Figure 6-1. Equation 6-1 now can be expanded into Equation 6-11:

$$\mathbf{z}_2 + \mathbf{d}_2 + \alpha_2 \frac{\mathbf{v}_2^2}{\mathbf{2g}} = \mathbf{z}_1 + \mathbf{d}_1 + \alpha_1 \frac{\mathbf{v}_1^2}{\mathbf{2g}} + \mathbf{h}_f + \textbf{other losses}$$

Equation 6-11.

where:

z = elevation of the streambed (ft. or m)

d = depth of flow (ft. or m)

α = kinetic energy coefficient

v = average velocity of flow (fps or m/s)

h_f= friction head loss from upstream to downstream (ft. or m)

g = acceleration due to gravity = 32.2 ft/ s^2 or 9.81 m/s^2.

The energy grade line (EGL) is the line that joins the elevations of the energy head associated with a water surface profile (see Figure 6-1).

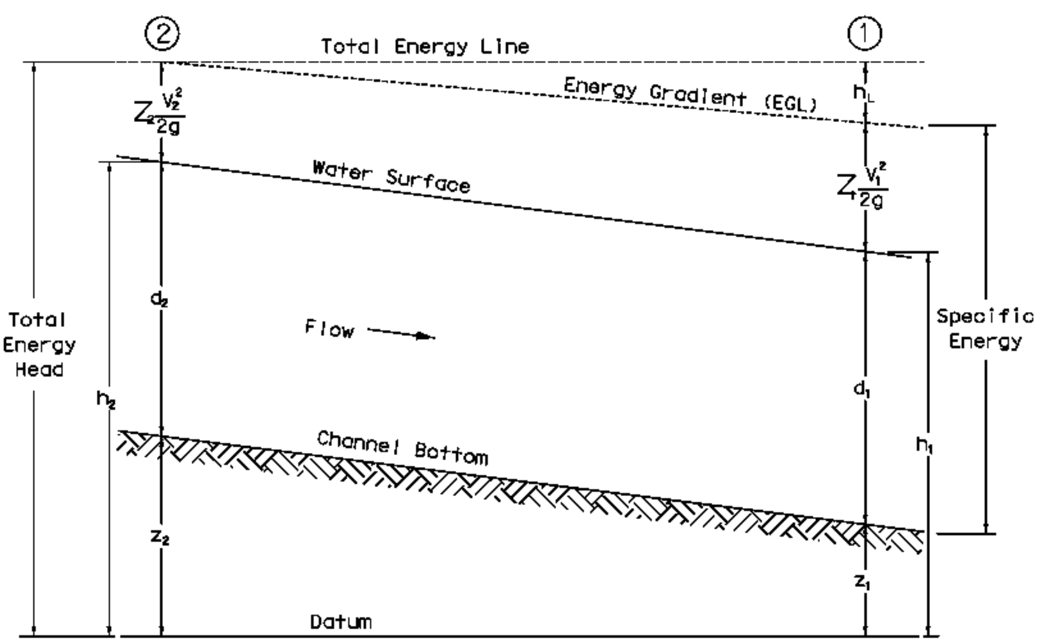


Figure 6-1. EGL for Water Surface Profile

Depth of Flow

Uniform depth (d_u) of flow (sometimes referred to as normal depth of flow) occurs when there is uniform flow in a channel or conduit. Uniform depth occurs when the discharge, slope, cross-sectional geometry, and roughness characteristics are constant through a reach of stream. See Slope Conveyance Method for how to determine uniform depth of flow in an open channel (Chapter 7).

By plotting specific energy against depth of flow for constant discharge, a specific energy diagram is obtained (see Figure 6-2). When specific energy is a minimum, the corresponding depth is critical depth (d_c). Critical depth of flow is a function of discharge and channel geometry. For a given discharge and simple cross-sectional shapes, only one critical depth exists. However, in a compound channel such as a natural floodplain, more than one critical depth may exist.

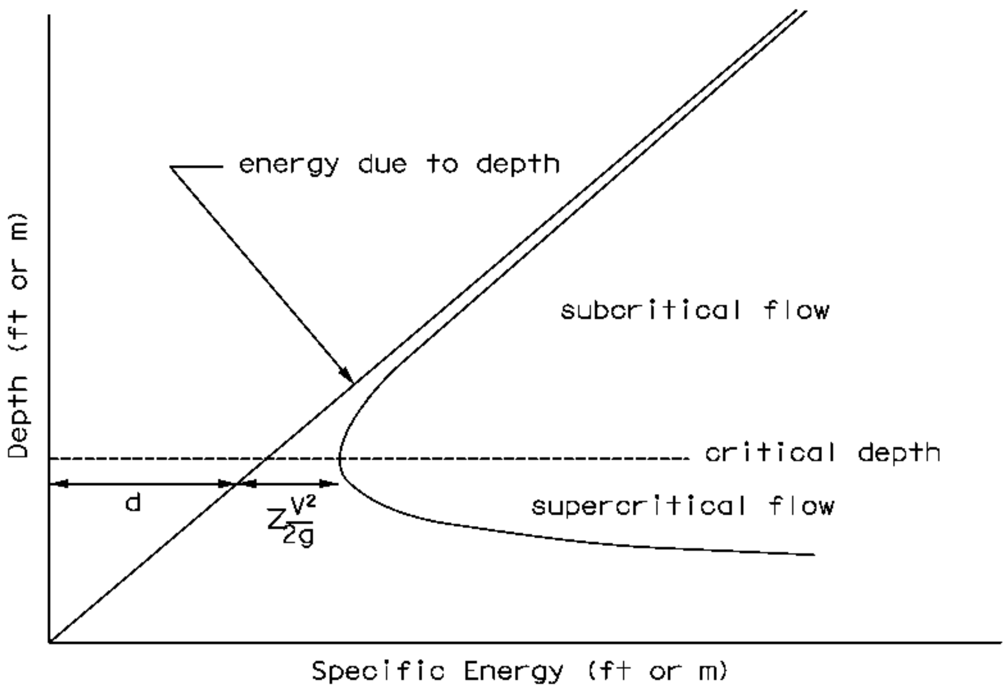


Figure 6-2. Typical Specific Energy Diagram

You can calculate critical depth in rectangular channels with the following Equation 6-12:

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

Equation 6-12.

where:

q = discharge per ft. (m) of width (cfs/ft. or $m^3/s/m$).

You can determine the critical depth for a given discharge and cross section iteratively with Equation 6-13:

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

Equation 6-13.

where:

T_c = water surface width for critical flow (ft. or m)

A_c = area for critical flow (sq. ft. or m^2).

Froude Number

The Froude Number (F_r) represents the ratio of inertial forces to gravitational forces and is calculated using Equation 6-14.

$$\mathbf{F_r = \frac{v}{\sqrt{g\,d_m}}}$$

Equation 6-14.

where:

v = mean velocity (fps or m/s)

g = acceleration of gravity (32.2 ft/ s^2 or 9.81 m/s^2)

d_m = hydraulic mean depth = A / T (ft. or m)

A = cross-sectional area of flow (sq. ft. or m^2)

T = channel top width at the water surface (ft. or m).

The expression for the Froude Number applies to any single section of channel. The Froude Number at critical depth is always 1.0.

Flow Types

Several recognized types of flow are theoretically possible in open channels. The methods of analysis as well as certain necessary assumptions depend on the type of flow under study. Open channel flow is usually classified as uniform or non-uniform, steady or unsteady, or or critical or supercritical.

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

Steady, Uniform Flow. Steady flow implies that the discharge at a point does not change with time, and uniform flow requires no change in the magnitude or direction of velocity with distance along a streamline such that the depth of flow does not change with distance along a 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 practical highway applications, the flow is steady, and changes in width, depth, or direction (resulting in non-uniform flow) are sufficiently small so that flow can be considered uniform. A further assumption of rigid, uniform boundary conditions is necessary to satisfy the conditions of constant flow depth along the channel. Alluvial, sand bed channels do not exhibit rigid boundary characteristics.

Steady, Non-uniform Flow. Changes in channel characteristics often occur over a long distance so that the flow is non-uniform and gradually varied. Consideration of such flow conditions is usually reasonable for calculation of water surface profiles in Texas streams, especially for the hydraulic design of bridges.

Subcritical/Supercritical Flow. Most Texas streams flow in what is regarded as a subcritical flow regime. Subcritical flow occurs when the actual flow depth is higher than critical depth. A Froude Number less than 1.0 indicates subcritical flow. This type of flow is tranquil and slow and implies flow control from the downstream direction. Therefore, the analysis calculations are carried out from downstream to upstream. In contrast, supercritical flow is often characterized as rapid or shooting, with flow depths less than critical depth. A Froude Number greater than 1.0 indicates supercritical flow. The location of control sections and the method of analysis depend on which type of flow prevails within the channel reach under study. A Froude number equal to, or close to, 1.0 indicates instability in the channel or model. A Froude number of 1.0 should be avoided if at all possible.

Cross Sections

A typical cross section represents the geometric and roughness characteristics of the stream reach in question. Figure 6-3 is an example of a plotted cross section.

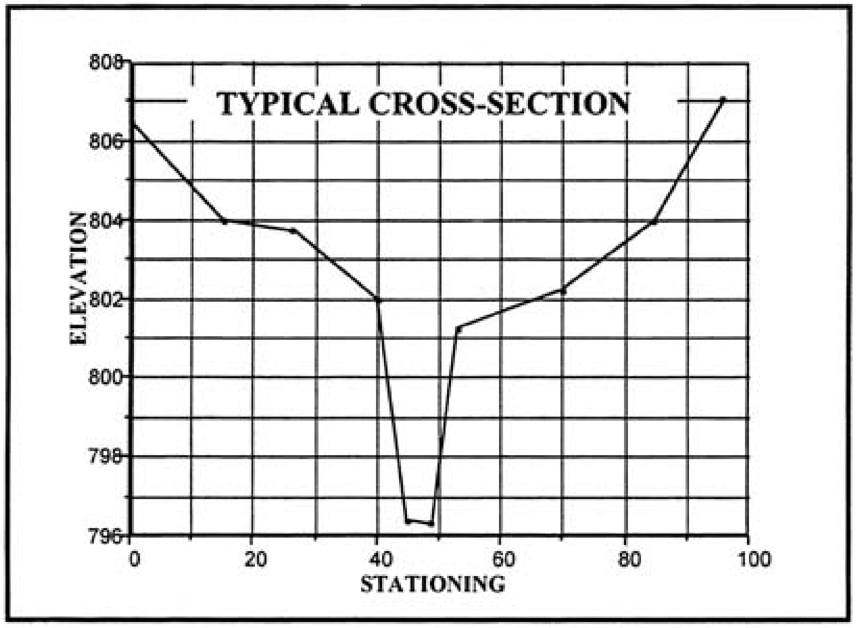


Figure 6-3. Plotted Cross Section

Most of the cross sections selected for determining the water surface elevation at a highway crossing should be downstream of the highway because most Texas streams exhibit subcritical flow. Calculate the water surface profile through the cross sections from downstream to upstream. Generate enough cross sections upstream to determine properly the extent of the backwater created by the highway crossing structure. See Chapter 4 for details on cross sections.

Roughness Coefficients

All water channels, from natural stream beds to lined artificial channels, exhibit some resistance to water flow, and that resistance is referred to as roughness. Hydraulic roughness is not necessarily synonymous with physical roughness. All hydraulic conveyance formulas quantify roughness subjectively with a coefficient. In Manning's Equation, the roughness coefficients, or n-values, for Texas streams and channels range from 0.200 to 0.012; values outside of this range are probably not realistic.

Determination of a proper n-value is the most difficult and critical of the engineering judgments required when using the Manning's Equation.

You can find suggested values for Manning's roughness coefficient ("n" values) in design charts such as the one shown in the file named nvalues.doc (NVALUES). Any convenient, published design guide can be referenced for these values. Usually, reference to more than one guide can be productive in that more opinions are collected. You can find a productive and systematic approach for this task in the FHWA publication *TS-84-204, Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*.

However inexact and subjective the n-value determination may be, the n-values in a cross section are definite and unchangeable for a particular discharge and flow depth. Therefore, once you have carefully chosen the n-values, do not adjust them just to provide another answer. If there is uncertainty about particular n-value choices, consult a more experienced designer.

In some instances, such as a trapezoidal section under a bridge, the n-value may vary drastically within a section, but you should not subdivide the section. If the n-value varies as such, use a weighted n-value (n_w). This procedure is defined by Equation 6-15 as follows:

$$n_w = \frac{\sum (n \, WP)}{\sum WP}$$

Equation 6-15.

where:

WP = subsection wetted perimeter

n = subsection n-value.

Subdividing Cross Sections

Because any estimating method involves the calculation of a series of hydraulic characteristics of the cross section, arbitrary water-surface elevations are applied to the cross section. The computation of flow or conveyance for each water-surface application requires a hydraulic radius, as seen in Figure 6-4. The hydraulic radius is intended as an average depth of a conveyance. A hydraulic radius and subsequent conveyance is calculated under each arbitrary water surface elevation. If there is significant irregularity in the depth across the section, the hydraulic radius may not accurately represent the flow conditions. Divide the cross section into sufficient subsections so that realistic hydraulic radii are derived.

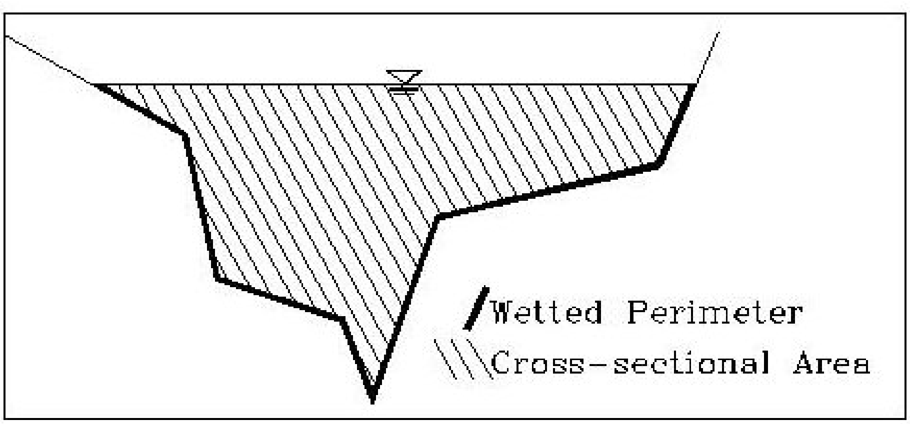


Figure 6-4. Cross Section Area and Wetted Perimeter

Subsections may be described with boundaries at changes in geometric characteristics and changes in roughness elements (see Figure 6-5). Note that the vertical length between adjacent subsections is not included in the wetted perimeter. The wetted perimeter is considered only along the solid boundaries of the cross sections, not along the water interfaces between subsections.

Adjacent subsections may have identical n-values. However, the calculation of the subsection hydraulic radius will show a more consistent pattern as the tabulation of hydraulic characteristics of the cross section is developed.

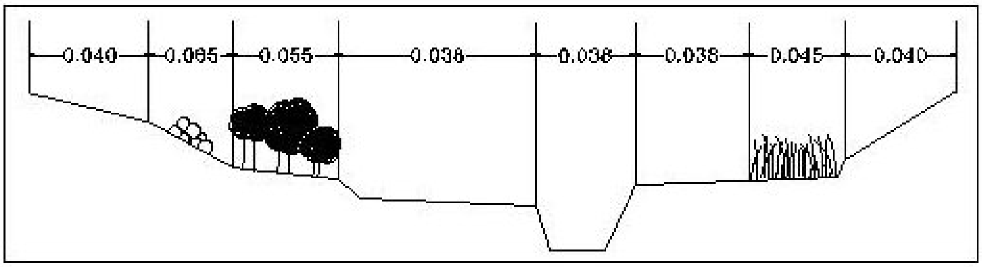


Figure 6-5. Subdividing With Respect to Geometry and Roughness

Subdivide cross sections primarily at major breaks in geometry. Additionally, major changes in roughness may call for additional subdivisions. You need not subdivide basic shapes that are approximately rectangular, trapezoidal, semicircular, or triangular.

Subdivisions for major breaks in geometry or for major changes in roughness should maintain these approximate basic shapes so that the distribution of flow or conveyance is nearly uniform in a subsection.

Importance of Correct Subdivision

The importance of proper subdivision as well as the effects of improper subdivision can be illustrated dramatically. Figure 6-6 shows a trapezoidal cross section having heavy brush and trees on the banks and subdivided near the bottom of each bank because of the abrupt change of roughness.

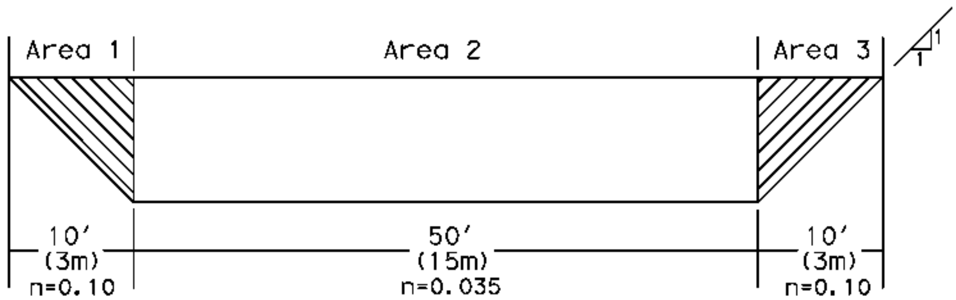


Figure 6-6. Subdivision of a Trapezoidal Cross Section

The conveyance for each subarea is calculated as follows:

$\mathbf{A_1 = A_3 = 50\ ft^2}$	$\mathbf{A_1 = A_3 = 4.5\ m^2}$
$P_1 = P_3 = 14.14$ ft	$P_1 = P_3 = 4.24$ m
$R_1 = R_3 = A_1/P_1 = 3.54$ ft	$R_1 = R_3 = A_1/P_1 = 1.06$ m
$K_1 = K_3 = 1.486A_1R_1^{2/3}/n = 1724.4$ cfs	$K_1 = K_3 = A_1R_1^{2/3}/n = 46.8\ m^3/s$
$A_2 = 500\ ft^2$	$A_2 = 45\ m^2$
$P_2 = 50$ ft	$P_2 = 15$ m
$R_2 = A_2/P_2 = 10$ ft	$R_2 = A_2/P_2 = 3$ m
$K_2 = 1.486A_2R_2^{2/3}/n = 98534.3$ cfs	$K_2 = A_2R_2^{2/3}/n = 2674.4\ m^3/s$

When the subareas are combined, the effective n-value for the total area can be calculated.

$\mathbf{A_c = A_1 + A_2 + A_3 = 600\ ft^2}$	$\mathbf{A_c = A_1 + A_2 + A_3 = 54\ m^2}$
$P_c = P_1 + P_2 + P_3 = 78.28$ ft	$P_c = P_1 + P_2 + P_3 = 23.5$ m
$R_c = A_c/P_c = 7.66$ ft	$R_c = A_c/P_c = 2.3$ m
$K_T = K_1 + K_2 + K_3 = 101983$ cfs	$K_T = K_1 + K_2 + K_3 = 2768\ m^3/s$

$A_c = A_1 + A_2 + A_3 = 600\ ft^2$	**$A_c = A_1 + A_2 + A_3 = 54\ m^2$**
$n = 1.486 A_c R_c^{2/3} / K_T = 0.034$	$n = A_c R_c^{2/3} / K_T = 0.034$

A smaller wetted perimeter in respect to area abnormally increases the hydraulic radius (R = A / P), and this results in a computed conveyance different from that determined for a section with a complete wetted perimeter. As shown above, a conveyance (K_T) for the total area would require a composite n-value of 0.034. This is less than the n-values of 0.035 and 0.10 that describe the roughness for the various parts of the basic trapezoidal shape. Do not subdivide the basic shape. Assign an effective value of n somewhat higher than 0.035 to this cross section, to account for the additional drag imposed by the larger roughness of the banks.

At the other extreme, you must subdivide the panhandle section in Figure 6-7, consisting of a main channel and an overflow plain, into two parts. The roughness coefficient is 0.040 throughout the total cross section. The conveyance for each subarea is calculated as follows:

$A_1 = 195\ ft^2$	**$A_1 = 20\ m^2$**
$P_1 = 68$ ft	$P_1 = 21$ m
$R_1 = A_1/P_1 = 2.87$ ft	$R_1 = A_1/P_1 = 0.95$ m
$K_1 = 1.486 A_1 R_1^{2/3}/n = 14622.1$ cfs	$K_1 = A_1 R_1^{2/3}/n = 484.0\ m^3/s$
$A_2 = 814.5\ ft^2$	$A_2 = 75.5\ m^2$
$P_2 = 82.5$ ft	$P_2 = 24.9$ m
$R_2 = A_2/P_2 = 9.87$ ft	$R_2 = A_2/P_2 = 3.03$ m
$K_2 = 1.486 A_2 R_2^{2/3}/n = 139226.2$ cfs	$K_2 = A_2 R_2^{2/3}/n = 3954.2\ m^3/s$

The effective n-value calculations for the combined subareas are as follows:

$A_c = A_1 + A_2 = 1009.5\ ft^2$	**$A_c = A_1 + A_2 = 95.5\ m^2$**
$P_c = P_1 + P_2 = 150.5$ ft	$P_c = P_1 + P_2 = 45.9$ m
$R_c = A_c/P_c = 6.71$ ft	$R_c = A_c/P_c = 2.08$ m
$K_T = K_1 + K_2 = 153848.3$ cfs	$K_T = K_1 + K_2 = 4438.2\ m^3/s$
$n = 1.486 A_c R_c^{2/3}/K_T = 0.035$	$n = A_c R_c^{2/3}/K_T = 0.035$

If you do not subdivide the section, the increase in wetted perimeter of the floodplain is relatively large with respect to the increase in area. The hydraulic radius is abnormally reduced, and the calculated conveyance of the entire section (K_c) is lower than the conveyance of the main channel, K_2.

You should subdivide irregular cross sections such as that in Figure 6-7 to create individual basic shapes.

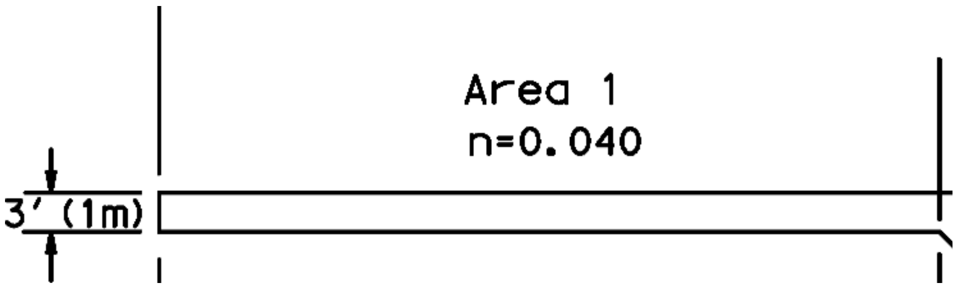


Figure 6-7. Subdividing a "Panhandle" Cross Section

The cross section shapes in Figure 6-6 through Figure 6-9 represent extremes of the problems associated with improper subdivision. A bench panhandle, or terrace, is a shape that falls between these two extremes (see Figure 6-8). Subdivide bench panhandles if the ratio L/d is equal to five or greater.

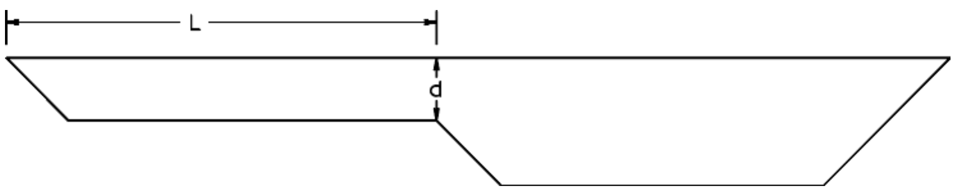


Figure 6-8. Bench Panhandle Cross Section

The following guidelines apply to the subdivision of triangular sections (see Figure 6-9):

- Subdivide if the central angle is 150 or more (L/d is five or greater).
- If L/d is almost equal to five, then subdivide at a distance of L/4 from the edge of the water.
- Subdivide in several places if L/d is equal to or greater than 20.
- No subdivisions are required on the basis of shape alone for small values of L/y, but subdivisions are permissible on the basis of roughness distribution.

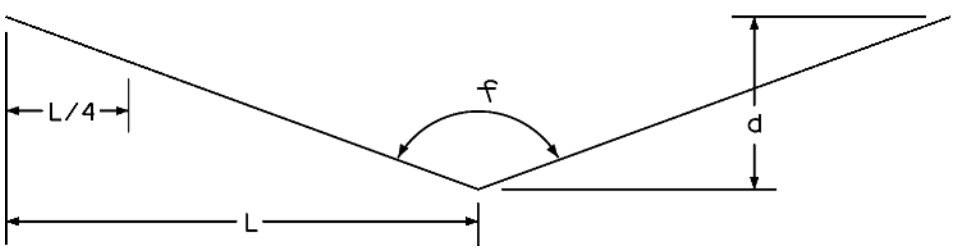


Figure 6-9. Triangular Cross Section

Figure 6-10 shows another shape that commonly causes problems in subdivision. In this case, subdivide the cross section if the main-channel depth (d_{max}) is more than twice the depth at the stream edge of the overbank area (d_b).

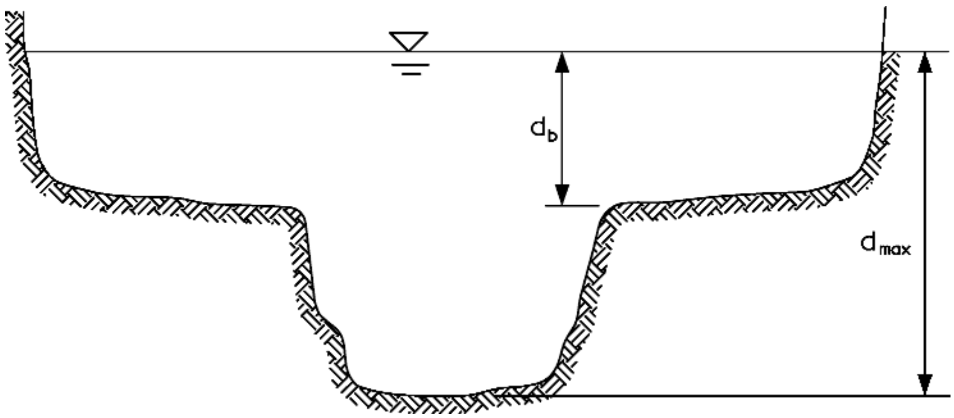


Figure 6-10. Problematic Cross Section

In some cases the decision to subdivide is difficult. Subdivisions in adjacent sections along the stream reach should be similar to avoid large differences in the kinetic energy coefficient (α). Therefore, if a borderline case is between sections not requiring subdivision, do not subdivide the borderline section. If it is between sections that must be subdivided, subdivide this section as well.

Section 2 — Flow in Conduits

Open Channel Flow or Pressure Flow

When a conduit is not submerged, the principles of open channel flow apply. When the conduit is submerged, pressure flow exists because the water surface is not open to the atmosphere, and the principles of conduit flow apply. For circular pipes flowing full, Equation 6-3 becomes:

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

Equation 6-16.

where:

Q = discharge (cfs or m^3/s)

z = 0.4644 for English measurement or 0.3116 for metric.

n = Manning's roughness coefficient

D = pipe diameter, ft. or m

S = slope of the energy gradeline (ft./ft. or m/m) (For uniform, steady flow, S = channel slope, ft./ft. or m/m).

Depth in Conduits

The equations for critical depth apply to conduits, too. Determine critical depth for a rectangular conduit using Equation 6-12 and the discharge per barrel. Calculate critical depth for circular and pipe-arch or irregular shapes by trial and error use of Equation 6-13. For a circular conduit, use Equation 6-17 and Equation 6-18 to determine the area, A, and top width, T, of flow, respectively. For other shapes, acquire or derive relationships from depth of flow, area, and top width.

$$A = \frac{D^2}{8}\left[2\cos^{-1}\left(1-\frac{2d}{D}\right)-\sin\left(2\cos^{-1}\left(1-\frac{2d}{D}\right)\right)\right]$$

Equation 6-17.

$$T = D\sin\left(\cos^{-1}\left(\frac{2d-D}{D}\right)\right)$$

Equation 6-18.

where:

A = section area of flow, sq. ft. or m^2

T = width of water surface, ft. or m

d = depth of flow, ft. or m

D = pipe diameter, ft. or m

the $\cos^{-1}(\theta)$ is the principal value in the range $0 \le \theta \le \pi$.

Use Equation 6-3 to determine uniform depth. For most shapes, a direct solution of Equation 6-3 for depth is not possible. The Slope Conveyance Procedure discussed in Chapter 7 is applicable. For rectangular shapes, area, A, and wetted perimeter, WP are simple functions of flow depth. For circular pipe, compute area using Equation 6-17, and compute wetted perimeter using Equation 6-19. For other shapes, acquire or derive the relationship from depth of flow, area, and wetted perimeter.

Refer to the table below for recommended Manning's roughness coefficients for conduit.

$$WP = D\cos^{-1}\left(1 - \frac{2d}{D}\right)$$

Equation 6-19.

Roughness Coefficients

The following table provides roughness coefficients for conduits.

Recommended Culvert Conduit Roughness Coefficients

Type of Conduit	n-Value
Concrete Box	0.012
Concrete Pipe	0.012
Smooth-lined metal pipe	0.012
Smooth lined plastic pipe	0.012
Corrugated metal pipe	0.015-0.027
Structural plate pipe	0.027-0.036
Long span structural plate	0.031
Corrugated metal (paved interior)	0.012
Plastic	0.012-0.024

Energy

The energy equation, Equation 6-6, applies to conduit flow, too. Additionally, the following concepts apply to conduit flow.

- For pressure flow, the depth, d, represents the distance from the flowline to the hydraulic grade line.
- For pressure flow, the slope of the energy grade line and hydraulic grade line through the conduit are parallel and are represented by the friction slope.
- Compute friction losses, h_f, as the product of friction slope and length of conduit.
- Consider the kinetic energy coefficient (α) equal to unity.
- Other losses include entrance losses, exit losses, and junction losses.

Refer to Chapter 8 for directions to accommodate such losses for culvert design and Chapter 10 for storm drain design.

Compute the velocity head at any location in a conduit using Equation 6-20.

where:

$$h_v = \left[\frac{v^2}{2g}\right]$$

Equation 6-20.

where:

v = flow velocity in culvert (ft./s or m/s).

g = the gravitational acceleration = 32.2 ft/ s^2 or 9.81 m/s^2.

The friction slope represents the slope of the energy grade line and is based upon Manning's Equation, rearranged as follows:

$$S_f = \left(\frac{Qn}{zR^{2/3}A}\right)^2$$

Equation 6-21.

where:

S_f = friction slope (ft./ft. or m/m)

z = 1.486 for English measurements and 1.0 for metric.

Steep Slope versus Mild Slope

When critical depth (d_c) is higher than uniform depth (d_u), the slope is steep. The conduit may flow completely full (pressure flow) or partly full (free surface flow). The free surface flow may be supercritical or subcritical depending on tailwater conditions.

When critical depth is lower than uniform depth, the slope is termed mild. Pressure flow or free surface flow may occur. Free surface flow is most likely to be subcritical within the conduit.

The shape of the free water surface is dependent on whether the conduit slope is steep or mild and on the tailwater conditions. The Standard Step Procedure described in Chapter 7 accommodates the differences in water surface shape.

Section 3 — Hydraulic Grade Line Analysis

Introduction

Analyze the system's hydraulic grade line (sometimes referred to as the HGL) to determine if you can accommodate design flows in the drainage system without causing flooding at some location or causing flows to exit the system at locations where this is unacceptable.

Hydraulic Grade Line Considerations

Develop the hydraulic grade line for the system to determine probable water levels that may occur during a storm event. You can then evaluate these water levels with respect to critical elevations within the designed facility. The development of the hydraulic grade line is a last step in the overall design of a storm drain system.

The hydraulic grade line is the locus of elevations to which the water would rise if open to atmospheric pressure (e.g., piezometer tubes) along a pipe run (see Figure 6-11). The difference in elevation of the water surfaces in successive tubes separated by a specific length usually represents the friction loss for that length of pipe, and the slope of the line between water surfaces is the friction slope.

If you place a pipe run on a calculated friction slope corresponding to a certain rate of discharge, a cross section, and a roughness coefficient, the surface of flow (hydraulic grade line) is parallel to the top of the conduit.

If there is reason to place the pipe run on a slope less than friction slope, then the hydraulic gradient would be steeper than the slope of the pipe run (pressure flow).

Depending on the elevation of the hydraulic grade line at the downstream end of the subject run, it is possible to have the hydraulic grade line rise above the top of the conduit. That is, the conduit is under pressure until, at some point upstream, the hydraulic grade line is again at or below the level of the soffit of the conduit.

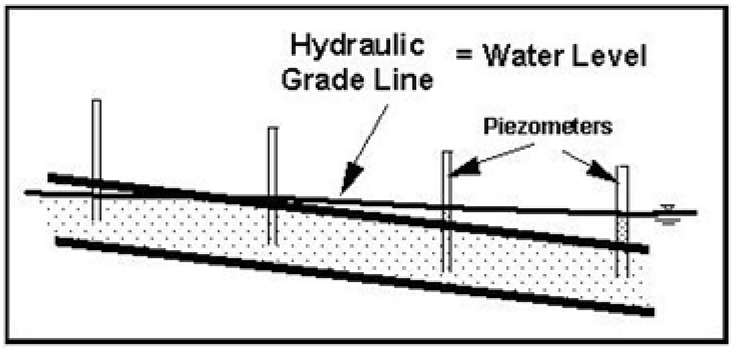


Figure 6-11. Hydraulic Grade Line

Analyze to determine the flow characteristics of the outfall channel. Use the tailwater level occurring in the outfall to the storm drain system in the development of a hydraulic grade line.

Use a realistic tailwater elevation as the basis for the hydraulic grade line calculation. If the outfall tailwater is a function of a relatively large watershed area (such as a large stream) and you base the contribution from the storm drain system on a relatively small total watershed area, then it is not realistic to use a tailwater elevation based on the same frequency as the storm drain design frequency. Refer to Section 3 of Chapter 5 for the design frequency in the hydraulic grade line development of a storm drain system.

Stage versus Discharge Relation

Generally a stage versus discharge relation for the outfall channel is useful. Refer to the Slope Conveyance Procedure in Chapter 7 for considerations and a procedure leading to the development of a stage versus discharge relation in an outfall channel.

As a normal design practice, calculate the hydraulic grade line when the tailwater surface elevation at the outlet is greater than the soffit elevation of the outlet pipe or boxes. If you design the system as a non-pressure system, ignoring junction losses, the hydraulic grade line eventually will fall below the soffit of the pipe somewhere in the system, at which point the hydraulic grade line calculation is no longer necessary. Generally, check the hydraulic grade line. However, such calculations are not needed if the system has all of the following characteristics:

- All conduits are designed for non-pressure flow.
- Potential junction losses are insignificant.
- Tailwater is below the soffit of the outfall conduit.

If the proposed system drains into another enclosed system, analyze the downstream system to determine the effect of the hydraulic grade line.

Conservation of Energy Calculation

When defining the hydraulic grade line, calculations proceed from the system outfall upstream to each of the terminal nodes. For department practice, base calculation of the hydraulic grade line on conservation of energy as shown in Equation 6-22 which includes major and minor energy losses within the system. For conduit, d=1.

$$HGl_{us} + \frac{v_{us}^{2}}{2g} = HGL_{ds} + \frac{v_{ds}^{2}}{2g} + h_f + h_m$$

Equation 6-22.

where:

HGL_{us} = 2 + d = elevation of the hydraulic grade line at upstream node (ft. or m)

v_{us} = upstream velocity (fps or m/s)

v_{ds} = downstream velocity (ft./s or m/s)

h_m = minor (junction/node) head loss (ft. or m)

h_f = friction head loss (ft. or m)

HGL_{ds} = elevation of hydraulic grade line at downstream node (ft. or m)

g = 32.2 ft./ s^2 or 9.81 m/s^2.

Minor Energy Loss Attributions

Major losses result from friction within the pipe. Minor losses include those attributed to junctions, exits, bends in pipes, manholes, expansion and contraction, and appurtenances such as valves and meters.

Minor losses in a storm drain system are usually insignificant. In a large system, however, their combined effect may be significant. Methods are available to estimate these minor losses if they appear to be cumulatively important. You may minimize the hydraulic loss potential of storm drain system features such as junctions, bends, manholes, and confluences to some extent by careful design. For example, you can replace severe bends by gradual curves in the pipe run where right-of-way is sufficient and increased costs are manageable. Well designed manholes and inlets, where there are no sharp or sudden transitions or impediments to the flow, cause virtually no significant losses.

Entrance Control

Generally treat a storm drain conduit system as if it operates in subcritical flow. As such, entrance losses of flow into each conduit segment are mostly negligible. However, if discharge enters into the system through a conduit segment in which there must be supercritical flow, significant head losses are encountered as the discharge builds enough energy to enter the conduit. This situation is most likely where a lateral is located on a relatively steep slope. On such slopes, evaluate the type of flow (subcritical or supercritical).

With supercritical flow, the lateral may be operating under entrance control. When a lateral is operating under entrance control as described above, the headwater level is usually much higher than a projection of the hydraulic grade line.

If the entrance control headwater submerges the free fall necessary for the inlet to function properly, it may be necessary to reconfigure the lateral by increasing its size or changing its slope. Some improvement to the inlet characteristics may help to overcome any unfavorable effects of entrance control. Usually, entrance control does not affect steep units in the trunk lines because the water is already in the conduit; however, you may need to consider velocity head losses.

Use the following procedure to determine the entrance control head:

1. Calculate critical depth as discussed in Critical Depth in Conduit earlier in this section.
2. If critical depth exceeds uniform depth, go to step 3; otherwise, no entrance control check is necessary.
3. Calculate entrance head in accordance with the Headwater Under Inlet Control subsection in Chapter 8.
4. Add entrance head to flowline and compare with the hydraulic grade line at the node.
5. Take the highest of the two values from step 4. Check to ensure that this value is below the throat of the inlet.

Hydraulic Grade Line Procedure

Use the following procedure to determine the entrance control head:

1. Determine an appropriate water level in the outfall channel or facility. For an open channel outfall, the appropriate water level will be a function of the stage vs. discharge relation of flow in the outfall facility and designer's selection of design frequency for the storm drain facility. If the outfall tailwater level is lower than critical depth at the exiting conduit of the system, use the elevation associated with critical depth at that point as a beginning water surface elevation for the Hydraulic Grade Line calculation.
2. Compute the friction loss for each segment of the conduit system, beginning with the most downstream run. The friction loss (h_f) for a segment of conduit is defined by the product of the friction slope at full flow and the length of the conduit as shown in Equation 6-23.

$$h_f = S_f L$$

Equation 6-23.

The friction slope, S_f, is calculated by rearranging Manning's Equation to Equation 6-24.

$$S_f = \frac{Q^2 n^2}{z^2 A^2 R^{4/3}}$$

Equation 6-24.

where:

S_f = friction slope (ft./ft. or m/m)

Q = discharge (cfs or m^3/s)

n = Manning's roughness coefficient

z = 1.486 for use with English measurements only.

A = cross-sectional area of flow (sq. ft. or m^2)

R = hydraulic radius (ft. or m) = A / WP

WP = wetted perimeter of flow (the length of the channel boundary in direct contact with the water) (ft. or m).

Combining Equation 6-23 with Equation 6-24 yields Equation 6-25 for friction loss.

$$h_f = \frac{Q^2 n^2}{z^2 A^2 R^{4/3}} L$$

Equation 6-25.

where:

z = 1.486 for use with English measurements units only.

L = length of pipe (ft. or m).

For a circular pipe flowing full, Equation 6-25 becomes Equation 6-26.

$$h_f = \left(\frac{Qn}{zD^{8/3}}\right)^2 L$$

Equation 6-26.

where:

z = 0.4644 for English measurement or 0.3116 for metric.

D = Pipe diameter (ft. or m).

For partial flow, you could use Equation 6-25 to approximate the friction slope. However, the backwater methods, such as the (Standard) Step Backwater Method outlined in Chapter 7, provide better estimates of the hydraulic grade line.

1. Using the downstream Hydraulic Grade Line elevation as a base, add the computed friction loss h_f. This will be the tentative elevation of the Hydraulic Grade Line at the upstream end of the conduit segment.
2. Compare the tentative elevation of the Hydraulic Grade Line as computed above to the elevation represented by uniform depth of flow added to the upstream flow line elevation of the subject conduit.
3. The higher of the two elevations from step 2 above will be the controlling Hydraulic Grade Line elevation (HGL_{us}) at the upstream node of the conduit run. (If you perform backwater calculations, the computed elevation at the upstream end becomes the Hydraulic Grade Line at that point).
4. If other losses are significant, calculate them using the procedures outlined below. Use Equation 6-27 to determine the effect of the sum of minor losses (h_m) on the Hydraulic Grade Line.

$$HGL_i = HGL_o + \frac{v_o^2}{2g} + h_m - \frac{v_i^2}{2g}$$

Equation 6-27.

5. If the upstream conduit is on a mild slope (i.e., critical depth is lower than uniform depth), set the starting Hydraulic Grade Line for the next conduit run (HGL_{ds}) to be the higher of critical depth and the Hydraulic Grade Line from step 3 (or 4 if minor losses were considered).
6. Go back to step 2 and continue the computations in an upstream direction into all branches of the conduit system. The objective is to compare the level of the Hydraulic Grade Line to all critical elevations in the storm drain system.
7. Check all laterals for possible entrance control head as described in the subsection below.
8. If the Hydraulic Grade Line level exceeds a critical elevation, you must adjust the system so that a revised Hydraulic Grade Line level does not submerge the critical elevation (this condition is sometimes referred to as a "blowout.") Most adjustments are made with the objective of increasing capacity of those conduit segments causing the most significant friction losses. If the developed Hydraulic Grade Line does not rise above the top of any manhole or above the gutter invert of any inlet, the conduit system is satisfactory.

NOTE: If the conduit system does not include any pressure flow segments but the outlet channel elevation is higher than the top of the conduit at the system exit, compute the Hydraulic Grade Line through the system until the Hydraulic Grade Line level is no higher than the soffit of the conduit. At this point, continuance of the Hydraulic Grade Line is unnecessary, unless other losses are likely to be significant.

Chapter 7 — Channels

Contents:

Section 1 — Introduction

Open Channel Types

In this chapter, the term open channel includes the total conveyance facility (the floodplain and stream channel). This chapter addresses required design criteria, design philosophy, and channel design and analysis procedures.

The various types of open channels include stream channels, roadside channels or ditches, and artificial channels such as irrigation channels or drainage ditches. The hydraulic design process for open channels consists of establishing criteria, developing and evaluating alternatives, and selecting the alternative that best satisfies the criteria. Plan for capital investment and probable future costs, including maintenance and flood damage to property, traffic service requirements, and stream and floodplain environment. Evaluate risks warranted by flood hazard at the site, economics, and current engineering practices.

Use channel design to determine the channel cross section required to accommodate a given discharge. This includes sizing outfall channels and various roadway ditches. Channel design involves selection of trial channel characteristics, application of channel analysis methods, and then iteration until the trial characteristics meet the desired criteria.

Analyze the channel to determine the depth and average velocity at which the discharge flows in a channel with an established cross section. Use channel analysis most frequently to establish a water surface elevation that influences the design or analysis of a hydraulic structure or an adjacent roadway profile scheme.

Assess the following when designing transportation drainage systems:

- potential flooding caused by changes in water surface profiles
- disturbance of the river system upstream or downstream of the highway right-of-way
- changes in lateral flow distributions
- changes in velocity or direction of flow
- need for conveyance and disposal of excess runoff
- need for channel linings to prevent erosion.

Methods Used for Depth of Flow Calculations

Use the Slope Conveyance Method and Standard Step Backwater Method), described in this chapter, for calculating depth of flow for analyzing an existing channel or for designing a new or improved channel.

Section 2 — Stream Channel Planning Considerations and Design Criteria

Location Alternative Considerations

The planning phase for a highway section usually involves consideration of a number of alternate highway locations, which often require construction across or along streams and floodplains. During the planning phase, evaluate the effects that location alternatives would have on stream systems. (See the Project Development Process Manual for more details.) Include a preliminary hydraulic study of the various alternatives because the type and cost of drainage facilities required can determine location selection. As project development proceeds, you may find that locations selected without adequate hydraulic consideration to floodplain encroachments and extensive channel modifications are unacceptable.

Consider the environmental effects, risks, and costs of required drainage facilities in the final selection of an alternative. Analysis of alternative alignments may reveal possibilities for reducing construction costs, flood damage potential, maintenance problems, and adverse environmental impacts.

Detailed information and survey data are seldom available for an in-depth hydraulic study during the planning phase; however, it is possible to ascertain basic requirements and consequences of a particular location or alignment and the relative merits of alternatives. Topographic maps, aerial photography, stream gage data, floodplain delineation maps, and a general knowledge of the area often provide the basis for preliminary evaluations of alternatives.

Phase Planning Assessments

Consider the following factors:

- water quality standards
- stream stability
- heavy debris discharge
- highly erodible banks
- fish and wildlife resources.

Assessments may require the cooperative efforts of Area Office designers and Division personnel as well as others with experience on similar projects or specialized expertise in the particular area. Design all projects to comply with Federal and State regulations. As such, it is necessary to consider the implications of the following:

- Federal Emergency Management Agency National Flood Insurance Program (FEMA NFIP)
- U.S. Corps of Engineers (USACE) 404 permit

- U.S. Fish and Wildlife requirements
- Environmental Protection Agency (EPA) National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System permit requirements
- EPA NPDES permit for industrial activity (construction)
- EPA Endangered Species Act provisions

Refer to the Project Development Process Manual for more information on the above regulations.

Environmental Assessments

Consult the Chapter 3, Environmental in *Project Development Process Manual* for environmental concerns. (See Texas Parks and Wildlife Department (TPWD), Clean Water Act (CWA), in the Environmental Procedures in Project Development Process Manual.) Consider stream channel modification only after examining all other alternatives. Regulatory requirements invoked by stream channel modifications can be substantial.

Consider the U.S. Fish and Wildlife Service (USFWS) review requirements where review may result in recommendations to avoid, minimize, or compensate for the adverse effects to wildlife habitat.

Refer to Chapter 3, Environmental for more information. It is prudent to plan measures to avoid, minimize, or compensate for stream modifications.

Justify the selection of a stream modification alternative. Consult with resource agencies early in design planning, and include these consultations in the Environmental Assessment Statement (EAS) or Environmental Impact Statement (EIS) with supporting documentation. (See Chapter 3, Environmental, in the *Project Development Process Manual* for more details.) The EA should also contain compensation plans for replacing any removed habitats. Avoid or minimize adverse effects, or implement mitigation plans to the best of your ability when transportation projects impact riparian corridors as described in the Fish and Wildlife Coordination Act (FWCA). (See Chapter 3, Environmental, in the *Project Development Process Manual* for more details.) If the department cannot offer mitigation for riparian corridor impacts, offer an explanation as justification in the environmental documentation.

Consultations with Respective Agencies

During the planning phase, contact Federal, State, and local agencies in regard to plans or land uses such as the following that could affect the highway drainage design:

- dams and reservoirs
- irrigation

- flood control levees or channel modifications
- navigation
- floodplain management
- zoning
- recreational use
- fish or wildlife management.

Consult the four agencies having regulatory authority over navigation and construction activities in waters of the United States and agencies with special expertise, such as in the limits and classification of wetlands, for preliminary information that may affect location decisions. The four agencies are as follows:

- U.S. Coast Guard (USCG), U.S. Department of Transportation
- U.S. Army Corps of Engineers (USACE), Department of Army
- Federal Highway Administration (FHWA)
- Environmental Protection Agency (EPA).

See References for contact information.

Other Agency Requirements

An increasing number of federal and state permits are required for construction activities that may involve navigation and water quality. Program application for permits and approvals by Federal and State agencies having reulatory authority over streams early in the project development process. See Chapter 2, Section 2, Federal Laws, Regulations, and Agencies Governing Hydraulic Design.

Stream Channel Criteria

Stream channel criteria include the following:

- Evaluate the hydraulic effects of floodplain encroachments for the peak discharges of the design AEP and the 1% AEP on any major highway facility.
- Avoid relocation or realignment of a stream channel wherever practicable.
- Match the cross-sectional shape, plan-view, roughness, sediment transport, and slope to the original conditions insofar as practicable.
- Include some means of energy dissipation when velocities through the structure are excessive or when the original conditions cannot be duplicated.

- Provide stream bank stabilization, when appropriate, to counteract any stream disturbance such as encroachment. Stabilize both upstream and downstream banks, as well as the local site. Refer to "Stream Stability at Highway Bridges," FHWA-IP-90-014 for guidance.
- Provide a sufficient top width with access for maintenance equipment for features such as dikes and levees associated with natural channel modifications. Provide turnaround points throughout and at the end of these features.

Section 3 — Roadside Channel Design

Roadside Drainage Channels

According to the AASHTO Roadside Design Guide, roadside drainage channel is an open channel usually paralleling the highway embankment and within limits of the ROW. The primary function of a drainage channel is to collect surface runoff from the roadway and areas that drain to the ROW and to convey the accumulated runoff to acceptable outlet points. Drainage channels must be designed to carry the design runoff and to accommodate excessive storm water with minimal roadway flooding or damage. The design frequency should correspond with the storm drain frequency. For details of roadway safety design which governs ditch shape design, see the Roadway Design Manual, Chapter 2, Section 6, Slopes and Ditches, and Median Design, and Chapter 2, Section 7, Side Ditches. Where the Roadway Design Manual requirements can't be met, the channel will have to be enclosed in a pipe or box. See Chapter 10, Storm Drains.

Channel Linings

Channel lining may be desirable or necessary to minimize maintenance, resist the erosive forces of flowing water, improve hydraulic efficiency, and/or limit the channel size for right-of-way or safety considerations. The considerations of flow volumes, topography, and soil conditions may dictate the channel lining material to be used. Wherever possible, highway drainage channel design should make use of native, natural materials such as grass, crushed rock, and earth. Other types of materials for reasons of hydraulics, economics, safety, aesthetics, and environment may be considered.

The following section contains a short discussion on channel linings. For comprehensive descriptions, advantages, and disadvantages of different types of channel linings, refer to the FHWA Hydraulic Engineering Circular No. 15 (HEC-15).

Rigid versus Flexible Lining

Engineers may design roadside channels with rigid or flexible linings. Flexible linings in channels conform better to a changing channel shape than rigid linings. However, a rigid lining may resist an erosive force of high magnitude better than a flexible one.

The following types of rigid linings are common:

- cast-in-place concrete
- soil cement
- fabric form work systems for concrete
- grouted riprap.

Rigid channel linings have the following disadvantages when compared to natural or earth-lined channels:

- Initial construction cost of rigid linings is usually greater than the cost of flexible linings.
- Maintenance costs may also be high because rigid linings are susceptible to damage by undercutting, hydrostatic uplift, and erosion along the longitudinal interface between the lining and the unlined section.
- Inhibition of natural infiltration in locations where infiltration is desirable or permissible.
- Smooth linings usually cause high flow velocities with scour occurring at the terminus of the sections unless controlled with riprap or other energy dissipating devices
- Contaminants may be transported to the receiving waters in areas where water quality considerations are of major concern. A vegetative or flexible type of lining may filter the contaminants from the runoff.

Permanent flexible linings include the following:

- rock riprap
- wire enclosed riprap (gabions)
- vegetative lining
- geotextile fabrics.

Flexible linings generally have the following advantages:

- less costly to construct
- have self-healing qualities that reduce maintenance costs
- permit infiltration and exfiltration
- present a more natural appearance and safer roadsides.

Various species of grass may be used as permanent channel lining if flow depths, velocities, and soil types are within acceptable tolerances for vegetative lining. The turf may be established by sodding or seeding. Sod is usually more expensive than seeding, but it has the advantage of providing immediate protection. Some type of temporary protective covering is often required for seed and topsoil until vegetation becomes established.

The following are classified as temporary flexible linings:

- geotextile fabrics
- straw with net
- curled wood mat
- jute, paper, or synthetic net

- synthetic mat
- fiberglass roving.

Temporary channel lining and protective covering may consist of jute matting, excelsior mats, or fiberglass roving. Straw or wood-chip mulch tacked with asphalt is usually not well suited for channel invert lining but may be used for side slopes. Geotextile materials, known as soil stabilization mats, may be used for protective linings in ditches and on side slopes. These materials are not biodegradable and serve as permanent soil reinforcement while enhancing the establishment of vegetation.

Channel Lining Design Procedure

Use the following design procedure for roadside channels. Even though each project is unique, these six basic design steps normally apply:

1. Establish a roadside plan. Collect available site data:
   - Obtain or prepare existing and proposed plan/profile layouts including highway, culverts, bridges, etc.
   - Determine and plot on the plan the locations of natural basin divides and roadside channel outlets.
   - Lay out the proposed roadside channels to minimize diversion flow lengths.
2. Establish cross section geometry: Identify features that may restrict cross section design including right-of-way limits, trees or environmentally sensitive areas, utilities, and existing drainage facilities. Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects. Choose channel side slopes based on the following geometric design criteria: safety, economics, soil, aesthetics, and access. Establish the bottom width of trapezoidal channel.
3. Determine initial channel grades. Plot initial grades on plan-profile layout (slopes in roadside ditch in cuts are usually controlled by highway grades) by establishing a minimum grade to minimize ponding and sediment accumulation, considering the influence of type of lining on grade, and where possible, avoiding features that may influence or restrict grade, such as utility locations.
4. Check flow capacities, and adjust as necessary. Compute the design discharge at the downstream end of a channel segment (see Chapter 5). Set preliminary values of channel size, roughness, and slope. Determine the maximum allowable depth of channel including freeboard. Check the flow capacity using Manning's Equation for Uniform Flow and single-section analysis (see Equation 7-1 and Chapter 6). If the capacity is inadequate, possible adjustments are as follows:
   - increase bottom width
   - make channel side slopes flatter

- make channel slope steeper
- provide smoother channel lining
- install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity
- provide smooth transitions at changes in channel cross sections
- provide extra channel storage where needed to replace floodplain storage or to reduce peak discharge

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

Equation 7-1.

where:

Q = discharge (cfs or m^3/s)

A = cross-sectional area of flow (sq. ft. or m^2)

R = hydraulic radius (ft. or m)

Z = conversion factor; 1.486 for English units, and 1.0 for metric

5. Determine channel lining or protection needed. Calculate uniform flow depth (y_m in ft. or m) at design discharge using the Slope Conveyance Method. Compute maximum shear stress at normal depth (see Equation 7-2 and Equation 7-3). Select a lining and determine the permissible shear stress (in lbs./sq.ft. or N/m^2) using the tables titled Retardation Class for Lining Materials and Permissible Shear Stresses for Various Linings. If $\tau_d < \tau_p$, then the lining is acceptable. Otherwise, consider the following options: choose a more resistant lining, use concrete or gabions or other more rigid lining as full lining or composite, decrease channel slope, decrease slope in combination with drop structures, or increase channel width or flatten side slopes.
6. Analyze outlet points and downstream effects. Identify any adverse impacts to downstream properties that may result from one of the following at the channel outlet: increase or decrease in discharge, increase in velocity of flow, confinement of sheet flow, change in outlet water quality, or diversion of flow from another watershed. Mitigate any adverse impacts identified in the previous step. Possibilities include enlarging the outlet channel or installing control structures to provide detention of increased runoff in channel, installing velocity control structures, increasing capacity or improving the lining of the downstream channel, installing sedimentation/infiltration basins, installing sophisticated weirs or other outlet devices to redistribute concentrated channel flow, and eliminating diversions that result in downstream damage and that cannot be mitigated in a less expensive fashion.

$$\tau_d = \gamma R S$$

Equation 7-2.

where:

τ = average shear stress at normal depth (lb./sq.ft. or N/m^2)

γ = unit weight of water (62.4 lb./ft.^{3}or 9810 N./m.^{2})

R = hydraulic radius (ft. or m.) at uniform depth (y_m)

S = channel slope (ft./ft. or m./m.)

The maximum shear stress for a straight channel occurs on the channel bed.

$$\tau_d = \gamma d S$$

Equation 7-3.

where:

τ_d = maximum sheer stress (lb./sq ft. or N/m^2)

γ = unit weight of water (62.4 lb./ft.^{3}or 9810 N./m.^{2})

d = maximum depth of flow (ft. or m.)

S = channel slope (ft./ft. or m./m.)

Retardation Class for Lining Materials

Retardance Class	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30 in. or 760 mm)
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36 in. or 915 mm)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12 in. or 305 mm)
	Native grass mixture little bluestem, bluestem, blue gamma, other short and long stem midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good Stand, tall (average 24 in. or 610 mm)
	Lespedeza sericea	Good stand, not woody, tall (average 19 in. or 480 mm)
	Alfalfa	Good stand, uncut (average 11 in or 280 mm)
	Weeping lovegrass	Good stand, unmowed (average 13 in. or 330 mm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13 in. or 330 mm)
C	Crabgrass	Fair stand, uncut (10-to-48 in. or 55-to-1220 mm)
	Bermuda grass	Good stand, mowed (average 6 in. or 150 mm)

Retardation Class for Lining Materials

Retardance Class	Cover	Condition
	Common lespedeza	Good stand, uncut (average 11 in. or 280 mm)
	Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6-8 in. or 150-200 mm)
	Centipedegrass	Very dense cover (average 6 in. or 150 mm)
	Kentucky bluegrass	Good stand, headed (6-12 in. or 150-305 mm)
D	Bermuda grass	Good stand, cut to 2.5 in. or 65 mm
	Common lespedeza	Excellent stand, uncut (average 4.5 in. or 115 mm)
	Buffalo grass	Good stand, uncut (3-6 in. or 75-150 mm)
	Grass-legume mixture: fall, spring (orchard grass Italian ryegrass, and common lespedeza	Good Stand, uncut (4-5 in. or 100-125 mm)
	Lespedeza sericea	After cutting to 2 in. or 50 mm (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5 in. or 40 mm
	Bermuda grass	Burned stubble

Permissible Shear Stresses for Various Linings

Protective Cover	(lb./sq.ft.)	t_p (N/m^2)
Retardance Class A Vegetation (See the “Retardation Class for Lining Materials” table above)	3.70	177
Retardance Class B Vegetation (See the “Retardation Class for Lining Materials” table above)	2.10	101
Retardance Class C Vegetation (See the “Retardation Class for Lining Materials” table above)	1.00	48
Retardance Class D Vegetation (See the “Retardation Class for Lining Materials” table above)	0.60	29
Retardance Class E Vegetation (See the “Retardation Class for Lining Materials” table above)	0.35	17
Woven Paper	0.15	7
Jute Net	0.45	22
Single Fiberglass	0.60	29
Double Fiberglass	0.85	41
Straw W/Net	1.45	69
Curled Wood Mat	1.55	74
Synthetic Mat	2.00	96

Permissible Shear Stresses for Various Linings

Protective Cover	(lb./sq.ft.)	t_p (N/m^2)
Gravel, D_{50} = 1 in. or 25 mm	0.40	19
Gravel, D_{50} = 2 in. or 50 mm	0.80	38
Rock, D_{50} = 6 in. or 150 mm	2.50	120
Rock, D_{50} = 12 in. or 300 mm	5.00	239
6-in. or 50-mm Gabions	35.00	1675
4-in. or 100-mm Geoweb	10.00	479
Soil Cement (8% cement)	>45	>2154
Dycel w/out Grass	>7	>335
Petraflex w/out Grass	>32	>1532
Armorflex w/out Grass	12-20	574-957
Erikamat w/3-in or 75-mm Asphalt	13-16	622-766
Erikamat w/1-in. or 25 mm Asphalt	<5	<239
Armorflex Class 30 with longitudinal and lateral cables, no grass	>34	>1628
Dycel 100, longitudinal cables, cells filled with mortar	<12	<574
Concrete construction blocks, granular filter underlayer	>20	>957
Wedge-shaped blocks with drainage slot	>25	>1197

Trial Runs

To optimize the roadside channel system design, make several trial runs before a final design is achieved. Refer to HEC-15 for more information on channel design techniques and considerations.

Section 4 — Stream Stability Issues

Stream Geomorphology

Planning and location engineers should be conscious of fluvial geomorphology and request the services of hydraulics engineers to quantify natural changes and changes that may occur as a result of stream encroachments, crossings, or channel modifications.

Fluvial geomorphology and river mechanics are not new subjects; however, methods of quantifying the interrelation of variables are relatively recent developments. The theories and knowledge available today make it possible to estimate and predict various reactions to changes and, more importantly, to establish thresholds for tolerance to change.

Streams have inherent dynamic qualities by which changes continually occur in the stream position and shape. Changes may be slow or rapid, but all streams are subjected to forces that cause changes to occur. In these streams, banks erode, sediments are deposited, and islands and side channels form and disappear in time. The banks and adjacent floodplains usually contain a large proportion of sand, even though the surface strata may consist of silt and clay; thus, the banks erode and cave with relative ease.

Most alluvial channels exhibit a natural instability that results in continuous shifting of the stream through erosion and deposition at bends, formation and destruction of islands, development of oxbow lakes, and formation of braided channel sections.

The degree of channel instability varies with hydrologic events, bank and bed instability, type and extent of vegetation on the banks, and floodplain use.

The identification of these characteristics and understanding of the relationship of the actions and reactions of forces tending to effect change enables the design engineer to estimate the rates of change and evaluate potential upstream and downstream effects of natural change and proposed local channel modifications.

The potential response of the stream to natural and proposed changes may be quantified with the basic principles of river mechanics. The design engineer must understand and use these principles to minimize the potential effect of these dynamic systems on highways and the adverse effects of highways on stream systems.

Non-alluvial channels have highly developed meanders in solid rock valleys and may be degrading their beds. An example of such a stream is the Guadalupe River as it passes through the Edwards Aquifer recharge zone. Many mountain streams are classified as non-alluvial, and in these cases the design engineer may perform a hydraulic analysis utilizing rigid boundary theory.

Stream Classification

Figure 7-1 illustrates the three main natural channel patterns: straight, meandering, braided, and their relationships to each other. For a more complete explanation of this classification system, see FHWA/RD-80/160 “Methods for Assessment of Stream Related Hazards to Highways and Bridges”, Shen, et. al, 1981.

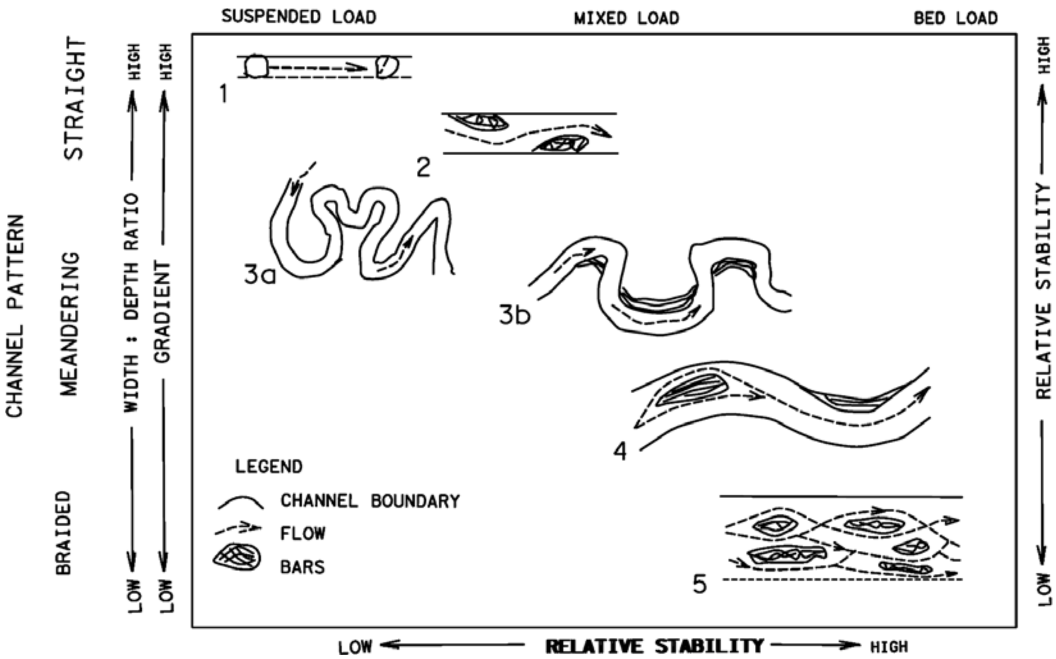


Figure 7-1. Natural Stream Patterns

Straight Streams. A stream is classified as straight when the ratio of the length of the thalweg (path of deepest flow; see Figure 7-2) to the length of the valley is less than 1.05. This ratio is known as the sinuosity of the stream. Degrees of sinuosity are illustrated in Figure 7-3.

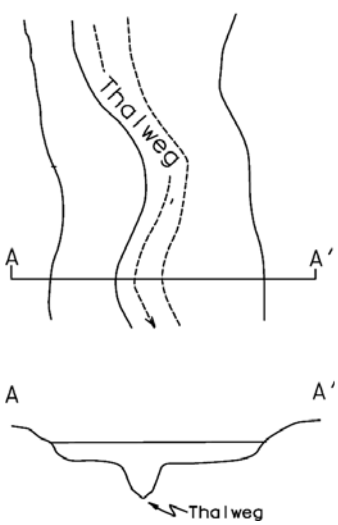


Figure 7-2. Thalweg Location in Plan View and Cross Section

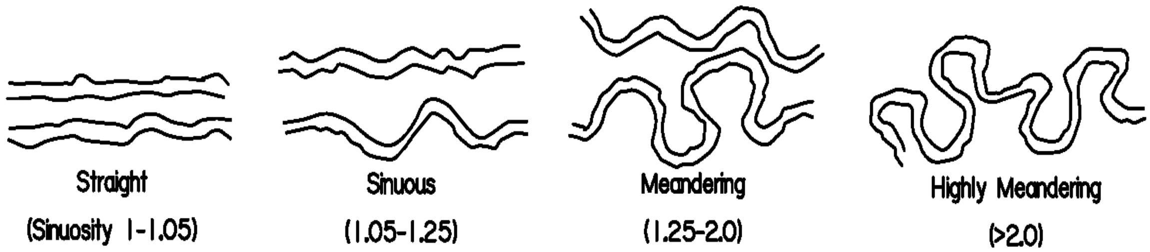


Figure 7-3. Various Degrees of Sinuosity

Straight channels are sinuous to the extent that the thalweg usually oscillates transversely within the low flow channel, and the current is deflected from one side to the other. The current oscillation usually results in the formation of pools on the outside of bends while lateral bars, resulting from deposition, form on the inside of the bends as shown in Figure 7-1, diagrams 2 and 3b.

Straight reaches of alluvial channels may be only a temporary condition. Aerial photography and topographic maps may reveal former locations of the channel and potential directions of further movement.

Braided Streams. Braiding is caused by bank caving and by large quantities of sediment load that the stream is unable to transport (see Figure 7-4). Deposition occurs when the supply of sediment exceeds the stream's transport capacity. As the streambed aggrades from deposition, the downstream channel reach develops a steeper slope, resulting in increased velocities. Multiple channels develop on the milder upstream slope as additional sediment is deposited within the main channel.

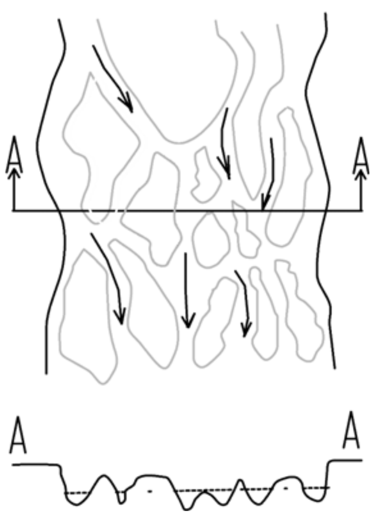


Figure 7-4. Plan View and Cross Section of a Braided Stream

The interlaced channels cause the overall channel system to widen, resulting in additional bank erosion. The eroded material may be deposited within the channel to form bars that may become stabilized islands. At flood stage, the flow may inundate most of the bars and islands, resulting in the complete destruction of some and changing the location of others. A braided stream is generally unpredictable and difficult to stabilize because it changes alignment rapidly, is subject to degradation and aggradation, and is very wide and shallow even at flood stage.

Meandering Streams. A meandering stream consists of alternating S-shaped bends (see Figure 7-5). In alluvial streams, the channel is subject to both lateral and longitudinal movement through the formation and destruction of bends.

Bends are formed by the process of erosion and sloughing of the banks on the outside of bends and by the corresponding deposition of bed load on the inside of bends to form point bars. The point bar constricts the bend and causes erosion in the bend to continue, accounting for the lateral and longitudinal migration of the meandering stream (Figure 7-5).

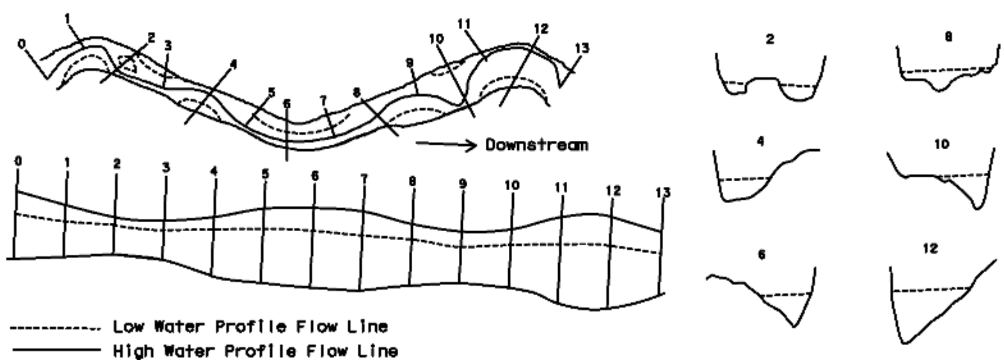


Figure 7-5. Plan View and Cross Section of a Meandering Stream

As a meandering stream moves along the path of least resistance, the bends move at unequal rates because of differences in the erodibilty of the banks and floodplain. Bends are ultimately cut off, resulting in oxbow lakes (see below).

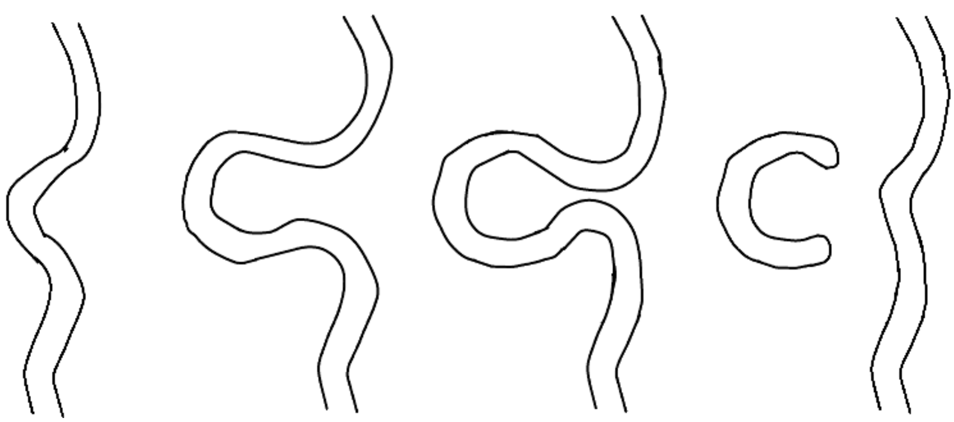


Figure 7-6. Migration Leading to Formation of Oxbow Lake

After a cutoff is formed, the stream gradient is steeper, the stream tends to adjust itself upstream and downstream, and a new bend may develop. Compare aerial photographs taken over a period of years to estimate the rate and direction of the meander movement. Local history may also help to quantify the rate of movement.

Modification to Meandering

Modification of an alluvial channel from its natural meandering tendency into a straight alignment usually requires confinement within armored banks because the channel may be very unstable. Straightening meandering channels can result in steeper gradients, degradation, and bank caving upstream as the stream attempts to reestablish equilibrium. The eroded material will be deposited downstream, resulting in reduced stream slopes, reduced sediment transport capacity, and possible braiding. When a channel is straightened without armor banks, the current will tend to oscillate transversely and initiate the formation of bends. Eventually, even protected straight channel reaches may be destroyed as a result of the natural migration of meanders upstream of the modified channel.

Graded Stream and Poised Stream Modification

Graded streams and poised streams are dynamically balanced, and any change altering that condition may lead to action by the stream to reestablish the balance. For example, if the channel gradient is increased, as occurs with a cutoff, the sediment transport capacity of the flow is increased and additional scouring results, thereby reducing the slope. The transport capacity of the downstream reach has not been altered; therefore, the additional sediment load carried downstream is deposited as a result of upstream scour. As the aggradation progresses, the stream slope below the deposition is increased, and the transport capacity is adjusted to the extent required to carry the additional material through the entire reach. This process continues until a new balance is achieved, and the effect could extend a considerable distance upstream and downstream of the cutoff.

Modification Guidelines

It may be necessary to modify a stream in order to make it more compatible with the highway facility and the physical constraints imposed by local terrain or land use. The modifications may involve changes in alignment or conveyance. Changes may be necessary to accommodate the highway requirements, but they must be evaluated to assess short-term and long-term effects on the stream system.

Background data on the existing stream should be available from previously completed planning and location studies, and a preliminary highway design should be available in sufficient detail to indicate the extent of required channel modifications.

Certain types of streams may have a very wide threshold of tolerance to changes in alignment, grade, and cross-section. In contrast, small changes can cause significant impacts on sensitive waterways. An analysis of the tolerance to change may reveal that necessary modifications will not have detrimental results.

If you recognize detrimental effects, develop plans to mitigate the effects to within tolerable limits. You can enhance certain aspects of an existing stream system, often to the economic benefit of the highway. The following are examples of ways to enhance stream systems:

- Control active upstream headcutting (degradation due to abrupt changes in bed elevation) with culverts or check dams so that many hectares of land along the stream banks will not be lost and the highway facility will be protected from the headcutting.
- Coordinate and cooperate with fish and wildlife agency personnel, adapt or modify stabilization measures necessary to protect the highway while improving aquatic habitat.

Realignment Evaluation Procedure

The realignment of natural streams may disrupt the balance of the natural system. When evaluating stream modifications, use the following procedure:

1. Establish slope, section, meander pattern and stage-discharge relationship for present region.
2. Determine thresholds for changes in the various regime parameters.
3. Duplicate the existing regime, where possible, but keep within the established tolerances for change, where duplication is not practical or possible.

Stream realignment may occasionally decrease channel slope; more often, the modification will increase the channel gradient. A localized increase in channel slope may introduce channel responses that are reflected for considerable distances upstream and downstream of the project.

Response Possibilities and Solutions

Increased Slope. The following are possible responses to increased slope:

- The stream response may be in the form of a regime change from a meandering to a braided channel, or sediment transport through the steepened reach may be increased enough to cause degradation upstream of the realignment and aggradation downstream.
- Banks may become unstable and require structural stabilization measures to prevent erosion.
- Tributary channels entering the steepened main channel may be subject to headcutting, with deposition occurring at or downstream of the junction.

The following are possible solutions to increased slope:

- You may use grade control structures (such as check dams, weirs, or chutes) to minimize increases in gradient, provided there is some assurance that the normal meandering tendency of the channel will not bypass these structures in time.

- If topography permits, use meanders to reduce the stream gradient to existing or threshold levels. These meanders may require stabilization to assure continued effectiveness and stability.

Encroachment. Highway locations or modifications in certain terrain conditions may result in an encroachment such as that illustrated in Figure 7-7.

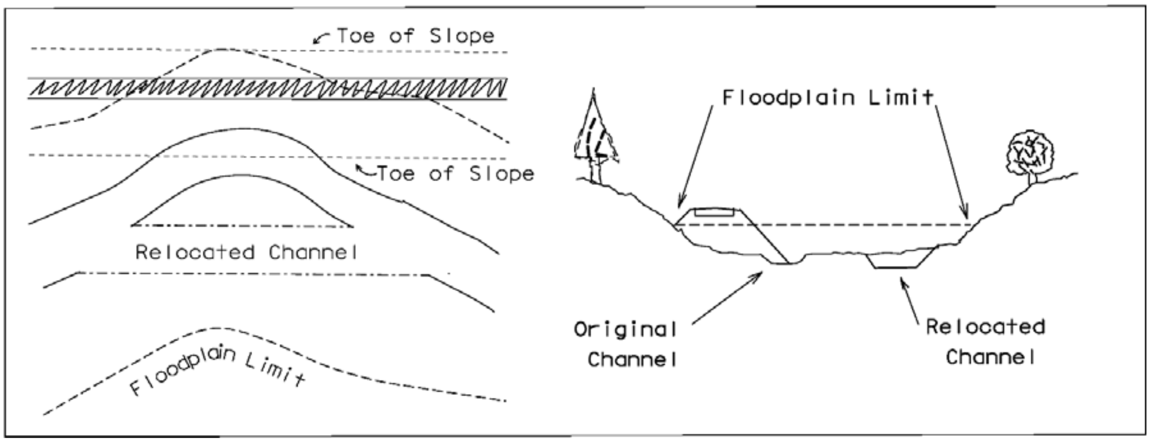


Figure 7-7. Highway Encroachment on Natural Streams and Stream Relocation

This type of channel realignment may require providing a channel of sufficient section to convey both normal and flood flow within the banks formed by the roadway and the floodplain. The low flow channel may require realignment, in which case a pilot channel could approximate the existing channel characteristics of width, depth, gradient, and bottom roughness. Where no pilot channel is provided, the average daily flow is likely to spread over a much wider section, and flow depth will be reduced in such a way that water temperature, pool formation, and sediment transport are adversely affected. These modifications may result in a braided channel condition and hamper the re-establishment of the natural aquatic environment.

Clearing of vegetation along stream banks may remove root systems that have contributed to bank stability. Clearing and grubbing reduces the bank and floodplain roughness and contributes to higher velocities and increased erosion potential for those areas. However, the limited clearing of adjacent right-of-way involved with transverse encroachments or crossings does not normally affect the overall conveyance capacity of a channel to any significance.

A water surface profile analysis is necessary to establish the stage-discharge relationship for channels with varying roughness characteristics across the channel. The Slope Conveyance Method of estimating stage-discharge relationships can be subject to significant error if the typical section used does not represent the actual conditions upstream and downstream of the crossing site. Therefore, the Standard Step Backwater Method is recommended. (See Section 6 for more details on these methods.)

Channel enlargement or cleanout through a limited channel reach is sometimes proposed in an effort to provide additional stream capacity. If the stage of the stream at the proposed highway site is controlled by downstream conditions, there can be limited or possibly no benefits derived from localized clearing.

Environmental Mitigation Measures

The potential environmental impacts and the possible need for stream impact mitigation measures should be primary considerations. (See Environmental Assessments in Section 2 for more information.) Mitigation practices are not generally warranted but may be mandated by the cognizant regulatory agency. As such, you may need to coordinate with Texas fish and wildlife agencies before determining mitigation. Consult the Environmental Affairs Division and the Design Division, Hydraulic Branch, to determine the need for mitigation when you deem stream modifications necessary.

Channel modifications may be necessary and also can provide environmental enhancement (see the previous Modification Guidelines subsection). Also, channel modifications that are compatible with the existing aquatic environment can sometimes be constructed at little or no extra cost.

There will be less aquatic habitat where a channel is shortened to accommodate highway construction. This not only decreases the aquatic biomass, but also reduces the amount of surface water available for recreation and sport fishing. Estimate the significance of this effect by comparing the amount of surface water area, riparian and upland wetland area, and stream length that will be lost with the existing amount in the geographic area. If there will be a loss, particularly of wetlands, resource and regulatory agencies may raise objections in light of the national “no net loss” policy currently prevailing. In some instances, such habitat loss may be acceptable when combined with mitigation measures, but such measures should prevent habitat damage beyond the channel change limits.

Enhancement of the channel may be accomplished during stream reconstruction at little additional cost, and perhaps at less cost where reconstruction is essential to the needs of the highway project. It may even be possible to reconstruct the surface water resource in one of the following manners that eliminates an existing environmental problem:

- incorporating sinuosity into a straight stream reach
- relocating the channel to avoid contamination from minerals or other pollution sources
- adjusting flow depth and width to better utilize low flows
- providing an irregular shaped channel section to encourage overhanging bank
- improving the riparian vegetation.

The most common practices are using a drop-type grade control structure (check dam), maintaining the existing channel slope, and increasing the channel change length by constructing an artificial meander.

Culverts can provide another alternative similar to using drop structures. You can increase the culvert flowline slope to accommodate the elevation difference caused by shortening a channel. The increased erosion associated with steep culverts is localized at the outlet that can be protected.

Simulate the existing channel cross section if it is relatively stable, has low flow depths and velocities, or has adequate minimum flow requirements.

Determine the cross-sectional shape by hydraulically analyzing simple and easy to construct shapes that approximate the preferred natural channel geometry. The analysis generally compares the stage-discharge, stage-velocity, and stage-sediment relationships of the natural channel with the modified channel.

Stream relocations may temporarily impair water quality. The problem is primarily sediment-related, except for those rare instances where adverse minerals or chemicals are exposed, diverted, or intercepted. With a channel relocation, the new channel should be constructed in dry conditions wherever possible. Following completion, the downstream end should be opened first to allow a portion of the new channel to fill as much as possible. Next, the upstream end should be opened slowly to minimize erosion and damage to habitat mitigation.

Where the channel relocation interferes with the existing channel, it may be desirable to construct rock and gravel dikes or to use other filtering devices or commercially available dikes to isolate the construction site, thereby limiting the amount of sediment entering the water.

Countermeasures

Many streams have a strong propensity to meander. The sinuosity of the main channel is a general characteristic of a stream and can vary with the discharge and the type of soil that the stream passes through. The erosive force of the stream water forms meanders as it undercuts the main channel bank. The bank support is lost and material caves into the water to be deposited downstream. As the erosion on the outer bend of the meander migrates in a downstream direction, material from upstream deposits on the inside of the bend. This progression of stream meandering can have serious effects on highway crossings. This migration often threatens approach roadway embankment and bridge headers such as shown in Figure 7-8.

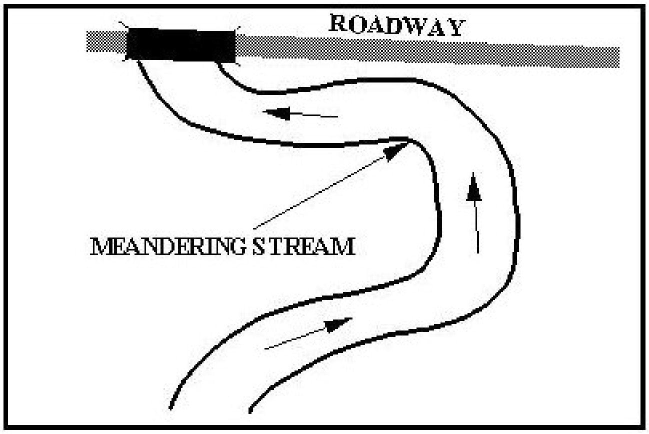


Figure 7-8. Meandering Stream Threatening Bridge and Approach Roadway

In order to protect the roadway from the threat of meanders, yet remain synchronous with nature, it is important to devise countermeasures that are environmentally sound, naturally acting, economically viable, and physically effective.

Possible countermeasures include the following:

- Bridge lengthening -- With reference to the example given in Figure 7-8, lengthening the bridge may not always be cost-effective as a countermeasure to the damage potential from the meander. In this example, the natural meandering course of the river threatens both the bridge and the approach roadway.
- Bridge relocation – In extreme situations, it may be necessary to relocate the bridge. Generally, it is good practice to locate the bridge crossing on a relatively straight reach of stream between bends.
- River training or some type of erosion control – River training or some type of erosion control may be more effective and economical. Designers have used several measures and devices successfully in Texas to counter the effects of serious stream meandering.
- Linear structures -- When it is not practical to locate the bridge on a relatively straight reach of stream, countermeasures such as spur or jetty type control structures may be needed (see Figure 7-9). These are sometimes referred to as linear structures, permeable or impermeable, projecting into the channel from the bank to alter flow direction, protect the channel bank, induce deposition, and reduce flow velocity along the bank.

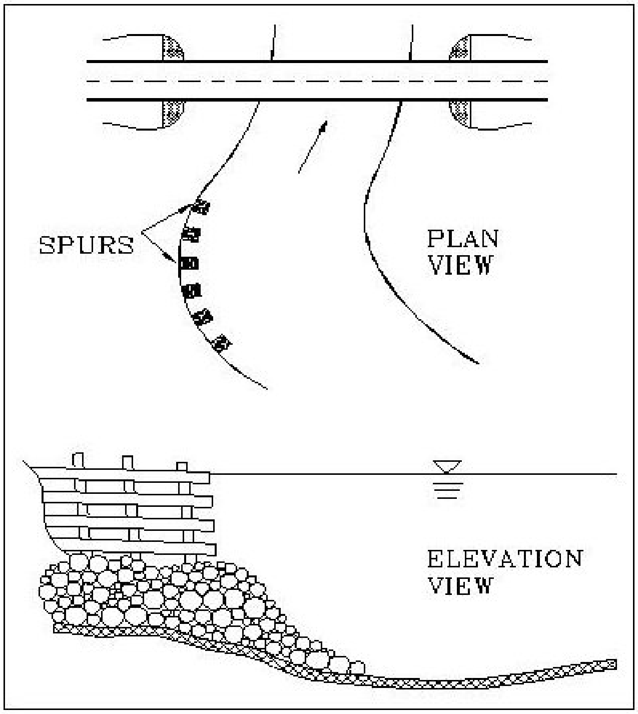


Figure 7-9. Permeable Fence Spurs as Meander Migration Countermeasures

Control structures may or may not cause the typical cross section of flow in a meandering stream to become more symmetrical. For many locations, countermeasures may not be required for several years because of the time required for the bend to move to a location where it begins to threaten the highway facility. In other streams, however, bends may migrate at such a rate that the highway is threatened within a few years or after a few flood events. In such cases, the countermeasure should be installed during initial construction.

Altered Stream Sinuosity

In some instances, stabilizing channel banks at a highway stream crossing can cause a change in the channel cross section and may alter the stream sinuosity winding upstream of the stabilized banks. Figure 7-10 illustrates meander migration in a natural stream. If sinuosity increases due to artificial stream stabilization, then meander amplitude may increase. Meander radii in other parts of the reach may become smaller and deposition may occur because of reduced slopes. The channel width-depth ratio may increase as a result of bank erosion and deposition. Ultimately, cutoffs can occur.

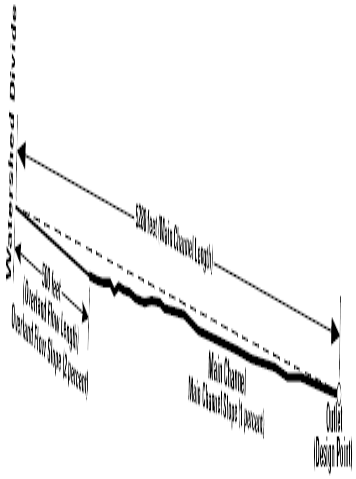


Figure 7-10. Meander Migration in a Natural Stream

Refer to *Design of Spur-Type Streambank Stabilization Structures*, FHWA/RD-84/101 and *Stream Stability at Highway Structures*, HEC-20, for further design considerations, guidelines, and procedures for the various types of stream stabilization and meander countermeasures used and recommended by the department.

Stabilization and Bank Protection

Highway embankments constructed within a floodplain may require stabilization to resist erosion during flood events. You may design and construct embankment stabilization with the initial roadway project where the need is obvious or the risk of damage is high. In other locations the following factors may warrant that installation of embankment stabilization to be delayed until a problem actually develops as follows:

- economic considerations
- availability of materials
- probability of damage.

Highway channel stabilization measures are usually local in nature. Engineers design them primarily to protect the highway facility from attack by a shifting channel or where the floodplain adjacent to the facility is highly erodible.

If a highway location adjacent to a stream cannot be avoided, you should evaluate protective measures to determine the measure best suited to the situation. These alternatives may include channel change, roadway embankment protection, stream bank stabilization, and stream-training works.

Channel stabilization should be considered only when it is economically justified and one or more of the following basic purposes will be accomplished:

- prevent loss or damage of the highway facility and associated improvement
- reduce maintenance requirements
- achieve secondary benefits such as beautification, recreation, and the preservation or establishment of fish and wildlife habitat.

Stabilization measures at the highway site may not be successful if the section is located within long reaches of unstable channel. Local stabilization often results in high maintenance costs and repetitive reconstruction. A stream may respond to local stabilization by changing flow regime or attacking the unprotected bed or opposite bank. The potential for these occurrences should be considered. However, if bank erosion occurs only at isolated locations, stabilization measures at these locations are probably an economical solution even though a period of repetitive maintenance may follow.

Revetments

Generally, revetments are located on the outside bank of bends where bank recession or erosion is most active as a result of impinging flow (see Figure 7-11). They may be required elsewhere to protect an embankment from wave wash or flood attack.

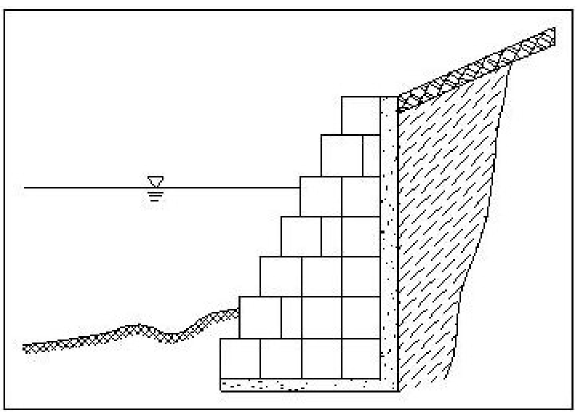


Figure 7-11. Gabions Used as Revetment

The segment of revetment placed above the annual flood elevation may differ in design from the segment located below that elevation due to the conditions affecting construction, the types of materials available, and the differences in the duration and intensity of attack. The higher segment is termed upper bank protection, and the lower segment is called subaqueous protection. Both are required to prevent bank recession, and the upper bank protection may be extended to a sufficient height to protect against wave action. For smaller streams and rivers, the upper and subaqueous protections are usually of the same design and are placed in a single operation.

The banks on which revetments will be placed should be graded to slopes that will be stable when saturated, and an adequate filter system should be incorporated to prevent loss of bank material through the protective revetment.

The type of filter system used depends on slope stability, bank material, type of revetment, and availability of filter materials.

Filter materials may consist of sand, gravel, or woven or non-woven synthetic filter cloth.

Numerous materials have been used for bank protection, including dumped rock, Portland cement concrete, sacked sand-cement, soil cement, gabions, and precast blocks.

Section 5 — Channel Analysis Guidelines

Stage-Discharge Relationship

A stage-discharge curve is a graph of water surface elevation versus flow rate in a channel. A stage-discharge curve is shown in Figure 7-12. You may compute various depths of the total discharge for the stream, normal flow channel, and floodplain.

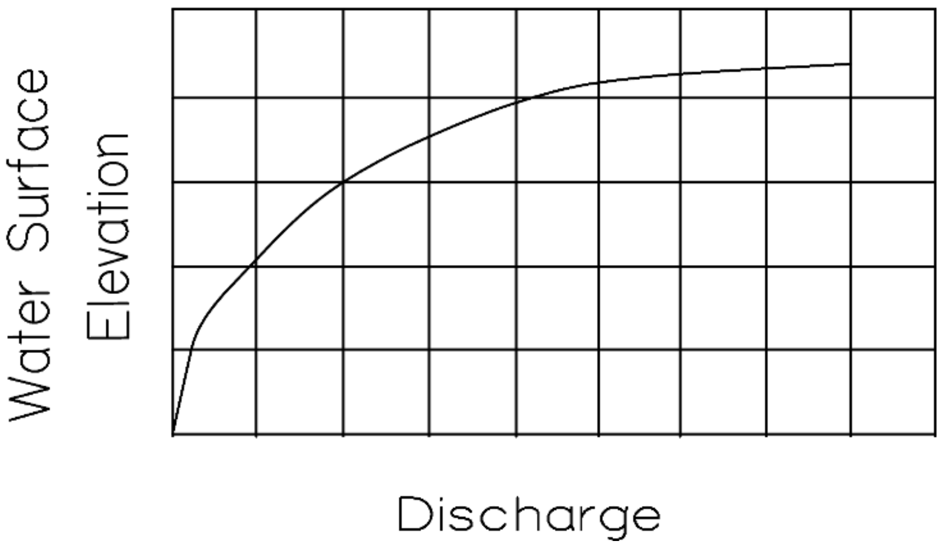


Figure 7-12. Typical Stage Discharge Curve

(See Manning's Equation for Uniform Flow and Stage-Discharge Determination.) The data, plotted in graphic form (sometimes termed a "rating curve"), gives you a visual display of the relationship between water surface elevations and discharges.

An accurate stage-discharge relationship is necessary for channel design to evaluate the interrelationships of flow characteristics and to establish alternatives for width, depth of flow, freeboard, conveyance capacity and type, and required degree of stabilization.

The stage-discharge relationship also enables you to evaluate a range of conditions as opposed to a preselected design flow rate.

Examine the plot of stage-discharge carefully for evidence of the "switchback" characteristic described below. Also, examine the plot to determine whether or not it is realistic. For example, a stream serving a small watershed should reflect reasonable discharge rates for apparent high water elevations.

Switchback

If you improperly subdivide the cross section, the mathematics of Manning's Equation may cause a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation or depth (see Manning's Equation for Uniform Flow in Chapter 6, Equation 6-3 and Figure 7-13). A small increase in depth can result in a small increase in cross-sectional area and large increase in wetted perimeter and a net decrease in the hydraulic radius. The discharge computed using the smaller hydraulic radius and the slightly larger cross-sectional area is lower than the previous discharge for which the water depth was lower. Use more subdivisions within such cross sections in order to avoid the switchback.

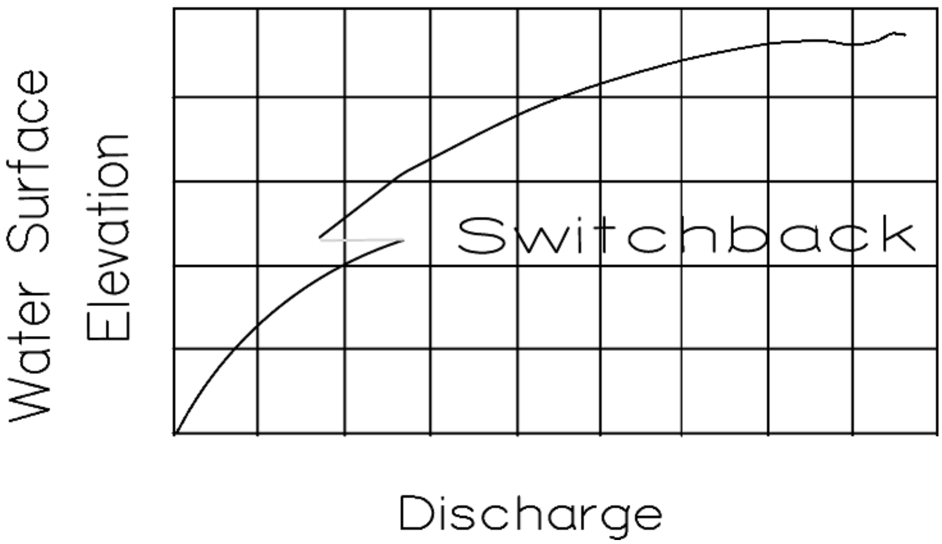


Figure 7-13. Switchback in Stage Discharge Curve

A switchback can occur in any type of conveyance computation. Computer logic can be seriously confused if a switchback occurs in any cross section being used in a program. For this reason, always subdivide the cross section with respect to both roughness and geometric changes. Note that the actual n-value may be the same in adjacent subsections. However, too many subdivisions can result in problems, too. (See Chapter 6 for more information.)

Section 6 — Channel Analysis Methods

Introduction

The depth and velocity of flow are necessary for the design and analysis of channel linings and highway drainage structures. The depth and velocity at which a given discharge flows in a channel of known geometry, roughness, and slope can be determined through hydraulic analysis. The following two methods are commonly used in the hydraulic analysis of open channels:

- Slope Conveyance Method
- Standard Step Backwater Method.

Generally, the Slope Conveyance Method requires more judgment and assumptions than the Standard Step Method. In many situations, however, use of the Slope Conveyance Method is justified, as in the following conditions:

- standard roadway ditches
- culverts
- storm drain outfalls.

Slope Conveyance Method

The Slope Conveyance Method, or Slope Area Method, has the advantages of being a relatively simple, usually inexpensive and expedient procedure. However, due to the assumptions necessary for its use, its reliability is often low. The results are highly sensitive to both the longitudinal slope and roughness coefficients that are subjectively assigned. This method is often sufficient for determining tailwater (TW) depth at non-bridge class culvert outlets and storm drain outlets.

The procedure involves an iterative development of calculated discharges associated with assumed water surface elevations in a typical section. The series of assumed water surface elevations and associated discharges comprise the stage-discharge relationship. When stream gauge information exists, a measured relationship (usually termed a “rating curve”) may be available.

You normally apply the Slope Conveyance Method to relatively small stream crossings or those in which no unusual flow characteristics are anticipated. The reliability of the results depends on accuracy of the supporting data, appropriateness of the parameter assignments (n-values and longitudinal slopes), and your selection of the typical cross section.

If the crossing is a more important one, or if there are unusual flow characteristics, use some other procedure such as the Standard Step Backwater Method.

A channel cross section and associated roughness and slope data considered typical of the stream reach are required for this analysis. A typical section is one that represents the average characteristics of the stream near the point of interest. While not absolutely necessary, this cross section should be located downstream from the proposed drainage facility site. The closer to the proposed site a typical cross section is taken, the less error in the final water surface elevation.

You should locate a typical cross section for the analysis. If you cannot find such a cross section, then you should use a "control" cross section (also downstream). (Known hydraulic conditions, such as sluice gates or weirs exist in a control cross section.) The depth of flow in a control cross section is controlled by a constriction of the channel, a damming effect across the channel, or possibly an area with extreme roughness coefficients.

The cross section should be normal to the direction of stream flow under flood conditions.

After identifying the cross section, apply Manning's roughness coefficients (n-values). (See Equation 6-3 and Chapter 6 for more information.) Divide the cross section with vertical boundaries at significant changes in cross-section shape or at changes in vegetation cover and roughness components. (See Chapter 6 for suggestions on subdividing cross sections.)

Manning's Equation for Uniform Flow (see Chapter 6 and Equation 6-3) is based on the slope of the energy grade line, which often corresponds to the average slope of the channel bed. However, some reaches of stream may have an energy gradient quite different from the bed slope during flood flow.

Determine the average bed slope near the site. Usually, the least expensive and most expedient method of slope-determination is to survey and analyze the bed profile for some distance in a stream reach. Alternately, you may use topographic maps, although they are usually less accurate.

Slope Conveyance Procedure

The calculation of the stage-discharge relationship should proceed as described in this section. The Water Surface Elevation tables represent the progression of these calculations based on the cross section shown in Figure 7-14. The result of this procedure is a stage-discharge curve, as shown in Figure 7-15. You can then use the design discharge or any other subject discharge as an argument to estimate (usually done by interpolation) an associated water surface elevation.

1. Select a trial starting depth and apply it to a plot of the cross section.
2. Compute the area and wetted perimeter weighted n-value (see Chapter 6) for each submerged subsection.
3. Compute the subsection discharges with Manning's Equation. Use the subsection values for roughness, area, wetted perimeter, and slope. (See Equation 7-1). The sum of all of the incremental discharges represents the total discharge for each assumed water surface elevation.

Note. Compute the average velocity for the section by substituting the total section area and total discharge into the continuity equation.

$$V = \frac{Q}{A}$$

Equation 7-4.

4. Tabulate or plot the water surface elevation and resulting discharge (stage versus discharge).
5. Repeat the above steps with a new channel depth, or add a depth increment to the trial depth. The choice of elevation increment is somewhat subjective. However, if the increments are less than about 0.25 ft. (0.075 m), considerable calculation is required. On the other hand, if the increments are greater than 1.5 ft. (0.5 m), the resulting stage-discharge relationship may not be detailed enough for use in design.
6. Determine the depth for a given discharge by interpolation of the stage versus discharge table or plot.

The following x and y values apply to Figure 7-14:

X and Y Values for Figure 7-14

X	Y
0	79
2	75
18	72
20	65
33	65
35	70
58	75
60	79

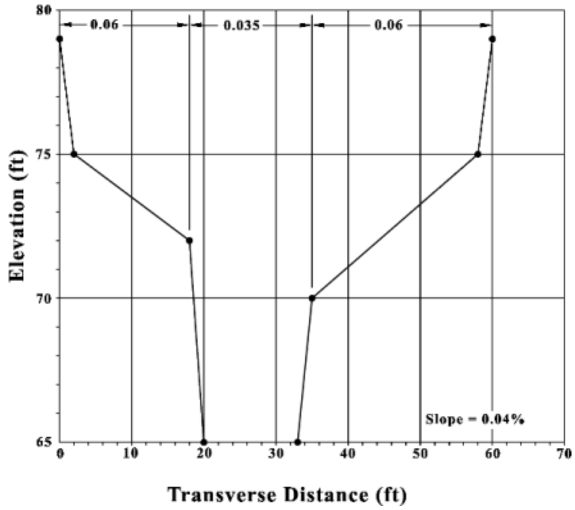


Figure 7-14. Slope Conveyance Cross Section

Water Surface Elevation of 66 ft.

	Subsection L	Subsection C	Subsection R	Full Section
Area (ft2)	0	13.34	0	13.34
Wetted Perimeter (ft)	0	15.12	0	
Hydraulic Radius (ft)		0.88		
n	0.060	0.035	0.060	
Q (cfs)		10.43		10.43
Velocity (fps)		0.78		0.78

Water Surface Elevation of 79 ft.

	Subsection L	Subsection C	Subsection R	Full Section
Area (ft2)	92.00	226.00	153.50	471.5
Wetted Perimeter (ft)	20.75	25.67	28.01	
Hydraulic Radius (ft)	4.43	8.81	5.48	
n	0.060	0.035	0.060	
Q (cfs)	122.98	818.33	236.34	1177.66
Velocity (fps)	1.34	3.62	1.54	2.50

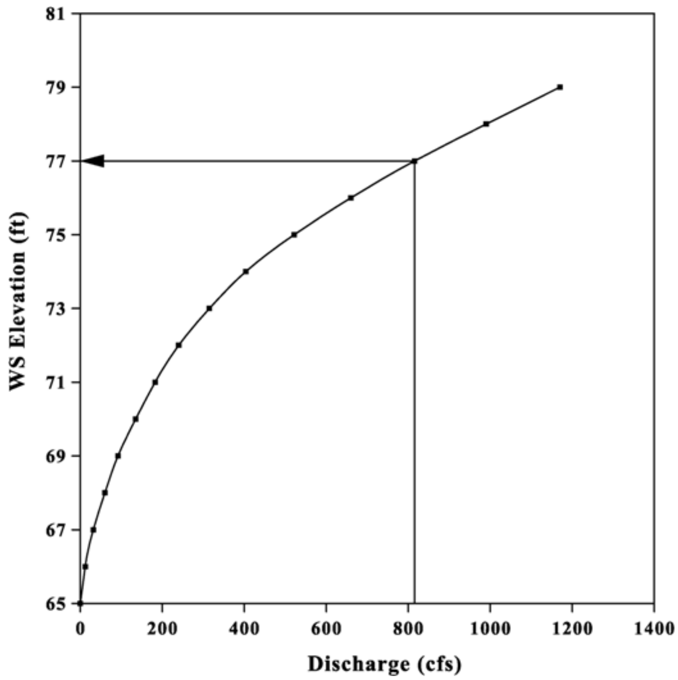


Figure 7-15. Stage Discharge Curve for Slope Conveyance

Standard Step Backwater Method

The Step Backwater Method, or Standard Step Method, uses the energy equation to “step” the stream water surface along a profile (usually in an upstream direction because most Texas streams exhibit subcritical flow). This method is typically more expensive to complete but more reliable than the Slope-Conveyance Method.

The manual calculation process for the Standard Step Method is cumbersome and tedious. With accessibility to computers and the availability of numerous algorithms, you can accomplish the usual channel analysis by Standard Step using suitable computer programs.

A stage-discharge relationship can be derived from the water surface profiles for each of several discharge rates.

Ensure that the particular application complies with the limitations of the program used.

Use the Standard Step Method for analysis in the following instances:

- results from the Slope-Conveyance Method may not be accurate enough
- the drainage facility’s level of importance deserves a more sophisticated channel analysis
- the channel is highly irregular with numerous or significant variations of geometry, roughness characteristics, or stream confluences
- a controlling structure affects backwater.

This procedure applies to most open channel flow, including streams having an irregular channel with the cross section consisting of a main channel and separate overbank areas with individual n-values. Use this method either for supercritical flow or for subcritical flow.

Standard Step Data Requirements

At least four cross sections are required to complete this procedure, but you often need many more than three cross sections. The number and frequency of cross sections required is a direct function of the irregularity of the stream reach. Generally speaking, the more irregular the reach, the more cross sections you may require. The cross sections should represent the reach between them. A system of measurement or stationing between cross sections is also required. Evaluate roughness characteristics (n-values) and associated sub-section boundaries for all of the cross sections. Unfortunately, the primary way to determine if you have sufficient cross sections is to evaluate the results of a first trial.

The selection of cross sections used in this method is critical. As the irregularities of a stream vary along a natural stream reach, accommodate the influence of the varying cross-sectional geometry. Incorporate transitional cross sections into the series of cross sections making up the stream reach. While there is considerable flexibility in the procedure concerning the computed water surface profile, you can use knowledge of any controlling water surface elevations.

Standard Step Procedure

The Standard Step Method uses the Energy Balance Equation, Equation 6-11, which allows the water surface elevation at the upstream section (2) to be found from a known water surface elevation at the downstream section (1). The following procedure assumes that cross sections, stationing, discharges, and n-values have already been established. Generally, for Texas, the assumption of subcritical flow will be appropriate to start the process. Subsequent calculations will check this assumption.

1. Select the discharge to be used. Determine a starting water surface elevation. For subcritical flow, begin at the most downstream cross section. Use one of the following methods to establish a starting water surface elevation for the selected discharge: a measured elevation, the Slope-Conveyance Method to determine the stage for an appropriate discharge, or an existing (verified) rating curve.
2. Referring to Figure 6-1 and Equation 6-11, consider the downstream water surface to be section 1 and calculate the following variables:
    - z_1 = flowline elevation at section 1
    - y_1 = tailwater minus flowline elevation
    - α = kinetic energy coefficient (For simple cases or where conveyance does not vary significantly, it may be possible to ignore this coefficient.)

3. From cross section 1, calculate the area, A_1. Then use Equation 6-1 to calculate the velocity, v_1, for the velocity head at A_1. The next station upstream is usually section 2. Assume a depth y_2 at section 2, and use y_2 to calculate z_2 and A_2. Calculate, also, the velocity head at A_2.
4. Calculate the friction slope (s_f) between the two sections using Equation 7-5 and Equation 7-6:

$$s_f = \left(\frac{Q}{K_{ave}}\right)^2$$

Equation 7-5.

where:

$$K_{ave} = \frac{K_1 + K_2}{2} = 0.5\left(\frac{ZA_1R_1^{\frac{2}{3}}}{n_1} + \frac{ZA_2R_2^{\frac{2}{3}}}{n_2}\right)$$

Equation 7-6.

5. Calculate the friction head losses (h_f) between the two sections using

$$h_f = SaveL$$

Equation 7-7.

where:

L = Distance in ft. (or m) between the two sections

6. Calculate the kinetic energy correction coefficients (α_1 and α_2) using Equation 6-10.
7. Where appropriate, calculate expansion losses (h_e) using Equation 7-8 and contraction losses (h_c) using Equation 7-9 (Other losses, such as bend losses, are often disregarded as an unnecessary refinement.)

$$h_e = K_e \frac{\Delta V^2}{2g}$$

Equation 7-8.

where:

K_e = 0.3 for a gentle expansion

K_e = 0.5 for a sudden expansion

$$h_c = K_c \frac{\Delta V^2}{2g}$$

Equation 7-9.

where:

$K_c = 0.1$ for a gentle contraction

$K_c = 0.3$ for a sudden contraction

8. Check the energy equation for balance using Equation 7-10 and Equation 7-11.

$$L = z_2 + y_2 + \alpha_2 \frac{V_2^2}{2g}$$

Equation 7-10.

$$R = z_1 + y_1 + \alpha_1 \frac{V_1^2}{2g} + h_f + h_e + h_c$$

Equation 7-11.

The following considerations apply:

- if L=R within a reasonable tolerance, then the assumed depth at Section 1 is okay. This will be the calculated water surface depth at Section 1; proceed to Step (9)
- if L≠R, go back to Step (3) using a different assumed depth.

9. Determine the critical depth (d_c) at the cross section and find the uniform depth (d_u) by iteration. If, when running a supercritical profile, the results indicate that critical depth is greater than uniform depth, then it is possible the profile at that cross section is supercritical. For subcritical flow, the process is similar but the calculations must begin at the upstream section and proceed downstream.
10. Assign the calculated depth from Step (8) as the downstream elevation (Section 1) and the next section upstream as Section 2, and repeat Steps (2) through (10).
11. Repeat these steps until all of the sections along the reach have been addressed.

Profile Convergence

When you use the Standard Step Backwater Method and the starting water surface elevation is unknown or indefinite, you can use a computer to calculate several backwater profiles based on several arbitrary starting elevations for the same discharge. If you plot these profiles ,as shown in Figure 7-16, they will tend to converge to a common curve at some point upstream because each successive calculation brings the water level nearer the uniform depth profile.

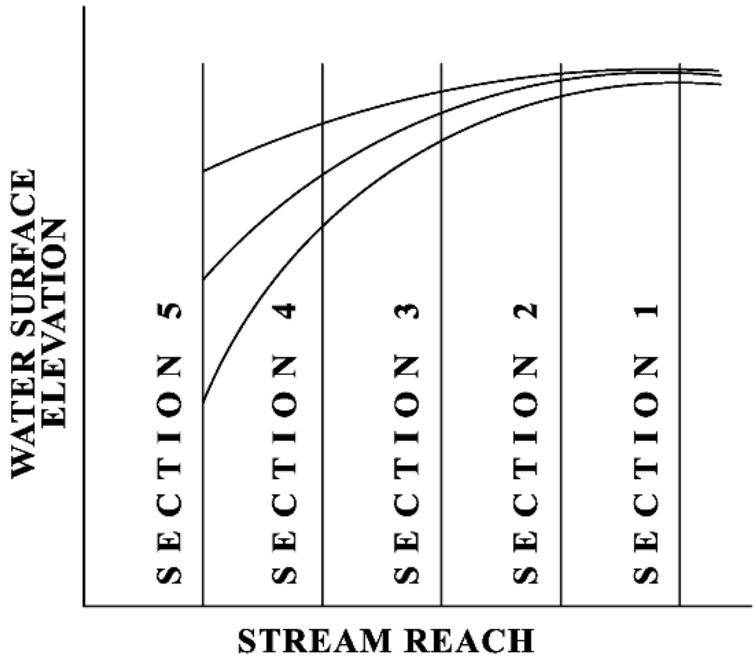


Figure 7-16. Water Surface Profile Convergence

The purpose of plotting the curves and finding the convergence point is to determine where the proposed structure site is in reference to the convergence point. If the site is in the vicinity or upstream of the convergence point, you have started the calculations far enough downstream to define a proper tailwater from an unknown starting elevation. Otherwise, you may have to begin the calculations at a point further downstream by using additional cross sections.

Chapter 8 — Culverts

Contents:

Section 1 — Introduction

Definition and Purpose

A culvert conveys surface water through a roadway embankment or away from the highway right-of-way (ROW) or into a channel along the ROW. In addition to the hydraulic function, the culvert must also support construction and highway traffic and earth loads; therefore, culvert design involves both hydraulic and structural design. The hydraulic and structural designs must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. Culverts are considered minor structures, but they are of great importance to adequate drainage and the integrity of the facility. This chapter describes the hydraulic aspects of culvert design, construction and operation of culverts, and makes references to structural aspects only as they are related to the hydraulic design.

Culverts, as distinguished from bridges, are usually covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed or concrete riprap channel serving as the bottom of the culvert. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. Any culvert with a clear opening of more than 20-feet, measured along the center of the roadway between inside of end walls, is considered a bridge by FHWA, and is designated a bridge class culvert. (See Chapter 9, Section 1). This chapter addresses structures designed hydraulically as culverts, regardless of length.

At many locations, either a bridge or a culvert fulfills both the structural and hydraulic requirements for the stream crossing. The appropriate structure should be chosen based on the following criteria:

- construction and maintenance costs
- risk of failure
- risk of property damage
- traffic safety
- environmental and aesthetic considerations
- construction expedience.

Although the cost of individual culverts is usually relatively small, the total cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, culvert maintenance may account for a large share of the total cost of maintaining highway hydraulic features. Improved traffic service and reduced cost can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

Before starting culvert design, the site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation) must be considered.

Construction

Culverts are constructed from a variety of materials and are available in many different shapes and configurations. When selecting a culvert, the following should be considered:

- roadway profiles
- channel characteristics
- flood damage evaluations
- construction and maintenance costs
- estimates of service life.

Numerous cross-sectional shapes are available. The most commonly used shapes are circular, pipe-arch and elliptical, box (rectangular), modified box, and arch. Shape selection should be based on the cost of construction, limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance. Commonly used culvert materials include concrete (reinforced and non-reinforced), steel (smooth and corrugated), aluminum (smooth and corrugated), and plastic (smooth and corrugated).

The selection of material for a culvert depends on several factors that can vary considerably according to location. The following groups of variables should be considered:

- structure strength, considering fill height, loading condition, and foundation condition
- hydraulic efficiency, considering Manning’s roughness, cross section area, and shape
- installation, local construction practices, availability of pipe embedment material, and joint tightness requirements
- durability, considering water and soil environment (pH and resistivity), corrosion (metallic coating selection), and abrasion
- cost, considering availability of materials.

The most economical culvert is the one that has the lowest total annual cost over the design life of the project. Culvert material selection should not be based solely on the initial cost. Replacement costs and traffic delay are usually the primary factors in selecting a material that has a long service life. If two or more culvert materials are equally acceptable for use at a site, including hydraulic performance and annual costs for a given life expectancy, bidding the materials as alternates should be considered, allowing the contractor to make the most economical material selection.

Inlets

Several inlet configurations are utilized on culvert barrels. These include both prefabricated and constructed-in-place installations. Commonly used inlet configurations include the following:

- projecting culvert barrels
- cast-in-place concrete headwalls
- pre-cast or prefabricated end sections
- culvert ends mitered to conform to the fill slope.

When selecting various inlet configurations, structural stability, aesthetics, erosion control, and fill retention should be considered.

Culvert hydraulic capacity may be improved by selecting appropriate inlets. Because the natural channel is usually wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. A more gradual flow transition lessens the energy loss and thus creates a more hydraulically efficient inlet condition. Beveled inlet edges are more efficient than square edges. Side-tapered inlets and slope-tapered inlets, commonly referred to as improved inlets, further reduce head loss due to flow contraction. Depressed inlets, such as slope-tapered inlets, increase the effective head on the flow control section, thereby further increasing the culvert efficiency.

Section 2 — Design Considerations

Economics

A wide spectrum of flood flows with associated probabilities occurs at the culvert site during its service life. The benefits of constructing a large capacity culvert to accommodate all of these events with no detrimental flooding effects are normally outweighed by the initial construction costs. Therefore, an economic analysis of the trade-offs should be conducted.

The initial cost is only a small part of the total cost over the lifetime of the culvert. Understanding how the culvert operates at discharges other than the design discharge can help you define some of the longer-term operational costs.

The cost of traffic detours can be the most important if factors such as the cost of emergency vehicle response time or detour distance and cost of operation per vehicle mile are considered, especially if there is a large average daily traffic rate.

Reduced to an annual cost on the basis of the anticipated service life, the long-term costs of a culvert operation include the following:

- initial cost of the culvert
- cost of damage to the roadway
- cost of damage to the culvert and associated appurtenances
- cost of damage to the stream (approach and exit)
- cost of damage to upstream and downstream private or public property.

For minor stream crossings, the use of the Design Flood and Check Flood Standards table may preclude the need for a detailed economic analysis (see Chapter 4). A more rigorous investigation, such as a risk analysis may be needed for large culvert installations or when deviations from recommended design frequencies are indicated. Refer to Chapter 3 for discussion design by Evaluation of Risk assessment.

Site Data

The survey should provide you with sufficient data for locating the culvert and identifying information on all features affected by installation of the culvert, such as elevations and locations of houses, commercial buildings, croplands, roadways, and utilities. See Chapter 3 Process and Procedures and Chapter 4 Hydrology.

Culvert Location

Culvert location involves the horizontal and vertical alignment of the culvert with respect to both the stream and the highway. The culvert location affects hydraulic performance of the culvert, stream and embankment stability, construction and maintenance costs, and safety and integrity of the highway.

Ideally, you place a culvert in the natural channel (see Figure 8-1). This location usually provides good alignment of the natural flow with the culvert entrance and outlet. It usually requires little structural excavation or channel work, which requires a USACE permit.

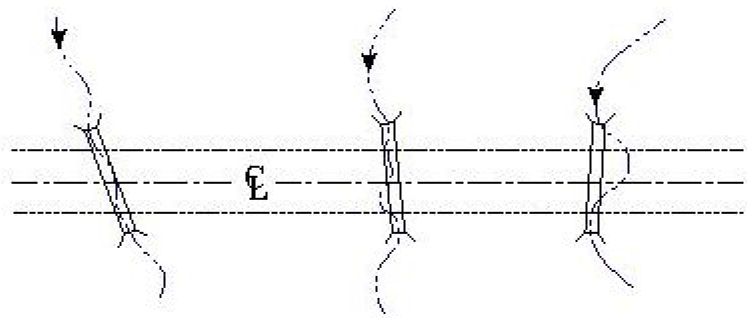


Figure 8-1. Culvert Placement Locations

Establishing the culvert's vertical orientation is usually a matter of placing the upstream flow line and downstream flow line elevations of the culvert at the same elevations as the existing streambed.

In some instances, the upstream flowline may need to be lowered or raised. Lowering the upstream flowline can provide an improved hydraulic operation but may create maintenance problems due to a higher potential for both sedimentation and scour. However, lowering the upstream flowline can also decrease the slope of the culvert, thereby slowing the velocity and decreasing the potential for downstream scour.

The placement of the downstream flowline of the culvert at a level higher than the roadway embankment toe of slope should be avoided. Such a configuration results in a waterfall that increases the potential for erosion.

Sometimes, extending a culvert to accommodate a widened roadway requires changing the flowline slope at one or both ends. Such a configuration is called a broken back culvert. In some cases, a broken back configuration can be designed to reduce the outlet velocity by introducing a hydraulic jump inside the culvert.

Waterway Considerations

The installation of a culvert through a highway embankment may significantly constrict the floodplain. Therefore, pre-construction data should be collected to predict the consequences of the culvert alteration. Refer to Chapters 4, 5, and 7 for information on site surveys and data collection, hydrology, NFIP criterion, and channel properties.

The longitudinal slope of the existing channel in the vicinity of the proposed culvert should be determined in order to establish culvert vertical profile and to define flow characteristics in the natural stream. Often, the proposed culvert can be positioned at the same longitudinal slope as the streambed.

The existing channel must be evaluated for downstream obstructions that will affect the tailwater. Obstructions may include a narrowing of the channel or another roadway crossing or railroad crossing. Other phenomena which may affect the tailwater may be represented by a high roughness coefficient such as vegetation or excessive sinuosity, usually expressed as Mannings "n" (See Chapter 6, Roughness Coefficients). Other sources may include a decreasing channel slope, or water backed up from another source. The tailwater elevation will affect culvert capacity under outlet control conditions or may even drive a culvert into outlet control which may otherwise operate as inlet control.

The storage capacity upstream of (behind) the culvert may need to be considered, especially if the culvert is to be altered in a way that may increase the headwater.

The channel must be scrutinized downstream for adequate capacity, especially ifa culvert will be replaced with a larger barrel. Increased flow through the larger barrel could be perceived as causing flooding that had not occurred before. Although rare, the situation has been known to occur.

Roadway Data

The proposed or existing roadway affects culvert cost, hydraulic efficiency, and alignment.

Information from the roadway profile and the roadway cross section should be obtained from preliminary roadway drawings or from standard details on roadway sections. If the culvert must be sized prior to the development of preliminary plans, a best estimate of the roadway section must be developed, and the culvert design must be confirmed after the roadway plans are completed.

Roadway cross sections normal to the centerline are typically available from highway plans. However, the required cross section at the stream crossing may be skewed with reference to the roadway centerline. To obtain this section for a proposed culvert, combine roadway plan, profile, and cross-sectional data as necessary.

Preliminary dimensions and features of the culvert should become evident when the desired roadway cross section has been evaluated or established. The dimensions may be obtained by superimposing the estimated culvert barrel on the roadway cross section and the streambed profile, which will establish the inlet and outlet invert elevations. The elevations and the resulting culvert length are approximate since the final culvert barrel size must still be determined.

The roadway embankment represents an obstruction to the flowing stream, much like a dam. The culvert is similar to the normal release structure, and the roadway crest acts as an emergency spillway in the event that the upstream pool (headwater) attains a sufficient elevation. The location of initial overtopping depends on the roadway geometry. Generally, the location of overtopping (roadway sag) should coincide as closely as possible to the location of the majority of flood flow under existing conditions. Since the roadway centerline profile may not represent the high point in the highway cross section, location of the actual low point is critical.

Allowable Headwater

Energy is required to force flow through a culvert. Energy takes the form of an increased water surface elevation on the upstream side of the culvert. The depth of the upstream water surface measured from the invert at the culvert entrance is generally referred to as headwater depth.

The headwater subtended by a culvert is a function of several parameters, including the culvert geometric configuration. The culvert geometric configuration elevation consists of the number of barrels, barrel dimensions, length, slope, entrance characteristics, and barrel roughness characteristics.

Selection of a design flood and allowable design headwater elevation are restricted by the potential for damage to adjacent property, damage to the culvert and the roadway, traffic interruption, hazard to human life, and damage to stream and floodplain environment. Potential damage to adjacent property or inconvenience to owners should be of primary concern in the design of all culverts. By definition, the allowable headwater associated with the design discharge must also be below the roadway, that is, the roadway must be passable. Other possible critical elevations on the highway itself include edge of pavement, sub-grade crown, and top of headwall. In addition, the allowable change in headwater of the 1% AEP should be limited to 1.0 foot if at all practicable. For roadways encroaching on a FEMA-designated floodplain or Special Flood Hazard Area, refer to Chapter 5 for information on FEMA NFIP criteria and procedures.

Culvert installations under high fills may present an opportunity to use a high headwater or ponding to attenuate flood peaks. The possibility of catastrophic failure should be investigated prior to considering deep ponding because a breach in the highway fill could be quite similar to a dam failure:

Culverts should be located and designed for the least disruption of the existing flow distribution. Culvert headwater study should include verification that watershed divides are higher than design headwater elevations. If the divides are not sufficiently high to contain the headwater, if at all possible, culverts of lesser depths or earthen training dikes should be used to avoid diversion across watershed divides. In flat terrain, watershed divides are often undefined or nonexistent.

Outlet Velocity

The two basic culvert design criteria are allowable headwater and allowable velocity. Similar to the allowable headwater, the allowable outlet velocity is a design criterion that is unique to each culvert site. Allowable headwater usually governs the overall configuration of the culvert. However, the allowable outlet velocity only partially governs the overall culvert configuration but is the governing criterion in the selection and application of various downstream fixtures and appurtenances.

The velocity in the culvert is likely to be higher than that in the channel because the culvert usually constricts the available channel area. This increased velocity can cause streambed scour and bank erosion in the vicinity of the culvert outlet. There may also be eddies resulting from flow expansion. It is important to control the amount of scour at the culvert outlet because of the possibility of undermining of the headwall and loss of support of the culvert itself. Bank erosion may threaten nearby structures and may also disrupt the stability of the channel itself.

Scour prediction is somewhat subjective because the velocity at which erosion will occur is dependent upon many variables such as the characteristics of the bed and bank material, depth of flow in the channel and at the culvert outlet, velocity, velocity distribution, and the amount of sediment and other debris in the flow. Scour developed at the outlet of similar existing culverts in the vicinity is always a good guide in estimating potential scour at the outlet of proposed culverts.

Scour does not develop at all suspected locations because the susceptibility of the stream to scour is difficult to assess and the flow conditions that will cause scour do not occur at all flow rates. At locations where scour is expected to develop only during relatively rare flood events, the most economical solution may be to repair damage after it occurs.

At many locations, use of a simple outlet treatment (e.g., cutoff walls, aprons of concrete or riprap) will provide adequate protection against scour. At other locations, adjustment of the barrel slope may be sufficient to prevent damage from scour.

When the outlet velocity will greatly exceed the erosive velocity in the downstream channel, considerations should be given to energy dissipation devices (e.g., stilling basins, riprap basins). It should be recognized, however, that such structures are costly, many do not provide protection over a wide range of flow rates, some require a high tailwater to perform their intended function, and the outlet velocity of most culverts is not high enough to form a hydraulic jump that is efficient in dissipating energy. Therefore, selection and design of an energy dissipation device to meet the needs at a particular site requires a thorough study of expected outlet flow conditions and the performance of various devices. The cost of dissipation devices may dictate the design that provides outlet protection from low-frequency (high AEP) discharges and accepts the damage caused by larger floods. See Section 5 Velocity Protection and Control Devices.

Velocities of less than about 3 fps usually foster deposition of sediments. Therefore, 3 fps is recommended as a minimum for culvert design and operation.

End Treatments

End treatments serve several different purposes but typically act as a retaining wall to keep the roadway embankment material out of the culvert opening. Secondary characteristics of end treatments include hydraulic improvements, traffic safety, debris interception, flood protection, and prevention of piping (flow through the embankment outside of the culvert).

Traffic Safety

Cross-drainage and longitudinal drainage facilities are usually necessary in any highway project to relieve drainage from the natural phenomenon of runoff to the highway. However, due to their inherent mass and fixed nature, they can pose somewhat of a safety threat to errant vehicles and associated drivers and passengers.

Figure 8-2 shows sketches of various end treatment types. The Bridge Division maintains standard details of culvert end treatments. For requirements and applications, see the Roadway Design Manual.

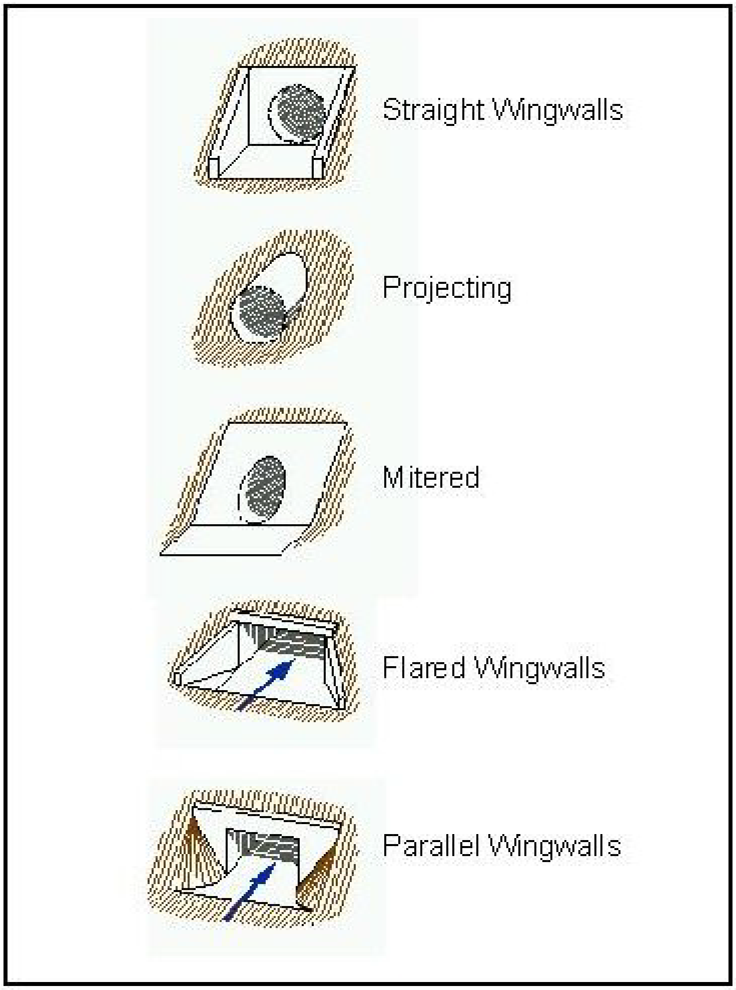


Figure 8-2. Typical Culvert End Treatments

Safety end treatment (SET) of a culvert provides a method of mitigating a less safe condition without interfering with the hydraulic function of the culvert. SETs such as those used with driveway and other small diameter culverts may be more hydraulically efficient by providing both tapered wingwalls and a beveled edge instead of using a mitered section. SETs for larger culverts that are not protected by a railing or guard fence use pipe runners arranged either horizontally or vertically.

Figure 8-3. Pipe mitered to conform to fill slope.

Figure 8-4. Pipe mitired to conform to fill slope.

Figure 8-5. End section conforming to fill slope

Figure 8-6. End section conforming to fill slope with pipe runners

Shielding by metal beam guard fence is a traditional protection method and has proven to be very effective in terms of safety. However, metal beam guard fence also can be more expensive than safety end treatment.

Generally, if clear zone requirements can be met, neither safety end treatment nor protection such as guard fence is necessary. However, some site conditions may still warrant such measures. See the Design Clear Zone Requirements in the Roadway Design Manual for more information.

Culvert Selection

Total culvert cost can vary considerably depending upon the culvert type. Generally, the primary factors affecting culvert type selection in Texas are economics, hydraulic properties, durability, and strength.

The following factors influence culvert type selection:

- fill height
- terrain
- foundation condition
- shape of the existing channel
- roadway profile
- allowable headwater
- stream stage discharge
- frequency-discharge relationships
- cost
- service life

- fish passage.

Culvert type selection includes the choice of materials to meet design life, culvert shapes, and number of culvert barrels. First, select a material that satisfies hydraulic and structural requirements at the lowest cost. Keep in mind that material availability and ease of construction both influence the total cost of the structure.

Second, choose culvert components that are readily available to construction contractors. Such material choices usually will assure better bid prices for the project.

Some commonly used combinations are as follows:

- Pipe (concrete, steel, aluminum, plastic): circular or pipe-arch and elliptical (CMP only).
- Structural-plate (steel or aluminum): circular, pipe-arch, elliptical, or arch.
- Box (or rectangular) (single or multiple barrel boxes or multiple boxes): concrete box culvert or steel or aluminum box culvert.
- Long span (structural-plate, steel or aluminum): low-profile arch, high profile arch, elliptical, or pear.

Culvert Shapes

Typically, several shapes provide hydraulically adequate design alternatives:

- Circular -- The most common shape used for culverts, this shape is available in various strengths and sizes. The need for cast-in-place construction is generally limited to culvert end treatments and appurtenances.
- Pipe-arch and elliptical – Generally used in lieu of circular pipe where there is limited cover or overfill, structural strength characteristics usually limit the height of fill over these shapes when the major axis of the elliptical shape is laid in the horizontal plane. These shapes are typically more expensive than circular shapes for equal hydraulic capacity.
- Box (or rectangular) -- A rectangular culvert lends itself more readily than other shapes to low allowable headwater situations. The height may be lowered and the span increased to satisfy hydraulic capacity with a low headwater. In addition, multiple barrel box culverts accommodate large flow rates with a low profile.
- Modified box -- Economical under certain construction situations, the longer construction time required for cast-in-place boxes can be an important consideration in the selection of this type of culvert. Pre-cast concrete and metal box sections have been used to overcome this disadvantage.
- Arch -- Arch culverts span a stream using the natural streambed as the bottom. As a result arch culverts serve well in situations where the designer wishes to maintain the natural stream bottom for reasons such as fish passage. Nevertheless, the scour potential and the structural

stability of the streambed must be carefully evaluated. Structural plate metal arches are limited to use in low cover situations.

The terrain often dictates the need for a low profile due to limited fill height or potential debris clogging.

Multiple Barrel Boxes

Culverts consisting of more than one box are useful in wide channels where the constriction or concentration of flow must be kept to a minimum. Alternatively, low roadway embankments offering limited cover may require a series of small openings. In addition, the situation may require separating the boxes to maintain flood flow distribution. As a general recommendation, where a culvert consists of more than one barrel, shapes of uniform geometry and roughness characteristics should be used to maintain uniform flow distribution. Locations where debris flow may obstruct the culvert entrance may be better served with a clear span bridge.

Certain situations warrant placing boxes at various elevations. Placing one box at the natural stream flowline and placing additional boxes slightly higher is good practice for the following reasons:

- the configuration does not require widening the natural channel
- the side boxes provide overflow (flood) relief when needed but do not silt up or collect debris when dry
- the minimal stream modification supports environmental preservation.

Design versus Analysis

Culvert design is an iterative process that starts with reasonable assumptions and culminates with a final selection of material, shape, and placement that satisfies the requirements of function and safety. Culvert analysis is a straight forward process that determines the functioning status of an existing culvert structure or a proposed design.

Culvert analysis includes determination of flow rates, velocities, and water surface elevations for the full range of probabilities (50%, 20%, 10%, 4%, 2%, and 1% AEPs) for both the existing and the proposed conditions. A complete list of the requirements for design can be found in the Documentation Reference Tables in Chapter 3.

Differences exist between computer programs. Simple culvert computer programs have limitations such as how they handle roadway overtopping and upstream momentum. More complex hydraulic programs are not so limited because they include features such as backwater calculations and more data input capabilities. Unless the culvert is hydraulically simple, the more complex hydraulic programs are recommended for use. For situations where the roadway is overtopped at the structure, the simplified computer methods become unstable when overtopping occurs. These errors can be critical when a FEMA NFIP analysis of water surface elevation is required. (See Chapter 5, Federal

Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP) Compliant Design Of Floodplain Encroachments And Minor Structures.)

Culvert Design Process

The culvert design process includes the following basic stages:

1. Define the location, FEMA NFIP status, orientation, shape, and material for the culvert to be designed. In many instances, consider more than a single shape and material.
2. With consideration of the site data, establish allowable outlet velocity (v_{max}) and maximum allowable depth of barrel.
3. Based upon subject discharges (Q), associated tailwater levels (TW), and allowable headwater level (HW_{max}), select an overall culvert configuration -- culvert hydraulic length (L), entrance conditions, and conduit shape and material to be analyzed.
4. Determine the flow type (supercritical or subcritical) to establish the proper approach for determination of headwater and outlet velocity.
5. Determine the headwater elevation and outlet velocity.
6. Adjust slope or shape for excessive outlet velocity if necessary. Check effect on headwater elevation.
7. Continue to adjust configuration until headwater elevation and outlet velocity are within allowable limits. It may be necessary to treat any excessive outlet velocity separately from headwater and control by other means such as velocity controls.

Design Guidelines and Procedure for Culverts

The flow charts of Figure 8-7 and Figure 8-8 can guide the hydraulic designer in computing for the vast majority of culvert design situations.

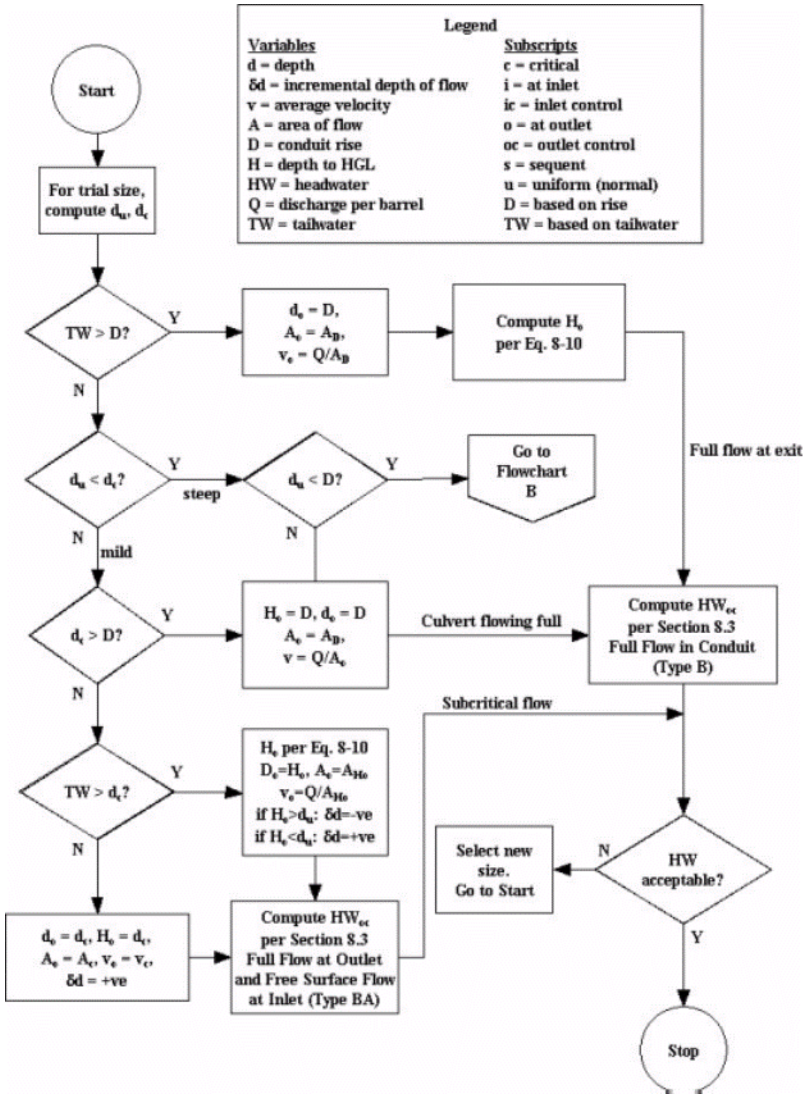


Figure 8-7. Flow Chart A - Culvert Design Procedure

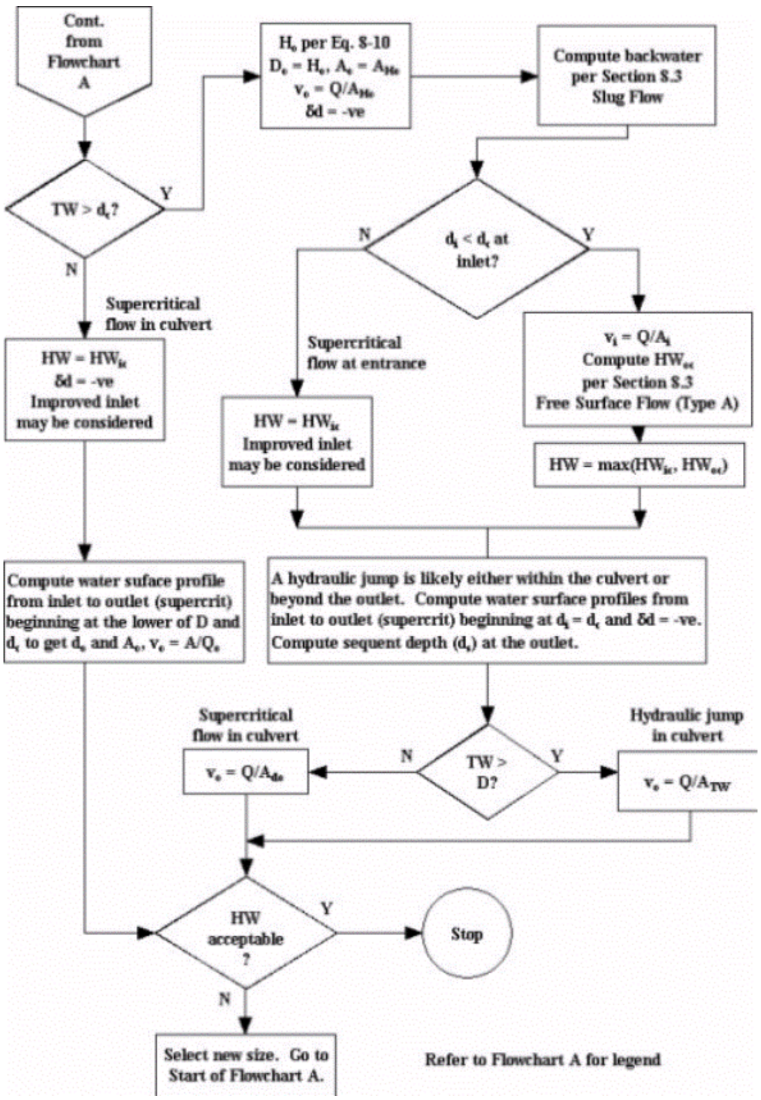


Figure 8-8. Flow Chart B - Culvert Design Procedure (cont.)

The following is a step-by-step culvert design procedure for a standard culvert configuration , i.e. straight in profile with one or more barrels of equal size. Each of the configurations considered in the iterative process of design process influences a unique flow type. Each new iteration requires a determination of whether there is inlet or outlet control.

1. Establish an initial trial size. The trial size may be picked at random or judiciously, based on experience. However, one convenient rule-of-thumb is to assume inlet control and proceed as follows: Determine the maximum practical rise of culvert (D_{max}) and the maximum allowable headwater depth (HW_{max}). Determine a trial head using Equation 8-1.

$$h = HW_{max} - \frac{D_{max}}{2}$$

Equation 8-1.

where:

h = allowable effective head (ft. or m)

HW_{max} = allowable headwater depth (ft. or m)

D_{max} = maximum conduit rise (ft. or m).

Use Equation 8-2 (a form of the orifice equation) to determine the required area, A, for the design discharge, Q. This assumes an orifice coefficient of 0.5, which is reasonable for initial estimates only.

$$A = 0.45\frac{Q}{h^{0.5}}$$

Equation 8-2.

where:

A = approximate cross-sectional area required (sq.ft. or m^2)

Q = design discharge (cfs or m^3/s).

Decide on the culvert shape:

- A properly designed culvert has an effective flow area similar in height and width to the approach channel section so that approach velocities and through-culvert velocities are similar.
- For a box culvert, determine the required width, W, as A/D_{max}. Round W to the nearest value that yields a whole multiple of standard box widths. Divide W by the largest standard span S for which W is a multiple. This yields the number of barrels, N. At this point, the determination has been made that the initial trial configuration will be N - S D_{max} L, where L is the length of the barrel in feet.
- For a circular pipe culvert, determine the ratio of area required to maximum barrel area according to Equation 8-3.

$$\frac{4A}{\pi D_{max}^{2}} \leq N$$

Equation 8-3.

- Round this value to the nearest whole number to get the required number of barrels, N. At this point, the determination has been made that the initial trial size culvert will be N - D L circular pipe.
- For other shapes, provide an appropriate size such that the cross section area is approximately equal to A.

2. Determine the design discharge per barrel as Q/N. This assumes that all barrels are of equal size and parallel profiles with the same invert elevations. The computations progress using one barrel with the appropriate apportionment of flow.

3. Perform a hydraulic analysis of the trial configuration. Generally, a computer program or spreadsheet would be used. Nomographs and simplified hand methods should be used only for preliminary estimates. For the trial configuration determine the inlet control headwater (HW_{ic}), the outlet control headwater (HW_{oc}) and outlet velocity (v_o) using Flow Chart A shown in Figure 8-7. Flow Chart A references Flow Chart B, which is shown in Figure 8-8.

4. Evaluate the trial design. At this step in the design process, you have calculated a headwater and outlet velocity for the design discharge through a trial culvert configuration has been calculated.

- If the calculated headwater is equal to or is not appreciably lower than the allowable headwater (an indication of culvert efficiency), the design is complete. A good measure of efficiency is to compare the calculated headwater with the culvert depth D. If the headwater is less than the depth, the configuration may not be efficient.
- If the calculated headwater is considerably lower than the allowable headwater or lower than the culvert depth D, a more economical configuration may be possible. Choose the trial culvert configuration by reducing the number of barrels, span widths, diameter, or other geometric or material changes. Repeat the calculations from step 2.
- If the calculated headwater is equal to or is not appreciably lower than the actual headwater and the culvert is operating as inlet control, an improved inlet may be in order.
- If the calculated headwater is greater than the actual headwater, change the trial culvert configuration to increase capacity by adding barrels, widening spans, and increasing diameter. Repeat the calculations from step 2.
- If the operation is not inlet control, then the culvert geometry design is complete.
- If the culvert is operating with inlet control, the possibility exists for improving the entrance conditions with the aim of reducing the overall cost of the structure. Investigate the design of a flared (or tapered) inlet and associated structure.Because of the cost of the improved inlet, make a careful economic comparison between the design with a normal entrance and the design with an improved inlet.
- Check outlet velocities against the predetermined maximum allowable for the site. The culvert for which the calculated headwater is satisfactory may have an excessive outlet velocity. Excessive velocities are usually caused by a steep slope or a computational error. The definition of "excessive" is usually an engineering judgment based on local and soil conditions, but as a general rule, anything over 12 fps is considered excessive.

Consider any required outlet control or protection device as part of the hydraulic design. It is normal for a properly designed culvert to have an outlet velocity that is greater than the natural stream velocity.

1. Develop a hydraulic performance curve using the procedures outlined in the Hydraulic Operation of Culverts section. An overall hydraulic performance curve for the designed culvert indicates headwater and outlet velocity characteristics for the entire range of discharges. At an

absolute minimum, the additional analysis of the 1% AEP discharge is required for both the existing and the proposed conditions.

- The design can be completed if the results of the headwater and outlet velocity represent an acceptable risk and conform to FEMA NFIP requirements. (See Chapter 2 and pertinent parts of the Project Development Policy Manual for more details.)
- However, if any of the hydraulic characteristics are unacceptable, some adjustment to the culvert design may be in order.

Evaluate other culvert performance risks. Identify and evaluate the potential for increased impact associated with different flood conditions.

Section 3 — Hydraulic Operation of Culverts

Parameters

The hydraulic operation and performance of a culvert involve a number of factors. You must determine, estimate, or calculate each factor as part of the hydraulic design or analysis.

The following procedures assume steady flow but can involve extensive calculations that lend themselves to computer application. The procedures supersede simplified hand methods of other manuals. TxDOT recommends computer models for all final design applications, although hand methods and nomographs may be used for initial planning.

Headwater under Inlet Control

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. Inlet control is possible when the culvert slope is hydraulically steep ($d_c > d_u$). The control section of a culvert operating under inlet control is located just inside the entrance. When the flow in the barrel is free surface flow, critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Depending on conditions downstream of the culvert inlet, a hydraulic jump may occur in the culvert. Under inlet control, hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. Upstream water surface elevation and inlet geometry (barrel shape, cross-sectional area, and inlet edge) are the major flow controls.

A fifth-degree polynomial equation based on regression analysis is used to model the inlet control headwater for a given flow. Analytical equations based on minimum energy principles are matched to the regression equations to model flows that create inlet control heads outside of the regression data range. Equation 8-4 only applies when $0.5 \le HW_{ic}/D \le 3.0$.

$$HW_{ic} = \left[a + bF + cF^2 + dF^3 + eF^4 + fF^5\right]D - 0.5DS_0$$

Equation 8-4.

where:

HW_{ic} = inlet control headwater (ft. or m)

D = rise of the culvert barrel (ft. or m)

a to f = regression coefficients for each type of culvert (see the following table)

S_0 = culvert slope (ft./ft. or m/m)

F = function of average outflow discharge routed through a culvert; culvert barrel rise; and for box and pipe-arch culverts, width of the barrel, B, shown in Equation 8-5.

$$F = 1.8113 \frac{Q}{WD^{3/2}}$$

Equation 8-5.

where:

W = width or span of culvert (ft. or m).

Table 8-1: Regression Coefficients for Inlet Control Equations

Shape and Material	Entrance Type	a	b	c	d	e	f
RCP	Square edge w/headwall	0.087483	0.706578	-0.2533	0.0667	-0.00662	0.000251
-	Groove end w/headwall	0.114099	0.653562	-0.2336	0.059772	-0.00616	0.000243
-	Groove end projecting	0.108786	0.662381	-0.2338	0.057959	-0.00558	0.000205
-	Beveled ring	0.063343	0.766512	-0.316097	0.08767	-0.00984	0.000417
-	Improved (flared) inlet	0.2115	0.3927	-0.0414	0.0042	-0.0003	-0.00003
CMP	Headwall	0.167433	0.53859	-0.14937	0.039154	-0.00344	0.000116
-	Mitered	0.107137	0.757789	-0.3615	0.123393	-0.01606	0.000767
-	Projecting	0.187321	0.567719	-0.15654	0.044505	-0.00344	0.00009
-	Improved (flared) inlet	0.2252	0.3471	-0.0252	0.0011	-0.0005	-0.00003
Box	30-70º flared wingwall	0.072493	0.507087	-0.11747	0.02217	-0.00149	0.000038
-	Parallel to 15º wingwall	0.122117	0.505435	-0.10856	0.020781	-0.00137	0.0000346
-	Straight wingwall	0.144138	0.461363	-0.09215	0.020003	-0.00136	0.000036
-	45º wingwall w/top bevel	0.156609	0.398935	-0.06404	0.011201	-0.00064	0.000015
-	Parallel headwall w/ bevel	0.156609	0.398935	-0.06404	0.011201	-0.00064	0.000015
-	30º skew w/chamfer edges	0.122117	0.505435	-0.10856	0.020781	-0.00137	0.000034
-	10-45º skew w/bevel edges	0.089963	0.441247	-0.07435	0.012732	-0.00076	0.000018
Oval B>D	Square edge w/headwall	0.13432	0.55951	-0.1578	0.03967	-0.0034	0.00011
-	Groove end w/headwall	0.15067	0.50311	-0.12068	0.02566	-0.00189	0.00005
-	Groove end projecting	-0.03817	0.84684	-0.32139	0.0755	-0.00729	0.00027

Table 8-1: Regression Coefficients for Inlet Control Equations

Shape and Material	Entrance Type	a	b	c	d	e	f
Oval D>B	Square edge w/headwall	0.13432	0.55951	-0.1578	0.03967	-0.0034	0.00011
-	Groove end w/headwall	0.15067	0.50311	-0.12068	0.02566	-0.00189	0.00005
-	Groove end projecting	-0.03817	0.84684	-0.32139	0.0755	-0.00729	0.00027
CM Pipe arch	Headwall	0.111261	0.610579	-0.194937	0.051289	-0.00481	0.000169
-	Mitered	0.083301	0.795145	-0.43408	0.163774	-0.02491	0.001411
-	Projecting	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293
Struct plate Pipe arch	Projecting—corner plate (17.7 in. or 450 mm)	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293
-	Projecting—corner plate (30.7 in. or 780 mm)	0.12263	0.4825	-0.00002	-0.04287	0.01454	-0.00117
CM arch (flat bottom)	Parallel headwall	0.111281	0.610579	-0.1949	0.051289	-0.00481	0.000169
-	Mitered	0.083301	0.795145	-0.43408	0.163774	-0.02491	0.001411
-	Thin wall projecting	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293

For $HW_i/D > 3.0$, Equation 8-6, an orifice equation, is used to estimate headwater:

- Determine the potential head from the centroid of the culvert opening, which is approximated as the sum of the invert elevation and one half the rise of the culvert. The effective area, A, and orifice coefficient, C, are implicit.
- Determine the coefficient, k, by rearranging Equation 8-6 using the discharge that creates a HW/D ratio of 3 in the regression equation, Equation 8-7 (i.e., the upper limit of the Equation 8-1):

$$HW_i = \left[\frac{Q}{k}\right]^2 + \frac{D}{2}$$

Equation 8-6.

where:

HW_i = inlet control headwater depth (ft. or m)

Q = design discharge (cfs or m^3/s)

k = orifice equation constant

D = rise of culvert (ft. or m).

$$k = 0.6325 \frac{Q_{3.0}}{D^{1/2}}$$

Equation 8-7.

where:

$Q_{3.0}$ = discharge (cfs or m^3/s) at which HW/D = 3.

Generally for TxDOT designs, it is not considered efficient to design culverts for $HW_i/D < 0.5$. However, if such a condition is likely, an open channel flow minimum energy equation (weir equation) should be used, with the addition of a velocity head loss coefficient. The minimum energy equation, with the velocity head loss adjusted by an entrance loss coefficient, generally describes the low flow portion of the inlet control headwater curve. However, numerical errors in the calculation of flow for very small depths tend to increase the velocity head as the flow approaches zero. This presents little or no problem in most single system cases because the flows that cause this are relatively small.

In many of the required calculations for the solution of multiple culverts, the inlet control curve must decrease continuously to zero for the iterative calculations to converge. Therefore, computer models modify this equation to force the velocity head to continually decrease to zero as the flow approaches zero.

The “Charts” in HDS-5 (FHWA, Hydraulic Design of Highway Culverts) provide guidance for graphical solution of headwater under inlet control.

Headwater under Outlet Control

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. Outlet control is likely only when the hydraulic grade line inside the culvert at the entrance exceeds critical depth. (See Chapter 6 for Hydraulic Grade Line Analysis.) Therefore, outlet control is most likely when the culvert is on a mild slope ($d_n > d_c$). It is also possible to experience outlet control with a culvert on a steep slope ($d_n < d_c$) with a high tailwater such that subcritical flow or full flow exists in the culvert.

The headwater of a culvert in outlet control is a function of discharge, conduit section geometry, conduit roughness characteristics, length of the conduit, profile of the conduit, entrance geometry (to a minor extent), and (possibly) tailwater level.

The headwater of a culvert under outlet control can be adjusted, for practical purposes, by modifying culvert size, shape, and roughness. Both inlet control and outlet control need to be considered to determine the headwater. The following table provides a summary conditions likely to control the culvert headwater. Refer to Figure 8-4 and Figure 8-5 to identify the appropriate procedures to make the determination.

Table 8-2: Conditions Likely to Control Culvert Headwater

Description	Likely Condition
Hydraulically steep slope, backwater does not submerge critical depth at inside of inlet	Inlet control
Hydraulically steep slope, backwater submerges critical depth at inside of inlet	Outlet control
Hydraulically steep slope, backwater close to critical depth at inlet	Oscillate between inlet and outlet control.
Hydraulically mild slope	Outlet control

Outlet control headwater is determined by accounting for the total energy losses that occur from the culvert outlet to the culvert inlet. Figure 8-7 and Figure 8-8 and associated procedures in Section 4 should be used to analyze or design a culvert.

Outlet control headwater HW_{oc} depth (measured from the flowline of the entrance) is expressed in terms of balancing energy between the culvert exit and the culvert entrance as indicated by Equation 8-8.

$$HW_{oc} + h_{va} = h_e + h_{vi} + \sum h_f - S_o L + H_o$$

Equation 8-8.

where:

HW_{oc} = headwater depth due to outlet control (ft. or m)

h_{va}= velocity head of flow approaching the culvert entrance (ft. or m)

h_{vi}= velocity head in the entrance (ft. or m) as calculated using Equation 8-9.

h_e= entrance head loss (ft. or m) as calculated using Equation 8-16

h_f= friction head losses (ft. or m) as calculated using Equation 8-11

S_o= culvert slope (ft./ft. or m/m)

L = culvert length (ft. or m)

H_o = depth of hydraulic grade line just inside the culvert at outlet (ft. or m) (outlet depth).

$$h_v = \left[\frac{v^2}{2g}\right]$$

Equation 8-9.

where:

v = flow velocity in culvert (ft./s or m/s).

g = the gravitational acceleration = 32.2 ft/ s^2 or 9.81 m/s^2.

For convenience energy balance at outlet, energy losses through barrel, and energy balance at inlet should be considered when determining outlet control headwater.

When the tailwater controls the outlet flow, Equation 8-10 represents the energy balance equation at the conduit outlet. Traditional practice has been to ignore exit losses. If exit losses are ignored, the hydraulic grade line inside the conduit at the outlet, outlet depth, H_o, should be assumed to be the same as the hydraulic grade line outside the conduit at the outlet and Equation 8-10 should not be used.

$$H_o = TW + h_{TW} + h_o - h_{vo}$$

Equation 8-10.

where:

h_{vo} = velocity head inside culvert at outlet (ft. or m)

h_{TW} = velocity head in tailwater (ft. or m)

h_o = exit head loss (ft. or m).

The outlet depth, H_o, is the depth of the hydraulic grade line inside the culvert at the outlet end. The outlet depth is established based on the conditions shown below.

Table 8-3: Outlet Depth Conditions

If...	And...	Then...
Tailwater depth (TW) exceeds critical depth (d_c) in the culvert at outlet	Slope is hydraulically mild	Set H_o using Equation 8-10, using the tailwater as the basis.
Tailwater depth (TW) is lower than critical depth (d_c) in culvert at outlet	Slope is hydraulically mild	Set H_o as critical depth.
Uniform depth is higher than top of the barrel	Slope is hydraulically steep	Set H_o as the higher of the barrel depth (D) and depth using Equation 8-10.
Uniform depth is lower than top of barrel and tail-water exceeds critical depth	Slope is hydraulically steep	Set H_o using Equation 8-10.
Uniform depth is lower than top of barrel and tail-water is below critical depth	Slope is hydraulically steep	Ignore, as outlet control is not likely.

NOTE: For hand computations and some computer programs, H_o is assumed to be equal to the tailwater depth (TW). In such a case, computation of an exit head loss (h_o) would be meaningless since the energy grade line in the culvert at the outlet would always be the sum of the tailwater depth and the velocity head inside the culvert at the outlet (h_{vo}).

Energy Losses through Conduit

Department practice is to consider flow through the conduit occurring in one of four combinations:

- Free surface flow (Type A) through entire conduit.
- Full flow in conduit (Type B).
- Full flow at outlet and free surface flow at inlet (Type BA).
- Free surface at outlet and full flow at inlet (Type AB).

Free Surface Flow (Type A)

If free surface flow is occurring in the culvert, the hydraulic parameters are changing with flow depth along the length of the culvert as seen in Figure 8-9. It is necessary to calculate the backwater profile based on the outlet depth, H_o.

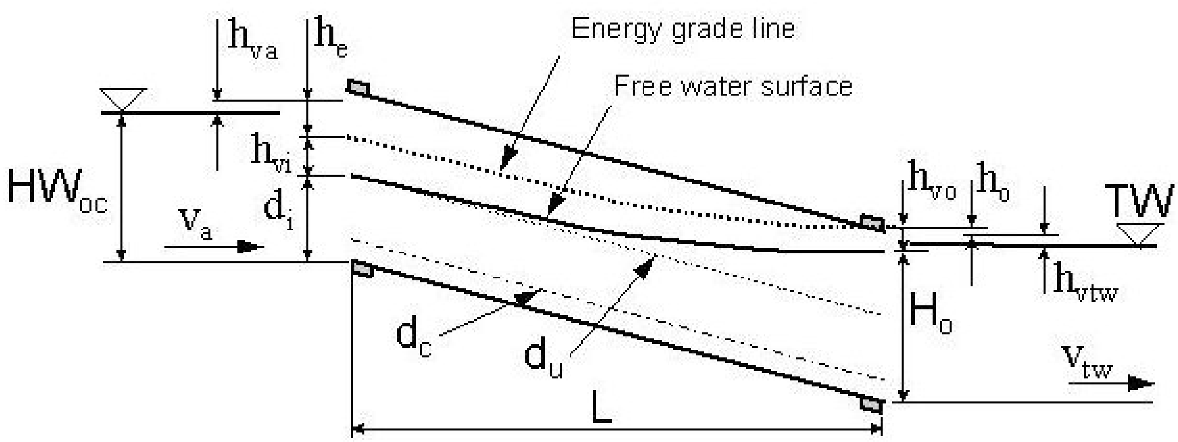


Figure 8-9. Outlet Control Headwater for Culvert with Free Surface

By definition, a free-surface backwater from the outlet end of a culvert may only affect the headwater when subcritical flow conditions exist in the culvert. Subcritical, free-surface flow at the outlet will exist if the culvert is on a mild slope with an outlet depth (H_o) lower than the outlet soffit or if the culvert is on a steep slope with a tailwater higher than critical depth at the culvert outlet and lower than the outlet soffit.

The Direct Step Backwater Method is used to determine the water surface profile (and energy losses) though the conduit. The depth, H_o, is used as the starting depth, d_1. For subcritical flow, the calculations begin at the outlet and proceed in an upstream direction. Use the depth, H_o, as the starting depth, d_1, in the Direct Step calculations.

When using the direct step method, if the inlet end of the conduit is reached without the calculated depth exceeding the barrel depth (D), it verifies that the entire length of the conduit is undergoing free surface flow. Set the calculated depth (d_2) at the inlet as H_i and refer to Energy Balance at Inlet to determine the headwater.

When using the direct step method, if the calculated depth (d_2) reaches or exceeds the barrel depth (D), the inside of the inlet is submerged. Refer to Type AB - Free surface at outlet and full flow at inlet for a description. This condition is possible if the theoretical value of uniform depth is higher than the barrel depth.

Full Flow in Conduit (Type B)

If full flow is occurring in the conduit, rate of energy losses through the barrel is constant (for steady flow) as seen in Figure 8-10. The hydraulic grade line is calculated based on outlet depth, H_o, at the outlet.

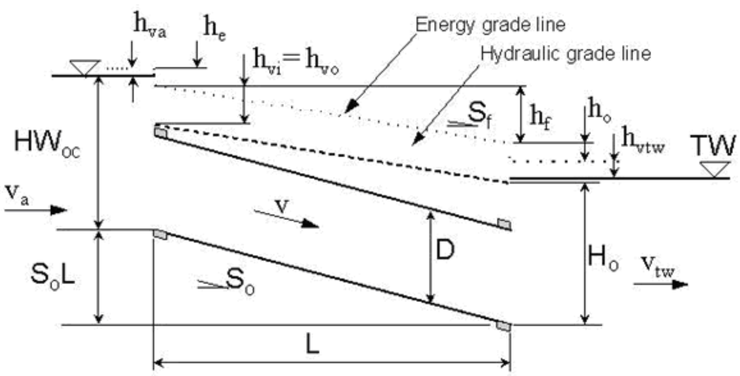


Figure 8-10. Outlet Control, Fully Submerged Flow

Full flow at the outlet occurs when the outlet depth (H_o) equals or exceeds barrel depth D. Full flow is maintained throughout the conduit if friction slope is steeper than conduit slope, or if friction slope is flatter than conduit slope but conduit is not long enough for the hydraulic grade line to get lower than the top of the barrel.

NOTE: Refer to Type BA – Submerged Exit, Free flow at Inlet to determine whether the entire conduit flows full.

Equation 8-11determines the energy loss (friction loss) through the conduit:

$$h_f = S_f L$$

Equation 8-11.

where:

h_f= head loss due to friction in the culvert barrel (ft. or m)

S_f= friction slope (ft. or m) (See Equation 8-13.)

L = length of culvert containing full flow (ft. or m).

Equation 8-12 is used to compute the depth of the hydraulic grade line at the inside of the inlet end of the conduit. Refer to Energy Balance at Inlet to determine the headwater.

$$H_i = H_o + h_f - S_o L$$

Equation 8-12.

where:

H_i = depth of hydraulic grade line at inlet (ft. or m)

h_f = friction head losses (ft. or m) as calculated using Equation 8-11.

S_o = culvert slope (ft./ft. or m/m)

L = culvert length (ft. or m)

Ho = outlet depth (ft. or m).

Equation 8-13 is used to calculate friction slope. If friction slope is flatter than the conduit slope, the hydraulic grade line may drop below the top of the barrel. If this occurs, refer to Type BA - Full Flow at the outlet and free surface flow at the inlet.

$$S_f = \left(\frac{Qn}{zR^{2/3}A}\right)^2$$

Equation 8-13.

where:

S_f = friction slope (ft./ft. or m/m)

z = 1.486 for English measurements and 1.0 for metric.

Full Flow at Outlet and Free Surface Flow at Inlet (Type BA)

If the friction slope is flatter than the conduit slope, it is possible that full flow may not occur along the entire length of the culvert (see the Table 8-5 on Entrance Loss Coefficients). The following steps should be followed:

1. Determine the length over which full flow occurs (L_f) is using the geometric relationship shown in Equation 8-14 (refer to Table 8-5 on Entrance Loss Coefficients):

$$L_f = \frac{H_o - D}{S_o - S_f}$$

Equation 8-14.

where:

L_f = length over which full flow occurs (ft. or m)

S_o= culvert slope (ft./ft. or m/m)

S_f= friction slope (ft./ft. or m/m)

H_o= outlet depth (ft. or m)

D = Conduit barrel height (ft. or m).

Use the following table to determine how to proceed considering a conduit length L.

Table 8-4: Conduit Length (L) Procedure Determination

If…	Then proceed to…	Comment
If $S_f \geq S_o$	Type B energy loss calculations	Entire length of culvert is full
If $L_f \geq L$	Type B energy loss calculations	Entire length of culvert is full
If $L_f < L$	Step 2.	Outlet is full but free surface flow at inlet

2. Determine Type BA free surface losses, if applicable. Free surface flow begins at the point of intersection of the hydraulic grade line and the soffit of the culvert barrel as shown in Figure 8-11. If this condition occurs, determine the depth of flow at the inlet using the Direct Step Method with the starting depth (d_1) equal to the barrel rise (D) and starting at the location along the barrel at which free surface flow begins.
3. Determine Type BA hydraulic grade line at inlet, if applicable. When the inlet end of the conduit is reached using the direct step method, set the calculated depth at the inlet as H_i and refer to Energy Balance at Inlet to determine the headwater.

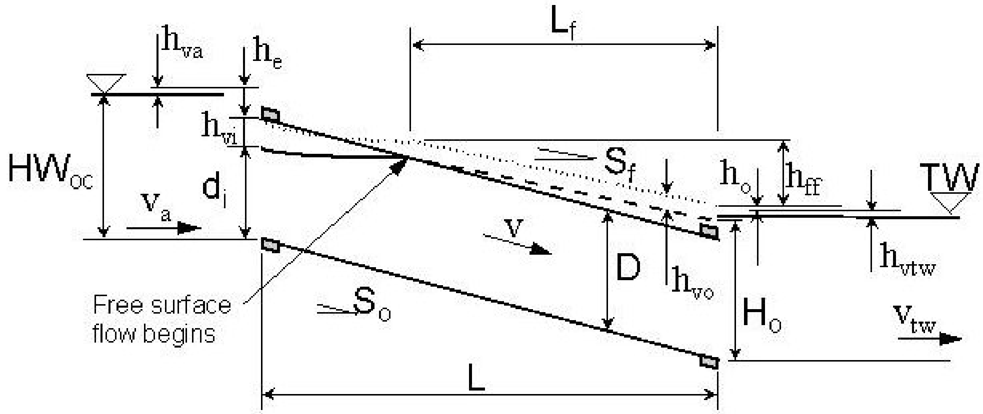


Figure 8-11. Point at Which Free Surface Flow Begins

Free Surface at Outlet and Full Flow at Inlet (Type AB)

When the outlet is not submerged, full flow will begin within the conduit if the culvert is long enough and the flow high enough. Figure 8-12 illustrates this condition. This condition is possible if the theoretical value of uniform depth is higher than the barrel depth. The following steps should be followed:

1. Check Type AB uniform depth. Compare calculated uniform depth and the barrel depth, D. If the theoretical value of uniform depth is equal to or higher than the barrel depth, proceed to Free Surface Losses. Otherwise, refer to Free Surface Flow (Type A).
2. Determine Type AB free surface losses, if applicable. Refer to Water Surface Profile Calculations, Free Surface Flow to determine the water surface profile in the conduit. If the computed depth of flow reaches or exceeds the barrel depth before the end of the conduit, note the position along the conduit at which this occurs and proceed to full flow losses below. Otherwise, complete the procedure described under Free Surface Flow.
3. Determine Type AB full flow losses, if applicable. Begin full flow calculations at the point along the conduit where the computed water surface intersects the soffit of the barrel as determined above. Determine the energy losses through the remainder of the conduit using Equation 8-11 but substituting L_f, the remaining conduit length, for L.
4. Determine Type AB hydraulic grade line at inlet, if applicable. Compute the depth of the hydraulic grade line, H_i, at the inside of the inlet end of the conduit using Equation 8-12. Use the barrel height D as the starting hydraulic grade line depth in place of H_o, and use the remaining length, L_f, in place of L. Refer to Energy Balance at Inlet to determine headwater depth.

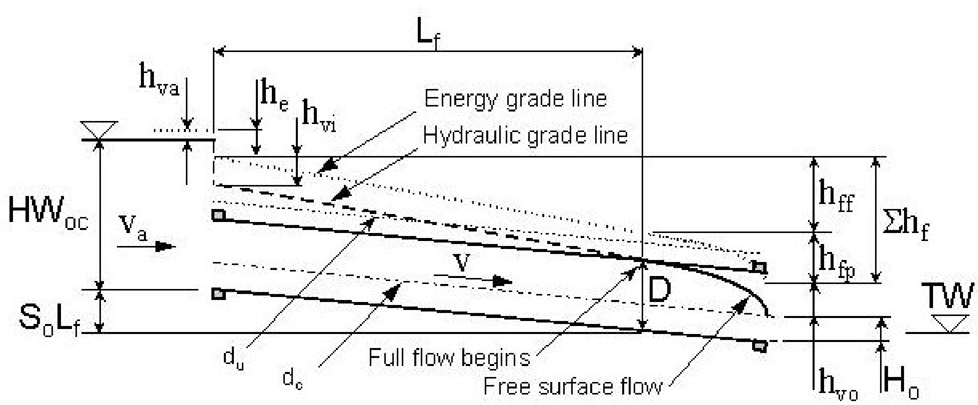


Figure 8-12. Headwater Due to Full Flow at Inlet and Free Surface at Outlet

Energy Balance at Inlet

The outlet control headwater, HW_{oc}, is computed by balancing the energy equation, depicted as Equation 8-15. The hydraulic grade at the inside face of the culvert at the entrance will need to be known. See Energy Losses through Conduit. The velocity at the entrance (v_i) is used to compute the velocity head at the entrance (h_{vi}).

$$HW_{oc} = H_i + h_{vi} + h_e - h_{va}$$

Equation 8-15.

where:

HW_{oc} = headwater depth due to outlet control (ft. or m)

h_{va} = velocity head of flow approaching the culvert entrance (ft. or m)

h_{vi} = velocity head in the entrance (ft. or m) as calculated using Equation 8-9

h_e = entrance head loss (ft. or m) as calculated using Equation 8-16

H_i = depth of hydraulic grade line just inside the culvert at inlet (ft. or m).

Generally, when using Equation 8-15, the velocity approaching the entrance may be assumed to be negligible so that the headwater and energy grade line are coincident just upstream of the upstream face of the culvert. This is conservative for most department needs. The approach velocity may need to be considered when performing the following tasks:

- determining the impact of a culvert on FEMA designated floodplains
- designing or analyzing a culvert used as a flood attenuation device where the storage volumes are very sensitive to small changes in headwater.

The entrance loss, h_e, depends on the velocity of flow at the inlet, v_i, and the entrance configuration, which is accommodated using an entrance loss coefficient, C_e.

$$h_e = C_e\left[\frac{v_i^2}{2g}\right]$$

Equation 8-16.

where:

C_e = entrance loss coefficient

V_i = flow velocity inside culvert inlet(fps or m/s).

NOTE: The pipes of pipe runner SETs have been proven to be within the tolerance of the entrance loss equations. Therefore, the entrance should be evaluated solely for its shape and the effect of the pipes should be ignored.

Values of C_e are shown on the following table (Entrance Loss Coefficients) based on culvert shape and entrance condition. (AASHTO Highway Drainage Manual Guidelines, 4th Ed, Table 4-1)

Table 8-5: Entrance Loss Coefficients (C_e)

Concrete Pipe	C_e
Projecting from fill, socket end (groove end)	0.2

Table 8-5: Entrance Loss Coefficients (C_e)

Concrete Pipe	C_e
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls:	-
◆ Socket end of pipe (groove end)	0.2
◆ Square-edge	0.5
◆ Rounded (radius 1/12 D)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Beveled edges, 33.7º or 45º bevels	0.2
Side- or slope-tapered inlet	0.2
Corrugated Metal Pipe or Pipe Arch	-
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, 33.7º or 45º bevels	0.2
Side- or slope-tapered inlet	0.2
Reinforced Concrete Box	-
Headwall parallel to embankment (no wingwalls):	-
◆ Square-edged on 3 edges	0.5
◆ Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30º to 75º to barrel:	-
◆ Square-edged at crown	0.4
◆ Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10º to 25º 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

Slug Flow

When the flow becomes unstable, a phenomenon termed slug flow may occur. In this condition the flow oscillates between inlet control and outlet control due to the following instances:

- Flow is indicated as supercritical, but the tailwater level is relatively high.
- Uniform depth and critical depth are relatively high with respect to the culvert barrel depth.
- Uniform depth and critical depth are within about 5% of each other.

The methods recommended in this chapter accommodate the potential for slug flow to occur by assuming the higher of inlet and outlet control headwater.

Determination of Outlet Velocity

The outlet velocity, v_o, depends on the culvert discharge (Q) and the cross-sectional area of flow at the outlet (A_o) as shown in Equation 8-17.

$$v_o = \frac{Q}{A_o}$$

Equation 8-17.

1. Assign the variable d_o as the depth with which to determine the cross-sectional area of flow at the outlet.
2. For outlet control, set the depth, d_o, equal to the higher of critical depth (d_c) and tailwater depth (TW) as long as the value is not higher than the barrel rise (D) as shown in Figure 8-13.
3. If the conduit will flow full at the outlet, usually due to a high tailwater or a conduit capacity lower than the discharge, set d_o to the barrel rise (D) so that the full cross-sectional area of the conduit is used as shown in Figure 8-14.

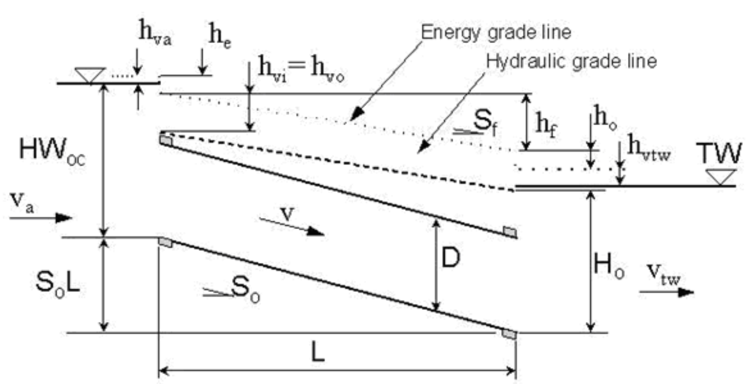


Figure 8-13. Cross Sectional Area based on the Higher of Critical Depth and Tailwater

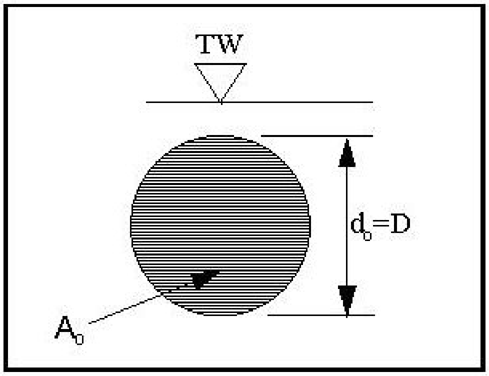


Figure 8-14. Cross Sectional Area Based on Full Flow

Depth Estimation Approaches

For inlet control under steep slope conditions, estimate the depth at the outlet using one of the following approaches:

- Use a step backwater method starting from critical depth (d_c) at the inlet and proceed downstream to the outlet: If the tailwater is lower than critical depth at the outlet, calculate the velocity resulting from the computed depth at the outlet. If the tailwater is higher than critical depth, a hydraulic jump within the culvert is possible. The Hydraulic Jump in Culverts subsection below discusses a means of estimating whether the hydraulic jump occurs within the culvert. If the hydraulic jump does occur within the culvert, determine the outlet velocity based on the outlet depth, $d_o = H_o$.
- Assume uniform depth at the outlet. If the culvert is long enough and tailwater is lower than uniform depth, uniform depth will be reached at the outlet of a steep slope culvert: For a short, steep culvert with tailwater lower than uniform depth, the actual depth will be higher than uniform depth but lower than critical depth. This assumption will be conservative; the estimate of velocity will be somewhat higher than the actual velocity. If the tailwater is higher than critical depth, a hydraulic jump is possible and the outlet velocity could be significantly lower than the velocity at uniform depth.

Direct Step Backwater Method

The Direct Step Backwater Method uses the same basic equations as the Standard Step Backwater Method but is simpler to use because no iteration is necessary. In the Direct Step Method, an increment (or decrement) of water depth (δd) is chosen and the distance over which the depth change occurs is computed. The accuracy depends on the size of δd. The method is appropriate for prismatic channel sections such as occur in most conduits. It is useful for estimating supercritical profiles and subcritical profiles.

1. Choose a starting point and starting water depth (d_1). This starting depth depends on whether the profile is supercritical or subcritical. Generally, for culverts, refer to outlet depth and set d_1 to the value of H_0. Otherwise, you may use the following conditions to establish d_1:
   - For a mild slope ($d_c < d_u$) and free surface flow at the outlet, begin at the outlet end. Select the higher of critical depth (d_c) and tailwater depth (TW). Supercritical flow may occur in a culvert on a mild slope. However, most often, the flow will be subcritical when mild slopes exist. Check this assumption.
   - For a steep slope ($d_c > d_u$), where the tailwater exceeds critical depth but does not submerge the culvert outlet, begin at the outlet with the tailwater as the starting depth.
   - For a steep slope in which tailwater depth is lower than critical depth, begin the water surface profile computations at the culvert entrance starting at critical depth and proceed downstream to the culvert exit. This implies inlet control, in which case the computation may be necessary to determine outlet velocity but not headwater.
   - For a submerged outlet in which free surface flow begins along the barrel, use the barrel depth, D, as the starting depth. Begin the backwater computations at the location where the hydraulic grade line is coincident with the soffit of the culvert.
2. The following steps assume subcritical flow on a mild slope culvert for a given discharge, Q, through a given culvert of length, L, at a slope, S_o. Calculate the following at the outlet end of the culvert based on the selected starting depth (d_1):
   - cross-section area of flow, A
   - wetted perimeter, WP
   - velocity, v, from Equation 8-17
   - velocity head, h_v, using Equation 8-9
   - specific energy, E, using Equation 8-18
   - friction slope, S_f, using Equation 8-13.

   Assign the subscript 1 to the above variables (A_1, WP_1, etc.).

$$E = d + \frac{v^2}{2g}$$

Equation 8-18.

where:

E = specific energy (ft. or m)

d = depth of flow (ft. or m)

v = average velocity of flow (fps or m/s)

g = gravitational acceleration = 32.2 ft/ s^2 or 9.81 m/s^2.

3. Choose an increment or decrement of flow depth, δd: if $d_1 > d_u$, use a decrement (negative δd); otherwise, use an increment. The increment, δd, should be such that the change in adjacent velocities is not more than 10%.
4. Calculate the parameters A, WP, v, E, and S_f at the new depth, $d_2 = d_1 + \delta d$, and assign the subscript 2 to these (e.g., A_2, WP_2, etc.).
5. Determine the change in energy, δE, using Equation 8-19.
6. Calculate the arithmetic mean friction slope using Equation 8-20.
7. Using Equation 8-21, determine the distance, δL, over which the change in depth occurs.
8. Consider the new depth and location to be the new starting positions (assign the subscript $_1$ to those values currently identified with the subscript $_2$) and repeat steps 3 to 7, summing the incremental lengths, δL, until the total length, ΣL, equals or just exceeds the length of the culvert. You may use the same increment throughout or modify the increment to achieve the desired resolution. Such modifications are necessary when the last total length computed far exceeds the culvert length and when high friction slopes are encountered. If the computed depth reaches the barrel rise (D) before reaching the culvert inlet, skip step 9 and refer to the Type AB full flow losses to complete the analysis.
9. The last depth (d_2) established is the depth at the inlet (H_i) and the associated velocity is the inlet, v_i. Calculate the headwater using Equation 8-15.

$$\delta E = E_2 - E_1$$

Equation 8-19.

$$S_f = \frac{(S_{f2} + S_{f1})}{2}$$

Equation 8-20.

$$\delta L = \frac{\delta E}{S_o - S_f}$$

Equation 8-21.

Subcritical Flow and Steep Slope

The procedure for subcritical flow ($d > d_c$) but steep slope ($d_c > d_u$) is similar with the following exceptions:

- Choose a decrement in depth, δd = negative.
- If the depth, d, reaches critical depth before the inlet of the culvert is reached, the headwater is under inlet control (Headwater Under Inlet Control) and a hydraulic jump may occur in the culvert barrel (refer to the following subsection for discussion of the hydraulic jump in culverts).

- If the depth at the inlet is higher than critical depth, determine the outlet control head using Equation 8-15 as discussed in the Energy Balance at Inlet subsection. A hydraulic jump may occur within the culvert.

Supercritical Flow and Steep Slope

The procedure for supercritical flow ($d < d_c$) and steep slope is similar with the following exceptions:

- Begin computations at critical depth at the culvert entrance and proceed downstream.
- Choose a decrement of depth, δd.
- If the tailwater is higher than critical depth, a hydraulic jump may occur within the culvert (refer to the following subsection for discussion of the hydraulic jump).

Hydraulic Jump in Culverts

Figure 8-15 provides a sample plot of depth and momentum function and an associated specific energy plot. For a given discharge and given energy and momentum, there exist two possible depths, one less than critical depth (supercritical flow) a sequent (or conjugate) depth greater than critical depth (subcritical flow). With a proper configuration, the water flowing at the lower depth in supercritical flow can “jump” abruptly to its sequent depth in subcritical flow. This is called a hydraulic jump. With the abrupt change in flow depth comes a corresponding change in cross-sectional area of flow and a resulting decrease in average velocity.

By comparing the two curves at a supercritical depth and its sequent depth, it is apparent that the hydraulic jump involves a loss of energy. Also, the momentum function defines critical depth as the point at which minimum momentum is established.

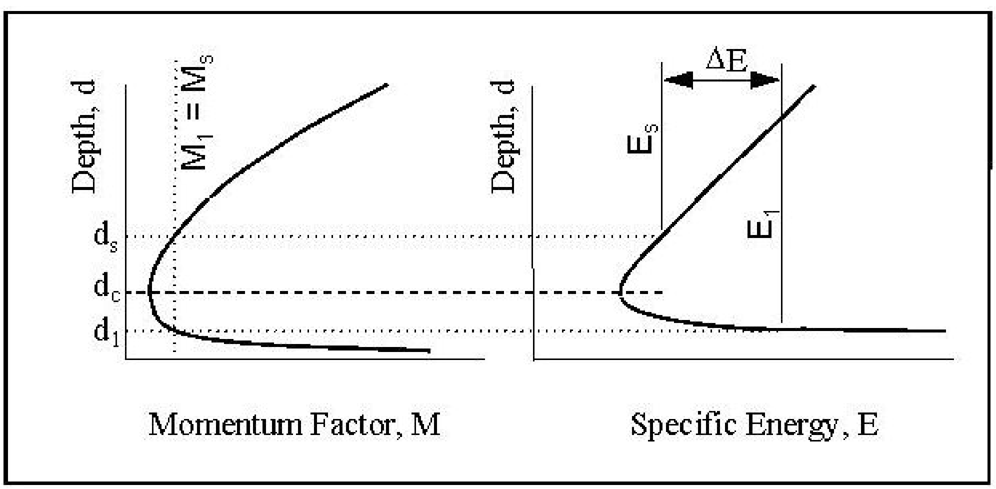


Figure 8-15. Momentum Function and Specific Energy

The balance of forces is represented using a momentum function (Equation 8-22):

$$M = \frac{Q^2}{gA} + A\bar{d}$$

Equation 8-22.

where:

M = momentum function

Q = discharge (cfs or m^3/s)

g = gravitational constant = 32 ft./sec^2

A = section area of flow (sq. ft. or m^2)

$\bar{d}$ = distance from water surface to centroid of flow area (ft. or m).

The term $A\bar{d}$ represents the first moment of area about the water surface. Assuming no drag forces or frictional forces at the jump, conservation of momentum maintains that the momentum function at the approach depth, M_1, is equal to the momentum function at the sequent depth, M_s.

The potential occurrence of the hydraulic jump within the culvert is determined by comparing the outfall conditions with the sequent depth of the supercritical flow depth in the culvert. The conditions under which the hydraulic jump is likely to occur depend on the slope of the conduit.

Under mild slope conditions ($d_c < d_u$) with supercritical flow in the upstream part of the culvert, the following two typical conditions could result in a hydraulic jump:

- The potential backwater profile in the culvert due to the tailwater is higher than the sequent depth computed at any location in the culvert.
- The supercritical profile reaches critical depth before the culvert outlet.

Under steep slope conditions, the hydraulic jump is likely only when the tailwater is higher than the sequent depth.

Sequent Depth

A direct solution for sequent depth, d_s is possible for free surface flow in a rectangular conduit on a flat slope using Equation 8-23. If the slope is greater than about 10 percent, a more complex solution is required to account for the weight component of the water. FHWA Hydraulic Engineering Circular 14 provides more detail for such conditions.

$$d_s = 0.5d_1\left(\sqrt{1+\frac{8v_1^2}{gd_1}} - 1\right)$$

Equation 8-23.

where:

d_s= sequent depth, ft. or m

d_1 = depth of flow (supercritical), ft. or m

v_1 = velocity of flow at depth d, ft./s or m/s.

A direct solution for sequent depth in a circular conduit is not feasible. However, an iterative solution is possible by following these equations:

- Select a trial sequent depth, d_s, and apply Equation 8-24 until the calculated discharge is equal to the design discharge. Equation 8-24 is reasonable for slopes up to about 10 percent.
- Calculate the first moments of area for the supercritical depth of flow, d_1, and sequent depth, d_s, using Equation 8-25.
- This equation uses the angle β shown in Figure 8-16, which you calculate by using Equation 8-26.

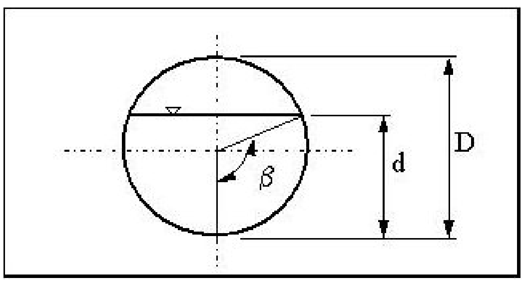


Figure 8-16. Determination of Angle b

CAUTION: Some calculators and spreadsheets may give only the principal angle for β in Equation 8-26 (i.e., -π/2 radians ≤ β ≤ π/2 radians).

- Use Equation 8-27 to calculate the areas of flow for the supercritical depth of flow and sequent depth.

$$Q^2 = \frac{g\left(A_s\bar{d}_s - A_1\bar{d}_1\right)}{\frac{1}{A_1} - \frac{1}{A_s}}$$

Equation 8-24.

where:

Q = discharge, cfs or m^3/s

A_s = area of flow at sequent depth, sq.ft. or m^2

$A_s\bar{d}_s$ = first moment of area about surface at sequent depth, cu.ft. or m^3

$A_1\bar{d}_1$ = first moment of area about surface at supercritical flow depth, cu.ft. or m^3.

$$A\overline{d} = \frac{D^3}{24}\left(3\sin\beta - \sin^3\beta - 3\beta\cos\beta\right)$$

Equation 8-25.

where:

Ad = first moment of area about water surface, cu.ft. or m^3

D = conduit diameter, ft. or m

β = angle shown in Figure 8-16 and calculated using Equation 8-26.

$$\beta = \cos^{-1}\left(1 - \frac{2d}{D}\right)$$

Equation 8-26.

$$A = \frac{D^2}{8}\left[2\cos^{-1}\left(1 - \frac{2d}{D}\right) - \sin\left(2\cos^{-1}\left(1 - \frac{2d}{D}\right)\right)\right]$$

Equation 8-27.

Equation 8-24 applies to other conduit shapes having slopes of about 10 percent or less. The first moment of area about the surface, $A\overline{d}$, is dependent on the shape of the conduit and depth of flow. A relationship between flow depth and first moment of area must be acquired or derived.

Roadway Overtopping

Where water flows both over the roadway and through a culvert (see Figure 8-17), a flow distribution analysis is required to define the hydraulic characteristics. This is a common occurrence where a discharge of low design AEP (low probability of occurrence) is applied to a facility designed for a lower design frequency.

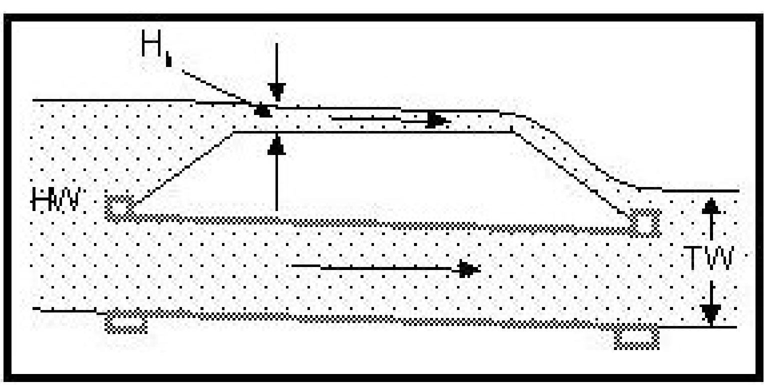


Figure 8-17. Culvert with Overtopping Flow

For example, a complete design involves the application and analysis of a 1% AEP discharge to a hydraulic facility designed for a much smaller flood. In such a case, the headwater may exceed the low elevation of the roadway, causing part of the water to flow over the roadway embankment while the remainder flows through the structure. The headwater components of flow form a common headwater level. An iterative process is used to establish this common headwater.

The following procedure is an iterative approach that is reasonable for hand computations and computer programs:

1. Initially assume that all the runoff (analysis discharge) passes through the culvert, and determine the headwater. Use the procedures outlined in the Culvert Design section. If the headwater is lower than the low roadway elevation, no roadway overtopping occurs and the analysis is complete. Otherwise, proceed to step 2.
2. Record the analysis discharge as the initial upper flow limit and zero as the initial lower flow limit. Assign 50% of the analysis discharge to the culvert and the remaining 50% to the roadway as the initial apportionment of flow.
3. Using the procedures outlined in the Design Guidelines and Procedure for Culverts section, determine the headwater with the apportioned culvert flow.
4. Compute the roadway overflow (discharge) required to subtend the headwater level determined in step 3 using Equation 8-28.

$$Q = k_t C L H_h^{1.5}$$

Equation 8-28.

where:

Q = discharge (cfs or m^3/s)

k_t = over-embankment flow adjustment factor (see Figure 8-18)

C = discharge coefficient (use 3.0 – English or 1.66 metric for roadway overtopping)

L = horizontal length of overflow (ft. or m). This length should be perpendicular to the overflow direction. For example, if the roadway curves, the length should be measured along the curve.

H_h = average depth between headwater and low roadway elevation (ft. or m).

- Base the value H_h on the assumption that the effective approach velocity is negligible. For estimation of maximum headwater, this is a conservative assumption. However, under some conditions, such as the need to provide adequate detention storage, you may need to consider the approach velocity head ($v^2/2g$). That is, replace H_h in Equation 8-28 with $H_h + v^2/2g$.
- With reference to Figure 8-19, the flow over the embankment will not be affected by tailwater if the excess (H_t) is lower than critical depth of flow over the road (approximately 0.67 H_h) . For practical purposes, H_t/H_h may approach 0.8 without any correction coefficient. For H_t/H_h values above 0.8 use Figure 8-18 to determine k_t.

- For most cases of flow over highway embankments, the section over which the discharge must flow is parabolic or otherwise irregular (see Figure 8-20). In such cases, it becomes necessary to divide the section into manageable increments and to calculate individual weir flows for the incremental units, summing them for total flow.
- If the tailwater is sufficiently high, it may affect the flow over the embankment. In fact, at high depth, the flow over the road may become open channel flow, and weir calculations are no longer valid. At extremely high depth of roadway overtopping, it may be reasonable to ignore the culvert opening and compute the water surface elevation based on open channel flow over the road.

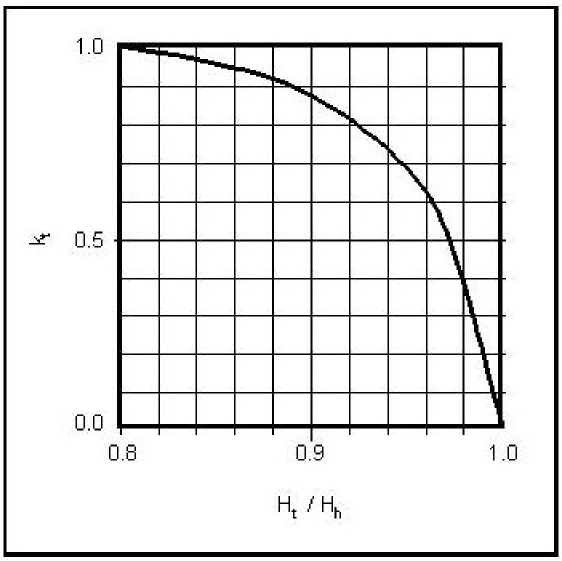


Figure 8-18. Over Embankment Flow Adjustment Factor

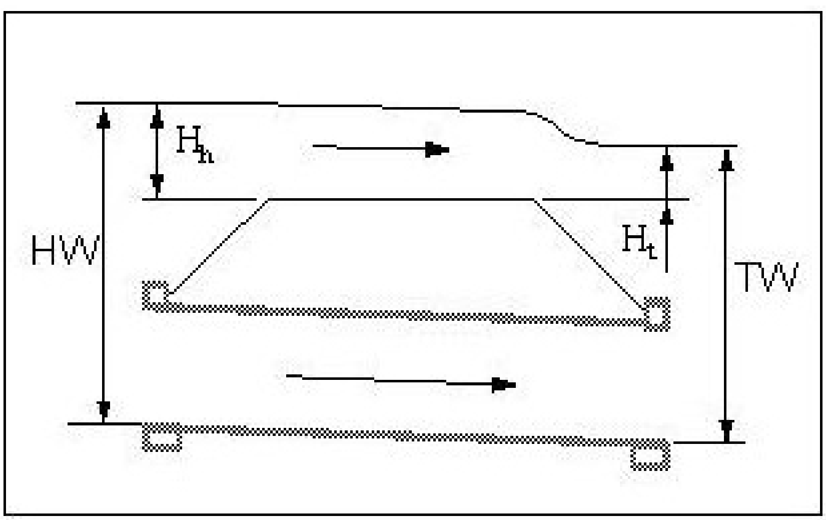


Figure 8-19. Roadway Overtopping with High Tailwater

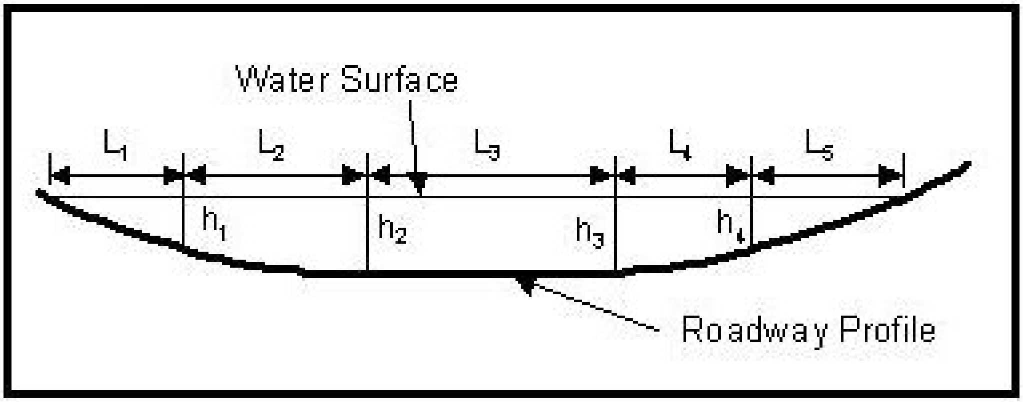


Figure 8-20. Cross Section of Flow over Embankment

5. Add the calculated roadway overflow to the culvert flow. If the calculated total is greater than the analysis discharge, record the current culvert flow apportionment as the current upper flow limit and set the new culvert flow apportionment at a value halfway between the current upper and lower flow limits. If the calculated total is less than the analysis discharge, record the current culvert flow apportionment as the lower flow limit for the culvert and set the new culvert flow apportionment at a value halfway between the current upper and lower flow limits.
6. Repeat steps 3 to 5, using the culvert flow apportionment established in step 5, until the difference between the current headwater and the previous headwater is less than a reasonable tolerance. For computer programs, the department recommends a tolerance of about 0.1 in. The current headwater and current assigned culvert flow and calculated roadway overflow can then be considered as the final values.

Performance Curves

For any given culvert, the control (outlet or inlet) might vary with the discharge. Figure 8-18 shows sample plots of headwater versus discharge for inlet and outlet control. The envelope (shown as the bold line) represents the highest value of inlet and outlet headwater for any discharge in the range. This envelope is termed a performance curve.

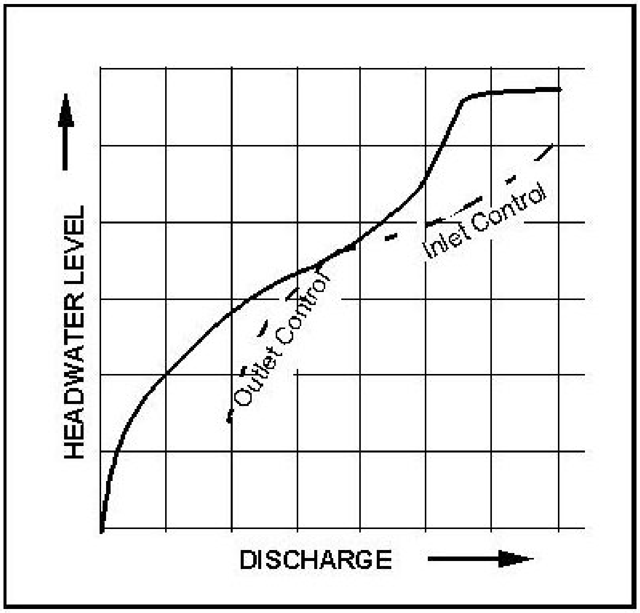


Figure 8-21. Typical Performance Curve

In this example, inlet control prevails at lower discharges and flow transitions to outlet control as the discharge increases. The flatter portion represents the effect of roadway overflow. The performance curve can be generated by calculating the culvert headwater for increasing values of discharge. Such information is particularly useful for performing risk assessments and for hydrograph routing through detention ponds and reservoirs.

Exit Loss Considerations

The traditional assumption in the design of typical highway culverts is continuity of the hydraulic grade line. At the outlet, this implies that when the tailwater is higher than critical depth and subcritical flow exists, the hydraulic grade line immediately inside the barrel is equal to the tailwater level. This is reasonable for most normal culvert designs for TxDOT application. However, by inference there can be no accommodation of exit losses because the energy grade line immediately inside the culvert can only be the hydraulic grade line plus the velocity head, no matter what the velocity is in the outfall.

Occasionally, an explicit exit loss may need to be accommodated. Some examples are as follows:

- conformance with another agency's procedures
- comparison with computer programs (such as HEC-RAS)
- design of detention pond control structures in which storage volumes are sensitive to small changes in elevation.

If such a need arises, base the starting hydraulic grade level (H_o) is based on balancing Equation 8-29 between the outside and inside of the culvert face at the outlet. A common expression for exit loss appears in Equation 8-30. This assumes that the tailwater velocity (v_{TW}) is lower than the culvert outlet velocity (v_o) and the tailwater is open to the atmosphere. If the above approach is used, it is most likely that the outlet depth (H_o) will be lower than the tailwater. This conforms to basic one-dimensional hydrostatic principles.

$$H_o + \frac{v_o^2}{2g} = TW + \frac{v_{TW}^2}{2g} + h_o$$

Equation 8-29.

where:

H_o = outlet depth - depth from the culvert flow line to the hydraulic grade line inside the culvert at the outlet (ft. or m)

v_o = culvert outlet velocity (ft./s or m/s)

v_{TW} = velocity in outfall (tailwater velocity) (ft./s or m/s)

h_o = exit loss (ft. or m).

$$h_o = K\frac{v_o^2 - v_{TW}^2}{2g}$$

Equation 8-30.

where:

K = loss coefficient which typically varies from 0.5 to 1.

Section 4 — Improved Inlets

Inlet Use

An improved inlet serves to funnel the flow into the culvert to remove the point of control from the face of the inlet to a throat located downstream from the face. The normal contraction of flow is included in the transition from the face to the throat. An improved inlet may be economical if the culvert is operating under inlet control, but not if the culvert is operating under outlet control.

An improved inlet may offer the advantage of increasing the capacity of an existing culvert that has become inadequate because of changes in the watershed which have increased the discharge to the culvert. Improved inlets are not recommended because of the following disadvantages:

- Available design procedures cannot accommodate an improved inlet when the face of the inlet is skewed instead of normal.
- Heavy debris loads that can pass through the inlet face may become lodged in the restriction at the throat.
- The reduction at the throat may push the culvert into outlet control which will negate any advantages of the improved inlet.
- Improved inlets are usually costly to construct when compared with standard inlets.

Careful consideration should be given before selecting and using an improved inlet design. Guidelines for design can be found in FHWA publication, Hydraulic Design of Highway Culverts, HDS-5.

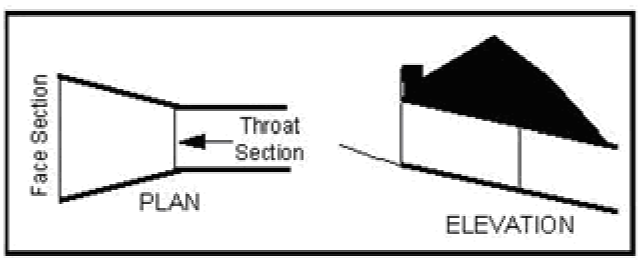


Figure 8-22. Side Tapered Inlet

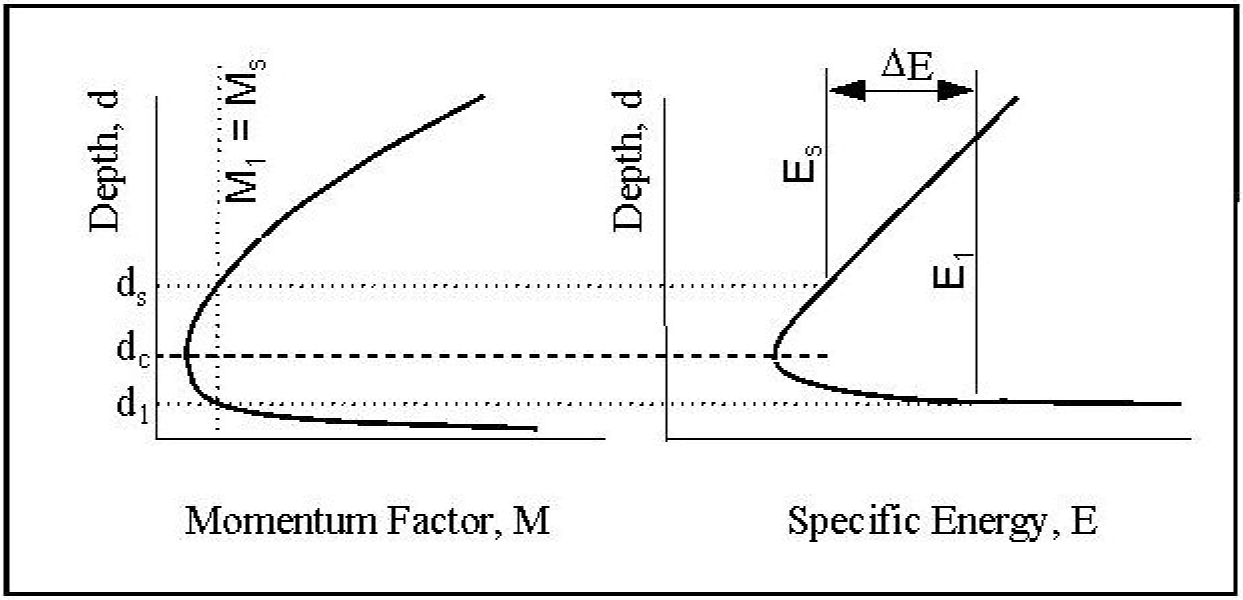


Figure 8-23. Slope Tapered Inlet

Section 5 — Velocity Protection and Control Devices

Excess Velocity

Several possible solutions are available for both protection and control to minimize the negative effects of excessive velocity. Solutions are categorized as either velocity protection devices or velocity control devices.

Velocity Protection Devices

A velocity protection device does not necessarily reduce excessive velocity but does protect threatened features from damage. Such devices are usually economical and effective in that they serve to provide a physical interim for the flow to return to a more natural velocity. The protection devices discussed here include the following:

- Channel liner -- Most of the various types of channel liner have proven effective for erosion protection. Some types of channel liner include low quality concrete (lightly reinforced), rock, soil retention blankets, articulated concrete blocks, and revetment mattresses. Channel liners, when used as an outlet velocity protection measure, should be applied to the channel area immediately downstream of the culvert outlet for some distance, possibly to the right of way line and beyond (with appropriate easement). (See Chapter 6 for types and guidelines.) These liners, however, are viewed as creating environment problems, including decreased habitat and increased water temperature. They also are viewed to increase impervious cover, decrease time of concentration, and change the hydrograph timing downstream. In many instances, the liner may stabilize the area in question, only to have the problem shift downstream to where the channel is not lined.
- Pre-formed outlets - Pre-formed outlets approximate a natural scour hole but protect the stream bed while dissipating energy. These have been shown to be effective protection in areas threatened by excessive outlet velocities. Such appurtenances should be lined with some type of riprap. (A velocity appurtenance for a culvert may be classified broadly as either a protection device or a control device.)
- Channel recovery reach -- Similar to a pre-formed outlet, a channel recovery reach provides a means for the flow to return to an equilibrium state with the natural, unconstricted stream flow. The recovery reach should be well protected against the threat of scour or other damage.

Velocity Control Devices

A velocity control device serves to effectively reduce an excessive culvert outlet velocity to an acceptable level. The design of some control devices is based analytically while, for others, the specific control may be unpredictable. Some velocity control devices are as follows:

- Natural hydraulic jumps (most control devices are intended to force a hydraulic jump) -- Most velocity control devices rely on the establishment of a hydraulic jump. Because a culvert being on a relatively steep slope usually results in excessive outlet velocity from the culvert, the depth downstream of the culvert exit is usually not great enough to induce a hydraulic jump. However, some mechanisms may be available to provide a simulation of a greater depth necessary to create a natural hydraulic jump.
- Broken-back culvert configuration -- One mechanism for creating a hydraulic jump is the broken back configuration, two types of which are depicted in Figure 8-26 and Figure 8-27. When used appropriately, a broken back culvert configuration can influence and contain a hydraulic jump. However, there must be sufficient tailwater, and there should be sufficient friction and length in unit 3 (see Figure 8-26 and Figure 8-27) of the culvert. In ordinary circumstances for broken back culverts, you may need to employ one or more devices such as roughness baffles to create a high enough tailwater.
- Sills -- The use of the sill is effective in forcing a hydraulic jump in culverts. One disadvantage of sills is the possible susceptibility for silting. Sills must usually be maintained frequently to keep it free of sediment deposition. Another disadvantage is the waterfall effect that they usually cause. Riprap should be installed immediately downstream of the sill for a minimum distance of 10 feet to protect features from the turbulence of the waterfall effect.

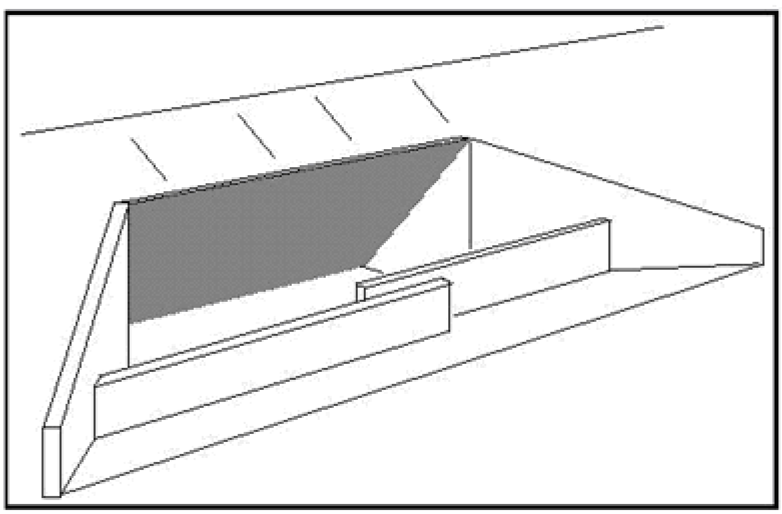


Figure 8-24. Sills

- Roughness baffles -- Roughness baffles, sometimes referred to as 'dragon's teeth', can be effective in inducing turbulence, dissipating energy, and reducing culvert outlet velocity (see Figure 8-25). Care must be taken in the design and placement of the baffles; if the baffles are too small or placed too far apart, they are ineffective. In addition, they may interfere with mowing operations around the culvert outlet. If these become damaged or broken from being hit by a mower, they are ineffective. To limit the amount of potential damage, baffles must be reinforced with rebar.

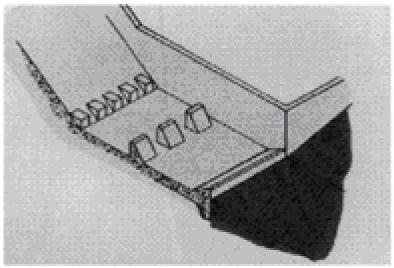


Figure 8-25. Roughness Baffles

- Energy dissipators -- An efficient but usually expensive countermeasure is an energy dissipator. Some energy dissipators have an analytical basis for design while others are intended to cause turbulence in unpredictable ways. With turbulence in flow, energy is dissipated and velocity can be reduced.

Other controls are described in the FHWA publication Hydraulic Design of Energy Dissipators for Culverts and Channels, HEC-14.

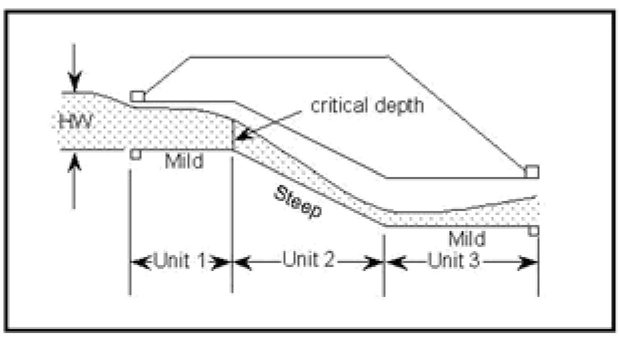


Figure 8-26. Three Unit Broken Back Culvert

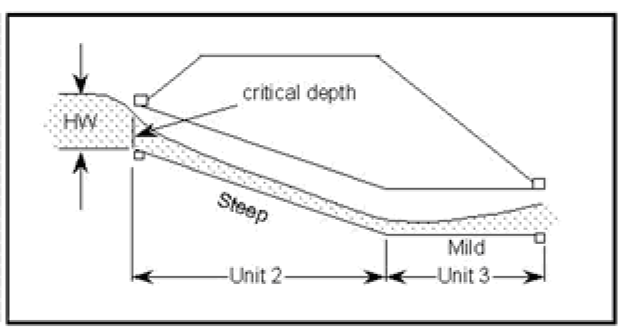


Figure 8-27. Two Unit Broken Back Culvert

Broken Back Design and Provisions Procedure

The design of a broken back culvert is not particularly difficult, but it requires reducing velocity at the outlet. Use the following procedure:

1. With design discharge and an associated tailwater, establish the flow line profile using the following considerations:
   - With reference to Figure 8-26 and Figure 8-27, unit 3 should be as long enough to ensure that the hydraulic jump occurs within the culvert.
   - For a given total drop, the resulting length of unit 2 is short, but this may cause the slope of unit 2 to be very steep.
   - Provided that unit 1 is on a mild slope, its length has no effect on the outlet velocity of any downstream hydraulic function. It is recommended that unit 1 either not be used or be very short; the result is additional latitude for adjustment in the profiles of units 2 and 3.
   - A longer unit 3 and a milder (but still steep) slope in unit 2 together enhance the possibility of a hydraulic jump within the culvert. However, these two conditions are contradictory and usually not feasible for a given culvert location. Make some compromise between the length of unit 3 and the slope of unit 2. Unit 3 must be on a mild slope ($d_u > d_c$). This slope should be no greater than necessary to prevent ponding of water in the unit. Do not use an adverse (negative) slope.
2. Size the culvert initially according to the directions outlined in step 1 under Design Guidelines and Procedure for Culverts.
   - If a unit 1 is used, the headwater will most likely result from the backwater effect of critical depth between units 1 and 2.
   - If a unit 1 is not used, the headwater will most likely result from inlet control.
3. Starting at the upstream end of unit 2, calculate a supercritical profile, beginning at critical depth and working downstream through unit 3. The Direct Step Backwater Method is appropriate. Note the following:
   - Critical depth will not change from one unit to the next, but uniform depth will vary with the slope of the unit.
   - The increment, δd, should be such that the change in adjacent velocities is not more than 10%.
   - The depth in unit 2 should tend to decrease towards uniform depth, so δd should be negative. The resulting profile is termed an S2 curve.
   - Also, δd should be small enough when approaching unit 3 such that the cumulative length does not far exceed the beginning of unit 3.
   - For hand computations, an acceptable expedient is to omit the profile calculation in unit 2 and assume that the exit depth from unit 2 is equal to uniform depth in unit 2.
4. When you reach unit 3, complete the profile computations with the following considerations.
   - Because uniform depth is now greater than critical depth (mild slope), and flow depth is lower than critical depth, the flow depth tends to increase towards critical depth. Therefore, in unit 3, δd should be positive.
   - The starting depth for unit 3 is the calculated depth at the end of unit 2.

- Reset the cumulative length, Σ L, to zero.
- The resulting water surface profile is termed an M3 curve.

As the profile is calculated, perform the checks outlined below:

- As each depth is calculated along unit 3, calculate the sequent depth, d_s. For more information, see the Direct Step Backwater Method, Hydraulic Jump in Culverts, and Sequent Depth subsections in Section 3.
- Calculate the elevation of sequent depth (d_s + flow line elevation) and compare it with the tailwater elevation. Tailwater elevation may be a natural stream flow elevation, or may produced artificially by installing a sill on the downstream apron between wingwalls. Design Division Hydraulics does not recommend the use of sills. (see Velocity Control Devices). If sills are used, the total vertical dimension of the artificial tailwater is determined by adding the elevation at the top of the sill and the critical depth of design discharge flow over the sill. Base this critical depth on the rectangular section formed by the top of the sill and the two vertical wingwalls. If the elevation of sequent depth is lower than the tailwater elevation, the following points apply; go to Step 5:
  - Hydraulic jump is likely to occur within the culvert.
  - Outlet velocity is based on the lower of tailwater depth, TW, and barrel height, D.
  - Profile calculations may cease even though the end of the barrel has not been reached.
- If the computed profile tends towards critical depth before reaching the end of the culvert, the following apply and you should go to Step 5:
  - Hydraulic jump is likely to occur within the culvert.
  - Outlet depth will be equal to critical depth and outlet velocity is based on critical depth.
  - Profile calculations may cease even though the end of the barrel has not been reached.
- Compare the cumulative length, Σ L, to unit 3 length. If Σ L ≥ length of unit 3, the following apply:
  - Hydraulic jump does not form within the length of unit 3.
  - Exit depth is the present value of d.
  - Exit velocity is based on exit depth.
  - The broken-back culvert configuration is ineffective as a velocity control device and should be changed in some manner. Alternatives include rearrangement of the culvert profile, addition of a sill, and investigation of another device. If the profile is reconfigured, go back to step 3. Otherwise, skip step 5 and seek alternative measures.

5. Consider hydraulic jump cautions. The hydraulic jump is likely to occur within the culvert for the design conditions. However, it is prudent to consider the following cautions:

- If tailwater is very sensitive to varying downstream conditions, it may be appropriate to check the occurrence of the hydraulic jump based on the lowest tailwater that is likely to occur.
- The hydraulic jump may not occur within the barrel under other flow conditions. It is wise to check the sensitivity of the hydraulic jump to varying flow conditions to help assess the risk of excessive velocities.
- If a sill has been employed to force an artificial tailwater, and the hydraulic jump has formed, the outlet velocity calculated represents the velocity of water as it exits the barrel. However, the velocity at which water re-enters the channel is the crucial velocity. This velocity would be the critical velocity of sill overflow.

Energy Dissipators

Impact basins are effective energy dissipators but are relatively expensive structures (see Figure 8-28).

Figure 8-28. Impact Basins

Stilling basins are hydraulically similar to sills (Figure 8-29). However, they are more expensive in construction and could present serious silting problems. A chief advantage in stilling basins is the lack of a waterfall effect.

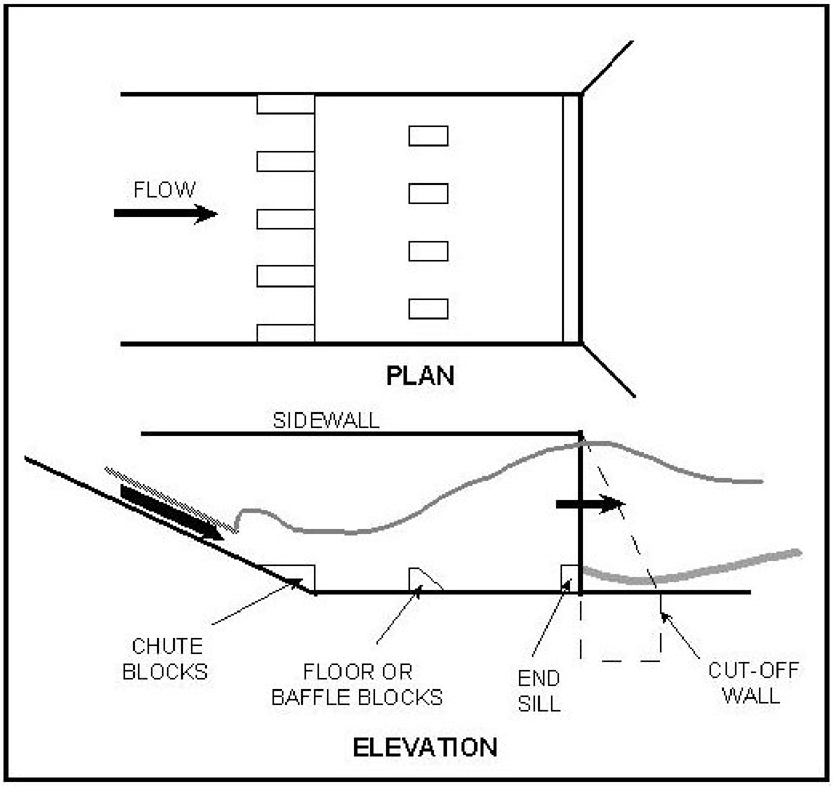


Figure 8-29. Stilling Basins

Radial energy dissipators are quite effective but extremely expensive to construct and, therefore, not ordinarily justified (Figure 8-30). They function on the principle of a circular hydraulic jump. For a detailed discussion on dissipator types, along with a variety of design methods for velocity control devices, refer to HEC-14.

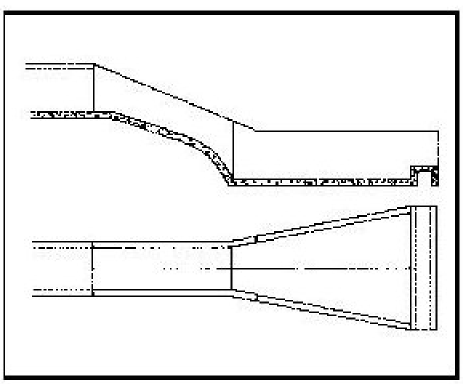


Figure 8-30. Radial Flow Energy Dissipator

Section 6 — Special Applications

Detour Culverts

Temporary culverts are usually installed in detours or emergency replacements of permanent culverts and bridges. The design must include consideration for soil protection to prevent erosion of the embankment and silting of the stream (Waters of the U.S.). Figure 8-31 shows and example of both problems on one temporary structure.

Figure 8-31. Temporary or Detour Culverts

Risk

Stream crossings for detours are normally built to higher AEP (lower flow) flood events than crossings designed for the highway. This may be good practice both hydraulically and economically. It follows that the hydraulic design of a detour stream crossing should be based on risk factors. Risk factors that should be evaluated include the probability of flooding during the anticipated service life of the detour (the construction period for the bridge or culvert), the risk to life and property from excessive backwaters and washouts, traffic service requirements, school bus routes, and emergency routes. Common sense and sound engineering judgment should prevail in making decisions.

Equation 8-31 describes the risk of the occurrence of a given AEP flood that a project incurs during its construction life.

$$\bar{R} = 1 - (1 - AE)^n$$

Equation 8-31.

Where:

R = Risk (probability of occurrence) in decimal form.
Æ = AEP of the flood event in decimal form.
n = duration in years of the project or the time requirement of the detour.

Equation 8-31 was used to generate the curves in Figure 8-32 for a family of curves for project lengths versus flood AEPs on the y axis and the resultant risk of occurrence on the x axis. Figure 8-32 demonstrates that during a one-year construction period, the odds are 4 to 1 against the occurrence of a flood as large as a 20% AEP event, and the chances are even (1 to 1) that the mean annual event will not be exceeded. The odds are 9 to 1 against the occurrence of a 10% AEP flood during a one-year construction and 2.7 to 1 against such an occurrence in a three-year construction period.

However, caution should be used with the graph. A one-year project which is to be started during one rainy or potential flood season and will finish during the next rainy or potential flood season really should be considered a two-year project for risk assessment, which means the odds are really even for risk instead of 4 to 1 against!

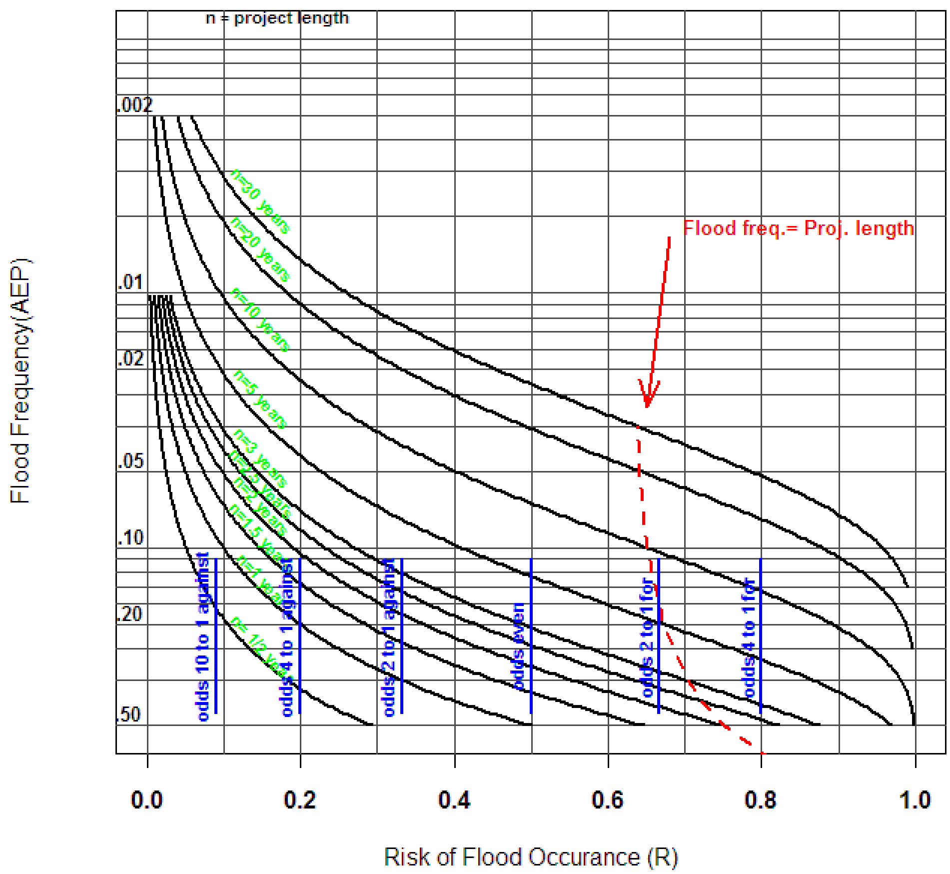


Figure 8-32. Graph was generated from Equation 8-31. The higher n values are not realistic, but were included for a sense of proportion.

Engineering Requirements

Detour culverts are not a contractor item, but shall be designed by a licensed Professional Engineer on sealed sheets which are part of the plan set. The Contractor shall not design a detour culvert as part of the construction project unless the design has been signed and sealed and submitted to TxDOT for approval.

Chapter 9 — Bridges

Contents:

Section 1 — Introduction and Definitions

Hydraulically Designed Bridges

A bridge is defined in the *TxDOT Standard Specifications For Construction and Maintenance of Highways, Streets, and Bridges 2004* as a structure, including supports, erected over a depression or an obstruction (e.g., water, a highway, or a railway) having a roadway or track for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between faces of abutments, spring lines of arches, or extreme ends of the openings for multiple box culverts. Bridges, as opposed to culverts, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions.

Bridges enable streams to maintain flow conveyance and to sustain aquatic life. They are important and expensive highway hydraulic structures vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing during the development, construction, and maintenance highway phases must be recognized and addressed.

This chapter addresses hydraulic engineering aspects of bridge stream crossings, including approach embankments and structures on floodplains. It does not provide detailed information on tidal areas such as bays and estuaries. Risk is discussed in Chapter 3, Section 3 Evaluation of Risk.

Definitions

One-Dimensional Analysis – A steady state or standard step model, meaning that there is no direct modeling of the hydraulic effect of cross section shape changes, bends, and other two- and three-dimensional aspects of flow. HEC-1, WSPRO, and HEC-RAS are examples of a one-dimensional analysis models.

Two-Dimensional Analysis – A spatially distributed hydraulic model which models dynamic unsteady flow and is therefore capable of delivering results far more accurately and closer to real life than a steady state model. Dynamic models allow the effects of storage and backwater in conduits and floodplains and the timing of the hydrographs to yield a true representation of the HGL at any point in space and time. Two-dimensional analysis models require such a high level of expertise and time to run effectively that they are used for unusual situations.

Section 2 — Planning and Location Considerations

Introduction

Generally, a stream crossing location is selected during the planning and location phase of a highway project. The final location should be selected only after obtaining detailed survey information and completing preliminary hydraulic studies. Although they are not the sole consideration in bridge location and design, hydraulic aspects should receive major attention in the initial planning of the highway. The location and alignment of the highway can either magnify or eliminate hydraulic problems at the crossing. Adverse conditions should be identified in the early stages of new location selection so that potential problems receive adequate consideration. If the cost of the required structures is prohibitive, consider rerouting the highway.

Location Selection and Orientation Guidelines

Bridge location and orientation requirements are covered in general in the Bridge Project Development Manual. The specific hydraulic requirements are covered below:

- The bridge should be centered on the main channel portion of the entire floodplain. This may mean an eccentricity in the location with respect to the entire stream cross section, but allows for better accommodation of the usual and low flows of the stream.
- The bridge waterway opening should be designed to provide a flow area sufficient to maintain the through-bridge velocity for the design discharge no greater than the allowable through-bridge velocity.
- The headers and interior bents should be oriented to conform to the streamlines at flood stage. Standard skew values of 15°, 30°, and 45° should be used where feasible. The piers and the toe of slope of the header must be located away from deep channels, cuts, and high velocity areas to avoid scour problems or interference with stream low flows.
- Consider including either relief openings or guide banks if the intrusion of either or both roadway headers into the stream floodplains is more than about 800-feet.
- Existing vegetation should be incorporated into the overall bridge plan. Where practicable, trees and shrubs should be left intact even within the right-of-way. Minimizing vegetation removal also tends to control turbulence of the flow into, through, and out of the bridge.
- For some configurations, roadway approaches may need to accommodate overflow. Such overflow approaches allow floods that exceed the design flow to overtop the roadway, thereby reducing the threat to the bridge structure itself. Protection of the approaches from overflow damage should be considered.

Environmental Considerations

Environmental impacts must be considered along with the hydraulic issues as one may directly affect the other. (See the *Environmental Management System Manual* for details.)

Biological considerations in site selection include the effects on habitat and ecosystems in the floodplain, stream, and associated wetlands. Biologists should assess this aspect of site selection, but provide much of the information necessary for a valid assessment of the biological effects and the available alternatives for mitigation, including the following:

- economic viability of using a bridge rather than filling in wetland areas
- cost to replace lost marsh or wetland areas
- circulation of fresh or brackish water in marshes and estuaries
- feasibility of providing mitigating measures for the loss of invertebrate population
- shade and resting areas for fish.

Water Resource Development Projects

Water resources development projects such as reservoirs or stream channel modifications, whether existing or only potentially planned, must be considered when selecting a stream crossing location. Planned resources development projects often require the relocation or reconstruction of existing highways and can interfere with the location or design of proposed highway-stream crossings. Many water resources development projects are planned or authorized for periods of years or even decades before construction begins. Others never come to fruition or may be permanently stopped by court decisions or regulatory agency actions.

The roadway designer must carefully plan and construct a highway facility near a water resources project location, designing the highway facility for both existing and future site conditions. The excess cost of building the facility due to the water resources project must be considered in selecting the stream-crossing site. See Chapter 12, Reservoirs.

FEMA Designated Floodplains

The majority of highway crossings involve floodplains that are in FEMA-participating communities. It is important to acknowledge FEMA floodplains in the planning phases of a project and accommodate them in design. Early coordination with the community's NFIP administrator is essential to identify and avert potential problems. See Chapter 5, *Federal Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP) Compliant Design of Floodplain Encroachments and Minor Structures*.

Stream Characteristics

All streams change with time. Planning, roadway and bridge design engineers should be conscious of stream morphology and be aware that methods are available for quantifying natural changes and changes that can occur as the result of stream encroachments and crossings. See Chapter 7, Channels.

Procedure to Check Present Adequacy of Methods Used

Methods to analyze the hydrology and hydraulics at bridge sites continue to improve. In many cases, a method used in the original analysis is no longer an appropriate method. The following steps should be used to examine the adequacy of the method:

1. Examine the adequacy of the analysis for the original crossing design before undertaking major reconstruction or replacement.
2. If the method originally used is no longer appropriate, recalculate the analysis for these crossings using an appropriate one.
3. Reconsider the risk of failure of the existing structure, including the following:
   - increased traffic volumes
   - changed traffic service requirements
   - increased highway construction and maintenance costs
   - liability for damages to property that could be attributed to the highway crossing.

Section 3 — Bridge Hydraulic Considerations

Bridge/Culvert Determination

The first step in analysis for a cross-drainage facilityis the establishment of the flood frequency curve and the stage-discharge curve according to Chapter 4, Hydrology Study Requirements, and Chapter 6, Open Channel Flow; and the second step is to make a decision concerning the type of cross-drainage structure. All types of facilities should be appraised based on performance and economics. The choice is usually between a bridge and culvert. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements for the stream crossing. The roadway designer should choose the appropriate structure based on the following criteria:

- construction and maintenance costs
- risk of failure
- risk of property damage
- traffic safety
- environmental and aesthetic considerations
- construction expedience.

Although the cost of an individual bridge is usually relatively small, the total cost of bridge construction constitutes a substantial share of the total cost of highway construction. Similarly, bridge maintenance may account for a large share of the total cost of maintaining highway hydraulic features. The roadway designer can achieve improved traffic service and reduced cost by judicious choice of design criteria and careful attention to the hydraulic design of each bridge.

Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves the following:

- determination of the backwater associated with each alternative profile and waterway opening(s)
- determination of the effects on flow distribution and velocities
- estimation of scour potential.

The hydraulic design of a bridge over a waterway involves the following such that the risks associated with backwater and increased velocities are not excessive:

- establishing a location
- bridge length
- orientation
- roadway and bridge profiles.

A hydrologic and hydraulic analysis is required for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely affect the floodplain even if no structural modifications are necessary. Typically, this should include the following:

- an estimate of peak discharge (sometimes complete runoff hydrographs)
- existing and proposed condition water surface profiles for design and check flood conditions
- consideration of the potential for stream stability problems and scour potential.

See the Documentation Reference Tables in Chapter 3 for a complete list of requirements.

Flow through Bridges

When flood flows encounter a restriction in the natural stream, adjustments take place in the vicinity of the restriction. The portion of flow not directly approaching the bridge opening is redirected towards the opening by the embankment. The flow contracts as it enters the bridge and then expands as it exits the bridge. Maintaining the contraction and expansion of flow and overcoming friction and disturbances associated with piers and abutments requires an exchange of energy. An increase in the depth of flow upstream of the encroachment, termed backwater, reflects this energy exchange, as shown in Figure 9-1.

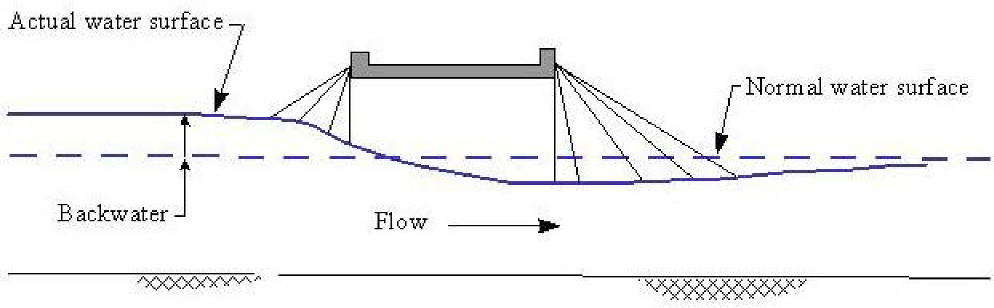


Figure 9-1. Backwater at a Stream Crossing

Backwater in Subcritical Flow

In subcritical flow conditions, the backwater tails off upstream until it reaches the normal water surface. The distance upstream over which backwater occurs depends on the channel conditions and flow conditions (see the Standard Step Procedure in Chapter 7). The maximum backwater tends to occur in an arc around the opening as Figure 9-2 shows. The relatively steep water surface gradient between the maximum backwater and the opening is termed the drawdown area.

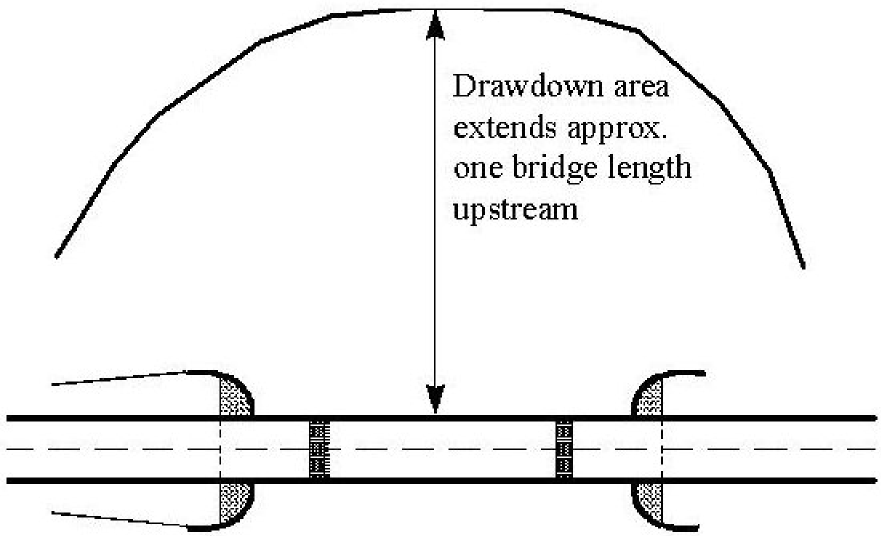


Figure 9-2. Extent of Backwater Drawdown

In a stream channel with supercritical flow conditions a constriction such as a bridge may not affect the upstream flow conditions. However, if the constriction is severe enough, it could cause a change in flow regime such that a backwater occurs upstream of the bridge and a hydraulic jump occurs near the bridge.

As the flow becomes constricted as it moves toward the bridge opening, the velocity increases, which can result in scour along the embankment and through the bridge. At the bridge headers, intersecting velocity vectors can cause severe turbulence and eddies as shown in Figure 9-3. Piers in the waterway create additional local turbulence and vortices. Turbulence, eddying, and vortices often result in scour.

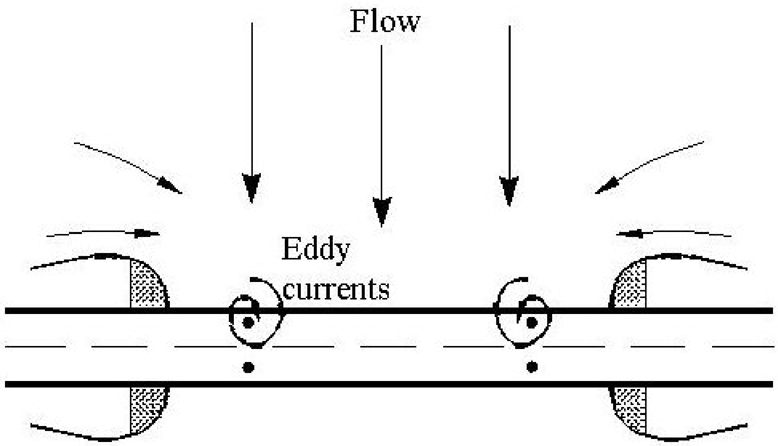


Figure 9-3. Typical Eddy Currents through Bridge Opening

Allowable Backwater Due to Bridges

For design frequency conditions, the allowable backwater should be established based on the risk of incurring flood-related damage to the highway and adjacent property. See Chapter 3 for discussion on Evaluation of Risk assessment. The allowable backwater should also consider the presence of FEMA mapped floodplains. See Chapter 5, for a more thorough explanation on the FEMA NFIP requirements.

Flow Distribution

Any stream crossing that uses a combination of fill and bridge within the floodplain disturbs flow distribution during some floods. However, the normal flow distribution should be preserved to the extent practicable in order to:

- avoid disruption of the stream-side environment
- preserve local drainage patterns
- minimize damage to property by either excessive backwater or high local velocities
- avoid concentrating flow areas that were not subjected to concentrated flow prior to construction of the highway facility
- avoid diversions for long distances along the roadway embankment.

Generally, the disturbance of flow distribution can be minimized by locating bridge openings at the areas of high conveyance.

For many situations, one-dimensional analysis techniques suffice for determining optimum bridge locations. When analyzing complex sites, such as those at a bend (Figure 9-4), and at skewed crossings (Figure 9-9), a great deal of intuition, experience, and engineering judgment are needed to supplement the one-dimensional analysis. Unfortunately, complex sites are frequently encountered in stream crossing design. The development of two-dimensional techniques of analysis greatly enhances the capabilities of hydraulics designers to deal with these complex sites. However, two-dimensional models required a great deal more data, intuition, experience and time than a one-dimensional model.

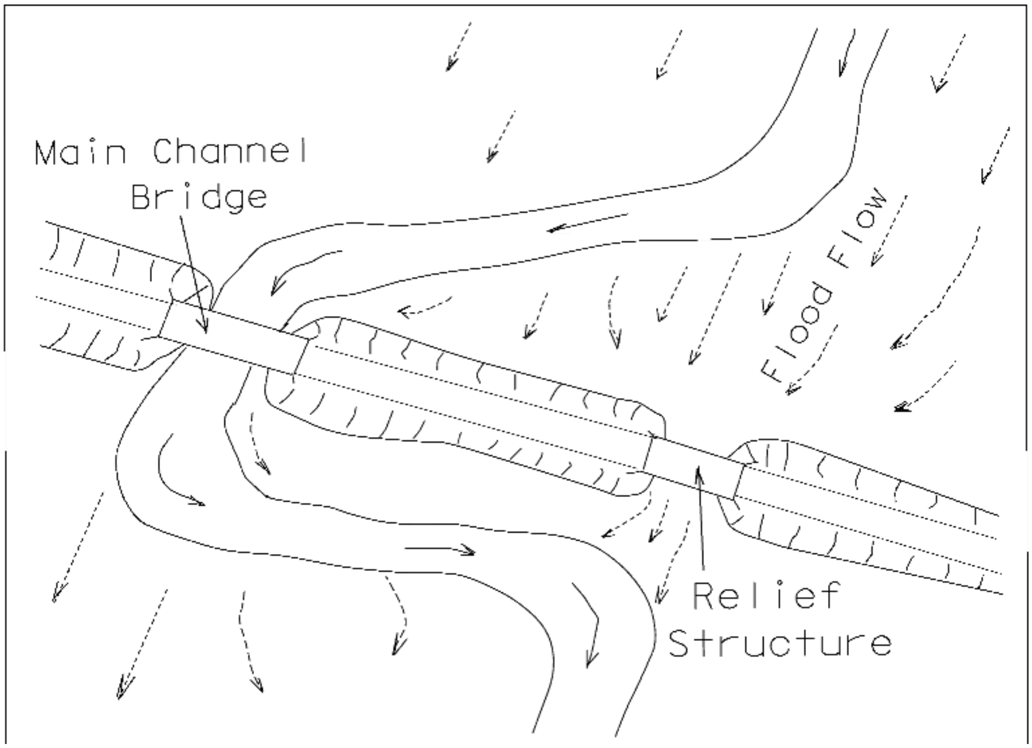


Figure 9-4. Highway Stream Crossing at a Bend

Velocity

While some bridge openings may have a relatively uniform velocity across the entire bridge opening, in most instances there are wide variations in the velocity profile. In some segments of the flow (e.g., near the center of the stream), the velocity may be considerably higher than the average velocity. In areas of shallow flow, the velocity may be quite low. The velocity profile may even include negative velocities (reverse flows). Figure 9-5 shows an example of a velocity profile through a bridge opening.

The through-bridge velocity is the basic sizing criterion used for span-type bridges. The average through-bridge velocity is described by the Continuity Equation (see Equation 9-1).

$$V = \frac{Q}{A}$$

Equation 9-1.

where:

V = average velocity (fps or m/s)

A = Normal cross-sectional area of the water (sq.ft. or m^2)

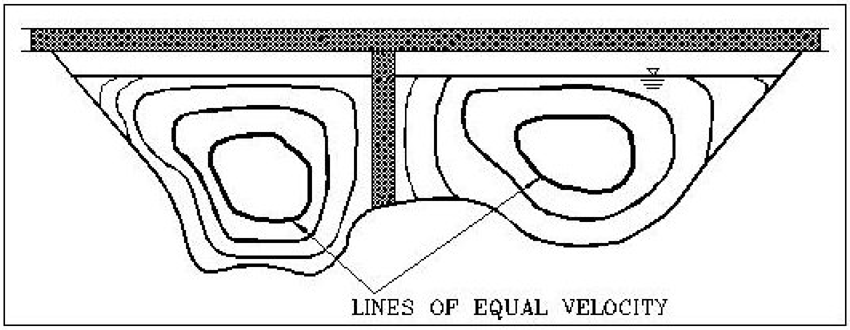


Figure 9-5. Velocity Profile Through Bridge Opening (heavier lines = higher velocity)

In general, waterway velocities should replicate the velocity of the natural channel. Higher velocities may be acceptable in certain cases where the streambed is rocky or the bridge headers are sufficiently removed from the erosive effects of floodwaters.

Bridge Scour and Stream Degradation

A scour analysis is required for new bridges, replacements, and widenings. Where a scour analysis indicates high depths of potential contraction scour, a structure larger than that required by the basic velocity and backwater criteria may be more cost effective than to designing foundations and armoring to withstand the scour. The potential for deep local scour can be reduced by enlarging the structure, but designing foundations and armoring to withstand local scour depths may be more cost-effective. Generally, a multi-disciplined team should assess the validity of calculated scour depths.

Stream stability issues such as potential vertical and horizontal degradation may warrant accommodations in the bridge design. If the channel is vertically degrading, it is likely that, as the channel deepens, the banks will slough resulting in a widening. Also, where significant meandering is occurring, meanders tend to migrate downstream and increase in amplitude. Structural options to accommodate either of these cases can include longer structures with deep enough foundations to accommodate anticipated degradation or deep enough foundations with abutment foundations designed to act as interior bents to allow future lengthening of the bridge.

Refer to the Geotechnical Manual and the Bridge Division Geotechnical Section for further information on bridge scour calculations and protection and for stream stability issues.

Freeboard

Navigational clearance and other reasons notwithstanding, the low chord elevation is established as the sum of the design normal water surface elevation (high water) and a freeboard.

For on-system bridges, the department recommends a suitable freeboard based on the flowing criteria:

- Higher freeboards may be appropriate for bridges over streams that are prone to heavy debris loads, such as large tree limbs, and to accommodate other clearance needs.
- Lower freeboards may be desirable, because of constraints such as approach geometry. However, the design high water must not impinge on the low chord.

Generally, for off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced unless the results of a risk assessment indicate a different structure is the most beneficial option.

Roadway/Bridge Profile

The bridge is integrated into both the stream and the roadway and must be fully compatible with both. Therefore, the alignment of the roadway and the bridge are the same between the ends of the bridge. Hydraulically, the complete bridge profile includes any part of the structure that stream flow can strike or impact in its movement downstream. If the stream rises high enough to inundate the structure, then the bridge and all parts of the roadway become the complete bridge profile.

It is not allowable for the design AEP flow to impinge on the bridge low chord or to inundate the roadway because it violates the definition of design frequency. However, flows exceeding the design AEP flow, including the 1% AEP flow, may inundate the structure and roadway. Unless the route is an emergency escape route, it is often desirable to allow floods in excess of the design flood to overtop the road. This helps minimize both the backwater and the required length of structure.

Several vertical alignment alternatives are available for consideration, depending on site topography, traffic requirements, and flood damage potential. The alternatives range from crossings that are designed to overtop frequently to crossings that are designed to rarely or to never overtop.

In Figure 9-6, the bridge is at the low point in a sag-vertical curve profile. An extreme example of this configuration is a bridge in rolling terrain on a low-traffic road which frequently overtop. Another example is a high bridge in rugged terrain that probably will never be threatened by floods. A distinctive feature of the sag-vertical profile is the certainty that the bridge structure will be submerged when any overflow of the roadway occurs.

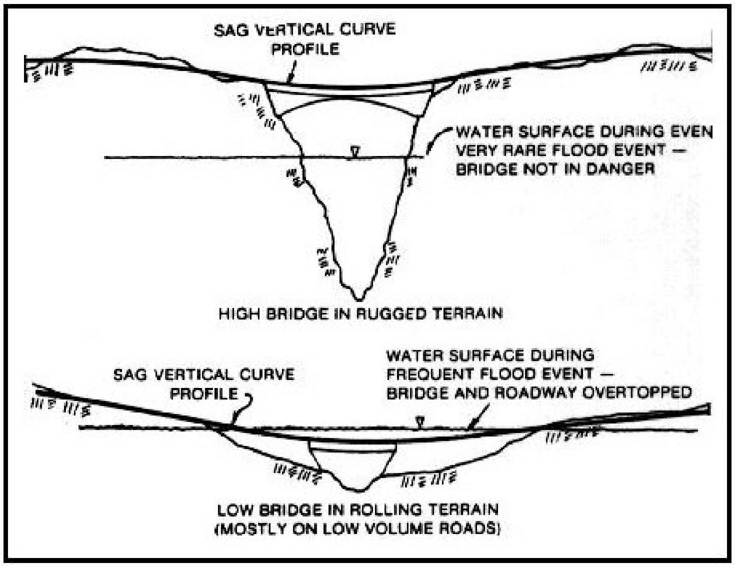


Figure 9-6. Sag-Vertical Curves

If accumulation of drift in the superstructure is likely, placement of bridges on sag-vertical curves should be avoided. Trapped debris can increase the potential for scour by creating eddies and turbulence. The accumulation of debris on the structure can also increase the effective depth of the superstructure, which would impose larger hydraulic forces on the superstructure and possibly cause structural failure, especially if scour has affected the foundations.

If a sag-vertical curve design has even a small probability of overtopping, open-type railing should be used and the use of curbs should be avoided to minimize damage from high velocity flow around the ends of the parapets.

Figure 9-7 illustrates a profile that may be used where the valley width is sufficient for a crest profile that allows the roadway to be overtopped without submerging the bridge superstructure. Use variations of this profile in locations where the stream channel is located on one side of the floodplain (i.e., an eccentric crossing) and the profile allows overtopping of the approach roadway only on one side. However, perching the structure any higher than required for freeboard offers no economic or hydraulic advantage unless other clearance requirements control the vertical position of the structure.

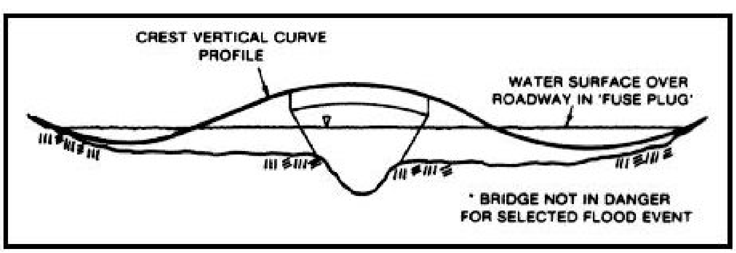


Figure 9-7. Crest Vertical Curve

You can vary the difference between the lowchord and the design water surface elevation, within geometric constraints, to meet requirements for maintaining free surface flow and to accommodate passage of debris and drift. However, perching the structure any higher than required for freeboard offers no economic or hydraulic advantage unless other clearance requirements control the vertical position of the structure.

Figure 9-8 illustrates a profile alternative. Variations of the level profile include a slight crest vertical curve on the bridge to establish a camber in the superstructure. With this profile, all floods with stages below the profile elevation of the roadway and bridge deck will pass through the waterway opening provided.

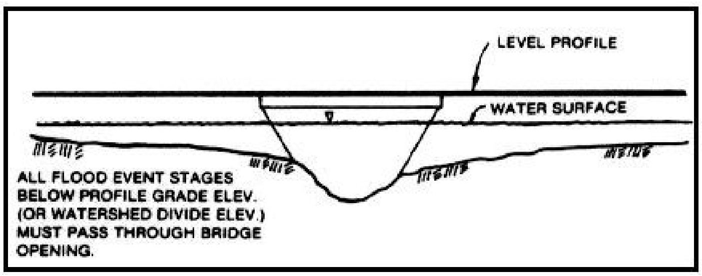


Figure 9-8. Level or Slight Crest Vertical Curve

The disadvantages of the near level profile are similar to those of a sag profile. With either profile configuration, severe contraction scour is likely to occur under the bridge and for a short distance downstream when the superstructure is partially or totally submerged.

Because no relief from these forces is afforded, crossings on zero gradients and in sag-vertical curves are more vulnerable than those with profiles that provide an alternative to forcing all water through the bridge waterway.

Crossing Profile

The horizontal alignment of a highway at a stream crossing should be considered in selecting the design and location of the waterway opening, as well as the crossing profile. Every effort should be made to align the highway so that the crossing will be normal to the stream flow direction (highway centerline perpendicular to the streamline). Often, this is not possible because of the highway or stream configuration.

When a skewed structure is necessary, such as appears in Figure 9-9, the substructure fixtures such as foundations, columns, piers, and bent caps must be designed to offer minimum resistance to the stream flow .at flood stage. The channel may meander within the floodplain and cross under the roadway at an angle different from the floodplain. The bents and headers should be aligned to the stream flow at flood stage because most damage to the bridge happens at flood stage. Flood stage flows also carry the most amount of debris. Bents not aligned with the flood flow will become an obstruction to the flood flow and increase the risk of scour or other failure. The standard skew angles, 15º, 30º, and 45º should be used unless the flow volume or some other problem renders them impractical.

In spite of the flood flow orientation, bents should not be located in the low flow channel if at all possible. As the flow is most concentrated in the channel, the piers would be subject to the highest hydraulic forces. The placement would also increase risk of scour by creating eddies and turbulence, and may encourage drift buildup.

Additionally, relief openings should be provided at the approximate location of point A in Figure 9-9 to reduce the likelihood of trapped flow and to minimize the amount of flow that would have to travel up against the general direction of flow along the embankment.

With the configuration shown in Figure 9-9, the difference in water surface on either side of the embankment at points A and B will be higher than water surface differential through the opening. Relief openings at A and B will help minimize the differential.

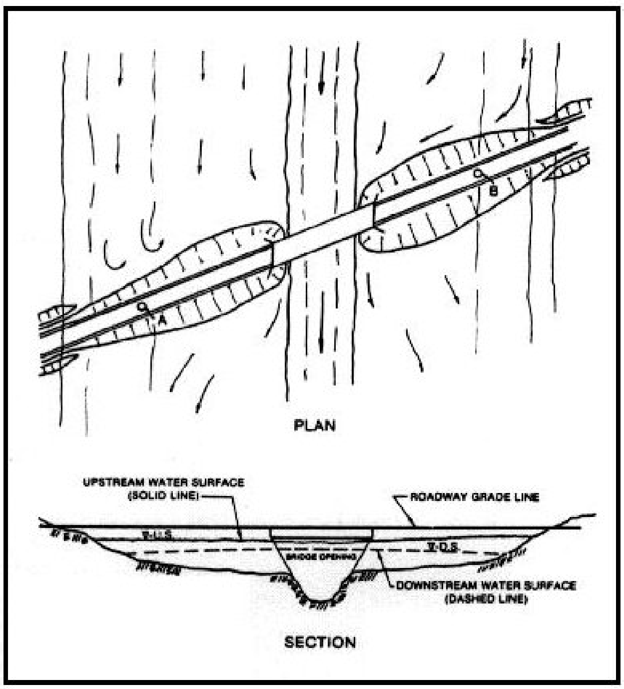


Figure 9-9. Skewed Stream Crossing and Water Surface Differentials

Single versus Multiple Openings

For a single structure, the flow will find its way to the opening until the roadway is overtopped. If two or more structures are available, the flow will divide and proceed to the structures offering the least resistance. The point of division is called a stagnation point.

In usual practice, TxDOT recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft. If flow distances along the embankment are greater than recommended, a relief structure that will provide an additional opening is recommended. A possible alternative to the provision of an additional structure is a guide bank (spur dike) to control the turbulence at the header as discussed in Section 7.

Natural vegetation between the toe of slope and the right-of-way line is useful in controlling flow along the embankment. Therefore, make special efforts to preserve any natural vegetation in such a situation.

Factors Affecting Bridge Length

Bridges over waterways are not always limited to the length of the hydraulic opening required.

- The roadway alignment is at a skew to the streambed, and normalizing the alignment would require unsafe or undesirable curves on the approaches to the bridge.
- Embankments may be limited to a certain location due to local soil instability or permitting requirements.
- Bridge costs might be cheaper than embankment costs.
- Matching the highway profile grade line.
- High potential for a meander to migrate, or other channel instabilities.

These and other aspects are valid considerations that affect bridge waterway openings. However, hydraulic computations are necessary to predict the performance and operation of the waterway opening at flood stages. Do not neglect hydraulic design. The design decisions, including the reasons for any excess opening, must be documented.

Section 4 — Hydraulics of Bridge Openings

Bridge Modeling Philosophy

Numerous methods exist for estimating the hydraulic impact of bridge openings on water surface profiles. TxDOT recommends that computer programs be employed to perform such estimates. Generally, the documentation of the specific computer program should be referred to for the theory employed.

Note. Previously, TxDOT employed a single energy loss equation, ($h = \Delta v^2/2g$), to estimate the backwater effect of bridge openings. It is no longer used as the basis for design of TxDOT bridges.

Bridge Alignment

The discussions of bridge design assume normal lengths and alignments perpendicular to the flow at flood stage even though the low flow streambed may be at a skew to the bridge. In actuality, many bridges are skewed to some degree which causes the hydraulic length of the bridge to be longer than the plan width. For example, a 60-foot wide bridge perpendicular to the stream has a hydraulic length of 60 feet. The same bridge at a 30º skew to the stream has a hydraulic length of 69.3 feet.

Hydraulic programs do not automatically account for the skew. The program operator is required to specifically account for the skew by putting in the correct hydraulic length.

Flow Zones and Energy Losses

Figure 9-10 shows a plan of typical cross section locations that establish three flow zones that should be considered when estimating the effects of bridge openings.

NOTE: These cross sections are related to the explanation of flow zones and energy losses and must not be confused with the cross sections required for analysis. The analysis cross sections may be further upstream and downstream.

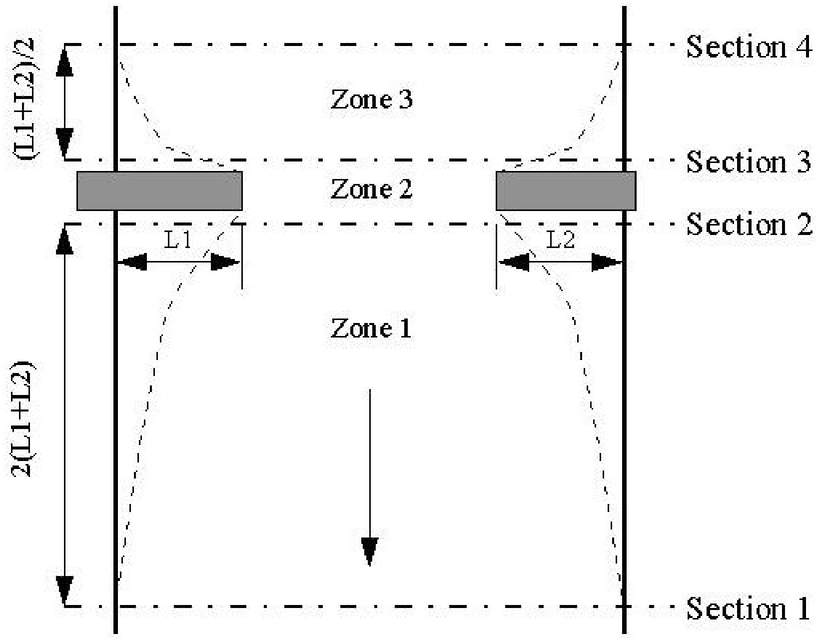


Figure 9-10. Flow Zones at Bridge

Zone 1 – Downstream. Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. No detailed guidance is available, but a distance equal to about four times the length of the average embankment constriction is reasonable for most situations. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 9-11.

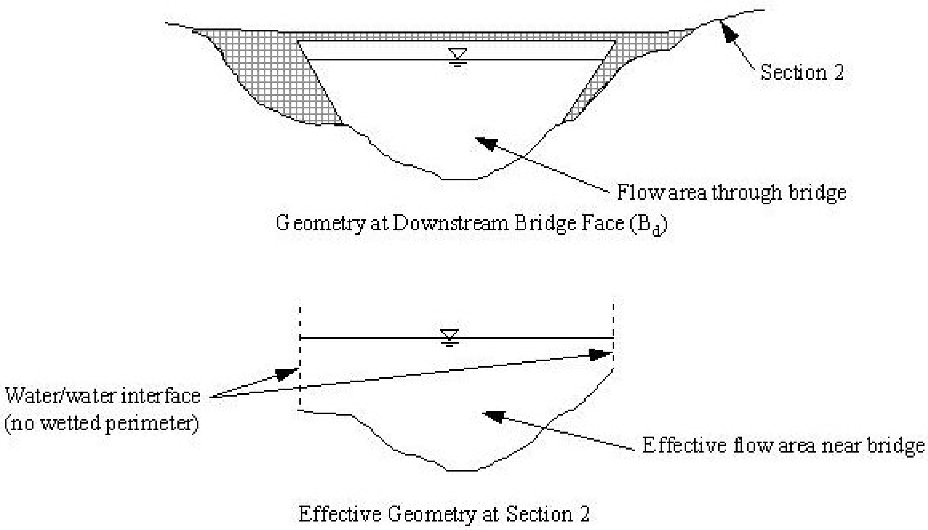


Figure 9-11. Effective Geometry for Bridge (Section 2 shown, Section 3 similar)

Zone 2 - Under Bridge Opening. Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally, consider the bridge opening by superimposing the bridge geometry on cross sections 2 and 3.

Zone 3 – Upstream. Zone 3 represents an area from the upstream face of the bridge to a distance upstream where contraction of flow must occur. A distance upstream of the bridge equal to the length of the average embankment constriction is a reasonable approximation of the location at which contraction begins. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the “approach” cross section.

Extent of Impact Determination

The maximum effect of the bridge should occur at cross section 4. However, in order to determine the extent of the impact, continue water surface profile computations upstream until the water surface does not differ significantly from the estimated pre-construction conditions.

Water Surface Profile Calculations

Calculate the water surface profile through Zones 1 and 3 using the Standard Step Backwater Method (see Chapter 7) with consideration of expansion and contraction losses. The table 9-1 below provides recommended loss coefficients.

Table 9-1: Recommended Loss Coefficients for Bridges

Transition Type	Contraction (K_c)	Expansion (K_e)
No losses computed	0.0	0.0
Gradual transition	0.1	0.3

Table 9-1: Recommended Loss Coefficients for Bridges

Transition Type	Contraction (K_c)	Expansion (K_e)
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

Bridge Flow Class

The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow.

Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. (This condition should exist for the design frequency of all new on-system bridges.) Low flow is divided into categories as described in Table 9-2 “Low Flow Classes”. Type I is the most common in Texas, although severe constrictions compared to the flow conditions could result in Types IIA and IIB. Type III is likely to be limited to steep hills and mountainous regions.

Table 9-2: Low Flow Classes

Type Designation	Description
I	Subcritical flow through Zones
IIA	Subcritical flow Zones 1 and 3, flow through critical depth Zone 2
IIB	Subcritical Zone 3, flow through critical Zone 2, hydraulic jump Zone 1
III	Supercritical flow through Zones 1, 2 and 3

High flow refers to conditions in which the water surface impinges on the bridge superstructure:

- When the tailwater does not submerge the lowchord of the bridge, the flow condition is comparable to a pressure flow sluice gate.
- At the tailwater, which submerges the lowchord but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the tailwater overtops the roadway, neither sluice gate flow nor orifice flow is reasonable, and the flow is either weir flow or open flow.

Zone 2 Loss Methods

Generally determine the losses in Zone 2 by one of the following methods depending on the flow characteristics and the engineer's judgment:

- Standard Step Backwater Method (based on balance of energy principle)
- Momentum Balance Method

- WSPRO Contraction Loss Method [This is a method, not limited to the WSPRO program.]
- Pressure Flow Method
- Empirical Energy Loss Method (HDS 1).

Standard Step Backwater Method (used for Energy Balance Method computations)

Refer to Chapter 7 for the Standard Step Backwater Method. Figure 9-12 shows the relative location of section geometry for profile computations. B_d and B_u refer to the bridge geometry at the downstream and upstream inside faces, respectively.

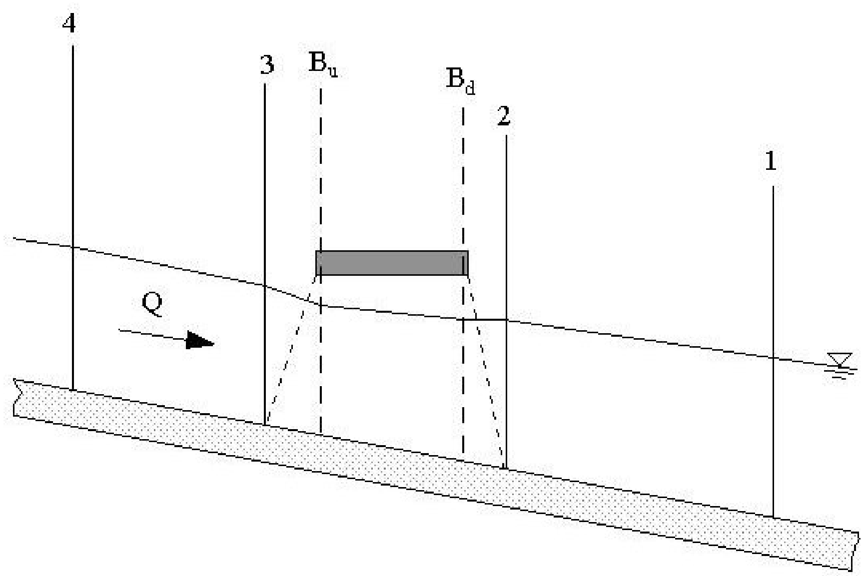


Figure 9-12. Relative Location of Section Geometry

1. Solve the energy equation (step backwater) between cross section 2 and the downstream bridge face (B_d). Use the water surface at cross section 2 determined from the previous backwater profile computations.
2. Proceed with the standard step backwater calculations from the downstream bridge face to the upstream face. Use the bridge geometry superimposed on cross sections 2 and 3 respectively.
3. Approximate the effects of piers and impingement of flow on the lowchord by reducing the section area and increasing the wetted perimeter accordingly.
4. Similarly, consider roadway overflow as open channel flow. Proceed with calculations from the upstream bridge face (B_u) to cross section 3.
5. As indicated in the previous Flow Zones and Energy Losses subsection, proceed with calculating the remainder of the bridge impact from cross section 3 upstream using step backwater calculations.

Under the right circumstances, you can consider the energy balance method for low flow and high flow.

Momentum Balance Method

This method computes the backwater through Zone 2 by balancing forces at three locations:

- between the inside, downstream face of the bridge (B_d) and cross section 2
- between the downstream and upstream ends of the bridge (B_d to B_u)
- between the inside, upstream face of the bridge (B_u) and cross section 3.

Refer to Figure 9-10 and Figure 9-12 for zone and cross section locations. Assuming hydrostatic pressure conditions, the forces acting on a control volume between two cross sections (1 and 2) must be in balance and are generalized in Equation 9-2.

$$F_{P2} + F_m = F_{P1} + F_f + F_d - F_w$$

Equation 9-2.

where:

F_{P1}, F_{P2} = force due to hydrostatic pressure at cross section = γAy

F_m = force causing change in momentum between cross sections = $\rho Q \Delta v$

F_f = force due to friction = $\gamma (A_1+A_2)LS_f/2$

F_d = total drag force due to obstructions (e.g., for piers = $\rho C_d A_o v/2$)

F_w = component of weight in direction of flow = $\gamma (A_1+A_2)LS_o/2$.

1. For subcritical flow, determine the water surface elevation and average velocity at section 2 from step backwater computations.
2. Determine the water surface elevation and average velocity at Section B_d by applying successive assumed water surface elevations to Equation 9-3 until equality is achieved within a reasonable tolerance.
3. Determine the momentum correction factor (B), which accommodates natural velocity distributions similar to the energy correction factor, α, using Equation 9-4.
4. Using the resulting water surface elevation at B_d, determine the water surface elevation and average velocity at Section B_u by applying successive assumed water surface elevations at Section B_u to Equation 9-5 until achieving equality within a reasonable tolerance. B_u refers to the upstream face of the bridge.
5. Determine the final momentum balance between the upstream face of the bridge and cross section 3 using Equation 9-6. Table 9-3 "Suggested Drag Coefficients for Bridge Piers" presents suggested drag coefficients for different pier types.

6. As discussed in the above Flow Zones and Energy Losses section, proceed with the remainder of the bridge impact computations from cross section 3 upstream using step backwater calculations.

$$A_{Bd}\bar{y}_{Bd} + \frac{\beta_{Bd}Q^2}{gA_{Bd}} = A_2\bar{y}_2 - A_{pd}\bar{y}_{pd} + \frac{\beta_2 Q^2}{gA_2} + \left(\frac{A_2 + A_{Bd}}{2}\right)LS_f - \left(\frac{A_2 + A_{Bd}}{2}\right)LS_o$$

Equation 9-3.

where:

Subscripts 2 and Bd refer to section 2 and the downstream bridge face, respectively.

A = effective flow area at cross sections (sq.ft. or m^2)

$\bar{y}$ = height from water surface to centroid of effective flow area (ft. or m)

g = acceleration due to gravity (ft./s^2 or m/s^2)

Q = discharge (cfs or m^3/s)

A_{pd} = obstructed area of pier at downstream side (sq. ft. or m^2)

L = distance between cross sections (ft. or m)

S_f = friction slope (ft./ft. or m/m) (see Chapter 6)

S_o = channel bed slope (ft./ft. or m/m)

β = momentum correction factor.

$$\beta = \frac{A_T \sum \left[K_i^2 / A_i\right]}{K_T^2}$$

Equation 9-4.

where:

K_i = conveyance in subsection (cfs or m^3/s)

A_i = area of subsection (sq. ft. or m^2)

K_T = total conveyance of effective area section (cfs or m^3/s)

A_T = total effective area (sq.ft. or m^2).

$$A_{Bu}\bar{y}_{Bu} + \frac{\beta_{Bu}Q^2}{gA_{Bu}} = A_{Bd}\bar{y}_{Bd} + \frac{\beta_{Bd}Q^2}{gA_{Bd}} + \left(\frac{A_{Bd} + A_{Bu}}{2}\right)LS_f - \left(\frac{A_{Bd} + A_{Bu}}{2}\right)LS_o$$

Equation 9-5.

$$A_3\bar{y}_3 + \frac{\beta_3 Q^2}{gA_3} = A_{Bu}\bar{y}_{Bu} + A_{pu}\bar{y}_{pu} + \frac{\beta_{Bu}Q^2}{gA_{Bu}} + \left(\frac{A_{Bu} + A_3}{2}\right)LS_f - \left(\frac{A_{Bu} + A_3}{2}\right)LS_o + \frac{C_d A_{pu} Q^2}{2gA_3^2}$$

Equation 9-6.

where:

Subscript 3 refers to cross section 3

A_{pu} = Obstructed area of piers at upstream side (sq.ft. or m^2)

C_d = drag coefficient

Table 9-3: Suggested Drag Coefficients for Bridge Piers

Pier Type	Drag Coefficient, C_d
Circular	1.20
Elongated with semi-circular ends	1.33
Elliptical (2:1 aspect ratio)	0.60
Elliptical (4:1 aspect ratio)	0.32
Elliptical (8:1 aspect ratio)	0.29
Square nose	2.00
Triangular nose (30° apex)	1.00
Triangular nose (60° apex)	1.39
Triangular nose (90° apex)	1.60
Triangular nose (120° apex)	1.72

WSPRO Contraction Loss Method

The Water Surface Profile (WSPRO) method is a contraction model that uses step backwater calculations and empirical loss coefficients. *This method is not limited to the WSPRO program.*

1. Base the model on providing approach and exit cross sections (cross sections 1 and 4) at distances from the downstream and upstream faces approximately equal to the bridge opening length.
2. Compute the flow in Zones 1 and 3 using step backwater computations with a weighted flow length based on 20 equal conveyance tubes. Refer to *Bridge Waterways Analysis Model* (Shearman et al., 1986) for details on this method. (See Reference for details on obtaining this document.)

Pressure Flow Method

By definition, pressure flow methods represent high flow conditions. Figure 9-13 shows a high flow condition in which the water surface at the upstream face of the bridge has impinged the low-chord but the downstream face is not submerged. You may approximate this condition as a sluice gate using Equation 9-7. You need to assume successive elevations at cross section 3 (y_3) until the calculated discharge in Equation 9-7 is equal to the design discharge within a reasonable tolerance.

$$Q = CA_b\left[2g\left(y_3 - \frac{D_b}{2} + \frac{\alpha_3 v_3^{\,2}}{2g}\right)\right]^{0.5}$$

Equation 9-7.

where:

Q = calculated discharge (cfs or m^3/s)

C = discharge coefficient (0.5 suggested)

A_b = net area under bridge (sq. ft. or m^2)

y_3 = depth of flow at cross section 3 (ft. or m)

D_b = height of lowchord from mean stream bed elevation (ft. or m).

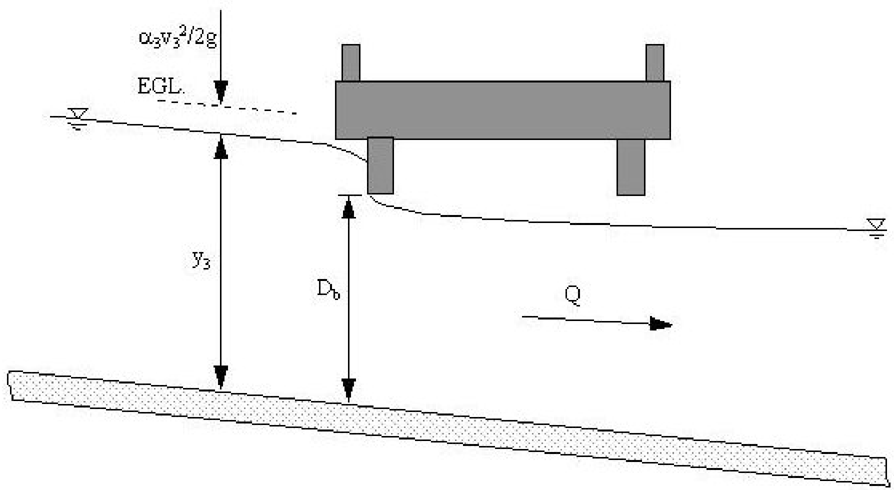


Figure 9-13. Sluice Gate Type Pressure Flow

Figure 9-14 shows a submerged bridge opening with a tailwater lower than the overtopping elevation Equation 9-8 represents orifice flow. You need to assume successive elevations at cross section 3 (y_3) until the calculated discharge in Equation 9-8 is equal to the design discharge within a reasonable tolerance.

$$Q = CA_b\sqrt{2gH}$$

Equation 9-8.

where:

C = discharge coefficient (0.8 typical)

H = difference between energy grade at cross section 3 and water surface at cross section 2 (ft. or m), Equation 9-9.

$$H = y_3 + \alpha_3 \frac{v_3^2}{2g} - y_2$$

Equation 9-9.

where:

α_3 = kinetic energy correction coefficient

C_d = coefficient of discharge, Equation 9-12.

$$C_d = 0.104 \frac{L_c}{b} + 0.7145$$

Equation 10-10.

where:

b = width of top of embankment at bridge abutment (ft. or m) (see Figure 9-15)

L_c = length of bridge opening between abutment faces (ft. or m).

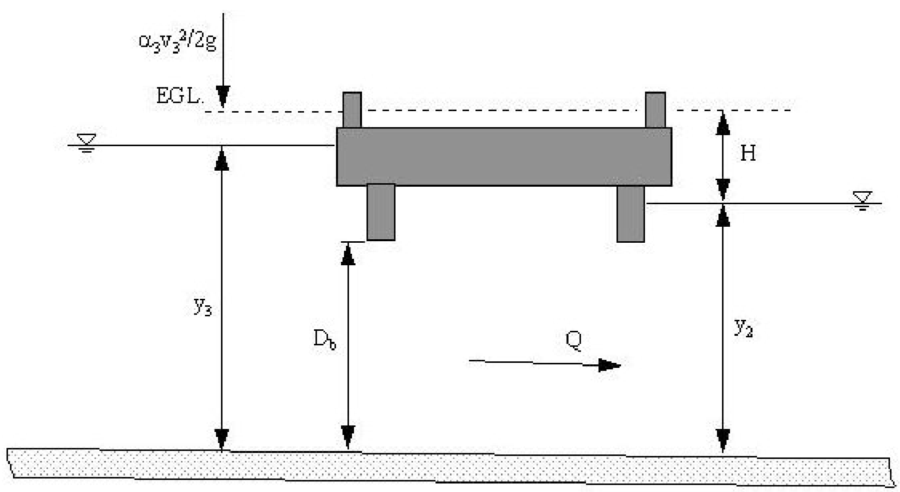


Figure 9-14. Orifice Type Pressure Flow

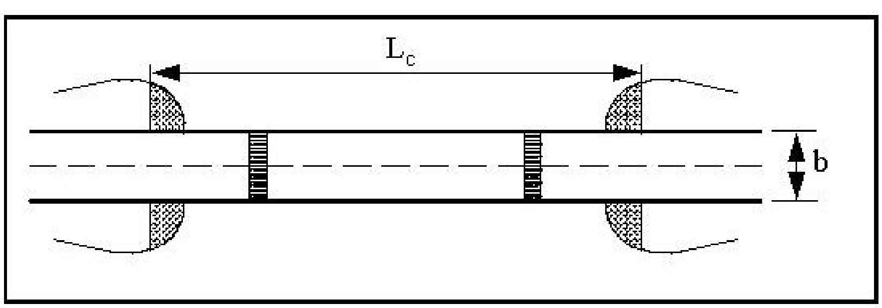


Figure 9-15. Bridge Dimensions for Pressure Flow Analysis

Empirical Energy Loss Method (HDS 1)

Although hand computations are used rarely, the FHWA publication *Hydraulics of Bridge Waterways* (HDS 1) presents methods for estimating bridge backwater effects. The HDS-1 Empirical Loss Method presents a summary of the method that is appropriate for low flow Type I. (This flow type should predominate for the design of bridges over Texas streams.)

Two-dimensional Techniques

Two-dimensional (2-D) horizontal flow, depth-averaged techniques are highly specialized. Contact the Design Division's Hydraulics Branch for consultation.

Roadway/Bridge Overflow Calculations

Consider flow over the bridge or roadway in one of two ways:

- Weir flow if the tailwater does not drown out critical depth of flow in the overtopping section -- the approach is similar to that in the Roadway Overtopping subsection of Chapter 8, except that the design engineer must use the bridge loss methods above instead of culvert head loss computations. That is, apportion flow between the bridge and the weir such that the head at cross section 3 results in a flow apportionment that sums to equal the design flow within a reasonable tolerance.
- Open channel flow if the tailwater is too high -- As the depth of flow over the road increases and the tailwater submerges the road, the design engineer considers the flow over the road as open channel flow and use step backwater computations across the road.

Backwater Calculations for Parallel Bridges

The backwater calculation for parallel bridges (depicted in Figure 9-16) requires the application of a coefficient. The chart in Figure 9-17 relates the value of the backwater adjustment coefficient (μ) to the ratio of the out-to-out dimension of the parallel bridges to the width of a single embankment (see Figure 9-18). Determine the backwater head calculation for a parallel bridge with Equation 9-11.

$$h = \mu h_1$$

Equation 9-11.

where:

h = total backwater head (ft. or m)

μ = backwater adjustment coefficient (see Figure 9-17)

h_1 = backwater head for one bridge as discussed in the Bridge Flow Class subsection above.

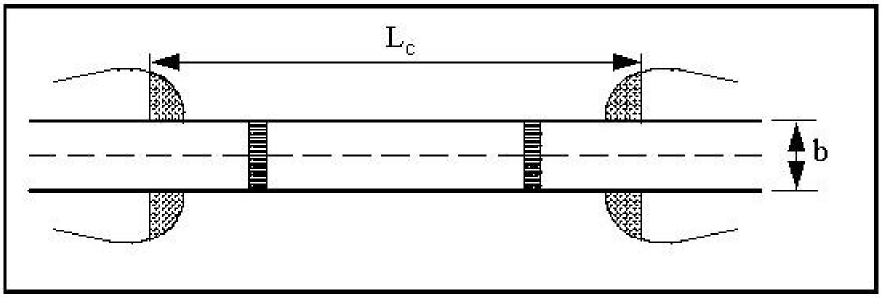


Figure 9-16. Parallel Bridges

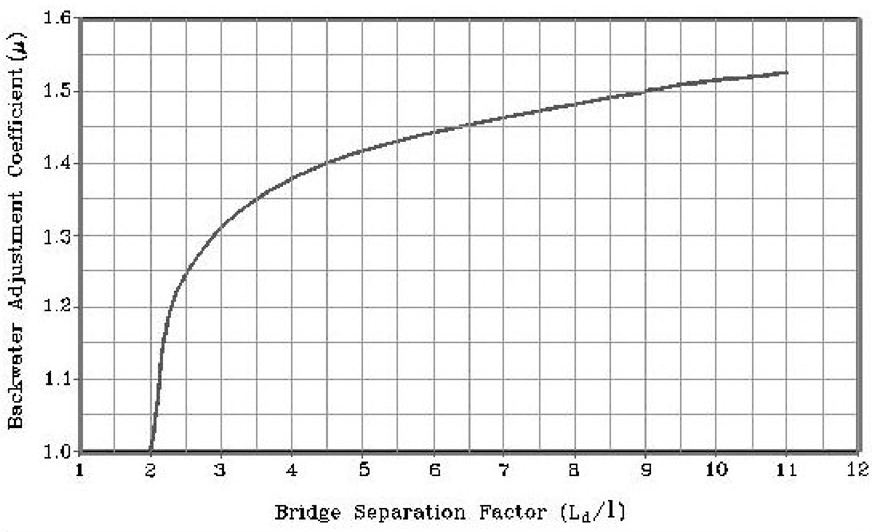


Figure 9-17. Parallel Bridges Backwater Adjustment

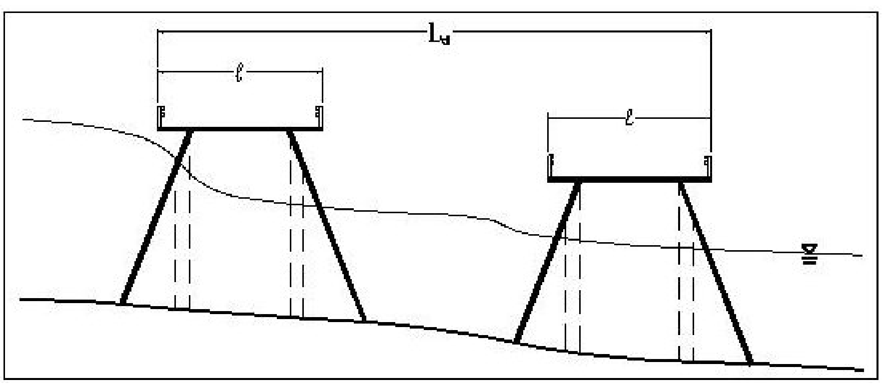


Figure 9-18. Definition of Parameters

Section 5 — Single and Multiple Opening Designs

Introduction

This section provides a means to establish an initial size of opening and lengths and locations of multiple openings.

- For a single opening, analyze the effect of the trial opening using the method selected from those outlined in Bridge Hydraulic Considerations. If the resulting backwater or through-bridge velocities are unacceptable, modify the opening until the estimated conditions are satisfactory for both the design and check flood conditions. The department recommends automated procedures for such analyses.
- Where a bridge must cross a relatively wide floodplain or multiple discharge concentrations, it may be necessary to design multiple openings. A multiple opening configuration usually constitutes a main channel bridge with relief openings. This type of crossing provides openings at or near the flow concentrations. The result is a reduction in along-embankment flow and backwater effects.

Single Opening Design Guidelines

To establish a single structure length and elevation of lowchord, begin by estimating the design flood, obtaining accurate controlling cross sections, and determining the design and check flood water surface profiles. For complete documentation, you may need a compilation of past flood history, existing structures, and other highway crossing characteristics of the stream.

1. Assume an average through-bridge velocity (v_t) that is less than the maximum allowable velocity but that is not lower than the unconstricted average velocity.
2. Apply the unconstricted design water surface elevation to the cross section, and find the area (A_t) subtended by this water surface that will satisfy the Continuity Equation (Equation 9-1, reworked as Equation 9-12) for trial velocity and design discharge.

$$A_t = \frac{Q}{v_t}$$

Equation 9-12.

3. Estimate an average depth of water (D_t) in the cross section where the bridge is to be located by inspecting the section.
4. Find the trial length (L_t) of the bridge using Equation 9-13.

$$L_t = \frac{A_t}{D_t}$$

Equation 9-13.

5. Position the headers in the stream cross section (same cross section as in Step 3) so that they are approximately L_t apart and at locations that appear to maximize the through-bridge area.
6. Find the exact waterway area (A_w) below the design high water within the structure limits.
7. Find the average through-bridge velocity (v_b) for the actual waterway area (A_w) by using the Continuity Equation.

$$v_b = \frac{Q}{A_w}$$

Equation 9-14.

8. Evaluate and establish allowable maximum velocity based on individual site characteristics. If v_b is close to the target average velocity, the initial bridge length may be reasonable. You must usually adjust this length slightly to fit standard span length requirements. If v_b is much lower or greater than the allowable maximum velocity, adjust the length as necessary, repeating steps 6 and 7. Repeat this routine until the average through-bridge velocity is close to the target velocity. To minimize the cost of the structure, it is usually desirable to adjust the bridge length so that the design velocity is at or very near the maximum allowable velocity.
9. Establish a lowchord (as discussed in the Freeboard subsection of Section 3).
10. For the design and 1% AEP discharges, estimate the backwater caused by the constriction of the bridge opening. Use the procedures outlined in the Bridge Hydraulic Considerations section (Section 3). You may need to adjust the bridge length to ensure that the backwater effects are not excessive and comply with FEMA NFIP criteria, where applicable.
11. Determine the maximum potential scour envelope. The Bridge Division Geotechnical Branch is the office of primary responsibility for bridge scour. See the Bridge Division Geotechnical Manual or contact the Geotechnical Branch for bridge scour policies.

Multiple Opening Design Approach

Design multiple structures so that each structure's carrying capacity (or conveyance) is approximately the same as the predicted discharge approaching the structure. Poorly sized structures could result in a reapportionment of the approach discharges. Reapportionment of flow, in turn, may cause excessive backwaters, unacceptable along-embankment velocities, and excessive velocities through some structures.

In addition to striving for balance in proportion (discussed in the Carrying Capacity Guidelines subsection above), satisfy average through-bridge velocity requirements. Unfortunately, widely disparate through-bridge velocities cause uneven backwaters that will likely redistribute of flow, upsetting the originally designed balance of structure conveyances. The goal is to balance conveyances and simultaneously try to assure that the resulting energy grade levels at the approach cross section (Section 4) are about the same for each bridge in the multiple opening facility. (See Bridge Sizing and Energy Grade Levels for more information.)

Multiple Bridge Design Procedural Flowchart

The flow chart for multiple bridge design (Figure 9-19) illustrates the steps and considerations recommended in TxDOT designs.

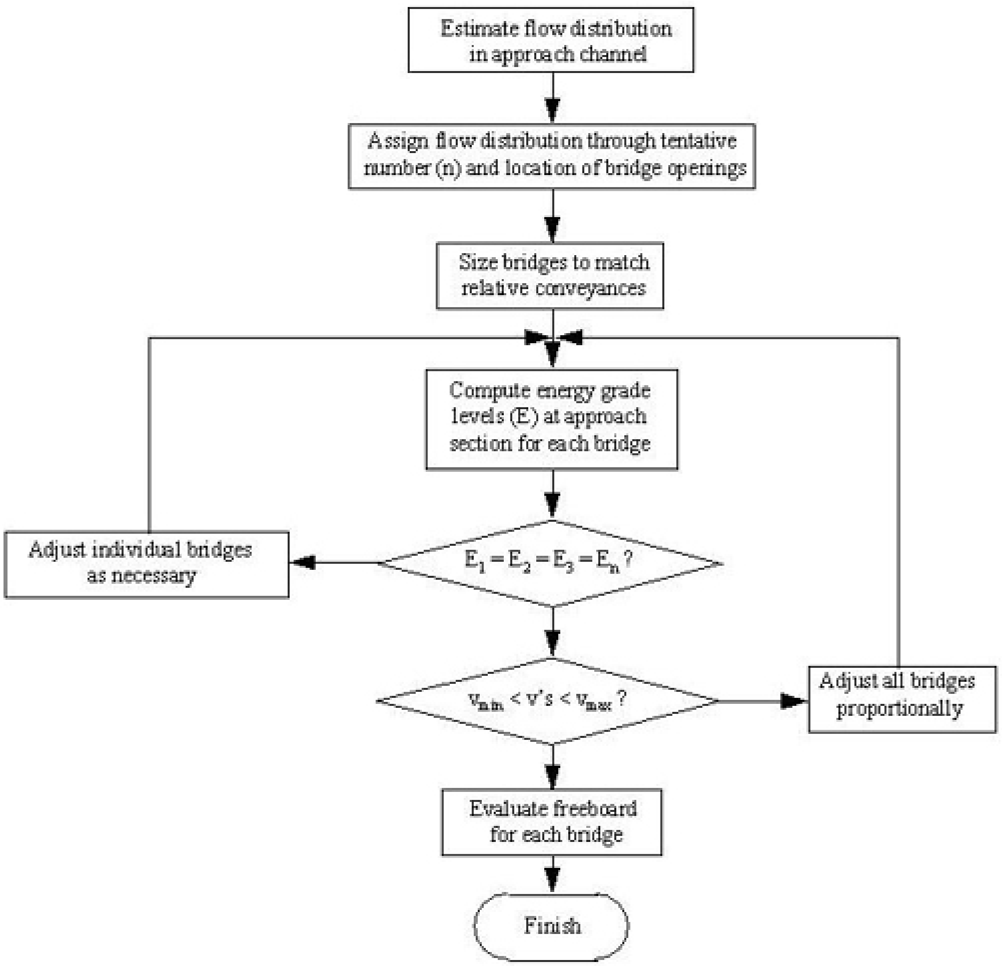


Figure 9-19. Multiple Bridge Design Flowchart

When estimating the design high water at a multiple structure location, the design engineer still needs to determine how the flow divides itself across the floodplain at flood stage. In the case of multiple structures, the flow division indicates the approximate portion of the total flood discharge that will be carried by each structure. One method for estimating flow division is by actually observing the flow at design discharge and design high water at the proposed site. However, the ability to make such an observation when the proper set of circumstances occurs would be rare. Therefore, use the following analytical method to determine flow distribution and establish flow division.

Cumulative Conveyance Curve Construction

Inspection of incremental discharges or conveyances across a floodplain cross section usually reveals the location of relatively heavier concentrations of flow. By determining these heavier concentrations of flow, the design engineer can usually find reasonable locations for each of the bridges. In some instances, the concentrations of flow and associated flow divides are quite obvious. In other cases, the distribution of flow may be subtler, and must be estimated analytically, which is most easily done with a hydraulic analysis program.

Bridge Sizing and Energy Grade Levels

When you have estimated relative approach discharges, you should have two, often contradictory objectives:

- Try to size the multiple structures so that they offer approximately the same relative carrying capacities as the relative flow distribution would indicate.
- To minimize cross flow, you need to obtain similar values of energy grade level at the approach section for all openings. Generally, if the relative velocity differentials are not approximately the same for all openings, head differentials develop, causing a redistribution of the approach flows.

Often, it is not possible to balance energy grade levels and conveyances simultaneously. Therefore, because of the importance of avoiding a redistribution of flow from natural conditions, place more emphasis on balancing energy grade levels by having velocity head differentials approximately the same for each of the openings.

Size the bridges in a multiple opening situation to avoid exceeding maximum allowable through-bridge velocities at any of the openings. Calculate backwater head for a multiple opening situation in the same manner as for single opening structures outlined in the Single Opening Design Procedure subsection and based on the appropriate floodplain subsection and flow apportionment. That is, consider each bridge separately using the flow apportionment and associated portion of cross section.

Freeboard Evaluation

Determine the distance between the lowchord and the water surface. Then, compare the result to the recommended freeboard considerations discussed under Freeboard in Section 3.

Analysis of Existing Bridges

One-dimensional analysis of an existing bridge involves the same concepts employed for designing a new bridge: assume that the flood flow will distribute itself to attain a constant energy grade at the approach section. The existing bridge will likely redistribute flow from what the approach channel conditions might otherwise imply. The stagnation points become functions of the bridge openings and the channel conditions. Until the computed energy levels at the approach section are approximately equal, you need considerable trial and error may be needed to adjust stagnation points, determine conveyance apportionment, and analyze each opening.

Section 6 — Flood Damage Prevention

Extent of Flood Damage Prevention Measures

The response of alluvial streams to floods is often unpredictable. Knowledge of the history of a stream and its response to floods is the best guide for determining the extent of flood damage prevention measures. When protection is needed, whether at the time of construction or at a later date, the cost of providing the control measures should be compared to the potential costs associated with flood damage without the prevention measures.

Flood-related damage results from a variety of factors including the following:

- scour around piers and abutments
- erosion along toe of highway embankment due to along-embankment flow
- erosion of embankment due to overtopping flow
- long term vertical degradation of stream bed
- horizontal migration of stream banks
- debris impact on structure
- clogging due to debris causing redirection of flow.

The designer should assess the potential for these and other conditions to occur and consider measures that reduce the potential for damage from flooding.

Pier Foundations

The primary flood-related concern at piers is the potential for scour. Two typical approaches are to design deep enough foundations to accommodate scour and to protect the streambed around the foundation to prevent or reduce the potential for scour.

Primary protection measures at piers include concrete riprap, stone protection, gabions, and grout-filled or sand/cement-filled bags. See FHWA IH-97-030, “Bridge Scour and Stream Instability Countermeasures” (HEC-23) for discussion on selection of measures.

The following should be considered the following to reduce the potential for pier scour:

- Reduce numbers of piers by increasing span lengths, especially where you expect large debris loads.
- Use bullet-nosed or circular-shaped piers.
- Use drilled shaft foundations.

- Align bents with flood flow to degree practicable.
- Increase bridge length to reduce through-bridge velocities.

Where there is a chance of submergence, a superstructure that is as slender as possible with open rails and no curb should be used.

Because of uncertainties in scour predictions, use extreme conservatism in foundation design. In other words, deeper foundations may be cheaper. The capital costs of providing a foundation secure against scour are usually small when compared to the risk costs of scour-related failure.

Approach Embankments

Embankments that encroach on floodplains are most commonly subjected to scour and erosion damage by overflow and by flow directed along the embankment to the waterway openings. Erosion can also occur on the downstream embankment due to turbulence and eddying as flow expands from the openings to the floodplain and due to overtopping flow.

The incidence of damage from flow along an approach embankment is probably highest in wooded floodplains where the right-of-way is cleared of all trees and where borrow areas are established upstream of the embankment. Damage to approach embankment is usually not severe, but scour at the abutments from the flow contraction may be significant if the abutment is not protected.

The potential for erosion along the toe of approach embankment can be minimized by avoiding extensive clearing of vegetation and avoiding the use of borrow areas in the adjacent floodplain. Embankment protection such as stone protection can be used, but stable vegetation on the embankment may suffice. Other measures that may be used are riprap, pervious dikes of timber, or finger dikes of earthen material spaced along and normal to the approach fill to impede flow along the embankment.

The embankment may need to be protected if significant overtopping of the approach embankment is anticipated during the life of the crossing. The embankment can be protected with soil cement or revetments, rock, wire-enclosed rock, or concrete.

Preventive measures are also needed at some crossings to protect the embankment against wave action, especially at reservoirs. Riprap of durable, hard rock should be used at such locations. The top elevation of the rock required depends on storage and flood elevations in the reservoir and wave height computed using wind velocities and the reservoir fetch.

Abutments

Protective measures used at abutments consist of the following:

- riprap header slopes and deep toe walls (stone protection is generally preferred to concrete)

- vertical abutment walls
- sheet pile toe walls
- deep foundations of piles or drilled shafts.

Vertical abutment walls will protect bridge ends and the embankment if the walls are extended around the fill slopes to below the depth of anticipated scour. Sheet pile toe walls are usually installed to repair scour damage after a flood. They are commonly used where rock is not available or access for placing rock is difficult. Sheet pile may be used only under guidance from the Bridge Division's Geotechnical Branch.

Revetment is usually placed at the abutment on the slopes under the bridge end and around the corners of the embankment to guard against progressive embankment erosion. Revetment on the fill slope may be susceptible to contraction scour. To prevent embankment failure from undermining by contraction scour, a toewall must be extended below the level of expected scour.

Two common types of revetments used to protect abutments are rigid (i.e. concrete riprap) and flexible (i.e. stone protection, articulated concrete blocks, and gabion mattresses). A unique feature of stone protection is can be designed to be self launching. That is, the rocks will shift to fill any area that scours and inhibit any further scour.

Guide Banks (Spur Dikes)

The twofold purpose of guide banks is to align flow from the floodplain with the waterway opening and minimize scour at the abutment by moving the scour-causing turbulence to the upstream end of the guide bank. Where the floodwater must flow along the embankment for more than 800 feet, guide banks should be considered. Figure 9-20 shows a typical plan form.

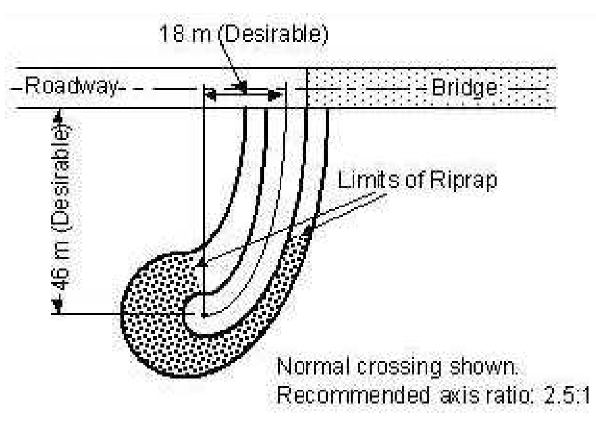


Figure 9-20. Typical Guide Bank

Guide banks are usually constructed of earthen embankment but are sometimes constructed from rock. The dike should be protected by revetment where scour is expected to occur, although a failure at the upstream end of a spur dike usually does not immediately threaten the bridge end.

Clearing around the end of the dike in wooded floodplains should be minimized to enhance the effectiveness. A drainage channel around the end of the dike for local drainage may induce turbulence from mixed flows. Instead, a small culvert through the dike will help minimize the turbulence of mixed flows from different directions.

The suggested shape of guide banks is elliptical with a major-to-minor axis ratio of 2.5:1. The suggested length varies with the ratio of flow diverted from the floodplain to flow in the first 100 feet of waterway under the bridge. The suggested shape is based on laboratory experiments, and the length is based on modeling and field data. The optimum shape and length may differ for each site and possibly for each flood at a site. However, field experience shows, however, that the recommended elliptical shape is usually quite effective in reducing turbulence. Should practical reasons require the use of another shape such as a straight dike, more scour may be expected at the upstream end of the guide banks. Guide banks can also be used at the downstream side of the bridge to help direct flow back into the overbanks.

Bank Stabilization and River Training Devices

Bank stabilization and river training devices are intended to inhibit the erosion and movement of stream banks. They may be needed either to defend against actions of the stream that threaten the highway crossing or to protect the stream banks and the highway from an anticipated response to highway construction.

Various materials and devices designers use include the following:

- stone protection
- concrete lining
- wood, steel, or rock jetties
- steel or concrete jack fields
- wire fences
- timber bulkheads
- articulated concrete mattresses
- guide banks, dikes, and spurs (usually constructed of earth and rock).

The choice of the appropriate device or devices for use depends on the geomorphology of the river. Futile attempts at localized control can be avoided where the river is in the midst of changes by studying long reaches. Regardless of the size of the stream and the control measures used, stream response to the measure must be considered. For instance, bank stabilization at a crossing may cause scour in the bed of the channel or redirect the current toward an otherwise stable bank downstream.

Bank stabilization and river training is a specialized field requiring familiarity with the stream and its propensity to change, knowledge of the bed load and debris carrying characteristics of the stream, and experience and experimentation at similar sites on the same or a similar stream.

The following are general principles for the design and construction of bank protection and training works:

- The cost of the protective measures should not exceed the cost of the consequences of the anticipated stream action.
- Base designs on studies of channel morphology and processes and on experience with compatible situations. Consider the ultimate effects of the work on the natural channel (both upstream and downstream).
- Inspect the work periodically after construction with the aid of surveys to check results and to modify the design, if necessary.
- Understand that the objective of installing bank stabilization and river training measures is to protect the highway. The protective measures themselves are expendable.

Refer to the FHWA publication *Stream Stability at Highway Structures* (HEC 20) for more detailed information regarding bank stabilization and stream training facilities.

The effectiveness of protective and training measures in many alluvial streams and the need for the measures may be short-term because of the dynamic nature of streams. The stream will move to attack another location or outflank the installation.

A cost comparison of viable options should be made. Alternatives to stream protection measures include the following:

- a continuing effort to protect the highway by successive installations intended to counter the most recent actions of the stream
- relocation of the roadway away from the river hazard
- a larger opening designed to accommodate the hazard
- abutment foundations designed sufficiently to allow them to become interior bents at a later date.

Minimization of Hydraulic Forces and Debris Impact on the Superstructure

The most obvious design guideline is to avoid the imposition of hydraulic forces on a bridge superstructure by placing the bridge at an elevation above which the probability of submergence is small. Obviously, this is not always economically or physically practical.

One design alternative is to make the superstructure as shallow as possible. Box girders that would displace great volumes of water and have a relatively small weight compared to the weight of water displaced are not a good design alternative unless the probability of submergence is very small. Solid parapets and curbs that increase the effective depth of the superstructure can give increased buoyancy over that of open rail designs. If submerged, the increased effective depth of the superstructure causes increased general scour, and drag forces on the superstructure are much greater than with open rails.

Another consideration is to provide a roadway approach profile that will be overtopped prior to the submergence of the bridge superstructure. This will reduce the probability of submergence of the bridge and help to reduce the potential for scour at the bridge . The consequence may be the need for repairs to the roadway approach.

Where large volumes of debris are likely to occur, longer spans and high freeboards may be warranted. In extreme situations, debris racks may be installed to stop the debris before it reaches the structure. Bridge designers should consult with Design Division Hydraulics prior to specifying or installing debris racks.

For even a small probability of total or partial submergence, see the Bridge Division Design Manual for guidance. If the dead load of the structure is not sufficient to resist buoyant, drag, and debris impact forces, the superstructure may need to be anchored to the substructure. Air holes may also be provided through each span and between each girder to reduce the uplift pressure.

Section 7 — Appurtenances

Bridge Railing

The type of railing used on a bridge is as much a hydraulic consideration as one of traffic safety and aesthetics. This is particularly true in instances where overtopping of the bridge is possible. The two types of rail discussed here are:

- Solid bridge railing -- Solid bridge rail should be used only where the bridge superstructure is in no danger of overtopping. A solid type of rail (e.g., a parapet wall) is useful from a safety standpoint but constitutes a significant impediment to flood flow.
- Open bridge railing -- The most desirable type of rail for accommodation of flood flow offers the floodwater an opening. An open slender type of bridge railing has a lower backwater and reduced lateral forces than a more solid type. A TxDOT research project was initiated to determine which of the standard TxDOT rails are the most hydraulically efficient. Results from this project are documented in a report, Hydraulic Performance of Bridge Rails (0-5492-1).

Deck Drainage

Effective deck drainage is necessary to minimize the possibilities of vehicular hydroplaning and corrosion of the bridge structure. Generally, it is more difficult to drain bridge decks than approach roadways for several reasons. Deck drainage can be improved by any of the following:

- providing a sufficient gradient to cause the water to flow to inlets or off the ends of the bridge
- avoiding zero gradients and sag vertical curves on bridges
- intercepting all flow from curbed roadways before it reaches the bridge
- using open bridge rails without curbs, where possible.

Currently, there is a trend toward using watertight joints and carrying all deck drainage to the bridge ends for disposal because of changes in environmental regulations.

Deck drains should be located so that water does not drain directly onto the roadway below. (See Ponding Considerations in Chapter 10 and *Bridge Deck Drainage Systems,* FHWA-SA-92-010 (HEC-21) for more information.)

When using downspouts, splash basins should be provided to minimize erosion or tie the downspouts into the storm drain conduit. Drainage should not be allowed to discharge against any part of the structure.

Where practicable, the need to suspend a conduit collection system on the superstructure should be avoided. Collection systems should be designed with cleanouts at all bends, runs as short as practicable, and sufficient gradients provided to minimize problems with debris.

Because of the vulnerability of approach roadway shoulders and foreslopes to erosion from concentrated flow, should be provided sufficient inlet capacity off the bridge ends to intercept flow from the bridge. A closed conduit is often preferable to an open chute down the foreslope because it controls the water in a more positive manner, is aesthetically more pleasing, and is less susceptible to damage by maintenance equipment.

When bridge end drains are not provided with the bridge construction, temporary provisions for protecting the approach fill from erosion should be utilized until permanent measures are installed and functional.

Chapter 10 — Storm Drains

Contents:

Section 1 — Introduction

Overview of Urban Drainage Design

The objective of urban storm drainage is to optimize safe passage of vehicle traffic by collecting stormwater from the roadway, and to convey it safely to an adequate receiving body without undue risk to pedestrian traffic or contributing to damage of adjacent private properties during the design storm event.

The flow of water along a roadway can interfere with or halt highway traffic. The most destructive effects of an inadequate drainage system are damage to surrounding or adjacent properties, deterioration of the roadway components, and hazard or delay to traffic caused by excessive ponding in sags or excessive flow along roadway grades.

Proper drainage of a roadway in an urban region can be more difficult than draining roadways in sparsely settled rural areas for the following reasons:

- heavy traffic and subsequent higher risks
- wide roadway sections
- relatively flat grades, both in longitudinal and transverse directions
- shallow water courses
- absence of side ditches and a presence of concentrated flow
- the potential for costly property damages that may occur from ponding of water or from flow of water through built-up areas
- a roadway section that must carry traffic and act as a channel to carry the water to some disposal point.

These conditions require sound and consistent engineering principles and the use of all available data to achieve an acceptable drainage design.

Overview of Storm Drain Design

Although the design of a storm drain system entails many conventional procedures, the design also requires engineering judgment. The proper design of any storm drainage system requires accumulation of basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design.

The development of a storm drain design requires a trial and error approach:

1. Analyze a tentative storm drain system.
2. Compare the system to design criteria.
3. Evaluate the system economically and physically.
4. Revise the system if necessary.
5. Analyze the revised system.
6. Make the design comparisons again.
7. Repeat the process until a storm drain system has been developed that satisfies the technical function of collecting and disposing of the runoff within budgeted allowances.

The hydraulic designer must establish design parameters and criteria, decide component location and orientation, determine appropriate design tools, and ensure comprehensive documentation.

Of major importance in the design of urban storm drainage facilities is the realization that all storm drainage systems are comprised of two separate and distinct systems, namely the *Primary System* (storm sewer) and the *Secondary System* (roadway).

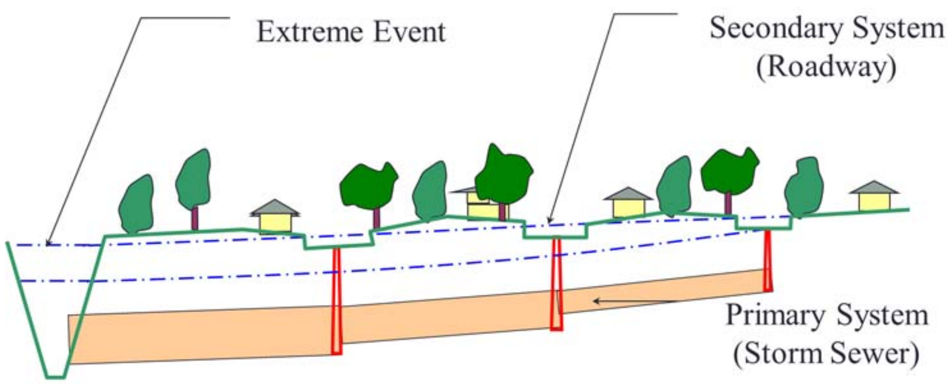


Figure 10-1. Drainage Systems (Storm Sewer and Roadway)

The *primary system* consists of a carefully designed storm drain with capacity to handle runoff for a relatively high frequency (e.g., 5–year event).

The *secondary system* is the conveyance of storm water when the primary system is inoperable or inadequate. This system usually accommodates flow exceeding the primary system's design flow up to the 100–year event.

The lack of a properly designed secondary system often leads to flooding causing severe damage.

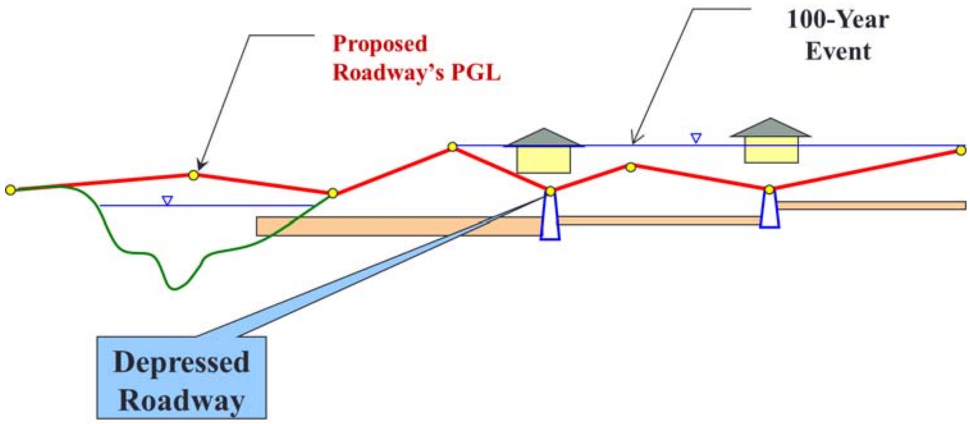


Figure 10-2. Improperly Designed Secondary System

It is economically impractical to enlarge the primary system to avoid the need for the secondary system; but by careful attention to the roadway profile during the initial project planning stage, a secondary system can usually be incorporated at no significant cost.

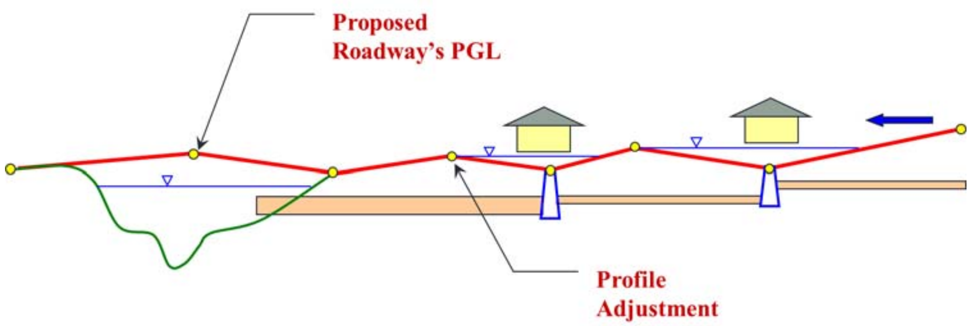


Figure 10-3. Properly Designed Secondary System

For this reason a sheet flow analysis is recommended for every storm drain project. A sheet flow map is an overall map showing all high points, grade breaks, cul-de-sac bulbs and sub-drainage area divides. This map should also include top of curb elevations at all high points. Flow direction arrows must be included. With this map, the designer should consider where the storm runoff would go as ponding levels increase, how deep the water can get at one inlet location before it starts to spill downhill toward the next ponded area, and ultimately, how the sheet flow is going to reach its outlet. With this tool the designer can make adjustments to the roadway's profile and terrain to accommodate the extreme flow event.

Section 2 — Preliminary Concept Development

Design Checklist

1. Identify the problem.
2. Develop a system plan.
3. Establish design criteria including:
   a. design frequency (design AEP),
   b. allowable ponded width,
   c. allowable ponded depth,
   d. pressure or non-pressure flow,
   e. critical elevations,
   f. suitable materials and conduit shapes,
   g. minimum and maximum allowable velocity, and
   h. minimum cover.
4. Develop sheet flow analysis of streets and terrain for existing and proposed conditions.
5. Determine outfall channel flow characteristics.
6. Identify and accommodate utility conflicts.
7. Consider the construction sequence and plan for temporary functioning.
8. Design the system, including determination of runoff, design of inlets and conduits, and analysis of the hydraulic grade line.
9. Check the final design, including check flood, and adjust if necessary.
10. Document the design.

1. Problem Identification

As with any kind of project, the hydraulic designer must first clearly define the problem that the proposed design is going to address. For storm drain design, the goal is to provide adequate drainage for a proposed roadway, while optimizing safety and minimizing potential adverse impacts.

2. System Plan

Preliminary or working drawings featuring the basic components of the intended design are invaluable in the development of a system plan. After design completion, the drawings facilitate documentation of the overall plan.

The following items should be included in the working drawings:

- a general layout
- basic hydrologic data
- pertinent physical features
- characteristics of flow diversion (if applicable)
- detention features (if applicable)
- outfall location and characteristics
- surface features (topography)
- utilities
- tentative component placement.

The final drainage design drawings should include the existing physical features of the project area and indicate the location and type of the following:

- streets
- driveways
- sidewalks and ADA ramps
- parking lots
- bridges
- adjacent areas indicating land use, such as undeveloped land, commercial land, industrial land, agricultural land, residential land, and park land
- detention facilities
- pump stations
- drainage channels
- drainage diversions
- off-site watershed boundaries.

3. Design Criteria

Design criteria includes such elements as design frequency (design AEP), allowable ponded width and allowable ponded depth, pressure or non-pressure flow, critical elevations, suitable materials and conduit shapes, minimum and maximum allowable velocity, and minimum cover.

a. The design AEP for a storm drain system design is based on the general nature of the system and the area it is to serve, the importance of the system and associated roadway, the function of the roadway, the traffic type (emergency/non-emergency), traffic demand, and a realistic assessment of available funds for the project. Chapter 4, Section 6 provides a discussion on design AEP and includes a table of recommended design AEPs.

b. The flow in the gutter should be restricted to a depth and corresponding width that will neither obstruct the roadway nor present a hazard to the motoring public at the design AEP. The depth and width of flow depend on the rate of flow, longitudinal gutter slope, transverse roadway slope, roughness characteristics of the gutter and pavement, and inlet spacing. Section 4, Ponding, provides more discussion on allowable ponded width and depth.

c. The standard practice of the Department is to design for a non-pressure flow network of collector conduits in most storm drain systems.

d. Typical critical elevations are at the throats of inlets and tops of manholes. Should the backwater (hydraulic grade line) within the system rise to a level above the curb and gutter grade, manhole, or any other critical elevation in a storm drain system, the system cannot perform as predicted by the calculations. Water will back out onto the roadway or runoff will be impeded from entering the system. The hydraulic designer must identify the critical elevations where these problems most likely will exist and compare the resultant hydraulic grade line to the system. The hydraulic grade line must not exceed the critical elevation for the design AEP at any point in the system.

e. The choice of material and component shape should be based on careful consideration of durability, hydraulic operation, structural requirements, and availability.

Both the shape and material of storm drain system components influence the system hydraulic capacity. Some shapes are more hydraulically efficient than others. Conduit roughness characteristics vary with conduit material; thus, the hydraulic capacity varies with the material type. For example, reinforced concrete pipe justifies a Manning's n-value of 0.012, while conventional corrugated metal pipe requires the use of an n-value of 0.024 or greater.

All possible storm drain materials should be considered with regard to the local environment of the system site. The durability of a drainage facility depends on the characteristics of soil, water, and air. Because these characteristics may vary from site to site, a rule of thumb to use one material exclusive of all others may not be cost-effective.

Durability of drainage facilities is a function of abrasion and corrosion. Except in some mountainous areas of the state, abrasion is not a serious problem. As a rule, durability does not directly affect the choice of shape. Refer to the Bridge Division for design considerations associated with durability. The roadway project's geotechnical report should be consulted for factors that affect material durability.

When choosing shape and material, the limitations of cover, headroom, and anticipated loading must be considered. Transportation costs are also important, as well as product availability in the geographic area of the project.

f. Flow velocities within a conduit network should be no less than 3 fps and no greater than about 12 fps. At velocities less than 3 fps, sediment deposit becomes a serious maintenance problem. When low velocities cannot be avoided, access for maintenance must be considered. At flow velocities greater than about 12 fps, the momentum of the flow can inflict a damaging impact on the components and joints within the system. When design velocities greater than 12 fps are necessary, countermeasures such as strengthened components and joints should be considered.

g. Both minimum and maximum cover limits for conduit must be considered. Minimum cover limits are established to ensure the conduit's structural stability under live and impact loads. For highway applications, a minimum cover depth of 3.0 ft should be maintained where possible in order to distribute live and impact loads. Where this criterion cannot be met, the conduit should be evaluated to determine if it is structurally capable of supporting the imposed loads.

With increasing fill heights, dead load becomes the controlling factor. Tables addressing maximum height of cover are available from the Bridge Division.

4. Outfall Considerations and Features

The outfall of the storm drain system is a key component which impacts the physical and hydraulic characteristics of the system. The hydraulic designer should consider the requirements and characteristics of the area in which the outfall facility is located. Important considerations in the identification of an appropriate system outfall include the following:

- the availability of the channel and associated right-of-way or easement
- the capacity of the existing or proposed channel or conduit
- the profile of the existing or proposed channel or conduit
- the flow characteristics under flood conditions
- the land use and soil type through the area of the channel.

Whether the outfall is enclosed in a conduit or is an open channel, the hydraulic designer should assess its ability to convey design flows. The hydraulic designer should consider that the outfall may need to be modified to minimize any significant impact to the receiving channel or the surrounding property. Detention upstream of the outfall may be an option to channel modification.

An outfall for a Department storm drain system must be operated for the life of the system. This implies that the Department must have access to all parts of the outfall for purposes of maintenance to ensure adequate operation of the drainage system. In some instances an outfall right-of-way (drainage easement) must be purchased to assure accessibility and that the discharge from the outfall will not be restricted.

Special Outfall Appurtenances

Backflow preventers such as flap gates may be installed when necessary to prevent the outfall tailwater from backing into a storm drain system. However, backflow preventers are also maintenance intensive items which should be avoided if at all possible. The best solution is to design the storm drain system to prevent backflow from causing damage.

5. Utility Conflicts

Hydraulic designers should minimize conflicts with existing utilities and potential conflicts with future utilities. During design, the order of consideration is as follows:

a. Carefully identify each utility and associated appurtenance that may be in conflict with any part of the storm drain system. Consider any utility that intersects, conflicts, or otherwise affects or is affected by the storm drain system. Determine the horizontal and vertical alignments of underground utilities to properly accommodate potential conflicts. The following are typical utilities that may be encountered in an urban situation:

- Electrical, overhead and underground
- telephone or cable television transmission lines, overhead and underground
- water lines
- wastewater lines
- gas lines
- irrigation ditches
- high-pressure fuel facilities
- communication transmission facilities.

b. Where reasonable, avoid utility conflicts.

c. Where utility conflicts cannot be avoided, arrange for the relocation or adjustment of the utility.

d. Make accommodations to the utility when adjustments are not feasible due to economics or other conditions. For example, it may be unreasonable to relocate a high-pressure gas line. In such a case, design an intersection of the unadjusted utility appurtenance and the subject component of the storm drain system. This may involve passing the utility through the storm drain component (e.g., through a junction box) or installing a siphon. The utility company may be on state right-of-way under the agreement that the Department may request utility adjustments. However, as a general objective, attempt to minimize the disruption to utilities within reasonable and feasible design alternatives.

6. Construction

The system must function, perhaps to a lesser extent, during the time of project construction. Storm drain lines should be built from downstream to upstream. For example, when inlets and laterals are built before the trunk line, the stormwater is trapped in the laterals. Therefore, it is important to consider construction sequencing during the design process.

7. System Design

System design includes issues such as determination of runoff, design of inlets and conduits, and analysis of the hydraulic grade line. System design is discussed in detail in the remainder of this chapter.

8. Check Flood

Once the final design is completed, the designer must review the design to assure that all the design criteria in Step 3 are still being met.

The intent of a check flood is to verify that the system will not experience problems for a frequency higher than the design AEP. Department policy is to use the 100-year frequency (1% AEP) as the check flood. This check is more than simply running the storm drain analysis with all the flow forced into the system; it is considering what will happen to excess flow which can't go into the inlets. Will it safely flow down the gutter to the outfall, or will it flow down a driveway and flood a structure? Are contingencies made for these possibilities? These are only a few of the circumstances which may be found on a project.

9. Documentation Requirements

The storm drain documentation requirements are presented in Chapter 3, Section 5 of this manual.

Section 3 — Runoff

Hydrologic Considerations for Storm Drain Systems

As inlet locations are established, the hydraulic designer can indicate intermediate drainage boundaries, and must show schematically or otherwise describe contributing watersheds. See Chapter 3 for discussion of preliminary design activities, and see Chapter 4 for hydrologic considerations.

Flow Diversions

A storm drain system should mimic the natural drainage pattern. Diversion of flow from one watershed to another should be avoided. When diversion is unavoidable, the impacts of the flow diversion must be considered. DES-HYD should be involved in the design.

Detention

Detention does not change the total runoff volume; however, detention does change the rate of discharge, depending on the characteristics of the runoff and the detention facility. Such facilities may be in the form of holding reservoirs, large borrow ditches, or underground storage.

In the past, detention was not typically incorporated into the design because the Department's policy was to remove and dispose of runoff as quickly and effectively as possible. However, with increased development in Texas, runoff rates and volumes have increased, causing the need for larger and more costly drainage structures. The greater rates and quantities of runoff may also damage downstream development.

A detention facility may decrease facility costs or diminish potential damages due to the increased runoff rates and volumes. With this intent, many municipalities, counties, and other entities in Texas have begun to require detention as an integral part of drainage design. While not specifically required to comply with municipal and county requirements, hydraulic designers for the Department should consider the need for detention in the design to avoid the risk of damage to adjacent properties. A detention system may also be necessary for water quality control.

Determination of Runoff

The first step in designing a storm drain system is to determine the peak runoff flow. The Rational Method, discussed in Chapter 4, is the method that applies to the vast majority of watersheds for storm drains.

The time of concentration in a storm drainage design is comprised of the time required for water to flow from the most distant point of the drainage area to the inlet (t_c, also called inlet time) and the travel time (t_t)as the water flows through the storm drain line under consideration (travel time through a conduit). See Time of Concentration in Chapter 4 for more information. The hydraulic designer must be careful to document the actual inlet time and travel time for each segment, because the total time is summed through the system and used for sizing the conduit. The temptation to use a default t_c of 10 minutes for every drainage area usually results in the flowrate in the conduit being underestimated.

Other Hydrologic Methods

On occasion, a special hydrologic method may need to be used. For example, if a city is funding the surface drainage facilities, that city may insist on using its own specific hydrologic method. Usually, such special methods are similar to the Rational Method with some minor variations.

Some situations may require the use of some variation of Natural Resources Conservation Service (NRCS) hydrologic estimating methods such as the NRCS TR-55 or TR-20 procedures. (See References for information on contacting this agency.) In other situations, the use of a unit hydrograph procedure may be in order. Refer to Hydrograph Methods in Chapter 4 for detailed information on the NRCS methods.

Where considerable storage is required in the storm drain system or detention is being designed, hydrologic routing methods should be employed to accommodate peak flow attenuation. Refer to Chapter 4 for information on Hydrograph Methods.

Section 4 — Pavement Drainage

Design Objectives

The objective of urban storm drainage is to provide safe passage of vehicle traffic by collecting stormwater from the roadway, and to convey it safely to an adequate receiving body while preventing damage to adjacent private properties or undue risk to pedestrian traffic during the design storm event.

Appropriate longitudinal and transverse slopes can serve to move water off the travel way to minimize ponding, sheet flow, and low crossovers. This means that the hydraulic designer must work with the roadway geometric designer to assure efficient drainage in accordance with the geometric and pavement design.

Ponding

The flow of water in the gutter should be restricted to a depth and corresponding width that will neither obstruct the roadway nor present a hazard to the motoring public at the design AEP. These restrictions are referred to as allowable depth and allowable ponded width. The depth and width of flow depend on the following:

- rate of flow
- longitudinal gutter slope
- transverse roadway slope
- roughness characteristics of the gutter and pavement
- inlet spacing.

Depth of flow should not exceed the curb height.

Ponded widths are limited to the following minimum acceptable standards for Department roadway design:

- Limit ponding to one-half the width of the outer lane for the main lanes of interstate and controlled access highways.
- Limit ponding to the width of the outer lane for major highways, which are highways with two or more lanes in each direction.
- Limit ponding to a width and depth that will allow the safe passage of one lane of traffic per direction for minor highways.

Inlets should be placed at all low points in the roadway and at suitable intervals along extended slopes as necessary to prevent excessive flow in the gutter or ponding on the roadway. An economical design uses a minimum number of inlets by allowing the ponded width and depth to approach the allowable limits. In instances such as a narrow shoulders or low grades, there may need to be a continuous removal of flow from the surface.

Longitudinal Slopes

Longitudinal gutter slopes should usually be not less than 0.3% for curbed pavements, although this minimum may be difficult to maintain in some locations. In such situations, a rolling (sawtooth) gutter profile may be necessary. The roadway designer may need to warp the longitudinal slope to achieve a rolling gutter profile as shown in Figure 10-1. Extremely long sag-vertical curves in the curb and gutter profile are discouraged because they incorporate relatively long, flat grades at the sag which tend to distribute runoff across the roadway surface instead of concentrating flow within a manageable area.

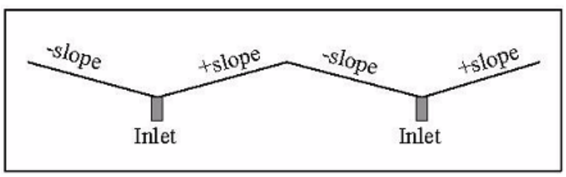


Figure 10-4. Rolling Gutter Profile

Transverse (Cross) Slopes

A steep cross slope provides for proper drainage, while flat cross slopes are amenable to driver safety and comfort. Except in cases of superelevated sections, the cross slope is usually a compromise between the two requirements. The Roadway Design Manual should be consulted for guidance on pavement cross slopes.

Drainage on multi-lane roadways can be enhanced by increasing the cross slope on the outer lanes, or by increasing the cross slope on each successive lane pair. Refer to the Roadway Design Manual for guidance. Drainage on very wide multi-lane roadways such as urban interstates may require special considerations such as porous pavements or transverse trench drains. DES-HYD should be consulted when these considerations arise.

Superelevated transitions should be carefully designed to minimize the extent of flat areas. Flat slopes should not be located in the sag of a vertical curve. It is usually these transition regions where small, shallow ponds of accumulated water, or “birdbaths,” occur. Aggressive drainage techniques such as porous pavement, rough texture, or additional drains must be used to minimize ponding in these “birdbaths.”

Hydroplaning

Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is exceeded; water builds up in front of the tire and creates a water wedge which can lift the tire off the pavement, thus reducing the tire/pavement friction to zero. Hydroplaning is a function of the water depth, roadway geometrics, vehicle speed, tread depth, tire inflation pressure, and conditions of the pavement surface, so it is difficult to calculate the exact conditions where hydroplaning will occur. The potential for hydroplaning increases as the depth of water over the roadway increases. Hydroplaning can occur at 55 mph with as little as 0.08 inches (2mm) of water.

Because the factors that influence hydroplaning are generally beyond the designer's control, hydroplaning is impossible to prevent. However, the physical characteristics that may influence hydroplaning can be minimized with the following considerations:

- Proper transverse slopes reduce the amount of water flowing over the pavement and prevent excessive ponding. The longitudinal slope is somewhat less influential in decreasing the potential for hydroplaning.
- Conscientious placement of inlets reduces or eliminates water flowing over the pavement and reduces excessive ponding. Transverse drains should not be used without serious consideration for small wheeled vehicles.
- Permeable surface courses and high macrotexture surface courses influence both water film thickness and the interaction of tires with the water film.
- Grooving may be a corrective measure for severe localized hydroplaning problems. Transverse grooving (perpendicular to the direction of traffic) produces better results that longitudinal grooving (parallel to the direction of traffic). In addition, longitudinal grooving has the potential to retard flow off the roadway.

The potential for hydroplaning can be evaluated using an empirical equation based on studies conducted for the FHWA publication “Bridge Deck Drainage Guidelines” (HEC-21).

Use of Rough Pavement Texture

The potential for hydroplaning may be minimized to some extent if the pavement has a rough texture. A very rough pavement texture benefits inlet interception. However, in a contradictory sense, very rough pavement texture is unfavorable because it causes a wider spread of water in the gutter. Rough pavement texture also inhibits runoff from the pavement.

Cross cutting (grooving) of the pavement is useful for removing small amounts of water such as in a light drizzle. The Department discourages longitudinal grooving because it usually causes problems in vehicle handling and tends to impede runoff from moving toward the curb and gutter.

Section 5 — Storm Drain Inlets

Inlet Types

Inlets used for the drainage of highway surfaces into four major classes:

- Curb opening inlets - See Figure 10-2.
- Grate inlets - See Figure 10-4.
- Linear drains - May be slotted drains (Figure 10-5) or trench drains (Figure 10-6).
- Combination inlets -- Combination inlets usually consist of some combination of a curb-opening inlet and a grate inlet. In a curb and grate combination, the curb opening may extend upstream of the grate.

Curb Opening Inlets

Figure 10-2 illustrates a generic example of a typical curb opening inlet. Curb inlets are used in urban sections of roadway along the curb line.

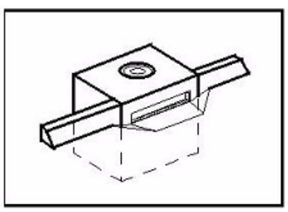


Figure 10-5. Curb Opening Inlet

Most curb opening inlets depend heavily upon an adjacent depression in the gutter for effective flow interception (see Figure 10-3). Greater interception rates result in shorter (and probably, more economical) inlet lengths. However, a large gutter depression can be unsafe for traffic flow and bicycle operation near the gutter line. Therefore, a compromise is in order when selecting an appropriate value for the gutter depression. The depth of the gutter depression should be:

- 0 to 1 in. (0 to 25 mm) where the gutter is within the traffic lane
- 1 to 3 in. (25 to 75 mm) where the gutter is outside the traffic lane or in the parking lane
- 1 to 5 in. (25 to 125 mm) for lightly traveled city streets that are not on a highway route.

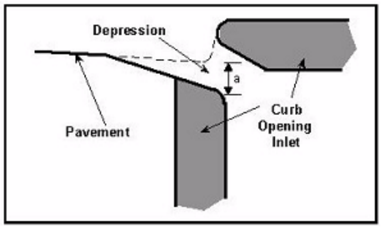


Figure 10-6. Curb Opening Inlet Depression

Some municipalities in the state prefer to recess curb inlets with significant depression to minimize interference with traffic flow. The inlet is recessed from the line of the curb and gutter such that the depression does not extend beyond the gutter line. This may improve driveability; however, the curb transition may pose a hazard to traffic.

Curb opening inlets are useful in sag and on-grade situations because of their self-cleansing abilities and hydraulic efficiency. Additionally, they are often preferred over grate inlets because the inlet is placed outside the travel way and poses less of a risk to motorists and bicycle traffic.

A drawback of curb opening inlets is that the flowline of the opening is fixed and not readily adaptable to changing pavement levels as occur in surface treatment overlays. Successive overlays can gradually reduce or even eliminate the original opening available for water removal, unless the pavement edge is tapered to the original gutter line.

Grate Inlets

Figure 10-4 illustrates a typical grate inlet. Water falls into the inlet through a grate instead of an opening in the curb. Designers use many variations of this inlet type, and the format of the grate itself varies widely as each foundry may have its own series of standard fabrication molds.

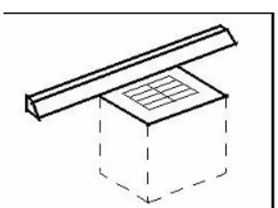


Figure 10-7. Grate Inlet Schematic

For the most part, grate inlets are used in sag configurations in gutters, adjacent to concrete traffic barriers or rails (where curb inlets would not be practicable), V-shaped gutters with no curb or barrier, and ditches. Grate inlets may also be used at on-grade situations combined with curb inlets.

Grate inlets adapt to urban roadway features such as driveways, street intersections, and medians. When grate inlets are specified, the grate configuration and orientation should be compatible with bicycle and wheelchair safety.

Access to the storm drain system through a grate inlet is excellent in that, usually, the grate is removable. On the other hand, maintenance of grate inlets can be a continuing problem during the life of the facility; the propensity to collect debris makes grate inlets a constant object of maintenance attention. As such debris accumulates it obstructs the flow of surface water into the inlet. Grate inlets also present potential interference with bicycles and wheelchairs.

Linear Drains

Linear drains were designed for the interception of wide spread, low flow situations. Applications include intercepting sheet flow from the roadway when collection at a concentration point is not practical, or providing a generalized inlet for stagnate flow on pavements without slope. Linear drains have the advantage that no depression is necessary for hydraulic efficiency.

Linear drains may be useful in problematic areas where curb and grate inlets are ineffective, such as along a median barrier, and at super elevation transitions. Linear drains can also be installed transversely across the roadway.

However, linear drains have several drawbacks:

- Linear drains have a high propensity to collect debris in sag configurations. Regular maintenance is required to clear debris from over the drain inlet.
- Installation can be difficult. Because the inlet is installed flush with the surface of the pavement, placement is critical.

Slotted Drains

Slotted drains consist of a corrugated pipe with an extended slot, or throat, at the top (see Figure 10-5). The throat of a slotted drain inlet is ordinarily reinforced for structural integrity.

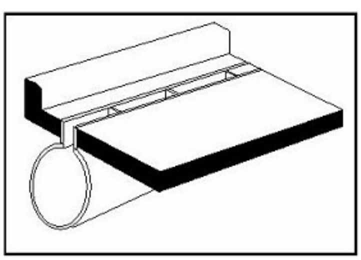


Figure 10-8. Slotted Drain Inlet

Slotted drains should be installed with sufficient slope to provide a cleaning velocity for the corrugations. If not, regular cleaning and maintenance must be scheduled on the slotted drain. Clean out access boxes are usually needed at the far end of each slotted drain run to facilitate regular maintenance and cleaning.

Trench Drains

Trench drains may be precast or cast-in-place. Figure 10-6 illustrates the body and grate of an installed precast trench drain.

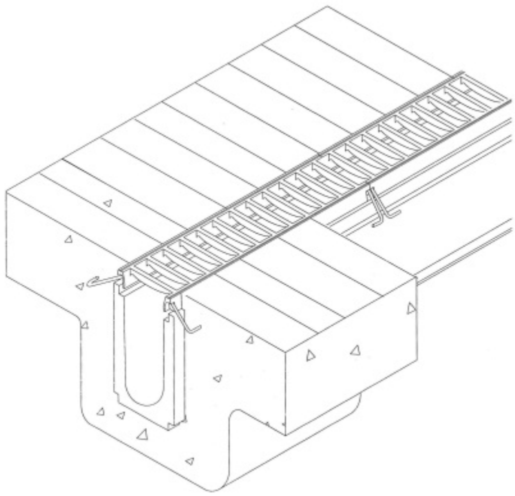


Figure 10-9. Precast trench drain.

Trench drains have the advantage of a shallower embedment depth and an extremely smooth invert that doesn't retain sediments, but the disadvantage of having a limited volume because of their size.

Trench drains on Department roadways are required to have non-removable grates because removable grates may come loose or move which will create a traffic hazard. The grates specified by the Department have a minimum of 60% open space which allows for cleaning with a vacuum truck or water truck without removing the grates.

Combination Inlets

Combination inlets such as curb and grate can be useful in many configurations, especially sag locations. Because of the inherent debris problem in sags, the combination inlet offers an overflow drain if part of the inlet becomes completely or severely clogged by debris. Maintenance of combination inlets is usually facilitated by the fact that the grate is removable, providing easy access to the inlet and associated storm drain system.

Combination inlets used on-grade generally are not cost-effective because of the relatively small additional hydraulic capacity afforded. Authentic data on such combinations are insufficient to establish accurate factors for determining the true capacity of a combination inlet.

For a combination curb and grate, assume that the capacity of the combination inlet comprises the sum of the capacity of the grate and the upstream curb opening length. Ignore the capacity of the curb opening that is adjacent to the grate opening.

Inlets in Sag Configurations

An inlet in a sag configuration is considered the "end of the line" because the water and its debris load have no other place to go. Because of this, failure of an inlet in a sag configuration often represents a threat to the successful operation of a storm drain system. Therefore, the hydraulic designer must consider some additional items such as complex ponded width and complex approach slopes.

In a sag configuration, the controlling ponded width can be from one of two origins. The inlet itself may cause a head that translates to a ponded width, and the flow in the curb and gutter from each direction subtends its own ponded width.

If the sag is in a vertical curve, the slope at the sag is zero, which would mean that there is no gutter capacity. In reality there is a three-dimensional flow pattern resulting from the drawdown effect of the inlet. As an approximation, one reasonable approach is to assume a longitudinal slope of one half of the tangent grade.

Because the water or its debris load can go no other place, an appropriate safety factor should be applied to the inlet size. For grate inlets in sags, the usual safety factor is approximately two; for curb inlets, the factor can be somewhat less. In application, the factor of safety for a grate inlet is applied as a safety reduction factor, or clogging factor. For example, a safety factor of 2 would result in a clogging factor of 50%, which assumes that half the grate is clogged by debris.

Where significant ponding can occur such as in an underpass and in a sag-vertical curve, good engineering practice is to place flanking inlets on each side of the sag location inlet to relieve some or most of the flow burden on the inlet in sag. Flanking inlets should be analyzed as inlets on-grade at some specified distance (usually 50 or 100 feet) away from the low point on the sag vertical curve.

Median/Ditch Drains

Drains or inlets appearing in ditches and medians are usually grate inlets and are also termed "drop inlets." The operation of the inlet is enhanced by a concrete riprap collar that forms a type of bowl around the inlet.

Often, such an inlet is in a sag (sump) configuration created by a ditch block. Department research (0-5823-1) indicates that a six inch ditch block may not be adequate to ensure complete capture of flow; a taller ditch block may be necessary. However, the designer must either determine the required depth for complete capture, or account for flow over the ditch block to the next inlet. Care must be taken to ensure that the ditch block slopes meet clear zone standards.

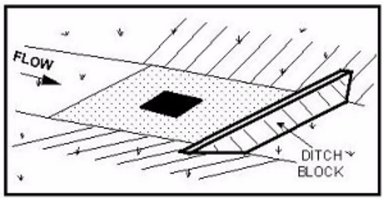


Figure 10-10. Median/Ditch Inlet

Drainage Chutes

Drainage chutes, also referred to as over-side drains or curb slots, are commonly used at the ends of bridges to either prevent flow from running onto a bridge deck, or to prevent flow from running off a bridge deck onto the pavement. There may be other locations where a drainage chute or curb cut may be useful in removing flow from the pavement where no storm drainage system is present and where suitable outfall is present behind the curb and gutter section. In most cases, an opening in the curb connects to a scour-resistant channel or chute which prevents erosion of the embankment or slope. In some instances, the channel or chute may be replaced with a pipe placed in the roadway embankment as illustrated in Figure 10-8. This treatment facilitates mowing and other maintenance but also introduces its own maintenance needs to keep it from clogging.

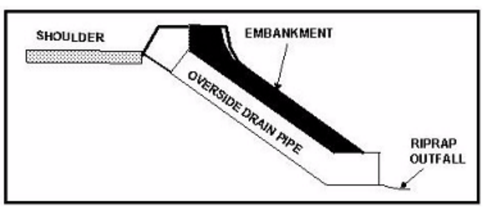


Figure 10-11. Over-Side Drains

Inlet Locations

The inlet location may be dictated either by roadway elements, hydraulic requirements, or both. Inlets should be placed upstream of roadway elements such as sags, street intersections, gore islands (see Figure 10-9), super elevation transitions, driveways, cross-walks, and curb ramps (see Bypass Flow Design Approach). Flow across intersections, ramps, and to a lesser extent, drive-ways, may cause a traffic hazard, while flow across cross-walks and curb ramps may cause a pedestrian hazard. Inlets at these locations should be designed to capture 100% of the flow. Inlets should also be located hydraulically to prevent excessive gutter flow and excessive ponding.

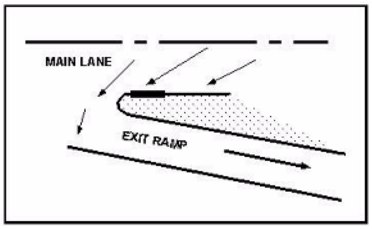


Figure 10-12. Inlet at a Gore Island

Section 6 — Gutter and Inlet Equations

Gutter Flow

The ponded width is a geometric function of the depth of the water (y) in the curb and gutter section. The spread is usually referred to as ponded width (T), as shown in Figure 10-10.

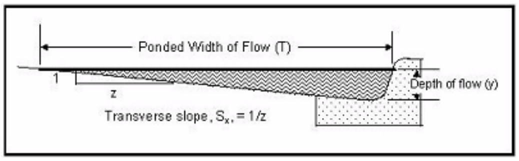


Figure 10-13. Gutter Flow Cross Section Definition of Terms

Using Manning's Equation for Depth of Flow as a basis, the depth of flow in a curb and gutter section with a longitudinal slope (S) is taken as the uniform (normal) depth of flow. (See Chapter 6 for more information.) For Equation 10-1, the portion of wetted perimeter represented by the vertical (or near-vertical) face of the curb is ignored. This justifiable expedient does not appreciably alter the resulting estimate of depth of flow in the curb and gutter section.

$$y = z\left(\frac{QnS_x}{S^{1/2}}\right)^{3/8}$$

Equation 10-1.

where:

y = depth of water in the curb and gutter cross section (ft. or m)

Q = gutter flow rate (cfs or m^3/s)

n = Manning’s roughness coefficient

S = longitudinal slope (ft./ft. or m/m)

S_x = pavement cross slope = 1/x (ft./ft. or m/m)

z = 1.24 for English measurements or 1.443 for metric.

The table below presents suggested Manning's “n” values for various pavement surfaces. Department recommendation for design is the use of the rough texture values.

Table 10-1 Manning’s n-Values for Street and Pavement Gutters

Type of gutter or pavement	**n**	
asphalt pavement:	Smooth texture	Rough texture
	0.013	0.016

Table 10-1 Manning's n-Values for Street and Pavement Gutters

Type of gutter or pavement	n	
Concrete gutter with asphalt pavement:	Smooth texture	Rough texture
	0.013	0.015
Concrete pavement:	Float finish	Broom finish
	0.014	0.016

Refer to Figure 10-10, and translate the depth of flow to a ponded width on the basis of similar triangles using Equation 10-2. Equation 10-2 can also be used to determine the ponded width in a sag configuration, where "y" is the depth of standing water or head on the inlet.

$$T = \frac{y}{S_x}$$

Equation 10-2.

where:

T = ponded width (ft. or m).

Equations 10-1 and 10-2 are combined to compute the gutter capacity.

$$Q = \frac{z}{n} S_x^{5/3} S^{1/2} T^{8/3}$$

Equation 10-3.

where:

z = 0.56 for English measurements or 0.377 for metric.

Rearranging Equation 10-3 gives a solution for the ponded width, "T".

$$T = z\left(\frac{Qn}{S_x^{5/3} S^{1/2}}\right)^{3/8}$$

Equation 10-4.

where:

z = 1.24 for English measurements or 1.443 for metric.

Equations 10-3 and 10-4 apply to roadway sections having constant cross slope and a vertical curb. The FHWA publication "*Urban Drainage Design Manual*" (HEC-22) should be consulted for parabolic and other shape roadway sections.

Ponding on Continuous Grades

Excessive ponding on continuous grades can be avoided by proper placement of storm drain inlets. The gutter ponding at a specific location may be determined on a continuous grade using the following steps:

1. Select a location of a proposed inlet. Start on the high point and work towards the low point. Take into consideration the discussion in Inlet Locations.
2. Calculate the total discharge in the gutter based on the drainage area to the selected location. See Determination of Runoff for methods to calculate discharge.
3. Determine the longitudinal slope, transverse slope and Manning's roughness coefficient of the gutter.
4. Compute the ponded depth and width. For a constant transverse slope, compute the ponded depth using Equation 10-1 and the ponded width using Equation 10-2. For parabolic gutters or sections with more than one transverse slope, refer to the FHWA publication "*Urban Drainage Design Manual*" (HEC-22).

Ponding at Approaches to Sag Locations

Because a sag location has a different amount of flow approaching from both directions, the gutter to each side of the inlet has a different ponded width and depth. At sag locations, the hydraulic designer must consider sag inlet capacity and allowable ponding using the following steps:

1. Estimate the apportionment of runoff to the left and right approaches. Next compute the discharge to the sag location based on the entire drainage area. Then determine the approximate fraction of area contributing to each side of the sag location. Multiply the total discharge by each fraction to determine the discharge to each side.
2. Determine the longitudinal slope of each gutter approach. For sawtooth profiles, the slopes will be the profile grades of the left and right approaches. However, if the sag is in a vertical curve, the slope at the sag will be zero, which would mean no gutter capacity. In reality there is a three-dimensional flow pattern resulting from the drawdown effect of the inlet. As an approximation, assume a longitudinal slope of one half of the tangent grade.
3. For each side of the sag, calculate the ponded depth and width using the appropriate flow apportionment, longitudinal slope, and Equation 10-1. Compute the ponded width using Equation 10-2.

Ponded Width Confirmation

Figure 10-11 shows the interdependence of inlet location, drainage area, discharge, and ponded width. A tentative inlet location is selected, drainage area determined for that location, discharge established, and ponded width calculated. Once the ponded width has been calculated for a tentative location, it must be compared to the allowable ponded width and depth.

If the ponded width is exceeded, the design must be adjusted by relocating the inlet to a point upstream in the curb and gutter section which will reduce the watershed area, the peak discharge, and thus the ponded width. This process is repeated until the ponded width is at or below the allowable ponded width.

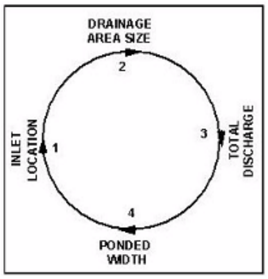


Figure 10-14. Relation of Inlet Location to Design Discharge

If the calculated ponded width is less than or equal to the allowable ponded width, the hydraulic designer must decide if the design is efficient. If all or most of the allowable ponded width is used, the location is probably efficient. If only a small portion of the allowable ponded width is used, a more efficient location may be possible. In extensive storm drain systems, a design objective should be to minimize the number of inlets without violating allowable ponded widths anywhere in the system. (See Ponding for guidelines on allowable ponding.)

Carryover Design Approach

An on-grade inlet may be much more efficient if it intercepts only a portion of the total flow in the gutter instead of all of the flow in the gutter. The gutter flow not intercepted is called bypass flow or carryover. This design approach can only be used for on-grade configurations and is recommended where interception of the total flow is not necessary.

Figure 10-12 illustrates (in profile) approximately what happens when the inlet is designed to intercept all of the approaching flow. Note the large area of inlet opening that is not utilized efficiently.

Figure 10-13 illustrates (in profile) approximately what happens when the inlet is designed for bypass flow. Note that the inlet opening is used much more efficiently than the inlet illustrated in Figure 10-12.

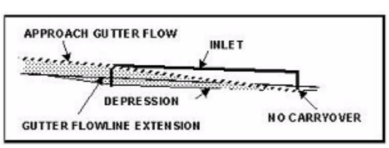


Figure 10-15. Inlet Designed with No Carryover

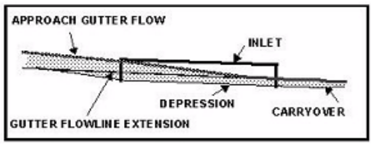


Figure 10-16. Inlet Designed with Carryover

Bypass flow is normally captured at some other location. The gutter between the two points must accommodate the additional flow. Bypass flow is not recommended upstream of intersections and driveways, at superelevation transitions where the cross slope begins to reverse, or below entrance/exit ramps. Bypass flow at these locations would be crossed by vehicular traffic and may pose a traffic hazard. Bypass flow is also not recommended to be allowed to flow where there is no outfall or designated capture point.

Curb Inlets On-Grade

The design of on-grade curb opening inlets involves determination of length required for total flow interception, subjective decision about actual length to be provided, and determination of any resulting carryover rate.

The following procedure is used to design curb inlets on-grade:

1. Compute depth of flow and ponded width (T) in the gutter section at the inlet.
2. Determine the ratio of the width of flow in the depressed section (W) to the width of total gutter flow (T) using Equation 10-5. Figure 10-14 shows the gutter cross section at an inlet.

$$E_0 = \frac{K_W}{K_W + K_0}$$

Equation 10-5.

where:

E_0 = ratio of depression flow to total flow

K_W = conveyance of the depressed gutter section (cfs or m^3/s)

K_0 = conveyance of the gutter section beyond the depression (cfs or m^3/s).

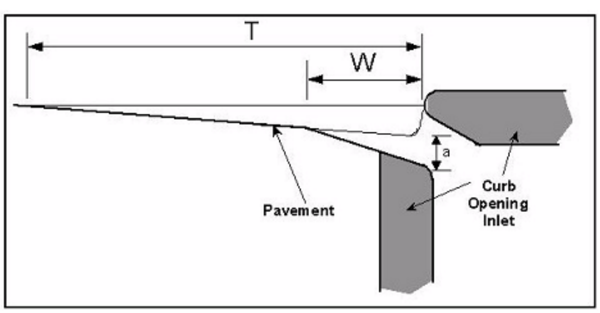


Figure 10-17. Gutter Cross-Section Diagram

Use Equation 10-6 to calculate conveyance, K_W and K_0.

$$K = \frac{zA^{5/3}}{nP^{2/3}}$$

Equation 10-6.

where:

K = conveyance of cross section (cfs or m^3/s)

z = 1.486 for English measurements and 1.0 for metric

A = area of cross section (sq.ft. or m^2)

n = Manning's roughness coefficient

P = wetted perimeter (ft. or m).

Use Equation 10-7 to calculate the area of cross section in the depressed gutter section.

$$A_W = WS_x\left(T - \frac{W}{2}\right) + \frac{1}{2}aW$$

Equation 10-7.

where:

A_W = area of depressed gutter section (ft^2 or m^2)

W = depression width for an on-grade curb inlet (ft. or m)

S_x = cross slope (ft./ft. or m/m)

T = calculated ponded width (ft. or m)

a = curb opening depression depth (ft. or m).

Use Equation 10-8 to calculate the wetted perimeter in the depressed gutter section.

$$P_W = \sqrt{(WS_x + a)^2 + W^2}$$

Equation 10-8.

where:

P_W = wetted perimeter of depressed gutter section (ft. or m)

W = depression width for an on-grade curb inlet (ft. or m)

S_x = cross slope (ft./ft. or m/m)

a = curb opening depression depth (ft. or m).

Use Equation 10-9 to calculate the area of cross section of the gutter section beyond the depression.

$$A_0 = \frac{S_x}{2}(T - W)^2$$

Equation 10-9.

where:

A_0 = area of gutter/road section beyond the depression width (ft^2 or m^2)

S_x = cross slope (ft./ft. or m/m)

W = depression width for an on-grade curb inlet (ft. or m)

T = calculated ponded width

Use Equation 10-10 to calculate the wetted perimeter of the gutter section beyond the depression.

$$P_0 = T - W$$

Equation 10-10.

where:

P_0 = wetted perimeter of the depressed gutter section (ft. or m)

T = calculated ponded width (ft. or m)

W = depression width for an on-grade curb inlet (ft. or m).

3. Use Equation 10-11 to determine the equivalent cross slope (S_e) for a depressed curb opening inlet.

$$S_e = S_x + \frac{a}{W}E_o$$

Equation 10-11.

where:

S_e = equivalent cross slope (ft./ft. or m/m)

S_x = cross slope of the road (ft./ft. or m/m)

a = gutter depression depth (ft. or m)

W = gutter depression width (ft. or m)

E_o = ratio of depression flow to total flow.

4. Calculate the length of curb inlet required for total interception using Equation 10-12.

$$L_r = zQ^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6}$$

Equation 10-12.

where:

L_r = length of curb inlet required (ft. or m)

z = 0.6 for English measurement and 0.82 for metric

Q = flow rate in gutter (cfs or m^3/s)

S = longitudinal slope (ft./ft. or m/m)

n = Manning's roughness coefficient

S_e = equivalent cross slope (ft./ft. or m/m).

If no bypass flow is allowed, the inlet length is assigned a nominal dimension of at least L_r, which should be an available (nominal) standard curb opening length. The exact value of L_r should not be used if doing so requires special details, special drawings, structural design, and costly construction.

If bypass flow is allowed, the inlet length is rounded down to the next available standard (nominal) curb opening length.

5. Determine bypass flow. In bypass flow computations, efficiency of flow interception varies with the ratio of actual length of curb opening inlet supplied (L_a) to required length (L_r) and with the depression to depth of flow ratio. Use Equation 10-13 to calculate bypass flow.

$$Q_{co} = Q\left(1 - \frac{L_a}{L_r}\right)^{1.8}$$

Equation 10-13.

where:

Q_{co} = carryover discharge (cfs or m^3/s)

Q = total discharge (cfs or m^3/s)

L_a = design length of the curb opening inlet (ft. or m)

L_r = length of curb opening inlet required to intercept the total flow (ft. or m).

Bypass flows usually should not exceed about 0.5 cfs (0.03 m^3/s). Greater rates can be troublesome and cause a significant departure from the principles of the Rational Method application. In all cases, the bypass flow must be accommodated at some other specified point in the storm drain system.

6. Calculate the intercepted flow as the original discharge in the approach curb and gutter minus the amount of bypass flow.

Curb Inlets in Sag Configuration

The capacity of a curb inlet in a sag depends on the water depth at the curb opening 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 and the capacity should be based on the lesser of the computed weir and orifice capacity. Generally, for Department design, this ratio should be less than 1.4 such that the inlet operates as a weir.

1. If the depth of flow in the gutter (y) is less than or equal to 1.4 times the inlet opening height (h), ($y \le 1.4h$), determine the length of inlet required considering weir control. Otherwise, skip this step. Calculate the capacity of the inlet when operating under weir conditions with Equation 10-14.

$$Q = C_W(L + 1.8W)y^{1.5}$$

Equation 10-14.

Equation 10-14 is rearranged to produce the following relation for curb inlet length required.

$$L = \frac{Q}{C_W y^{1.5}} - 1.8W$$

Equation 10-15.

where:

Q = total flow reaching inlet (cfs or m^3/s)

C_w = weir coefficient (ft.$^{0.5}$/s or m$^{0.5}$/s)

Suggested value = 2.3 ft.$^{0.5}$/s or 1.27 m$^{0.5}$/s for depressed inlets

Suggested value = 3.0 ft$^{0.5}$/s or 1.60 m$^{0.5}$ without depression

y = head at inlet opening (ft. or m), computed with Equation 10-1

L = length of curb inlet opening (ft. or m)

W = gutter depression width (perpendicular to curb)

If L > 12 ft. (3.6m), then W = 0 and C_W = 3.0 ft.$^{0.5}$/s or 1.60 m$^{0.5}$/s.

2. If the depth of flow in the gutter is greater than the inlet opening height (y > h), determine the length of inlet required considering orifice control. The equation for interception capacity of a curb opening operating as an orifice follows:

$$Q = C_o h L \sqrt{2gd_o}$$

Equation 10-16.

where:

Q = total flow reaching inlet (cfs or m^3/s)

C_o = orifice coefficient = 0.67

h = depth of opening (ft. or m)(this depth will vary slightly with the inlet detail used)

L = length of curb opening inlet (ft. or m)

g = acceleration due to gravity = 32.2 ft./s^2 or 9.81 m/s^2

d_o = effective head at the centroid of the orifice (ft. or m).

For curb inlets with an inclined throat such as Type C inlet, the effective head, d_o, is at the centroid of the orifice. This changes Equation 10-16 to:

$$Q = C_o hL\sqrt{2g\left(y + a - \frac{h}{2}\sin\theta\right)}$$

Equation 10-17.

where:
Q = total flow reaching inlet (cfs or m^3/s)
C_o = orifice coefficient = 0.67
h = depth of opening (ft. or m)(this depth will vary slightly with the inlet detail used)
L = length of curb opening inlet (ft. or m)
g = acceleration due to gravity = 32.2 ft/s^2 or 9.81 m/s^2
y = depth of water in the curb and gutter cross section (ft. or m)
a = gutter depression depth (ft.).

Rearranging Equation 10-17 allows a direct solution for required length.

$$L = \frac{Q}{C_o h\sqrt{2g\left(y + a - \frac{h}{2}\sin\theta\right)}}$$

Equation 10-18.

3. If both steps 1 and 2 were performed (i.e., $h < d \le 1.4h$), choose the larger of the two computed lengths as being the required length.
4. Select a standard inlet length that is greater than the required length.

Slotted Drain Inlet Design

The following procedure may be used for on-grade slotted drain inlets:

1. Determine the length of slotted drain inlet required for interception of all of the water in the curb and gutter calculated by Equation 10-19.

$$L_r = \frac{zQ_a^{0.442} S^E S_x^{-0.849}}{n^{0.384}}$$

Equation 10-19.

where:

L_r = length of slotted drain inlet required for total interception of flow (ft. or m)

z = 0.706 for English measurement or 1.04 for metric

Q_a = total discharge (cfs or m^3/s)

S = gutter longitudinal slope (ft./ft. or m/m)

E = function of S and S_x as determined by Equation 10-20

S_x = transverse slope (ft./ft. or m/m)

n = Manning's roughness coefficient.

Equation 10-19 is limited to the following ranges of variables:

total discharge $\leq$ 5.5 cfs (0.156 m^3/s)

longitudinal gutter slope $\leq$ 0.09 ft./ft. (0.09 m/m)

roughness coefficient (n) in the curb and gutter: $0.011 \leq n \leq 0.017$.

The longitudinal slope exponent (E) is determined with Equation 10-20:

$$E = 0.207 - 19.084S^2 + 2.613S - 0.0001S_x^{-2} + 0.007S_x^{-1} - 0.049SS_x^{-1}$$

Equation 10-20.

Because the equations are empirical, extrapolation is not recommended.

2. Select the desired design slotted drain length (L_a) based on standard inlet sizes. If $L_a < L_r$ the interception capacity may be estimated using Figure 10-15, multiplying the resulting discharge ratios by the total discharge. Alternatively, the carryover for a slotted drain inlet length may be directly computed using Equation 10-21.

$$Q_{co} = 0.918Q\left(1 - \frac{L_a}{L_r}\right)^{1.769}$$

Equation 10-21.

where:

Q_{co} = carryover discharge (cfs or m^3/s)

Q = total discharge (cfs or m^3/s)

L_a = design length of slotted drain inlet (ft. or m)

L_r = length of slotted drain inlet required to intercept the total flow (ft. or m).

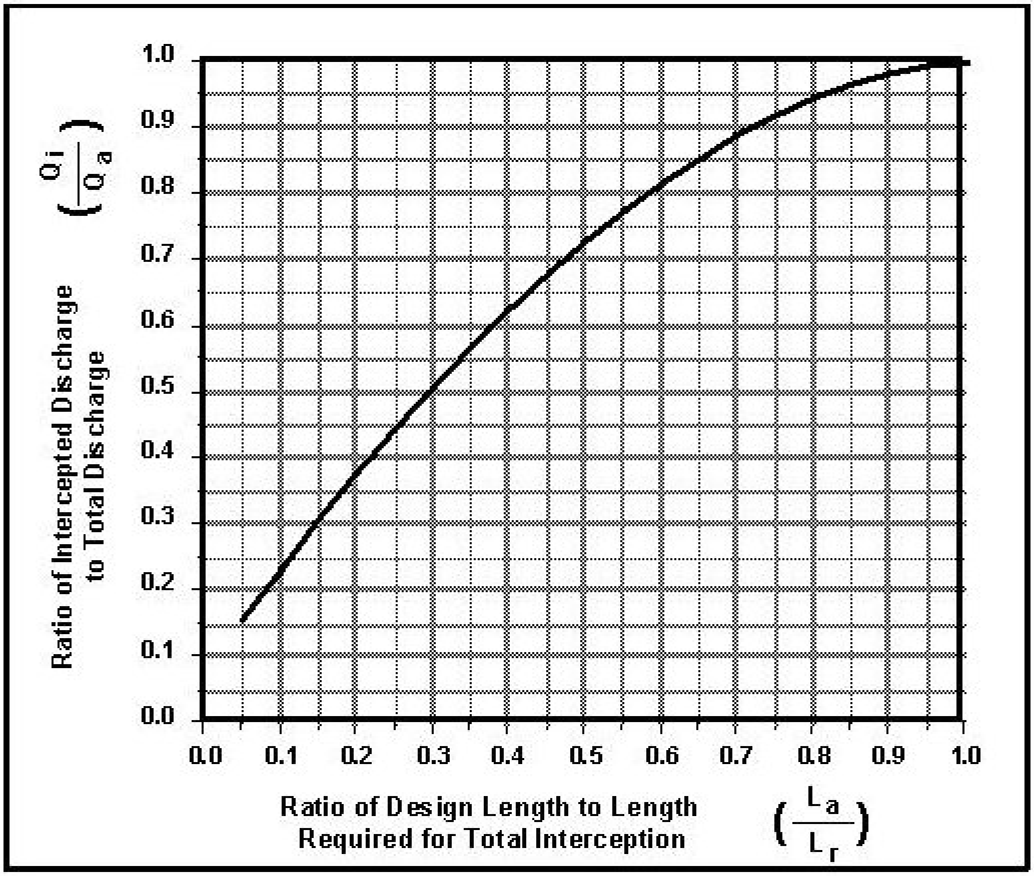


Figure 10-18.

As a rule of thumb, the hydraulic designer can optimize slotted drain inlet economy by providing actual lengths (L_a) to required lengths (L_r) in an approximate ratio of about 0.65. This implies a usual design with carryover for on-grade slotted drain inlets.

Trench Drain Inlet Design

The following procedure may be used for trench drain design:

1. Determine the location of trench drain to intercept the runoff.
2. If on a slope, calculate the runoff to be captured by the trench drain in terms of cfs per foot of slope width. The maximum intercept rate, calculated from the weir equation, is 1.4 cfs per foot of length per side of trench drain for the Department specified grate. Flows exceeding 1.4 cfs per foot of length per side will require a different method of interception or multiple rows of trench drain inlets.
3. Select the outfall location for the trench drain. The maximum length of the trench drain is measured from the outfall location because the depth of the outfall, whether natural ground or a storm drain pipe, will determine the maximum depth of the trench drain. Trench drain is normally available with either a fixed invert slope or neutral invert without slope.

4. Look at the manufacturer's data to select the segments needed by part number.
5. Select the trench drain outlet, horizontal or vertical, and size.
6. Calculate the allowable flow in the drain outlet using the orifice equation.
7. Select the method of connection to the storm drain pipe.

Grate Inlets On-Grade

The capacity of a grate inlet on-grade depends on its geometry and cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness.

The depth of water next to the curb is the major factor affecting the interception capacity of grate inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow is intercepted if the velocity is high or the grate is short because a portion tends to splash over the end of the grate ("splash-over"). For grates less than 2 ft. (0.6 m) long, intercepted flow is small.

Refer to HEC-12 or HEC-22 for inlet efficiency data for various sizes and designs of grates. Additionally, safety for small wheeled vehicles (bicycles, wheelchairs, etc.) must be considered in grate selection.

Design Procedure for Grate Inlets On-Grade

Use the following procedure for grate inlets on-grade:

1. Compute the ponded width of flow (T). Use the outline provided in Gutter Ponding on Continuous Grades.
2. Choose a grate type and size.
3. Find the ratio of frontal flow to total gutter flow (E_o) for a straight cross-slope using Equation 10-5. No depression is applied to a grate on-grade inlet.
4. Find the ratio of frontal flow intercepted to total frontal flow, R_f, using Equations 10-22, 10-23, and 10-24.

   If $v > v_o$, use Equation 10-22.

$$R_f = 1 - K_u(v - v_o)$$

Equation 10-22.

   If $v < v_o$, use Equation 10-23.

$$R_f = 1.0$$

Equation 10-23.

where:

R_f = ratio of frontal flow intercepted to total frontal flow

K_u = 0.09 for English measurement or 0.295 for metric

v = approach velocity of flow in gutter (ft./s or m/s)

v_o = minimum velocity that will cause splash over grate (ft./s or m/s)

For triangular sections, calculate the approach velocity of flow in gutter (v) using Equation 10-24.

$$v = \frac{2Q}{Ty} = \frac{2Q}{T^2 S_x}$$

Equation 10-24.

Otherwise, compute the section flow area of flow (A) and calculate the velocity using Equation 10-25:

$$v = \frac{Q}{A}$$

Equation 10-25.

where:

v_o = splash-over velocity (ft./s or m/s)

L = length of grate (ft. or m)

Calculate the minimum velocity (v_o) that will cause splash over the grate using the appropriate equation in Table 10-2 below.

Table 10-2. Splash-Over Velocity Calculation Equations (English)

Grate Configuration	Typical Bar Spacing (in.)	Splash-over Velocity Equation
Parallel Bars	2	$v_o = 2.218 + 4.031L - 0.649L^2 + 0.056L^3$
Parallel Bars	1.2	$v_o = 1.762 + 3.117L - 0.451L^2 + 0.033L^3$
Parallel bars w/ transverse rods	2 parallel/4 transverse	$v_o = 0.735 + 2.437L - 0.265L^2 + 0.018L^3$
Reticuline	n/a	$v_o = 0.030 + 2.278L - 0.179L^2 + 0.010L^3$

5. Find the ratio of side flow intercepted to total side flow, R_s.

$$R_s = \left[1 + \frac{zv^{1.8}}{S_x L^{2.3}}\right]^{-1}$$

Equation 10-26.

where:

R_S = ratio of side flow intercepted to total flow

z = 0.15 for English measurement or 0.083 for metric

S_x = transverse slope

v = approach velocity of flow in gutter (ft./s or m/s)

L = length of grate (ft. or m).

6. Determine the efficiency of grate, E_f. Use Equation 10-27.

$$E_f = [R_f E_o + R_s(1 - E_o)]$$

Equation 10-27.

7. Calculate the interception capacity of the grate, Q_i. Use Equation 10-28. If the interception capacity is greater than the design discharge, skip step 8.

$$Q_i = E_f Q = Q[R_f E_o + R_s(1 - E_o)]$$

Equation 10-28.

8. Determine the bypass flow (CO) using Equation 10-29.

$$CO = Q - Q_i$$

Equation 10-29.

9. Depending on the bypass flow, select a larger or smaller inlet as needed. If the bypass flow is excessive, select a larger configuration of inlet and return to step 3. If the interception capacity far exceeds the design discharge, consider using a smaller inlet and return to step 3.

Design Procedure for Grate Inlets in Sag Configurations

A grate inlet in sag configuration operates in weir flow at low ponded depths but transitions to orifice flow as the ponded depth increases. The following procedure is used for calculating the inlet capacity:

1. Choose a grate of standard dimensions to use as a basis for calculations.
2. Determine an allowable head (h) for the inlet location. For a grate in a curb and gutter section, this should be the lower of the curb height or the depth associated with the allowable ponded width. For a grate in a ditch (drop inlet), this should be the lower of the height of the ditch block, if any, or the allowable ponded depth. No gutter depression is applied at grate inlets.
3. Determine the capacity of a grate inlet operating as a weir. Under weir conditions, the grate perimeter controls the capacity. Figure 10-16 shows the perimeter length for a grate inlet located next to and away from a curb. The capacity of a grate inlet operating as a weir is determined using Equation 10-30.

$$Q_W = C_W P h^{1.5}$$

Equation 10-30.

where:

Q_W = weir capacity of grate (cfs or m^3/s)

C_W = weir coefficient = 3 for English measurement or 1.66 for metric

P = perimeter of the grate (ft. or m) as shown in Figure 10-16: A multiplier of about 0.5 is recommended to be applied to the measured perimeter as a safety factor.

h = allowable head on grate (ft. or m).

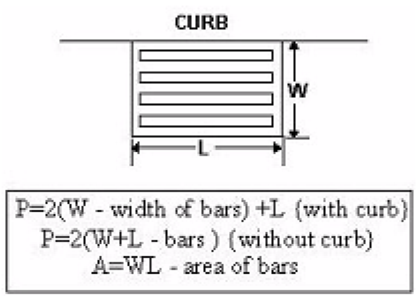


Figure 10-19. Perimeter Length for Grate Inlet in Sag Configuration

4. Determine the capacity of a grate inlet operating under orifice flow. Under orifice conditions, the grate area controls the capacity. The capacity of a grate inlet operating under orifice flow is computed with Equation 10-31.

$$Q_o = C_o A \sqrt{2gh}$$

Equation 10-31.

where:

Q_o = orifice capacity of grate (cfs or m^3/s)

C_o = orifice flow coefficient = 0.67

A = clear opening area (sq. ft. or m^2) of the grate (the total area available for flow). A multiplier of about 0.5 is recommended to be applied to the measured area as a safety factor

g = acceleration due to gravity (32.2 ft/s^2 or 9.81 m/s^2)

h = allowable head on grate (ft. or m).

5. Compare the calculated capacities from steps 3 and 4 and choose the lower value as the design capacity. The design capacity of a grated inlet in sag is based on the minimum flow calculated from weir and orifice conditions. Figure 10-17 demonstrates the relationship between weir and orifice flow. If Q_o is greater than Q_w (to the left of the intersection in Figure 10-17), then the designer would use the capacity calculated with the weir equation. If, however, Q_o is less than Q_w (to the right of the intersection), then the capacity as determined with the orifice equation would be used.

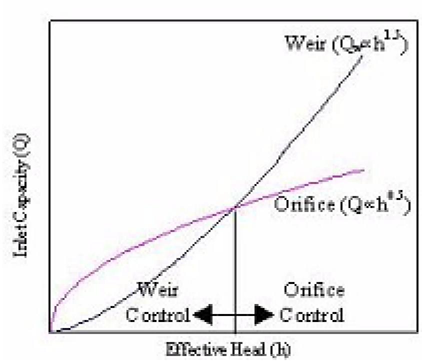


Figure 10-20. Relationship between Head and Capacity for Weir and Orifice Flow

Section 7 — Conduit Systems

Conduits

The storm drainage conduit system transports the runoff from the surface collection system (inlets) to the outfall. Conduit connections between points in the network such as junctions and inlets are referred to as "runs." Although it is an integral component, the conduit system is analyzed independently of the inlet system. However, the configuration of laterals and trunk lines is controlled by the locations of all inlet and roadway layouts, minimum cover requirements, and utility and foundation locations.

The longitudinal slope of the conduit affects the capacity. The slope of the subject conduit run is tentatively established during the system planning stage of design. Typically, the slope will be approximately parallel to the surface topography. However, the slope may have to be adjusted to adapt to critical elevations (such as outfall elevations or utilities) to increase capacity or to afford adequate cover for the conduit.

The Department minimum diameter for trunk lines and laterals is 18 inches because of difficulties in the construction and maintenance of smaller sizes. Some designers prefer to limit the minimum diameter to 24 inches. The following recommendations on conduit dimensions should be considered:

- Larger into smaller conduit dimensions -- Avoid discharging the flow of a larger conduit into a smaller conduit. The capacity of the smaller conduit may theoretically be greater due to a steeper slope; however, a reduction in size almost always results in operational problems and expenses for the system. Debris that may pass through a larger dimension may clog as it enters a smaller dimension.
- Soffit and flow line placement in conduits -- At changes in conduit size, the soffit (top inside surface), not the flow line, of the two conduits should be aligned. When flow lines are aligned, the smaller pipe often must discharge against a head. It may not be feasible to follow this guideline in every instance, but it should be the rule whenever practicable. It is acceptable to have the entire downstream conduit offset downward because the flow would still not discharge against a head.
- Conduit length -- The approximate length of conduit should be determined as the inlets and junctions are located. The length is the distance from the centerline of the upstream node to the centerline of the downstream node of the subject conduit run. The length and average flow velocity are used to estimate the travel time within the run. Establish the length of the run during the first phase of the storm drain system design in which the inlets are located.

NOTE: These lengths are hydraulic lengths of conduit, not pay lengths; the Department standard specifications provide that pay lengths include only the actual net length of pipe and not the distance across inlets or access holes where no conduit actually is placed.

Access Holes (Manholes)

Access holes or combination access hole/inlets should be placed at changes in direction, junctions of pipe runs, at intervals in long pipe runs, or wherever necessary for clean-out and inspection purposes. The table below provides maximum spacing criteria for access holes.

Table 10-3. Access Hole Maximum Spacing

Pipe Diameter	**Maximum Distance**
in.	**ft.**
12 - 24	300
27 - 36	375
39 - 54	450
$\geq$ 60	900

It is possible to reduce head losses caused by turbulence within an access hole by rounding the flowline (bottom) of the access hole to match the flowlines of the pipes attached to the manhole. See Inlet and Access Hole Energy Loss Equations for more details. For manholes larger than the incoming or outgoing pipes, expansion losses can sometimes be significant. Access holes that include other functions must be detailed to include facilities that will serve all the intended functions.

If the hydraulic grade line could rise above the access hole cover, the cover must be secured by bolting or with a locking mechanism to prevent removal by vandals or by a “blowout.” A “blowout” is caused when the hydraulic grade line rises higher than the access hole cover, which may force the cover to explode off.

Junction Angles

At junctions, right angle intersections are simpler to construct than acute angle junctions. However, acute angle junctions reduce head losses and also pass debris more easily, and should be used where practical. See Figure 10-18 for the contrast.

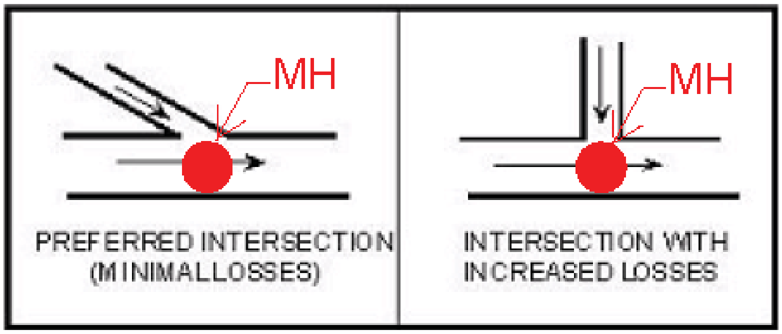


Figure 10-21. Acute Angle and Right Angle Intersections

Inverted Siphons

Inverted siphons carry flow under obstructions such as sanitary sewers, water mains, or other structure or utilities that may be in the path of the storm drain line. Siphons should be used only where avoidance or adjustment of the obstruction is not practical. Figure 10-19 shows a profile of an inverted siphon. A minimum flow velocity of 3 fps (1 m/s) is recommended to keep sediment suspended.

The conduit size through the inverted siphon used in a storm drain system should be the same size as either the approaching or exiting conduit. In no case should the size be smaller than the smallest of the approaching or exiting conduit.

Because inverted siphons include slopes of zero and adverse values, the head losses through the structure must be accounted for. The sources of the losses can be friction, bends, junctions, and transitions. See Chapter 6, Hydraulic Grade Line Analysis. Alternative means of avoiding the obstruction may be necessary if losses are unacceptable. Maintenance access should be provided at either or both ends of the inverted siphon as indicated in Figure 10-19.

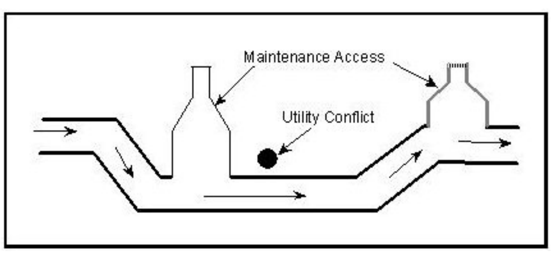


Figure 10-22. Inverted Siphon

Conduit Capacity Equations

Refer to Chapter 6 for calculating channel (conduit) capacity and critical depth.

Conduit Design Procedure

In this procedure, points in the network such as junctions and inlets are referred to as “nodes”. Conduit connections between nodes are referred to as “runs”. A storm drainage system is characterized as a link-node system with runoff entering the system at nodes (inlets) that are linked together (by pipe or conduit runs), all leading to some outfall (outlet node). The procedure entails proceeding progressively downstream from the most remote upstream node to the outlet. The peak discharge at each node is recomputed based on cumulative drainage area, runoff coefficient, and longest time of concentration contributing to the particular node.

The following steps are used for the design of conduit systems (a more detailed explanation and example are contained in HEC-22):

1. Determine the design discharge at each extreme node (inlet). Any bypass flow, either from or to the inlet, is ignored when considering the discharge into the conduit. Keep track of the cumulative runoff coefficient multiplied by the area (ΣCA) and the time of concentration. This time of concentration often is referred to as "inlet time," indicating it is the surface time of concentration in the watershed to the inlet.
2. Determine the design discharge for the first run (or any inlet lateral) based on the watershed area to the upstream node of the run (A), the associated weighted runoff coefficient (C), and the rainfall intensity based on the time of concentration (t_c) in the watershed. The rainfall intensity is calculated with Equation 10-33 using the larger of the actual t_c value or a t_c of 10 minutes. The discharge is computed using Equation 10-32. It is very important to record the actual time of concentration as this value will eventually become significant.

$$Q = \frac{CIA}{z}$$

Equation 10-32.

where:

Q = peak discharge (cfs or m^3/s)

C = runoff coefficient

I = rainfall intensity associated with a specific AEP (in./hr or mm/hr)

A = area of the watershed (ac. or ha)

z = 1.0 for English measurement and 360 for metric.

$$I_f = \frac{b}{(t_c + d)^e}$$

Equation 10-33.

where:

I_f = rainfall intensity for design AEP (in./hr or mm/hr)

t_c = time of concentration (min)

e, b, d = empirical factors that are tabulated for each county in Texas for frequencies of 2, 5, 10, 25, 50, and 100 years (50%, 20%, 10%, 4%, 2%, and 1% AEPs) in Hydrology. (See Rainfall Intensity-Duration-Frequency Coefficients.)

NOTE: Chapter 4 references the new rainfall atlas, Atlas of Depth-Duration Frequency (DDF) of Precipitation Annual Maxima for Texas (TxDOT 5-1301-01-1). A table of factors correlating to this atlas, similar to Rainfall Intensity-Duration-Frequency Coefficients, for use with Equation 10-33, will be developed at a later date. The new table will replace this reference to Rainfall Intensity-Duration-Frequency Coefficients at that time.

The intensity is based on the longest time of concentration leading to the upstream end of the run. This means that a recalculation of total discharge is necessary at each conduit run. It also means that the discharge rates from approaching pipe runs are not simply summed; instead, the sum of contributing CA values (ΣCA) are multiplied by an intensity based on the longest t_c leading to the point in question.

3. Size the conduit based on Manning's Equation and the design discharge. The Department recommended method is to design for non-pressure flow. Conduit size will likely be slightly larger than necessary to accommodate the design flow under the terms of Manning's Equation. To size circular pipe, use Equation 10-34:

$$D = z\left(\frac{Qn}{S^{1/2}}\right)^{3/8}$$

Equation 10-34.

where:

D = required diameter (ft. or m)

z = 1.3333 for English measurement or 1.5485 for metric

Q = discharge (cfs or m^3/s)

n = Manning’s roughness coefficient

S = slope of conduit run (ft./ft. or m/m).

For sizing other shapes, use trial and error by selecting a trial size and then computing the capacity. Adjust the size until the computed capacity is slightly higher than the design discharge.

4. Estimate the velocity of flow through the designed conduit. Determine the cross-section area, A_u, assuming uniform flow as an average depth of flow in the conduit as discussed in Section 2 of Chapter 6. This is a straightforward procedure for rectangular sections but much more complicated for circular and other shapes. Then calculate the average velocity of flow (V_a) using the continuity relation shown in Equation 10-35.

$$V_a = \frac{Q}{A_u}$$

Equation 10-35.

5. Calculate the travel time, t_t, for flow in the conduit from the upstream node to the downstream node by dividing the length of the conduit by the average velocity of flow. Add this travel time to the t_t at the upstream end of the subject run to represent the t_t at the downstream end of the run.

   NOTE: For this purpose, base the t_t on the actual calculated times, not the minimum of 10 minutes used to compute intensity.

6. Proceeding downstream through the system, determine the cumulative runoff coefficient multiplied by the area (ΣCA) and respective time of concentration at each node. Make sure to include all conduits and inlets coming to a particular node before sizing the conduit run out of that node. It may help to draw a stick diagram showing the cumulative CA and t_c/t_t values.

7. Compute the peak discharge for the next run downstream based on the ΣCA to the node and the intensity based on the longest value of t_c of all incoming conduits, and, if applicable, t_c of any inlet directly at the node. The discharge, so determined, is not the same as if all approaching discharges have been added.

   In some instances, an increase in t_t (which decreases I) with little or no additional CA can cause the calculated discharge to decrease as the analysis is carried downstream. In such cases, use the previous intensity to avoid designing for a reduced discharge, or consider using a hydrograph routing method.

8. Develop the hydraulic grade line (HGL) in the system as outlined in Chapter 6. Calculate minor losses according to Conduit Systems Energy Losses. If the system was designed for full flow, calculate other losses such as junction, manhole and exit losses according to Conduit Systems Energy Losses.

Conduit Analysis

The analysis of a conduit requires the same consideration of hydrology as does design. The difference is that geometry, roughness characteristics, and conduit slopes are already established.

The analysis and accumulation of discharge must proceed from upstream toward downstream in the system. Develop the discharges in this way so that appropriate discharge values are available for the development of the hydraulic grade line analysis.

Section 8 — Conduit Systems Energy Losses

Energy grade line (EGL) computations begin at the outfall and are worked upstream, taking each junction into consideration. Many storm drain systems are designed to function in subcritical flow. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL. In supercritical flow, pipe and access losses are not carried upstream.

Minor Energy Loss Attributions

Minor losses in a storm drain system are usually insignificant when considered individually. In a large system, however, the combined effects may be significant. The hydraulic loss potential of storm drain system features, such as junctions, bends, manholes, and confluences, can be minimized by careful design. For example, severe bends can be replaced by gradual bends where right-of-way is sufficient and increased costs are manageable. Well designed manholes and inlets without sharp or sudden transitions or impediments to the flow cause no significant losses.

Junction Loss Equation

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 in the form of the momentum equation. In Equation 10-36, the subscripts “i”, “o”, and “1” indicate the inlet, outlet, and lateral, respectively.

$$h_j = \frac{Q_o v_o - Q_i v_i - Q_1 v_1 \cos\theta}{0.5 g (A_o + A_i)}$$

Equation 10-36.

where:

h_j = junction head loss (ft. or m)

Q = flow (cfs or m^3/s)

v = velocity (fps or m/s)

A = cross-sectional area (sq. ft. or m^2)

θ = angle in degrees of lateral with respect to centerline of outlet pipe

g = gravitational acceleration = 32.2 ft/s^2 or 9.81 m/s^2.

The above equation applies only if $v_o > v_i$ and assumes that $Q_o = Q_i + Q_1$.

Exit Loss Equation

The exit loss, h_o, is a function of the change in velocity at the outlet of the pipe as shown in Equation 10-37.

$$h_o = C_o \frac{v^2 - v_d^2}{2g}$$

Equation 10-37.

where:

v = average outlet velocity (fps or m/s)

v_d = channel velocity downstream of the outlet (fps or m/s)

C_o = exit loss coefficient (0.5 typical).

The above assumes that the channel velocity is lower than the outlet velocity. Note that, for partial flow, where the pipe outfalls into a channel with water moving in the same direction, the exit loss may be reduced to virtually zero.

Inlet and Access Hole Energy Loss Equations

HEC-22, Chapter 7 presents a new method to compute energy losses for inlets and access holes.

As a starting point, the outflow pipe energy head (E_i) is the difference between the energy gradeline in the outflow pipe (EGL_i) and the outflow pipe flowline, as shown on Figure 10-20.

$$E_i = EGL_i - Z_i$$

Equation 10-38.

where:

E_i = Outflow pipe energy head (ft. or m)

EGL_i = Outflow pipe energy gradeline

Z_i = Outflow pipe flowline elevation

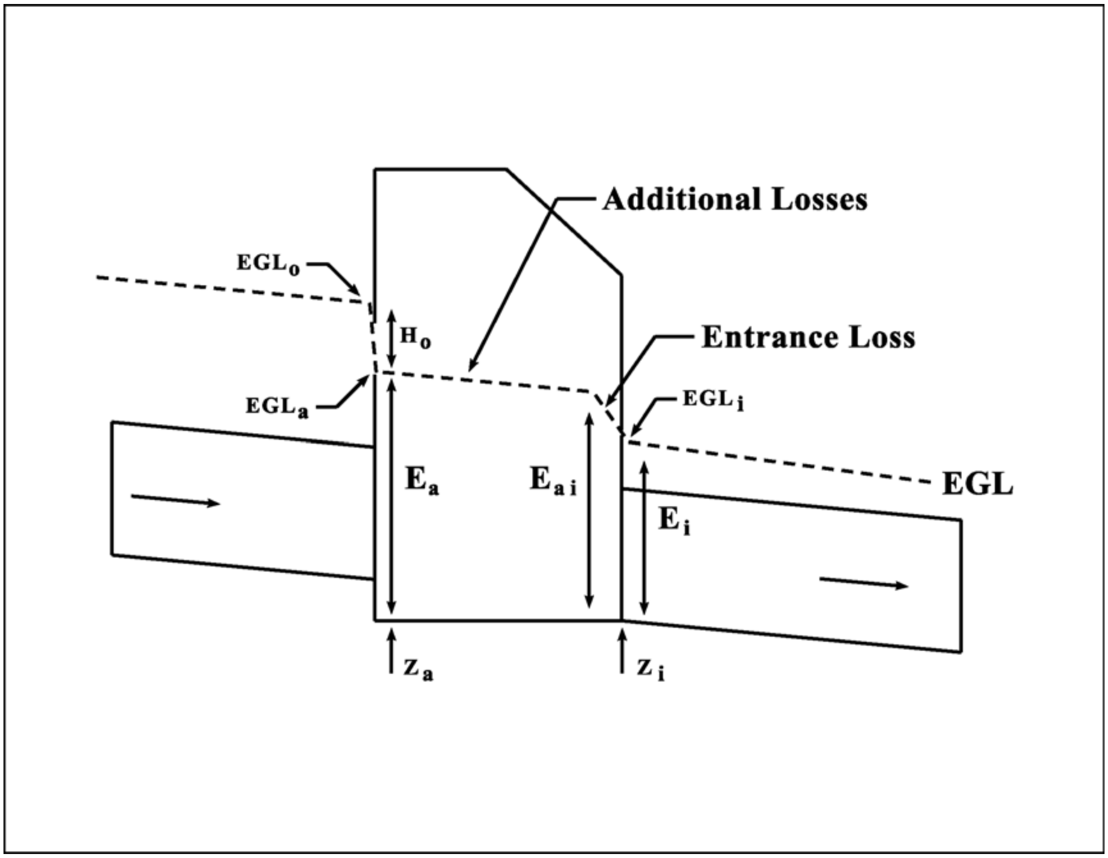


Figure 10-23. Access Hole Energy Level Definitions

Initial Access Hole Energy Level

The initial estimate of energy level (E_{ai}) is taken as the maximum of the three values, E_{aio}, E_{ais}, and E_{aiu}:

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

Equation 10-39.

where:

E_{aio} = Estimated access hole energy level for outlet control (full and partial flow)

E_{ais} = Estimated access hole energy level for inlet control (submerged)

E_{aiu} = Estimated access hole energy level for inlet control (unsubmerged)

E_{aio} -- Estimated Energy Level for Outlet Control

In the outlet control condition, flow out of the access hole is limited by the downstream storm drain system. The outflow pipe would be in subcritical flow and could be either flowing full or partially full.

Whether the outflow pipe is flowing full or partially full affects the value of E_{aio}. This can be determined by redescribing and rearranging the outflow pipe energy head, E_i. E_i can be described as the sum of the potential head, pressure head, and velocity head, as shown in Equation 10-40.

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

Equation 10-40.

where:

γ = Outflow pipe depth (potential head) (ft. or m)

(P/γ) = Outflow pipe pressure head (ft. or m)

$V^2/2g$ = Outflow pipe velocity head (ft. or m).

Rearranging Equation 40 to isolate the potential head and pressure head gives Equation 41:

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

Equation 10-41.

If $y + (P/\gamma)$ is less than the diameter of the outflow pipe, then the pipe is in partial flow and the estimated initial structure energy level (E_{aio}) is equal to zero ($E_{aio} = 0$).

If $y + (P/\gamma)$ is greater than the diameter of the outflow pipe, then the pipe is in full flow, and the estimated initial structure energy level (E_{aio}) is calculated using Equation 10-42:

$$E_{aio} = E_i + H_i$$

Equation 10-42.

where:

E_i = Outflow pipe energy head (ft. or m)

H_i = entrance loss assuming outlet control, using Equation 10-43

$$H_i = 0.2\left(\frac{V^2}{2g}\right)$$

Equation 10-43.

where:

$V^2/2g$ = Outflow pipe velocity head (ft. or m)

E_{ais} -- Estimated Energy Level for Inlet Control: Submerged

The submerged inlet control energy level (E_{ais}) checks the orifice condition and is estimated using Equation 10-44:

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

Equation 10-44.

where:

DI is the Discharge Intensity parameter, calculated by Equation 10-45:

$$DI = Q/[A(gD_o)^{0.5}]$$

Equation 10-45.

where:

DI = Discharge Intensity parameter

Q = flow in outfall pipe (cfs or m^3/s)

A = Area of outflow pipe (ft^2 or m^2)

D_o = Diameter of outflow pipe (ft. or m)

E_{aiu} -- Estimated Energy Level for Inlet Control: Unsubmerged

The unsubmerged inlet control energy level (E_{aiu}) checks the weir condition and is estimated using Equation 10-46:

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

Equation 10-46.

Adjustments for Benching, Angled Inflow, and Plunging Inflow

The revised access hole energy level (E_a) is determined by adding three loss factors for: (1) benching configurations; (2) flows entering the structure at an angle; and (3) plunging flows. Flows entering a structure from an inlet can be treated as plunging flows.

$$E_a = E_{ai} + H_a$$

Equation 10-47.

where:

E_a = the revised access hole energy level

E_{ai} = the initial estimate of access hole energy level, calculated using Equation 10-39

H_a = additional energy loss due to benching, angled inflow and plunging inflow, calculated using Equation 10-48.

If E_a is calculated to be less than the outflow pipe energy head (E_i), then E_a should be set equal to E_i.

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

Equation 10-48.

where:

C_B = Coefficient for benching (floor configuration)

C_θ = Coefficient for angled flows

C_P = Coefficient for plunging flows

Note that the value of H_a should always be positive. If not, H_a should be set to zero.

Additional Energy Loss: Benching

Benching serves to direct flow through the access hole, which reduces energy losses. Figure 10-21 illustrates some typical bench configurations. Department standard sheets do not show any benching practices other than methods (a) and (b).

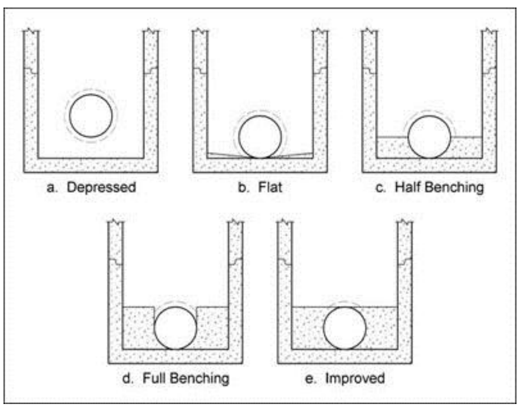


Figure 10-24. Access hole benching methods

The energy loss coefficient for benching, (C_B), is obtained from Table 10-4. A negative value indicates water depth will be decreased rather than increased.

Table 10-4. Values for the Coefficient, C_B

Floor Configuration	C_B
Flat (level)	-0.05
Depressed	0.0
Unknown	-0.05

Additional Energy Loss: Angled Inflow

The angles of all inflow pipes into the access hole are combined into a single weighted angle (θ_w) using Equation 10-49:

$$\theta_W = \Sigma((Q_J\theta_J)/(\Sigma Q_J))$$

Equation 10-49.

where:

Q_J = Contributing flow from inflow pipe, cfs

θ_J = Angle measured from the outlet pipe (degrees)(plunging flow is 180 degrees)

Figure 10-22 illustrates the orientation of the pipe inflow angle measurement. The angle for each inflow pipe is referenced to the outlet pipe, so that the angle is not greater than 180 degrees. A straight pipe angle is 180 degrees. If all flows are plunging, θ_w is set to 180 degrees; the angled inflow coefficient approaches zero as θ_w approaches 180 degrees and the relative inflow approaches zero. The angled inflow coefficient (C_θ) is calculated by Equation 10-50:

$$C_\theta = 4.5\frac{(\Sigma Q_J)}{Q_o}\cos(\theta_W/2)$$

Equation 10-50.

where:

Q_o = Flow in outflow pipe, cfs

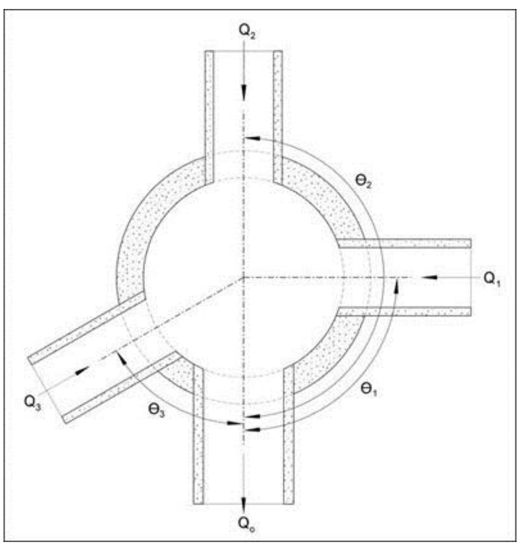


Figure 10-25. Access hole angled inflow definition.

Additional Energy Loss: Plunging Inflow

Plunging inflow is defined as inflow from an inlet or a pipe where the pipe flowline is above the estimated access hole water depth (approximated by E_{ai}).

The relative plunge height (h_k) for each inflow pipe is calculated using Equation 10-51:

$$h_k = (Z_k - E_{ai}) / D_o$$

Equation 10-51.

where:

Z_k = the difference between the inflow pipe flowline elevation and the access hole flowline elevation. If $Z_k > 10D_o$ it should be set to $10D_o$.

The relative plunge height for each inflow pipe is calculated separately and then combined into a single plunging flow coefficient (C_P):

$$C_P = \frac{\Sigma(Q_k H_k)}{Q_o}$$

Equation 10-52.

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

Access Hole Energy Gradeline

Knowing the access hole energy level (E_a) and assuming that the access hole flowline (Z_a) is the same elevation as the outflow pipe flowline (Z_i) allows determination of the access hole energy gradeline (EGL_a):

$$EGL_a = E_a + Z_a$$

Equation 10-53.

As described earlier, the potentially highly turbulent nature of flow within the access hole makes determination of water depth problematic. Research has shown that determining velocity head within the access hole is very difficult, even in controlled laboratory conditions. However, a reasonable assumption is to use the EGL_a as a comparison elevation to check for potential surcharging of the system.

Inflow Pipe Exit Losses

The final step is to calculate the energy gradeline into each inflow pipe, whether plunging or non-plunging.

Non-Plunging Inflow Pipe

Non-plunging inflow pipes are those pipes with a hydraulic connection to the water in the access hole. Inflow pipes operating under this condition are identified when the revised access hole energy gradeline (E_a) is greater than the inflow pipe flowline elevation (Z_o). In this case, the inflow pipe energy head (EGL_o) is equal to:

$$EGL_o = EGL_a + H_o$$

Equation 10-54.

where:

$H_o = 0.4(V^2/2g)$ = Inflow pipe exit loss

Exit loss is calculated in the traditional manner using the inflow pipe velocity head since a condition of supercritical flow is not a concern on the inflow pipe.

Plunging Inflow Pipe

For plunging inflow pipes, the inflow pipe energy gradeline (EGL_o) is logically independent of access hole water depth and losses. Determining the energy gradeline for the outlet of a pipe has already been described in Chapter 6.

Continuing Computations Upstream

For either the nonplunging or plunging flows, the resulting energy gradeline is used to continue computations upstream to the next access hole. The procedure of estimating entrance losses, additional losses, and exit losses is repeated at each access hole.

Energy Gradeline Procedure

1. Determine the EGL_i and HGL_i downstream of the access hole. The EGL and HGL will most likely need to be followed all the way from the outfall. If the system is being connected to an existing storm drain, the EGL and HGL will be that of the existing storm drain.
2. Verify flow conditions at the outflow pipe.
   a. If HGL_i is greater or equal to the soffit of the outflow pipe, the pipe is in full flow.
   b. If HGL_i is less than the soffit of the outflow pipe but greater than critical depth, the pipe is not in full flow but downstream conditions still control.
   c. If HGL_i is less than the soffit of the outflow pipe but greater than critical depth and less than or equal to normal depth, the pipe is in subcritical partial flow. EGL_i becomes the flowline elevation plus normal depth plus the velocity head.
   d. If HGL_i is less than critical depth, the pipe is in supercritical partial flow conditions. Pipe losses in a supercritical pipe section are not carried upstream.
3. Estimate E_i (outflow pipe energy head) by subtracting Z_i (pipe flowline elevation) from the EGL_i using Equation 10-38. Calculate $\gamma + P/\gamma$ using Equation 10-41. Compute DI using Equation 10-45.

4. Calculate E_{ai} as maximum of E_{aio}, E_{ais}, and E_{aiu} as below:
   a. If $(\gamma + P/\gamma) > D$, then the pipe is in full flow and $E_{aio} = E_i + H_i$ (Equation 10-42). If $(\gamma + P/\gamma) \leq D$, then the pipe is in partial flow and $E_{aio} = 0$.
   b. $E_{ais} = D_o(DI)^2$ (Equation 10-44)
   c. $E_{aiu} = 1.6\ D_o(DI)^{0.67}$ (Equation 10-46)

   If $E_{ai} < E_i$, the head loss through the access hole will be zero, and $E_{ai} = E_i$. Go to Step 10.
5. Determine the benching coefficient (C_B) using Table 10-4. Department standard sheets do not show any benching practices other than depressed (a) or flat (b). The values are the same whether the bench is submerged or unsubmerged.
6. Determine the energy loss coefficient for angle flow (C_θ) by determining θ_W for every pipe into the access hole.
   a. Is E_i < inflow pipe flowline? If so, then the flow is plunging and θ_W for that pipe is 180 degrees.
   b. If the pipe angle is straight, then θ_W for that pipe is 180 degrees.
   c. Otherwise, θ_W is the angle of the inflow pipe relevant to the outflow pipe. Maximum angle is 180 degrees (straight).

   Use Equation 10-49 and Equation 10-50 to calculate θ_W and C_θ.
7. Determine the plunging flow coefficient (C_P) for every pipe into the access hole using Equation 10-52. The relative plunge height (h_k) is calculated using Equation 10-51. Z_k is the difference between the access hole flowline elevation and the inflow pipe flowline elevation. If $Z_k > 10D_o$, Z_k should be set to $10D_o$.
8. If the initial estimate of the access hole energy level is greater than the outflow pipe energy head ($E_{ai} > E_i$), then $E_a = E_i$. If $E_{ai} \leq E_i$, then $H_a = (E_{ai} - E_i)(C_B + C_\theta + C_P)$. If $H_a < 0$, set $H_a = 0$.
9. Calculate the revised access hole energy level (E_a) using Equation 10-47. If $E_a < E_i$, set $E_a = E_i$.
10. Compute EGL_a by adding E_a to the outflow pipe flowline elevation. Assume HGL_a at the access hole structure is equal to EGL_a.
11. Compare EGL_a with the critical elevation (ground surface, top of grate, gutter elevation, or other limits). If EGL_a exceeds the critical elevation, modifications must be made to the design.

Chapter 11 — Pump Stations

Contents:

Section 1 — Introduction

Purpose of A Pump Station

A pump station mechanically lifts storm water runoff from a gravity fed collection cistern to a discharge place or outfall. In general, gravity outfalls are the primary and preferred means of releasing flow from storm drain systems. However, a pump station becomes necessary when gravity outfalls are not economically or engineeringly feasible.

The need for pump stations is a function of the highway geometric design rather than of climatic factors. Planners can design pump stations to be unobtrusive, efficient, and reliable.

In the planning stages, the roadway designer can obtain valuable advice and assistance from the following sources:

- manufacturer representatives for pumps,
- manufacturer representatives for generators,
- contractors who have had experience in pump station construction,
- utility representatives for electricity and natural gas.

Security and Access Considerations

Protection of the facilities is an important concern. The pump station facility should be protected and secured with fences, gates, and locks. When planning the fencing, adequate access for service and maintenance vehicles must be provided.

Safety and Environmental Considerations

Depending on the types and concentrations of runoff contaminants or pollutants that may be pumped by the facility, certain safety and environmental features may be necessary in the design. Consult the Design Division's Hydraulics Branch about the quality of the runoff discharge water. Refer to the TxDOT *Environmental Management System Manual* and the Environmental Division for more information on environmental concerns, policies, and agencies.

Section 2 — Pump Station Components

Overview of Components

A full discussion of the design and specification of a pump station is beyond the scope of this manual. However, this section attempts to bring to the design engineer's attention the various components and the considerations for those components. Appropriate design specialists for the control, electrical, mechanical, and structural components of a pump station must to be consulted early in the decision process. A common reference for design of pump stations is FHWA Hydraulic Engineering Circular number 24 (HEC-24).

The following are necessary considerations in pump station design:

- Property-An entire pump station generally requires more footprint than merely the pumps and wet well or sump. Other necessary parts of the station include the electrical service, system controller, motor control center cabinets, which must be in a separate, dry room, and standby power generation. Other considerations may be on-site storage and parking. A required consideration is maintenance access to the pumps and the standby generator; not just personnel access, but the ability and room to bring in suitable vehicles and equipment such as a boom crane to lift out pumps, generator, and electrical cabinets for repair or replacement.
- Arrangement-The wells and pumps may not need to be in the same place as the control house. An example of this is a set of wells with submerged pumps and discharge conduits located in a wide median of a depressed section of Interstate highway. The control house with the electrical service, standby generator, motor control center, and control circuitry is located along the frontage road out of the depressed section and away from buried or overhead utilities.
- Wet Well- The wet well receives the inflow of storm water prior to pumping. It must also be designed with a trash collection rack, room for sedimentation collection without diminishing the design capacity, and a sump pump to remove the bottom storage below the main pump level.
- Electrical-The appropriate electrical service for a pump station is usually 277/480-volt, 3-phase AC. For a typical pump station, the electrical service equipment includes large metal cabinets for the electrical metering, main circuit breaker, a transfer switch to isolate the station from the utility when the standby generator is powering the station, and the electrical distribution panel. The details of the electrical service equipment are the province of the electrical engineer. However, the project manager must understand that clearances and air space around electrical equipment are not options; they are mandatory safety requirements which may increase the footprint of the pump station, but cannot be ignored.
- Standby Power-The normal source of standby power is either a diesel or natural gas engine/generator set. Fuel cells are not suitable for pump stations because of the hours long start-up time they require. Battery technology is improving to the point where solar or wind power may

become viable. For TxDOT pump stations, natural gas powered engines should be considered over diesel. Diesel is the more efficient fuel which allows for a smaller engine than natural gas, but diesel has many other problems. Modern diesel is intended to be used within a few months of production. Diesel fuel that sits in a holding tank for a long period is at risk of gelling, particularly if the fuel is warmed during the monthly or weekly scheduled test run of the unit. Sitting diesel fuel is also highly subject to moisture contamination from the atmosphere. Natural gas is not subject to contamination or breakdown in the pipeline. Destructive storms which may cause road blockages and delivery problems usually do not interfere with natural gas service.

- Pumps-Pump selection depends on station layout, required pump rate, wet well depth, and pump maintenance considerations. Pump selection includes the size, type, and number of pumps. For the most part, department pump stations use vertical propeller and submersible pumps. Pump sizes are usually selected to use multiple pumps rather than a single pump of appropriate size. Smaller pumps are usually less expensive to buy and operate, and with multiple pumps the loss of one will not shut down the entire pump station. A single, large pump is more likely to have long term maintenance problems from the frequent start up required to handle flows from smaller events. The sump pump is a much smaller pump, usually designed to handle small amounts of trash or debris loading without failing.
- Motors-Pump motors for department pump stations are usually 480-volt, three-phase electric motors. However, the specific voltage selected depends on the power available from the utility and on what pump-motor combinations are commercially available. The size of each motor depends on the pump size, flow rate, pressure head, and duty cycle. The hydraulic engineer specifying the pumps must work together with the electrical engineer specifying the motors and the control system to insure compatibility of components.
- Control and Communication Systems-The control system for a pump station is more than the sensor and circuitry to activate the pumps when the water in the wet well reaches a predetermined height. The control system includes a large cabinet for the motor control center (MCC) to operate and protect all the motors in the station, separate cabinets for the variable frequency drives (VFD) for the pump motors or any motor that may be expected to operate at less than full speed, and a separate cabinet for the programmable logic controller (PLC). [NOTE: A traffic signal controller is a specialized PLC.] The PLC monitors all signals and controls the sequence of operation of the pumps, activation of the standby generator when necessary, deactivation when the flood event has passed, and operation of any night security lighting. The PLC may also include automatic communication with the District and/or Maintenance Office to report the station's status regarding water levels, pump readiness, utility electrical power status, standby generator battery status, fuel status, security, and other central office concerns. The PLC can be integrated with the ITS to warn motorists of water over the roadway in the event of extreme rain events that exceed the capacity of the pump station. The design of the controls and communications is also the province of the electrical designer. However, the design is dependent on the input information from the hydraulic designer such as wet well capacity, allowed pump discharge rate, desired pump discharge rate, and specific communications.

- Control Board-The pump station should have a central control board for starting or stopping some processes and verifying the various components' conditions, whether "running", "standby", or "off". In addition, although the station may be operated by a control system (PLC or other), a manual override for each component is highly recommended for maintenance and testing. This must be designed by the electrical engineer with input from TxDOT maintenance personnel.
- Structures-The structure must meet requirements for public safety, safety codes, local extreme weather conditions, site security, and maintenance operations. Maintenance requirements may be oversized doors to move equipment in and out or a movable roof to allow crane access. Aesthetics and the possibility of future expansion should also be considered.
- Discharge Conduits-The collected waters are usually discharged to a storm drain system, although sometimes the discharge point is a wetland, mud flat, or creek. The designer must consider whether the receiving location is suitable for the anticipated pump rate, whether it is available during flood events, and whether flood water discharges from the pump station are allowed.
- Acceptance test-A full run acceptance test should be performed successfully before the pump station is accepted. A full run test procedure consists of running the pumps at maximum capacity for at least 6 hours and testing the control systems. During this procedure, the standby generator should be used to power the full station for at least 6 hours which will test the pumps and generator at full load. The discharge conduits can be arranged with a diverter or bypass to pour the pumped water back into the wet well to maintain the full run test.
- Scheduled Maintenance-Pump stations, unlike other hydraulic structures, require scheduled cleaning and maintenance. The trash rack should be cleaned after each storm, while the wet well sump must be cleaned whenever the sediment reaches a set point. The standby generator must be exercised at least once a month for a minimum of 30-minutes run time. The entire system including pumps should be exercised under full load at the same schedule to assure reliability. The discharge diverter or bypass from the acceptance test should be maintained so that it can be used in the scheduled maintenance monthly test.

Section 3 — Pump Station Hydrology

Methods for Design

In order to design a pump station effectively, the inflow hydrology must be known. The hydrology developed for the associated storm drain system usually will not serve as a firm basis for discharge determination into the pump station. A hydrograph is required because the time component is critical in understanding the inflow which governs the sizing of the wet well. The designer needs to know not only the peak inflow, but the timing and volume. The difference between the input and the output hydrographs is the storage requirement of the pump station wet well. The hydrograph should consider the storage abilities of the storm drain system, which may reduce the required size of the wet well. Governmental regulations or the physical limitations of the receiving waters determine the output discharge from the pump station.

The storm drain system associated with the pump station may have a design basis of less than 2% AEP. However, TxDOT recommends at least a 2% AEP flood design because the pump station is generally used when drainage by gravity from a low point is inadequate or impractical.

Procedure to Determine Mass Inflow

A mass inflow curve represents the cumulative inflow volume with respect to time. In order to determine a mass inflow curve, the hydraulic designer must first develop an inflow hydrograph based on a design storm. The most typical design method is the NRCS Dimensionless Unit Hydrograph, discussed in detail in Chapter 4. For the following procedure taken from FHWA Hydraulic Engineering Circular 24 (HEC-24) example, the hydrograph data in Table 4-31 of this manual will be used.

1. Evaluate the time base of the hydrograph and select a time increment, usually the same time increment as that used for developing the inflow hydrograph.
2. Develop a table with columns for time, time increment, inflow rate, average inflow rate, incremental inflow rate, cumulative inflow volume, cumulative outflow volume, and storage difference as shown in Table 11-1.
3. At each time step, extract the inflow rate from the computed inflow hydrograph. (For this example, use Table 4-31, column Q_u).
4. Compute and tabulate the average inflow rate as half of the current and of the previous inflow rates for each time step. (i.e. time step 30: 188/2 cfs + 350/2 cfs = 269 cfs).
5. Compute the incremental volume for each time step as the average inflow rate multiplied by the time step in seconds.
6. Compute the cumulative inflow as the sum of each time step and the previous time step.

7. Plot a curve of cumulative volume versus time. The result is a mass inflow curve, shown as Figure 11-1.
8. Determine the allowable discharge to the receiving waters. The pump flow rate must be at or below the allowable discharge rate. For this example, assume the allowable discharge rate is 100 cfs. Notice that the pumping did not start until a sufficient volume was in the wet well.
9. Multiply the allowable discharge by the time step for the pump flow. Notice that the pumping cannot start until the inflow has developed. The greatest difference between inflow and pump flow is the required storage of the facility. The greatest difference in this example is at time step 80, which is about 691,200 cubic feet. The negative numbers at time steps 230 and 240 indicate that regular pumping should have stopped at about time step 220. The Pump Flow line is also plotted with the inflow curve in Figure 11-1.

Table 11-1: Mass Inflow Computation Table

1	2	3	4	5	6	7	8
Time (minutes)	**Time Increment (seconds)**	**Inflow Rate Q_u (cfs)**	**Average Inflow (cfs)**	**Incremental Inflow (cubic feet)**	**Cumulative Inflow (cubic feet)**	**Cumulative Outflow (pump flow in cubic feet)**	**Storage Difference (cubic feet)**
0		0	0	0	0	0	0
10	600	58	29.0	17,400	17,400	0	17,400
20	600	188	123.0	73,800	91,200	60,000	31,200
30	600	350	269.0	161,400	252,600	120,000	132,600
40	600	400	375.0	225,000	477,600	180,000	297,600
50	600	358	379.0	227,400	705,000	240,000	465,000
60	600	272	315.0	189,000	894,000	300,000	594,000
70	600	170	221.0	132,600	1,026,600	360,000	666,600
80	600	112	141.0	84,600	1,111,200	420,000	691,200
90	600	77	94.5	56,700	1,167,900	480,000	687,900
100	600	51	64.0	38,400	1,206,300	540,000	666,300
110	600	34	42.5	25,500	1,231,800	600,000	631,800
120	600	22	28.0	16,800	1,248,600	660,000	588,600
130	600	15	18.5	11,100	1,259,700	720,000	539,700
140	600	10	12.5	7,500	1,267,200	780,000	487,200
150	600	7	8.5	5,100	1,272,300	840,000	432,300
160	600	4	5.5	3,300	1,275,600	900,000	375,600

Table 11-1: Mass Inflow Computation Table

1	2	3	4	5	6	7	8
Time (minutes)	Time Increment (seconds)	Inflow Rate Q_u (cfs)	Average Inflow (cfs)	Incremental Inflow (cubic feet)	Cumulative Inflow (cubic feet)	Cumulative Outflow (pump flow in cubic feet)	Storage Difference (cubic feet)
170	600	3	3.5	2,100	1,277,700	960,000	317,700
180	600	2	2.5	1,500	1,279,200	1,020,000	259,200
190	600	1	1.5	900	1,280,100	1,080,000	200,100
200	600	0	0.5	300	1,280,400	1,140,000	140,400
210	600	0	0.0	0	1,280,400	1,200,000	80,400
220	600	0	0.0	0	1,280,400	1,260,000	20,400
230	600	0	0.0	0	1,280,400	1,320,000	-39,600
240	600	0	0.0	0	1,280,400	1,380,000	-99,600

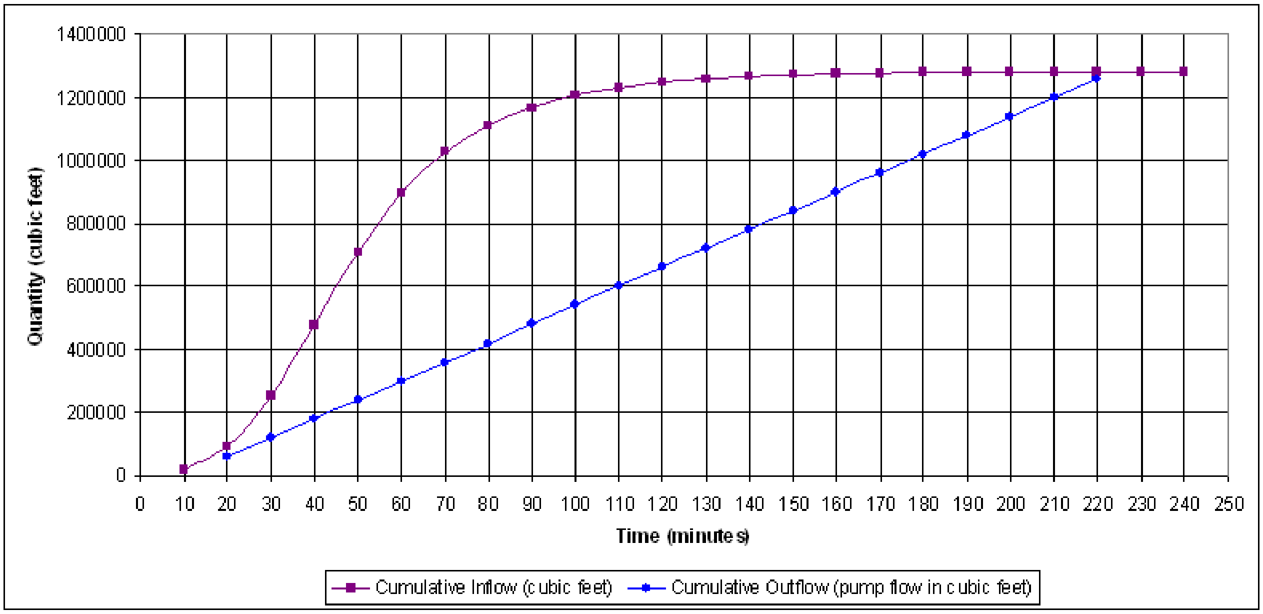


Figure 11-1. Inflow versus Pump Flow

Section 4 — Pump Station Hydraulic Design Procedure

Introduction

The hydraulic design of a pump stations has two major components, the storage design and the pump selection.

Storage Design Guidelines

The storage volume of the wet well should be less than the total volume of the wet well because allowances should be made for a sump and for freeboard. The sump is the volume of the wet well below the required minimum water level, which is the pump cutoff elevation. The wet well must maintain water above the pump inlet to keep the pump from attempting to pump dry or sucking air. The sump must also have room below the pump intake level for sedimentation and heavy trash that wash into the system.

The top of the storage volume determines the maximum water level, the level in the wet well above which the water should not be allowed to exceed. Any freeboard above the maximum water level is not included in the calculated storage volume. Pumping is initiated at or below the maximum water level, and is stopped when the water drops to the minimum water level.

Other spaces outside of the wet well which can store storm water before flooding occurs may also be considered part of the available storage volume. These include sumps, pipes, boxes, inlets, manholes, and ditches of the storm drain system. The storm drain system can represent a significant storage capacity.

Figures 11-2 and 11-3 are a pump station location plan and cross section. The cross section shows how the storm system can provide additional storage outside of the wet well.

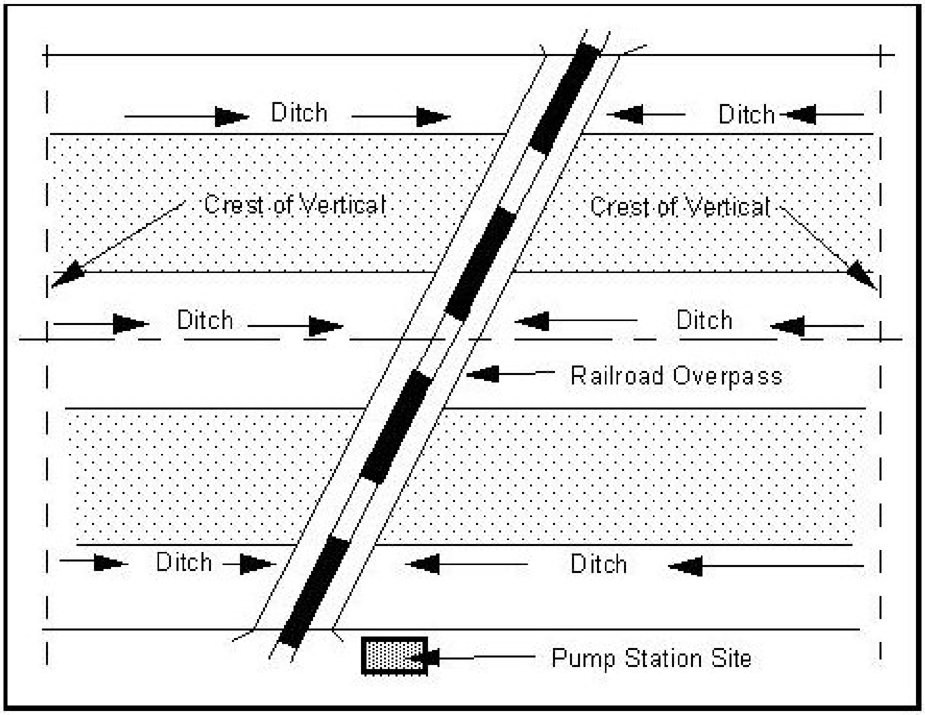


Figure 11-2. Pump Station Schematic

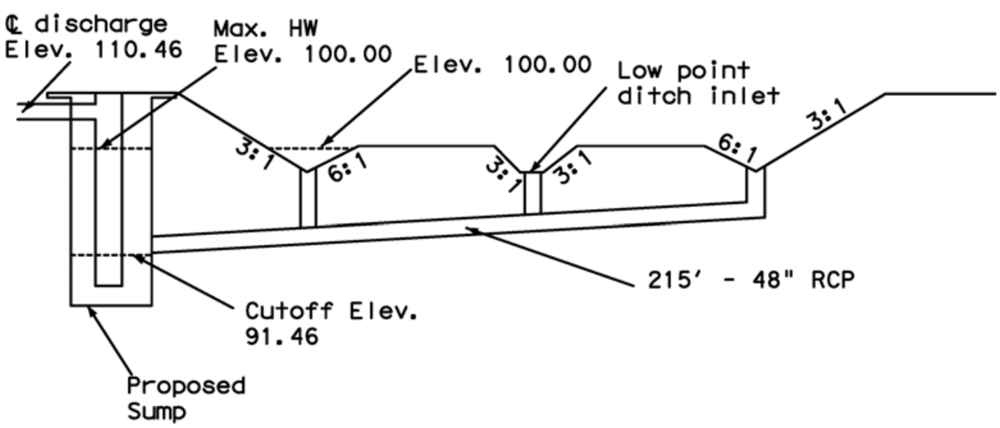


Figure 11-3. Typical Cross Section

Pump Selection

The selected rate of discharge from the pump station determines the number and size of pumps required for the facility. However, pump selection is a matter of economic analysis by the designer. To continue the example above using an allowable discharge of 100 cfs, Table 11-2 lists imaginary pumps data. For a real design, the designer must consult manufacturer's technical data to select a pump or combination of pumps to achieve the allowable discharge.

Table 11-2: Imaginary Pump Data

Pump Designation	Pump Capacity
Type AA	5,000 gpm = 11.1 cfs
Type BB	6,000 gpm = 13.4 cfs
Type CC	7,000 gpm = 15.6 cfs
Type DD	8,000 gpm = 17.8 cfs
Type EE	9,000 gpm = 20.0 cfs
Type FF	10,000 gpm = 22.3 cfs

From Table 11-2, five Type EE pumps (20 cfs x 5) will match the allowable pump rate of 100 cfs. Alternatively, four type FF pumps (22.3 cfs x 4) will yield a pump rate of 89.2 cfs, which is less than the allowable but still a significant discharge. However, the lower pump rate will require a larger wet well volume.

The designer must also consider the cost of construction and physical restrictions for the wet well. Enlarging the wet well and using fewer pumps might be a reasonable alternative to a larger wet well. In situations where one pump may be able to supply the entire discharge necessary, a minimum of two smaller pumps is recommended for reliability and maintenance. Multiple pumps also offer the opportunity for a staggered startup of pumps. Manufacturer's printed technical data and a sales or technical representative can be invaluable sources at this stage of the design in selecting the right pumps. The final design and pump selection must be based on all the considerations together.

Chapter 12 — Reservoirs

Contents:

Section 1 — Introduction

This chapter deals primarily with large reservoirs, the entities responsible for the reservoirs, and their impact on highway facilities and vice versa. TxDOT storm water detention is covered in Chapter 10.

Many TxDOT highways and roadways are located either alongside of a reservoir, cross upstream of a reservoir, or cross downstream of a reservoir. Reservoirs can impact highways by affecting the following:

- the natural storm runoff
- the highway alignment and/or location
- the risk of highway overtopping
- the embankment stability

Section 2 — Coordination with Other Agencies

Reservoir Agencies

Public agencies and entities that sponsor reservoirs include the following:

- U.S. Department of Army, Corps of Engineers (USACE)
- U.S. Department of the Interior, Bureau of Reclamation
- U.S. Department of Agriculture
- Texas Commission on Environmental Quality (TCEQ)
- Various river authorities, such as LCRA, TRA, BRA, and SARA.

See Contacts for information on contacting these and other agencies. Additional sponsors include counties, cities, and political subdivisions such as utility districts and drainage districts. These agencies provide reservoirs for flood control, hydroelectric power, water supply, recreation, and land conservation.

TxDOT Coordination

Any creation of a reservoir or the improvement of a reservoir that may have an impact on a TxDOT roadway requires coordination between the reservoir sponsoring agency and TxDOT. The two regulatory agencies most involved with reservoirs are the TCEQ and the Federal Emergency Management Agency (FEMA) (See Chapter 5). Large reservoir sponsors usually present comprehensive design packages. Private ventures have sponsored reservoirs in Texas, but TxDOT generally does not deal directly with private projects because TxDOT has no process to enforce the private sector's obligation to any contract. Therefore, a reservoir project supported by private funds usually requires a contract dealing with a third party (ordinarily a public agency or entity).

The sponsoring agency is required to analyze the proposal and evaluate all impacts to the roadway, and submit the same to TxDOT for review. Mitigation of adverse impacts to the roadway resulting from construction of a reservoir is the responsibility of the reservoir agency. Adverse impacts to the highway include constant or occasional flooding and roadway damage which require relocation, revision of the highway profile, embankment protection, or adjustment of structures in order to keep the roadway in service. Where a state highway is affected, sponsors should assure the department that they are in compliance with State and Federal permits, floodplain ordinances, and environmental clearances.

Section 3 — Reservoir Analysis Factors

The primary hydraulic factors involved in the analysis of a reservoir include hydrology methods, flood storage potential, and reservoir discharge facilities. Much of the necessary analysis data can be extracted or found in the reservoir design information which should be available through the controlling agency or owner. If the design information is not available, the roadway hydraulic designer may have to develop the necessary data by analyzing the reservoir independently.

Hydrology Methods

Several different methods are available for predicting runoff rates. Some of the more productive methods are described in Chapter 4; however, more sophisticated hydrologic methods may be used. For TxDOT consideration, the peak runoff rate for the drainage area served by a reservoir should be associated with a flood event having a 2% AEP (a minimum recurrence interval of 50 years). The hydraulic designer may determine the magnitude of the 2% AEP event by procedures provided in Chapter 4, specifically the following:

NRCS Curve Number Loss Model.

Texas Storm Hyetograph Development Procedure.

Hydrograph Routing.

A comprehensive hydraulic analysis of a reservoir operation requires a valid or reliable flood hydrograph. The peak discharge alone does not suffice.

Flood Storage Potential

Often, a comprehensive reservoir design provides for sediment storage in addition to flood water storage. Provision of sediment storage helps ensure that the proposed flood water storage is available for a minimum number of years. Nearly all major reservoirs and NRCS flood water retarding structures have sediment storage provisions. In analyzing the storage proposed, only the storage provided for flood water should be considered.

The adequacy of the proposed storage should be checked by routing the hydrograph through the proposed reservoir. Consider the following:

- ordinate/time association of the flood hydrograph
- available reservoir storage
- capacity of the reservoir outlet works.

The factors of the hydrograph, storage, and outlet relations should be considered simultaneously using a routing process. Several flood routing techniques are useful for department analysis. Chapter 4 discusses Reservoir versus Channel Routing.

Reservoir Discharge Facilities

For most reservoirs, the discharge capacity of the various outlet facilities influence flood routing. The administration of the discharge works is a function of the operating procedure for the reservoir. Therefore, it may be useful, in lieu of routing the flood, to secure the design notes and operating schedules from the agency responsible for operating the reservoir. Operational releases can exist for a long period of time and can even threaten the highway with sustained inundation. For this reason, the design notes and operating schedules should be carefully evaluated.

Section 4 — Highways Downstream of Dams

Peak Discharge

Urban development nearly always increases the runoff rate. Therefore, affected counties and municipalities often require that reservoirs be constructed on the primary and secondary drainage channels to minimize the effect that land development has on the storm runoff rate. This type of flood control requirement is a popular and permanent fixture in Texas.

Reservoirs upstream of a highway usually reduce the peak discharge reaching the highway for a selected frequency of storm runoff. This reduction is due to flood storage in the reservoir. Documentation for the design of large reservoirs is ordinarily complete and comprehensive; smaller reservoirs, however, often are not as well documented. Therefore, the TxDOT analysis often requires that the floods be analytically routed through the proposed storage areas to determine whether or not the required or desired reduction in the peak is accomplished.

Scour Considerations

Reservoirs can contribute to clear water scour downstream of the discharge point. Significant sediment deposition usually occurs within the reservoir whenever the reservoir tributary streams have appreciable sediment loading. As a result, water flowing out of the reservoir can be deprived of sediment, causing clear water scour to the banks and around bridge piers.

Design Adequacy

The TxDOT hydraulic designer should confirm with the reservoir agency that the reservoir has been inspected for structural adequacy and hydraulic adequacy. Unless the reservoir is consistently maintained and operated to reduce the flood peak, the reservoir should not be expected to provide consistent flood attenuation for a downstream culvert, bridge, or highway and should be ignored.

Section 5 — Highways Upstream of Dams

New Location Highways

Locating a new highway upstream of a dam and within the influence of a reservoir is usually not practicable for the department. However, if a reservoir must be crossed, the highway profile should be set high enough to reduce the risk of overtopping, and the embankment should be stabilized to prevent deterioration from water saturation and wind effects. This section provides specific criteria for setting the elevation and providing for protection of the highway embankment and structures.

Existing Highways

When a proposed reservoir is expected to impound floodwater on an existing highway location, ust the highway should be adjusted to meet the same conditions of structure size, embankment elevation, and protection that apply to new locations. The roadway should also be upgraded to meet current geometric design standards. All adjustments to the highway are usually the responsibility of the reservoir sponsor, as stated in the Texas Administrative Code 43 TAC 15.54(f). "Department" in this context refers specifically to TxDOT.

"(f) Highway adjustments for reservoir construction.

"(1) Where existing highways and roads provide a satisfactory traffic facility in the opinion of the department and no immediate rehabilitation or reconstruction is contemplated, it shall be the responsibility of the reservoir agency, at its expense, to replace the existing road facility disturbed by reservoir construction in accordance with the design standards of the department, based upon the road classification and traffic needs.

"(2) Where no highway or road facility is in existence but where a route has been designated for construction across a proposed reservoir area, the department will bear the cost of constructing a satisfactory facility across the proposed reservoir, on a line and grade for normal conditions of topography and stream flow, and any additional expense as may be necessary to construct the highway or road facility to line and grade to comply with the requirements of the proposed reservoir shall be borne by the reservoir agency."

Reservoirs that fall into this category are usually major facilities. The reservoir designs are usually well documented and available for the hydraulic designer's use in the analysis.

Minimum Top Establishment

The roadway embankment elevation should be measured at the point of the low shoulder (crown line), as shown in Figure 12-1.

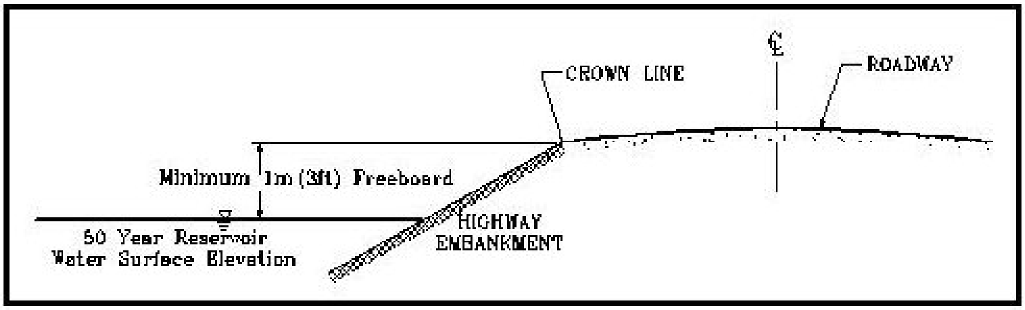


Figure 12-1. Reservoir Freeboard Requirement

The minimum top of embankment elevation should be set no lower than the elevation created by the higher of the following conditions:

- the 2% AEP reservoir surface elevation for the entire reservoir, plus a minimum freeboard of 3-feet,
- the elevation of the 2% AEP flood backwater curve as depicted in Figure 12-2, plus a minimum 3-feet freeboard to the low chord elevation of any structure,
- the elevation of the 0.2% AEP flood backwater for interstate highways and evacuation routes.

At times the crossing may be located on a tributary to the reservoir. Structures so located may at times operate independently from the reservoir, in which case the water surface elevation should be determined according to Chapter 9, Section 4, Hydraulics of Bridge Openings. In cases where the crossing is located on a tributary, the minimum top of embankment should be set to the higher of the reservoir requirements above and the results of the procedure in Chapter 9.

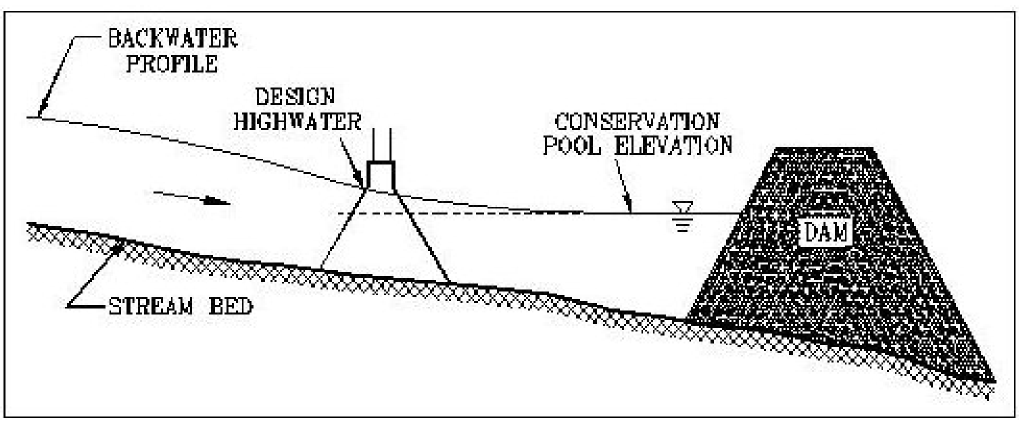


Figure 12-2.

Structure Location

Hydraulic structures located within the reservoir generally need not be situated in accordance with the stream crossing design process and guidelines outlined in Chapter 9 because the velocities are usually so low as to be insignificant. However, additional openings in the highway embankment may be needed for flood flow conditions, and near the borders of the reservoir to ensure reservoir normal circulation.

Scour Considerations

Scour is typically not an issue for a bridge over a reservoir because the velocities are usually so low as to be insignificant. The exception to this is a structure located at the upper end of, or on a tributary of, the reservoir. This structure may at times operate independently from the reservoir, therefore experiencing higher velocities. In this case, scour depths should be calculated as if the bridge were located solely on the tributary.

Section 6 — Embankment Protection

Introduction

The best slope protection type for a given situation depends on the conditions where the installation is to be made, availability of protection material, cost of the various types, and protection desired.

The major reservoir agencies usually can help select the best protection for a given situation.

Embankment Protection Location

Embankment protection is required from the toe of the highway embankment up to an elevation equal to the conservation pool elevation, plus the effects of wind tide and wave runup.

Where the toe of the roadway embankment is below the conservation pool elevation, the minimum elevation of the top of the protection should not be less than 3-feet above the conservation pool elevation. The remaining embankment above the limits of the required protection is an area of lower risk of damage from wind effects than the area affected by wind on the conservation pool. Generally, a vegetative cover with a strong root system is adequate and very economical.

Rock Riprap

The following elements of rock riprap should be considered:

- size – Rock riprap consists of loose rock that is dumped on the slope and distributed. The size of the rock should be large enough that it withstands the forces of wind and water directed at the slope.
- placement – The rock should be placed on a bedding of sand, engineering fabric pinned to the slope, or both a bedding of sand and engineering fabric pinned to the slope. Bedding is primarily for the purpose of keeping the embankment material in place as the embankment is saturated and drained.
- keyed rock riprap – An effective rock riprap variation is keyed riprap. Keyed riprap is rock that has been placed and distributed on bedding upon the slope and then slammed with a very heavy plate to set the rock riprap in place (i.e., to key the rock together). Rock riprap is considered a rough slope when computing wave runup on the slope.

Once the wind effects are known, the weight of the median stone and the total thickness of the riprap blanket can be established using the following equations:

$$W_{50} = \frac{\gamma_s H^3}{K_D \mathrm{Cot}(\alpha)(G-1)^3}$$

Equation 12-1.

$$T = \eta K_{\Delta}\left(\frac{W_{50}}{\gamma_s}\right)^{\frac{1}{3}}$$

Equation 12-2.

where:

W_{50} = weight of the median sized stone (lbs.)

γ_s = specific unit weight of the stone

H = design wave height (ft.)

K_D = riprap stability coefficient, 4.37 is appropriate for TxDOT

α = slope angle from the horizontal in degrees

G = specific gravity of the stone material

W_{max} = weight of the maximum sized stone (lbs.)

W_{min} = weight of the minimum sized stone (lbs.)

T = thickness of the riprap layer (in.)

ç = number of layers of W_{50} (typically taken as 2)

$K_{Ä}$ = layer thickness coefficient (typically taken as 1)

Soil-Cement Riprap

Soil-cement riprap consists of layers of soil cement on the slope placed in prescribed lifts (Figure 12-3). This type of protection provides excellent slope protection. However, inspection and maintenance is necessary, especially at the reservoir water surface elevation that exists most of the time.

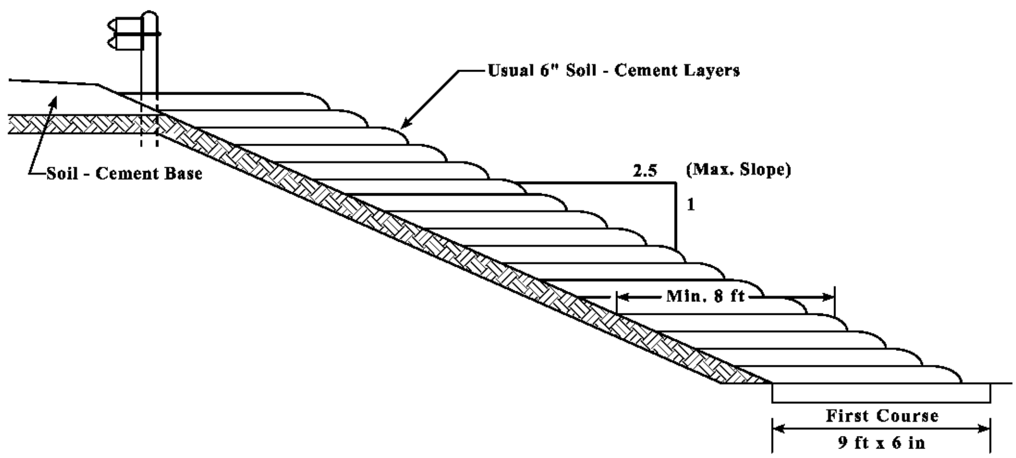


Figure 12-3. Soil Cement Riprap Specifications

Articulated Riprap

Articulated riprap is usually fabricated so that the individual elements are keyed together, and then secured by connecting cables or strands run in two directions to hold the units together. Articulated riprap is usually placed on a filter bed, engineering fabric, or both. The riprap is so named because it is flexible and can move as a unit with the slope and still remain intact. There are several commercial sources of articulated riprap. Each should be evaluated for price, performance, and experience.

Concrete Riprap

Concrete riprap usually consists of slope paving of 4 to 6-inches in thickness. Concrete riprap ordinarily is not recommended for embankment slope protection for highways within a reservoir. This is because the hydrostatic head that can exist in the embankment after it is wet cannot be relieved adequately through the concrete riprap. The riprap may bulge and fail because it does not have the structural integrity necessary to withstand the hydrostatic head of the trapped water.

Concrete riprap can be useful for short sections when placed on a bed of coarse filter material with numerous drain holes located in the riprap, and in an area where the embankment does not have standing water on the slope. There should not be constant differentials in the water surface that might cause prolonged periods of wetting and drying of the embankment.

Vegetation

The use of vegetation with large, strong root systems is a common and economical way to protect slopes. Vegetation protection can be useful on embankment slopes in a reservoir where wind effects are mild.

Chapter 13 — Storm Water Management

Contents:

Section 1 — Introduction

Storm Water Management and Best Management Practices

Storm water is defined in the Construction General Permit (CGP) as "Rainfall runoff, snow melt runoff, and surface runoff and drainage." For TxDOT purposes, storm water includes overland flow, and flow in ditches and storm drain systems.

Storm water management includes non-structural and structural measures such as the following:

- erosion control to minimize erosion and sediment transport
- storm water detention and retention systems to reduce peak runoff rates and improve water quality
- sedimentation and filtration systems remove debris, suspended solids, and insoluble pollutants
- vegetation buffers to reduce transport of pollutants.

Measures intended to mitigate storm water quantity and quality problems are termed “best management practices” (BMPs). These measures include detention and retention ponds which delay storm water flow and trap sediment, rock filter dams for the same reasons, silt fences to trap sediment, various filter materials in socks or tubes, and vegetation to retard flow and trap sediment.

Quantity

Urbanization, which includes transportation activities, increases storm water volume and velocity by increasing the amount of impervious cover. Improved storm drain systems increase the rate of runoff from a location such as a roadway or land development. Recognition is growing that rapid disposal of runoff from developing areas increases the frequency of flooding in downstream areas. The results can increase flooding, soil erosion, sedimentation, stream bank erosion and channel enlargement, and pollution of surface and subsurface waters.

Where developed areas already exist are downstream of more recent development, as is the predominant sequence of development in the United States, flooding reduces property values and may lead to abandonment of property. Massive investments in flood control works are sometimes required to reduce flood damage. The alternative is to provide flood protection by storm water management in the upstream developing areas. Where pollution abatement as well as flood control is an objective, additional or alternative storm water management measures may be necessary to provide source control of storm water pollution.

Water quality problems in surface waters often stem from nonpoint as well as point sources of pollution. A point source is a single identifiable localized source of pollution while a nonpoint source comes from diffuse sources, such as polluted runoff from agricultural areas draining into a river.

Water quality goals for surface waters cannot be achieved solely by separation of combined sewers but require abatement of pollution from nonpoint sources as well.

Highway construction, operation, and maintenance contribute a variety of pollutants to surface and subsurface water. Solids, nutrients, heavy metals, oil and grease, pesticides, and bacteria all can be associated with highway runoff. Although the impacts of highway runoff pollution on receiving waters may not be significant, it is generally recognized that responsible agencies may be required by federal and state regulations to apply the BMP available to reduce pollutant loads entering a water body. One of the primary objectives of an Environmental Impact Statement (EIS) is the quantification of possible pollutants emanating from the operation and maintenance of highway and other transportation facilities, so that a sound judgment can be made as to the overall usefulness of the facility. (For more information on EIS, refer to the *Environmental Documentation* in the *Project Development Process Manual*.)

Requirements for Construction Activities

Appropriate BMPs are recommended for all construction projects.

Storm Drain Systems Requirements

The U.S. Environmental Protection Agency (EPA) National Pollution Discharge Elimination System (NPDES) permit requirements for Municipal Separate Storm Sewer Systems (MS4) are the primary regulations that may affect the extent to which storm water BMPs are necessary. The Division of Environmental Affairs should be consulted to determine the status of the permit and the management plan for the municipality of interest.

In addition to NPDES permit requirements, over the Edwards Aquifer recharge zone, TxDOT is obligated to comply with a memorandum of understanding with the TNRCC that espouses the need for BMPs. Refer to the Division of Environmental Affairs for details of the most current agreement.

Section 2 — Soil Erosion Control Considerations

Erosion Process

Understanding erosion is necessary as a basis for adequate control measures. Erosion is caused by rainfall, which displaces soil particles on inadequately protected areas, and by water running over soil, carrying some soil particles away in the process. The rate of soil particle removal is proportional to the intensity and duration of the rainfall and to the volume and characteristics of the water flow and soil properties. Deposition of water-borne sediment occurs when the velocity decreases and the transport capacity of the flowing water becomes insufficient to carry its entire sediment load.

Schematically, Figure 13-1 illustrates the typical forces involved in soil erosion.

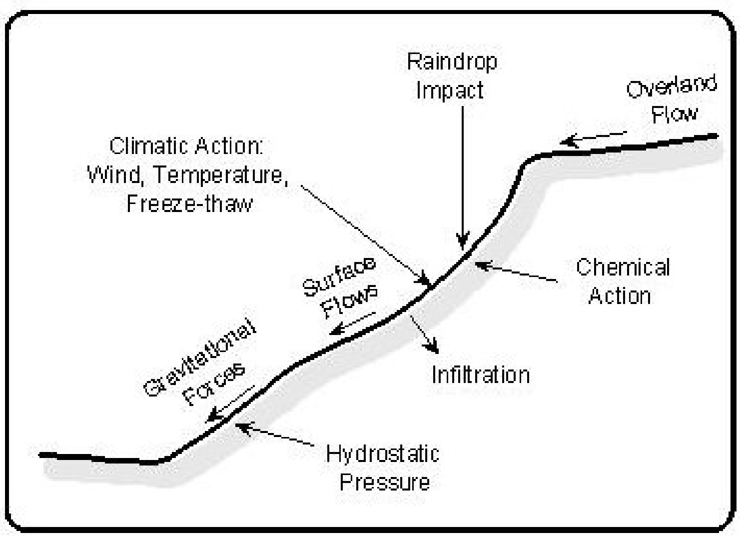


Figure 13-1. Typical Forces in Soil Erosion

It is usually not practical for the department to reduce erosion generated upstream of the highway. If possible, locations with high erosion potential should be avoided. In areas of considerable natural erosion and accelerated erosion, the quantity of sediment that reaches a stream before highway construction begins should be documented in a descriptive or qualitative way.

Damage that can occur on highway projects is not limited to the construction site. Sedimentation or degraded water quality may occur far downstream from the point where erosion occurs. The potential for damage exists because highways pass through watersheds, disrupting the natural drainage pattern. In addition, highway construction requires the removal of existing vegetation and the introduction of cuts and fills. This exposes large areas of disturbed soil, which increases the erosion hazard.

The potential for erosion is minimized by the following measures:

- flat side slopes, rounded and blended with natural terrain
- drainage channels designed with due regard to width, depth, slopes, alignment, and protective treatment
- protection at culvert outlets
- proper facilities for ground water interception
- dikes, berms, and other protective devices
- protective ground covers and plantings.

Erosion is a natural process that human activities often accelerate. Erosion and sedimentation are usually undesirable from an environmental standpoint. Technical competency in evaluating the severity of erosion problems and in planning and designing preventive and corrective measures is essential toward the goal of obtaining economical and environmentally satisfactory methods for erosion control.

Individuals involved in the process of controlling erosion and sedimentation include planners, designers, construction engineers, project inspectors, and contractors.

Effective and practical measures are available to minimize the erosion hazards and prevent sediment from reaching streams. Preventive measures taken during construction are more effective and economical than corrective measures. Erosion control involves the prevention of soil movement while sediment control deals with the interception of sediment-laden runoff and separation of soil particles already in motion or suspension. Erosion control at the source is the first consideration with sediment control the backup or last resort. Contact the DES-HYD for detailed information.

To deal adequately with the erosion and sediment problem, the erosion and sedimentation processes must be understood, erosion and sediment control plans must be developed, construction operations for erosion and sediment control must be scheduled, specific erosion and sediment control measures (when, where, and how) must be constructed, and water quality must be monitored and maintained.

The following general guidelines are considered BMPs:

- Select a route where erosion will not be a serious problem.
- Design slopes to be flatter than with soil limitations.
- Reduce the area of unprotected soil exposure.
- Reduce the duration of unprotected soil exposure.
- Protect soil with vegetative cover, mulch, or erosion resistant material.
- Retard runoff with planned engineering works.

- Trap sediment using temporary or permanent barriers, basins, or other measures.
- Maintain erosion control work, both during and after construction.
- Obtain easements for legal control, where necessary.

Natural Drainage Patterns

The natural drainage pattern, including subsurface flow, must be examined for the alternate routes considered. The drainage pattern beyond the vicinity of the proposed highway location must also be studied either to minimize and avoid damage to adjacent property or streams, or to anticipate expensive preventive or corrective measures. In consideration of design work on existing roadways, the established patterns of drainage (as contrasted to natural patterns) must be examined.

Stream Crossings

Crossings should be made as nearly as practical at a right angle to the direction of flow. Emphasis should be given to the direction of the flood flow where it is different from that of the low water. The direction, rate, and volume of flood flow at various stages in the location of bridge openings should always be considered. A highway built on the neck of a horseshoe bend that is subject to overflow is poorly located because the correct location of relief bridges sometimes varies with the flood stage.

Whenever practical, stream crossings should be at stable reaches of a stream. Meanders in the stream that are subject to shifting should be avoided. Meandering streams have inherent problems of having no stable place to cross because the sinusoidal pattern of the stream naturally tends to progress in a downstream direction.

The number of stream crossings and the disturbance of streambeds should be minimized. Crossing and then re-crossing the same stream should be avoided. Undue scour and erosion that might result in a complete change in the river channel should be avoided.

See Chapter 9, Planning and Location Considerations, for more details on planning and location.

Encroachments on Streams

If a proposed highway alignment will encroach upon a stream, consider moving the highway away from the stream to avoid erosion and sedimentation problems. For an existing roadway that already encroaches on or near a stream, improvements or rehabilitation work should be planned to minimize further encroachment. If the stream impinges and encroaches on the highway, the highway itself may need to be protected.

Public and Industrial Water Supplies and Watershed Areas

If possible, crossing of a catchment area of a water supply should be avoided. Such crossings could entail building costly temporary facilities for the water supply. Some industries require higher quality water than is required for drinking water, so problems with industrial water supplies may be as great as those with a public water supply. When crossing a water supply catchment area cannot be avoided, any corrective measures and their costs should be determined before making the choice of the route.

Geology and Soils

Knowledge of the area's geology allows the highway designer to detect potential problem areas and anticipate subsidence, landslides, and erosion problems. Terrain features are the result of past geologic and climatic processes. Erosion and deposition by running water are major geologic processes in shaping the terrain. A study of the terrain and the character of natural and accelerated erosion can aid in judging the complexity of the erosion and in estimating what erosion control measures may be required.

Some soil types are known to be more erosive than others, and their identification is a valuable aid in route selection and erosion control. The U.S. Department of Agriculture classification of soils is helpful. Soil survey maps, prepared by the Natural Resources Conservation Service (NRCS), show this classification as well as the engineering classification of soils. Local NRCS offices can give much assistance in both soil identification and erosion control measures applicable to the local area.

Problems in route selection for a new roadway can sometimes be avoided. For an existing roadway, however, problems must be recognized and precautions taken in the design.

Coordination with Other Agencies

Plans or projects of other agencies, such as the USACE, NRCS, and TCEQ, might affect or be affected by the location of a proposed highway, or by improvements or changes to an existing roadway. These agencies should be contacted to learn of their plans for controlling bank erosion, protective works, and stream grade control structures or channel modifications.

Roadway Guidelines

Independent roadway grade lines that fit the terrain with a minimum of cuts and fills reduce exposed areas subject to erosion. Alignment and grade, consistent with highway safety criteria, must be blended or fit to the natural landscape to minimize cut and fill sections and reduce erosion and costly maintenance. Slopes of the roadway cross section should consider soil stability, climatic exposure, geology, proposed landscape treatment, and maintenance procedures.

Depressed roadways and underpasses require careful consideration of drainage to avoid deposition of sediment and debris on the highway and in drainage facilities. Both ground and surface water can do the following:

- pass through the highway right-of-way
- be intercepted with minimum disturbance to streams
- be intercepted without causing serious erosion problems.

The cross section can be varied, if necessary, to minimize erosion and to facilitate safety and drainage. Generally, good landscaping and drainage design are compatible with both erosion control and safety to vehicles. Right-of-way constraints often prohibit extreme flattening of embankment slopes, but they should be an important consideration to the designer in their effect on erosion.

Severe Erosion Prevention in Earth Slopes

A concentration of storm water flowing from the area at the top of cut or fill slopes causes severe erosion of earth slopes. The concentration of storm water at the top of cuts should be avoided. These guidelines should be followed in areas of severe erosion prevention in earth slopes:

- Dike or berm construction – During project construction and immediately thereafter, construct a dike or berm at the top of the cut to prevent water from running down the slope. The dike or berm should be borrow material to avoid disturbing the natural ground, in conjunction with a grassed channel or paved ditch.
- Outlet protection – Water can be spread over the natural slope or carried to lower elevations in chutes or closed pipes. Protect outlets for such high velocity chutes from scour. Streams in cut sections require special attention.
- Serrated slopes – In some areas of Texas, serrated cut slopes help establish vegetative cover on decomposed rock or shale slopes. Serrate any material that is rippable or that will hold a vertical face for a few weeks until vegetation becomes established.
- Shoulder drains -- Where vegetation cannot be established or where flow down the fill slope is objectionable, collect the runoff at the shoulder edge and direct it to an adequate inlet and chute.

Channel and Chute Design

Surface channels, natural or man-made, are usually the most economical means of collecting and disposing of runoff in highway construction if concentration of flows cannot be avoided. A well-designed channel carries storm water without erosion or hazard to traffic and with the lowest overall cost, including maintenance. To minimize erosion and avoid a safety hazard, channels should have mild side slopes and wide rounded bottoms. Such channels can be protected from erosion by lining them with materials such as grass, rock, or concrete.

Chutes generally are applied to steep slopes and carry water at high velocities. Pipe chutes are preferable to open chutes because the water cannot jump out of the chute and erode the slope. Dissipation of energy along the chute or at the outlet is usually necessary. In highly erosive soil, watertight joints may need to be provided to prevent failure of the facility.

Variations in channel alignment should be gradual, particularly if the channel carries flow at high velocity. Whenever practical, changes in alignment should be located on the flatter gradients to prevent erosion caused by the overtopping of the channel walls and the associated erosion. Although rectangular channel sections are usually more expensive, they are preferred on bends of paved channels to give a more positive control of the flow.

If the bank and bed material will erode at the prevailing velocities, channel lining should be considered. Protective linings for channels and streams can be very expensive. A special effort should be made to develop the most cost-effective erosion protection, including maintenance, for the particular location.

Several applications are effective for both channel and bank protection, including spur dikes, permeable spur jetties, gabions and revetment mattresses, and sheet piling. For many of these protective appurtenances, no rigorous design is available, and experience or intuition is the best guides for their consideration and application. See Chapter 7, Channel Linings, for more information.

Culverts and bridges generally constrict the floodway and increase velocities, thus developing higher erosion potential. In many instances, erosion and scour at these locations damage the highway embankment, the structure itself, or the downstream channel. The energy of high velocity flow should be dissipated at the outlet of culverts and chutes where necessary, or the area protected by riprap or other types of protection. Some velocity control devices and methodologies are illustrated in Chapter 8, Velocity Protection and Control Devices.

Section 3 — Inspection and Maintenance of Erosion Control Measures

Inspections

Preventive maintenance built into the highway design and construction phases will decrease maintenance costs. Experts in soil conservation, agronomy, and drainage can assist in maintenance inspections and in recommending appropriate erosion control measures. Periodic inspections of drainage and erosion control measures should be conducted shortly after completion of construction so that deficiencies can be located and corrected before they develop into major problems. Deficiencies in design or in construction procedures should be discussed with the engineering staff to avoid similar deficiencies on future projects. Coordination of responsibilities for erosion control measures among design construction and maintenance sections is encouraged.

Embankments and Cut Slopes

Embankments and cut slopes are especially vulnerable to erosion. Maintenance equipment operators should be made aware that damage to ground cover at such locations can create serious erosion problems that are difficult to correct. Surveillance of these areas by maintenance personnel should be emphasized because such areas are not easily seen from the roadway.

Channels

Channels, whether active streams or open roadside ditches, are vulnerable to erosion, especially after construction. Maintenance personnel should inspect these facilities periodically and after significant storms for any erosion that will require remedial work.

Intercepting channels should be kept clean and free of brush, trees, tall weeds, and other material that decreases the capacity of the channel. When channel deterioration reduces channel capacity, overflow may occur more frequently, and erosion or deposition in the area adjacent to the channel may take place. Natural channels that are parallel to the roadway embankment may be best maintained in their natural state. This reduces the probability of embankment erosion.

High velocity flow in chutes or ditches often overtops the sides and erodes the adjacent area. Care must be taken to inspect for holes and eroded areas under paved channels to prevent collapse of rigid sections. Projections and joint offsets that cause splash and possible erosion should be removed or repaired. The channel entrance should not permit water to flow either along the side or underneath the channel.

Periodic inspection of channel changes is necessary to avoid costly repairs. Failures during construction should be carefully analyzed before performing remedial work because changes in the original construction may be indicated.

Repair to Storm Damage

Storm damage should be repaired as quickly as possible in order to avoid additional damage. Such damage may indicate that additional protection is needed. A damaged area restored only to its pre-flood condition usually will be damaged again when a flood of similar magnitude recurs.

Erosion/Scour Problem Documentation

When maintenance personnel discover excessive scour or erosion near a bridge or other major drainage structure, those responsible should be advised so that they can take proper actions to protect the structure. A system of record keeping and documentation regarding erosion/scour problems and flood events respective to highway facilities should be established and maintained.

Section 4 — Quantity Management

Impacts of Increased Runoff

For TxDOT applications, storm water quantity management mitigates the potential effects of increased runoff rates and volumes that can often accompany development, including highway construction. These effects include increased erosion and sedimentation, increased pollutant loads, and increased flood levels and velocities. By assessing the potential for increased runoff volume and, if necessary, taking measures to offset such increases, the department can minimize the potential for detrimental impact due to storm water runoff.

Storm Water Quantity Management Practices

Storm water runoff can be collected and disposed of through an integrated system of facilities. Storm drain systems collect the runoff water initially, and it is then handled by the following:

- pumping stations
- detention systems
- retention systems
- sedimentation basins
- hazard spill tanks
- bio-filtration systems
- outfall appurtenances
- outfall channels
- man-made wetlands.

The primary options for handling or mitigating increased runoff are detention, retention, outfall appurtenances, and outfall channels.

Measures for controlling urban storm runoff can be classified as structural or non-structural. Structural measures require the construction of certain facilities, such as detention basins for temporarily storing storm runoff, thus reducing and delaying runoff peaks. The hydrologic methods for analysis of detention and retention systems are detailed in Chapter 4, Reservoir Versus Channel Routing. Chapter 10 details storm drain system planning and design considerations, Chapter 11 gives pumping stations design and operation considerations, and outfall channel design and operation considerations and procedures are detailed in Chapter 7, Channel Analysis Methods.

Non-structural measures include such practices as land use management to strategically locate impervious areas so that the resulting total hydrograph peak is less severe. TxDOT rarely is involved in non-structural measures in association with transportation projects. Table 13-1 lists some of the measures for reducing and delaying urban storm runoff recommended by the NRCS.

Table 13-1: Measures for Reducing and Delaying Urban Storm Runoff

Area	Reducing runoff	Delaying runoff
Large flat roof	Cistern storage Rooftop gardens Pool storage or fountain storage Sod roof cover	Ponding on roof by constricted downspouts increasing roof roughness: ◆ Ripples roof ◆ Gravelled roof
Parking lots	Porous pavement: ◆ Gravel parking lots ◆ Porous or punctured asphalt Concrete vaults and cisterns beneath parking lots in high value areas Vegetated ponding areas around parking lots Gravel trenches	Grassy strips on parking Grassed waterways draining parking lot Ponding and detention measures for impervious area: ◆ Rippled pavement ◆ Depressions ◆ Basins Reservoir or detention basin
Residential	Cisterns for individual homes or group of homes Gravel driveways (porous) Contoured landscape Groundwater recharge: ◆ Perforated pipe ◆ Gravel (sand) ◆ Trench ◆ Porous pipe ◆ Drywells Vegetated depressions	Planting a high delaying grass (high roughness) Gravel driveways Grassy gutters or channels Increased length of travel of runoff by means of gutters, diversions, etc.
General	Gravel alleys Porous sidewalks Hed planters	Gravel alleys

Of the measures listed in Table 13-1, detention basins or ponds, either dry or wet, are the most commonly used practices for controlling storm runoff. These facilities serve to attenuate flood peaks and flood volumes. Retention basins also are used in some instances when the total runoff volume can be stored permanently.

Refer to Chapter 4 for details of hydrograph routing by the Reservoir Versus Channel Routing. The extent to which storage is provided is left to engineering judgment. You should aim to balance the risk of impact with the costs of providing storm water quantity control.

Chapter 14 — Conduit Strength and Durability

Contents:

Section 1 — Conduit Durability

Introduction

When designing a culvert or storm drainage system, you must evaluate aspects of structural design, hydraulic design, and durability design. The first two disciplines are quite familiar to most civil engineers. Durability design, however, is generally beyond the scope of civil engineering and is more closely aligned with the field of chemistry. Experience has shown that culverts most frequently fail as a result of durability problems. This is usually due to improper selection of materials to meet the project design life and site conditions.

Service Life

For permanent TxDOT hydraulic facilities, an ideal service life expectancy is generally 50 years. However, the scope and intended use of the facility and economic considerations may warrant longer or shorter service life. Many factors affect durability, each independently affecting different aspects of the facility:

- corrosion
- abrasion
- choice of material
- design of the facility
- maintenance practices
- consistency of the local site environment

With knowledge of these factors, the designer should exercise some control over choice of material, design of the facility, and maintenance practices.

Relative service life of conduit material is a function of the corrosion/abrasion cycle. You can predict the relative service life based on the evaluation of soil and water site characteristics such as the following:

- Acidity/alkalinity -- The universal measure for acidity/alkalinity is the pH scale. Acidity can result from either mineral or organic sources. Mineral acidity can be the result of leaching of acidic soil, runoff from mining activities, and acidic rainfall. Organic acidity may result from organic decay such as runoff from a large feedlot. Relative service life of materials used in conduits is a function of the pH value of the soil and water. High acidic values in the soil and water ($pH<4$) represent a greater threat to the conduit material service life. High alkalinity values in the soil and water ($pH>9$) also represent a significant threat to the conduit material service life.

- Resistivity -- Resistivity is a measure of the electrical current carrying capacity of a material. If the resistivity value (expressed in ohm-cm) is low, the current carrying capacity is high. In such a case, the potential for corrosion is also high. In general, the higher the resistivity, the lower the potential for corrosion due to resistivity.
- Abrasion -- Abrasion is a function of flow velocity and bedload. High flow velocity and the presence of an abrasive bedload in the water cause scour or erosion to the conduit material. Abrasive bedloads are typically not transported when flow velocities are less than 5 fps (1.5 m) per second. While this is a damaging mechanism leading to deterioration and further exposure for the mechanism of corrosion, it is not a common problem in most parts of Texas. In very hilly and rocky areas, consider abrasion as a possible threat to the expected service life of the conduit.

The hydrogen ion content (pH) of the soil and water and the resistivity of the soil and water determine the relative effect of a site on the durability of a drainage structure. The geotechnical report of the highway project may include information regarding pH values and resistivity values for soil and water associated with the project. Particularly sensitive cases may justify determining pH and resistivity values at specific facility sites.

Where corrosion is a threat, consider structure material choice and possibilities of material protection. Under no circumstances arbitrarily select the structure material. In some instances due to specific experiences with various materials, local practice or policy may dictate use of certain materials in drainage facilities. Where policy dictates selection of the material, document the basis of the policy.

For alkalinity or acidity and for resistivity consider all soils in contact with the culvert conduit, inside or outside, including:

- native soil at the culvert site
- soil used in the roadway embankment in the area
- soil used as culvert backfill

Acidity in the water may occur in either the runoff water or the ground water in the area of the facility.

The resistivity value correlates directly with the salt content of the soil or water. The presence of salts in the soil or water at a facility site can affect both the pH value and the resistivity. Calcium carbonate inhibits corrosion, and certain chlorides and sulfates increase the potential of corrosion. Generally, the project geotechnical report will address the salt characteristics of soils and water if the resistivity is greater than 7,500 ohm-cm.

Evaluate the abrasion level of the drainage facility. Select conduit material and conduit protection based on the abrasion level. Abrasion is classified by the following levels:

- Level 1 - non-abrasive - little or no bedload and very low velocities (less than 5 fps or 1.5 m per second)
- Level 2 - low abrasive - minor bedloads of sand and low velocities (less than 5 fps or 1.5 m per second)
- Level 3 - moderate abrasive - moderate bedloads of sand and gravel and average velocities (5 to 15 fps or 1.5 to 4.5 m per second)
- Level 4 - severe abrasive - heavy bedloads of sand, gravel, and rock, and high velocities (greater than 15 fps or 4.5 m per second)

Countermeasures to level 3 and level 4 abrasion may include one or a combination of the following:

- reducing the flow velocities in the conduit.
- for metal pipes, selecting a heavier gage metal (sacrificial material).
- burying the invert of the conduit.
- for metal pipes, installing invert protective linings such as bituminous paved invert, concrete paved invert, bituminous lining, and concrete lining.

Section 2 — Estimated Service Life

Corrugated Metal Pipe and Structural Plate

Determine the service life of corrugated metal structure by calculating the service life of the exterior and interior of the pipe using the site characteristics for the soil and water discussed in the previous section. The overall service life will be the lesser of the interior service life or exterior service life. The service life of a corrugated metal conduit is expressed by the sum of the base metallic coating, post applied coating, and paving or lining service life, as in Equation 14-1 and Equation 14-2:

$$SL_{INT} = \sum SL_{BMCI} + SL_{PACI} + SL_{LI}$$

Equation 14-1.

$$SL_{EXT} = \sum SL_{BMCE} + SL_{PACE}$$

Equation 14-2.

where:

SL_{INT} = service life of the interior of the pipe

SL_{EXT} = service life of the exterior of the pipe

SL_{BMCI} = service life of the base metallic coating interior

SL_{BMCE} = service life of the base metallic coating exterior

SL_{PACI} = service life of the post applied coating interior

SL_{PACE} = service life of the post applied coating exterior

SL_{LI} = service life of the paving or lining interior

Corrugated Steel Pipe and Steel Structural Plate

The base metallic coating data provided in this section are limited to the following values for galvanized metals:

- $6 < pH < 8$
- resistivity ≥ 2,000 ohm-cm
- soft waters considered hostile when resistivity ≥ 7,500 ohm-cm

For aluminized type 2, the following values apply:

- $5.0 < pH < 9.0$; Resistivity > 1,500 ohm-cm
- soft waters not considered to be a problem

Estimate the service life for the interior base metallic coating using Equation 14-3

$$SL_{BMCI} = (\text{basic interior service life}) \times (\text{thickness multiplier})$$

Equation 14-3.

The basic interior service life for 18-gage corrugated galvanized metal pipe is provided in the table following Equation 14-4 for pH values of 7.3 and lower and using the equation for pH values in excess of 7.3.

$$L_i = (1.25)(1.47)R^{0.41}$$

Equation 14-4.

where:

L_i = interior years

R = resistivity (ohm-mm)

Exterior Coating

Estimate the service life for the basic exterior base metallic coating using Equation14-5.

$$SL_{BMCE} = (\text{basic exterior service life}) \times (\text{thickness multiplier})$$

Equation 14-5.

The basic exterior service life (L_e) for 18-gage corrugated galvanized metal pipe is provided in the table following Equation 14-6 for pH values of 7.3 and lower and using the equation for pH values in excess of 7.3.

$$L_e = (2.0)(1.47)R^{0.41}$$

Equation 14-6.

Exterior Durability for 18-Gage CMP (years)

pH	**Resistivity (ohm-cm)**								
	1,000	**1,500**	**2,000**	**2,500**	**3,000**	**4,000**	**5,000**	**7,500**	**10,000**
7.3	54.8	59.6	63.1	65.8	67.9	71.4	74.1	78.9	82.4
7.0	34.6	39.4	42.9	45.6	47.7	51.2	53.9	58.7	62.2
6.5	23.9	28.8	32.2	34.9	37.1	40.5	43.2	48.0	51.5
6.0	18.0	22.9	26.3	29.0	31.2	34.6	37.3	42.1	45.6
5.8	16.2	21.0	24.5	27.2	29.3	32.8	35.5	40.3	43.8
5.5	13.8	18.6	22.1	24.8	26.9	30.4	33.1	37.9	41.4

Exterior Durability for 18-Gage CMP (years)

pH	Resistivity (ohm-cm)								
5.0	10.4	15.3	18.7	21.4	23.6	27.0	29.7	34.5	38.0

Heavier gage metal has more sacrificial metal and, therefore, a longer anticipated life under given conditions. The table below provides coating thickness/gage multipliers for use in Equation 14-1 and Equation 14-2 for the respective gage and metallic coating. The resulting values are not exact but allow a systematic comparison of relative durability of the various metals and gages used in design.

Thickness Multipliers for Steel Conduit

Gauge	Item 460 - CMP			Item 461 - Structural Plate	
	Thickness	Factor		Thickness	Factor
	in. (mm)	Galv	Alt 2	in. (mm)	Galv
18	0.052 (1.32)	1	3.6	**	**
16	0.064 (1.63)	1.3	3.9	**	**
14	0.079 (2.01)	1.6	4.2	**	**
12	0.109 (2.77)	2.2	4.8	0.109 (2.77)	2.24
10	0.138 (3.50)	2.8	5.4	0.138 (3.50)	2.84
8	0.168 (4.27)	3.4	6	0.168 (4.27)	3.54
7	**	**	**	0.188 (4.78)	3.81
5	**	**	**	0.218 (5.54)	4.42
3	**	**	**	0.249 (6.32)	5.05
1	**	**	**	0.280 (7.11)	5.68

Corrugated Aluminum Pipe and Aluminum Structural Plate

The service life of aluminum pipe and aluminum structural plate is a function of the pitting rate of the aluminum, which is less than 0.013 millimeter per year in the following environmental limits:

- $4.0 \le pH \le 9.0$
- resistivity ≥ 500 ohm-cm
- resistivity ≥ 25 ohm-cm (provided a free draining backfill material)
- no upper resistivity limits; soft waters not a problem

Estimate interior service life (SL_{BMCI}) and exterior service life (SL_{BMCE}) using Equation14-7.

$$SL_{BMCI} = SL_{BMCE} = \frac{(\text{metal thickness})}{0.0005\,\frac{\text{in.}}{\text{yr}} \text{ or } 0.0127\,\frac{\text{mm}}{\text{yr}}}$$

Equation 14-7.

The following table shows gage thickness and available structural plate thickness.

Aluminum Pipe Gage Thickness

Item 460 – CMP			Item 461 – Structural Plate		
Gage	**Thickness**		**Gage**	**Thickness**	
	(in)	(mm)		(in)	(mm)
18	0.048	1.22	**	**	**
16	0.06	1.52	**	**	**
14	0.075	1.91	**	**	**
12	0.15	2.67	**	0.1	2.54
10	0.135	3.43	**	0.125	3.18
8	0.164	4.17	**	0.15	3.81
**	**	**	**	0.175	4.45
**	**	**	**	0.2	5.08
**	**	**	**	0.225	5.72
**	**	**	**	0.25	6.35

Post-applied Coatings and Pre-coated Coatings

The following table provides anticipated additional service life for post-applied and pre-coated coatings (SL_{PACI} and SL_{PACE}) for use in Equation14-1 and Equation14-2.

Post-applied and Pre-coated Coatings Guide to Anticipated Service Life Add-On (additional years)

Coating	Interior (SL_{PACE})				Exterior (SL_{PACE})
	Abrasion Level				
	Level 1	Level 2	Level 3	Level 4	
Bituminous	8-10	5-8	0-2	0	30
Polymer 10/10	28-30	10-15	0-5	0	30

Paving and Lining

The following table provides additional service life for applied paving and lining (SL_l) for use in Equation14-1.

Post-applied Paving and Lining Guide to Anticipated Service Life Add-On

Paved Or Lined	Interior				Exterior
	Abrasion Level (SL_{LL})				
	Level 1	Level 2	Level 3	Level 4	
Bituminous Paved Invert	25	25	25	0	N/A
Concrete Paved Invert	40	40	40	25	N/A
100% Bituminous Lined	25	25	25	0	N/A
100% Concrete Lined	50	50	50	35	N/A

Reinforced Concrete

There is little technical data on methods to estimate service life for reinforced concrete. In department experience when cast-in-place and precast reinforced conduit is used in appropriate environments, service life exceeds the original design life of the project (typically in excess of 50 years).

Durability of reinforced concrete can be affected by acids, chlorides, and sulfate concentrations in the soil and water. If the pH value is 6.5 or less, the use of porous concrete pipe with shell thickness of 1 in. (25 mm) or less is not advisable. If the pH value is 5.5 or less, use of reinforced concrete without a protective coating of epoxy or other acceptable coating is not advisable.

Salt content of the soil and water can have a detrimental effect on reinforced concrete because the salt (with its chloride constituent) can permeate the concrete in time, threatening the embedded reinforcing steel. Sulfate content in the soil or water can have a detrimental effect on reinforced concrete facilities. The following table presents a guide for adjusting cement type and factor for sulfate content in soils and runoff.

Guide for Sulfate Resisting Concrete

Water-soluble sulfate in soil sample (%)	Sulfate in water sample (ppm)	Type of cement	Cement factor
0 - 0.20	0 - 2,000	II	Minimum required by specifications
0.20 - 0.50	2,000 - 5,000	V II	Minimum required by specifications 7 sacks

Guide for Sulfate Resisting Concrete

Water-soluble sulfate in soil sample (%)	Sulfate in water sample (ppm)	Type of cement	Cement factor
0.50 - 1.50	5,000 - 15,000	V II	Minimum required by specifications 7 sacks
over 1.50	over 15,000	V	7 sacks

Plastic Pipe

To date, the department has minimal long-term experience with plastic pipe applications. More information will be provided as the department becomes aware of appropriate information. However, this lack of information should not preclude the possible use of plastics that conform to AASHTO and ASTM specifications if there is solid indication that the particular installation will meet service life expectations.

Section 3 — Installation Conditions

Introduction

Pipe has four basic installation conditions, as illustrated in Figure 14-1.

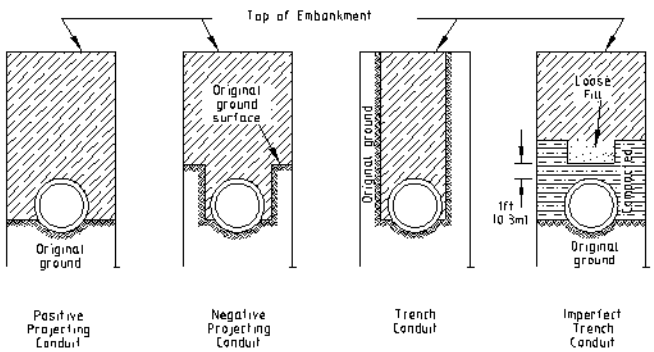


Figure 14-1. Pipe Installation Conditions

Trench

Trench installation of conduit is most preferred from the standpoint of structural advantage and long term operational costs. In order to establish trench conditions, the minimum trench shapes must conform to the diagrams shown in Figure 14-2.

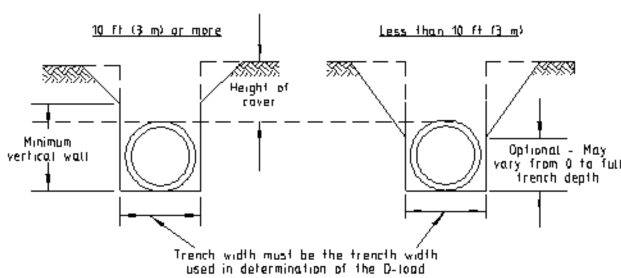


Figure 14-2. Permissible Trench Shapes

Positive Projecting (Embankment)

Positive projecting installation, sometimes termed "embankment installation," is the simplest technique and has the most economical first cost. However, operationally, it does not serve to relieve any structural loading from above the conduit and may result in failure or high maintenance costs during the life of the structure.

Negative Projecting (Embankment)

Negative projecting conditions are more costly than the positive projecting conditions. Negative projection provides some loading relief from the conduit due to the frictional interface between the trench boundaries and the backfill. See Figure 14-1 for a schematic of this effect. Negative projection conditions normally become cost-effective only when fill heights approach 30 ft. (10 m).

Imperfect Trench

The imperfect trench condition is usually more costly than any of the other three installation conditions shown. As with negative projection installation, imperfect trench installation normally becomes cost-effective only when fill heights approach 30 ft. (10 m).

Bedding for Pipe Conduits

In general, bedding for a conduit should comprise select, compact material that conforms to the external curvature of the conduit it supports. This is important for both flexible and rigid conduits.

For a flexible conduit, irregularities or imperfections in the bedding usually can be accommodated by minor shape deformations in the conduit without damage to the structural integrity of the pipe.

For a rigid conduit, such irregularities or imperfections in the bedding cannot be accommodated because the conduit cannot reshape itself without structural failure. Due to the compressive/tensile characteristics of rigid pipe under a load, critical shear zones can fail if bedding geometry is not in conformance with specifications. See Figure 14-3 for a schematic illustration of this characteristic.

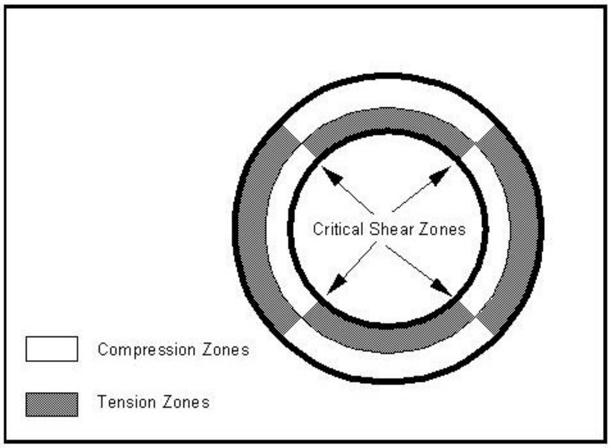


Figure 14-3. Critical Shear Stress Zones for Rigid Pipe

Planned bedding should be supported thoroughly by specifications.

Bedding affects required reinforced concrete pipe strength. The four recognized classes of bedding are shown in Figures 14-4 through 14-7. The most common classes of bedding are Class B and Class C. Class C is the most economical and Class A the most expensive. However, for a given fill height, Class A bedding requires the lowest reinforced concrete pipe strength, and Class C requires the greatest strength. Base selection of bedding on designing the most cost-effective facility.

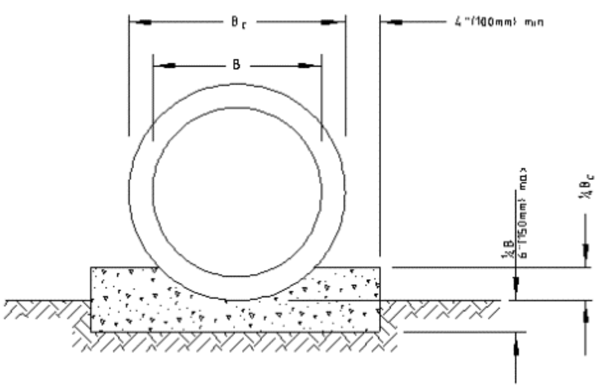


Figure 14-4. Class A Bedding

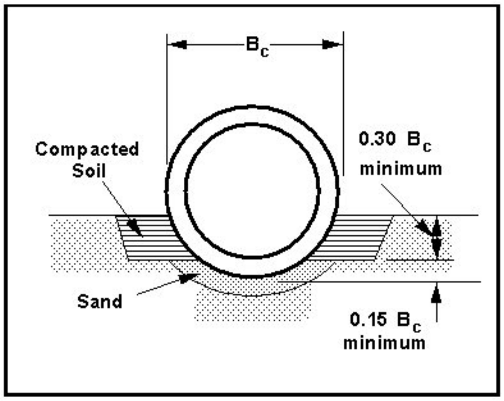


Figure 14-5. Class B Bedding

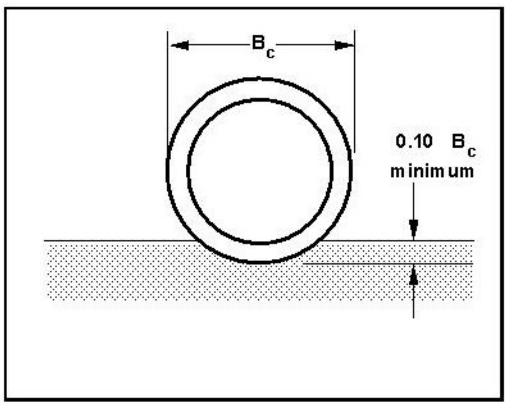


Figure 14-6. Class C Bedding

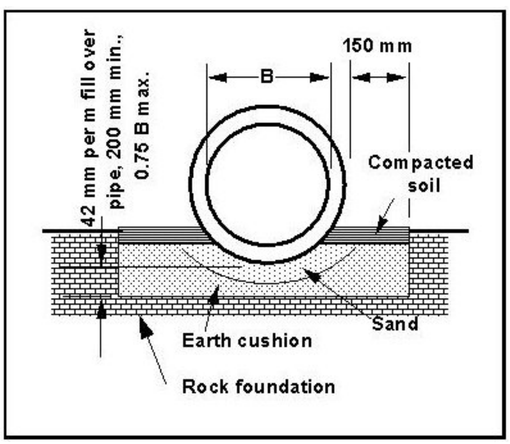


Figure 14-7. Class C Bedding on Rock Foundation

Section 4 — Structural Characteristics

Introduction

Flexible pipe and rigid pipe have some common structural characteristics. The following information provides general guidance on selecting appropriate strength of conduit. However, you may need to coordinate efforts with structural designers to ensure structural adequacy and compatibility.

Corrugated Metal Pipe Strength

Corrugated metal pipe (CMP) is structurally designed in accordance with AASHTO Section 12. Fill height tables are presented in the Conduit Strength and Durability document. These fill height tables are based on the following minimum parameters:

- AASHTO Section 12 Design Guide - Service Load Design
- soil unit mass of 120 lb./cu.ft. (1,922 kilograms per m^3)
- 90% standard density proctor AASHTO T99
- minimum internal factor of safety: wall area = 2.0, buckling = 2.0, and seam strength = 3.0.
- maximum height for pipe arch limited to 39,146 lb./sq.ft. (191,531 kilograms per m^2) of corner bearing pressure
- HS 20 and HS 25 live loading

For structures not represented by tables and conditions outside of above referenced conditions, contact the Bridge Division, Structures Section.

Concrete Pipe Strength

The final design of reinforced concrete pipe walls is not specified in detail on the plans. The required strength of the concrete pipe is indicated on the plans by the D-load that the pipe will be required to support in the test for acceptance. With this designated loading, the manufacturer can determine the most economical structural design of the pipe walls and reinforcement that comply with the applicable American Society for Testing and Materials (ASTM) specification.

The D-load is written as a number followed by (-D). For example, consider the shorthand notation of 1350-D, which represents 1350 lb./ft. of pipe length per foot of pipe diameter (lb./ft./ft.). For this example, multiply 1350 by the pipe diameter (in ft.) for the total allowable loading per foot of pipe length. (65-D represents 65 N/m of pipe length per millimeter of diameter (N/m/mm). For this example, multiply 65 by the pipe diameter in mm to obtain the total allowable loading per meter of pipe length.)

Design load (D-load) values have been computed for a range of conditions and are tabulated in the Conduit Strength and Durability document. The D-load values depend primarily on the following:

- soil unit weight and height of fill above the pipe (dead load)
- live loads
- installation conditions
- trench widths
- bedding

The soil weight used for preparing the tables is 120 lb./cu.ft. (18,857 kN/m^3). Live loads are determined using AASHTO methods, and the design loads for the various pipe diameters and corresponding fill heights are based upon the American Concrete Pipe Association Design Manual (Rev. 1978).

High Strength Reinforced Concrete Pipe

When the required pipe strength exceeds a D-load of 3000 lb./ft./ft. (140 N/m/mm), the structural design of the pipe can fall into a special design category. This can increase the cost because such pipe is usually not a standard stock item with the manufacturer.

Often, refinement of parameters for high-strength pipe, such as bedding, soil weight, and/or trench width, is warranted because the cost of stronger pipe justifies a more refined analysis. For such cases, even the use of Class A bedding may prove to be cost-effective.

Contact the concrete pipe manufacturer for assistance with estimates for the various design alternatives when earth loads require pipe strength greater than 3000 lb./ft./ft. (140 N/m/mm).

Recommended RCP Strength Specifications

Pipe strengths should be specified, as indicated in table below, to reduce the number of bid items and to simplify the administration of the project.

Recommended RCP Strength Specifications (Metric)

For D-loads (lb./ft./ft.) from...	...use	...or Equivalent Class
0 to 80	800	I
801 to 1,000	1,000	II
1001 to 1,350	1,350	III
1,351 to 2,000	2,000	IV
2001 to 3,000	3,000	V

Recommended RCP Strength Specifications (Metric)

For D-loads (N/m/mm) from…	…use	…or Equivalent Class
0 to 40	40.0	I
40.1 to 50.0	50.0	II
50.1 to 65.0	65.0	III
65.1 to 100.0	100.0	IV
100.1 to 140.0.	140.0	V

For some projects, it may be justified to indicate the actual computed D-load for bidding purposes without adhering to the suggested increments above. Generally, deviate from the suggested specification increments only when sufficient quantity of a pipe size warrants the special manufacturer of a specific D-load. Manufacturing conditions vary from company to company. Therefore, potential manufacturers should be contacted to confirm any suspected advantage.

Strength for Jacked Pipe

Pipe that must be jacked under an existing roadway embankment must endure an additional loading not considered for pipe that is simply placed during roadway construction. For jacked pipe, there is the additional load of the axial or thrust load caused by the jacking forces applied during the construction.

Often, ordinary reinforced concrete pipe will serve for the purpose of jacked pipe. Under some conditions, it may be worthwhile to consider specially fabricated fiberglass or synthetic material pipe for jacked pipe. Become acquainted with the availability of various special pipe types in the project area.

For axial loads, the cross-sectional area of a standard concrete pipe wall is adequate to resist stresses encountered in normal jacking operations, if the following construction techniques are used. To prevent localized stress concentrations, it is necessary to provide relatively uniform distribution of the axial loads around the periphery of the pipe. This requires the following:

- pipe ends be parallel and square for uniform contact
- jacking assembly be arranged so that the jacking forces are exerted parallel to the pipe axis

If excessive jacking pressures are anticipated due to long jacking distances, intermediate jacking stations should be provided.

Reinforced Concrete Box

The Bridge Division issues and maintains culvert standard details for cast-in-place and precast reinforced concrete culverts. These accommodate a range of fill heights from direct traffic up to as high as about 30 ft. (9 m) for some boxes. Consult the Bridge Division for conditions not covered by the standards.

Plastic Pipe

Consult the Bridge Division concerning strength requirements for plastic pipe.

Chapter 15 — Coastal Hydraulic Design

Contents:

Section 1 — Definitions

Below is a list of common design terminology and definitions for some of the basic coastal engineering concepts that are included in this chapter. For a more extensive list of coastal engineering terms, see the USACE Coastal Engineering Manual (CEM) Appendix A.

A

AE/VE Zone – Coastal hazard zones (flood zones) designated by the Federal Emergency Management Agency (FEMA) on Flood Insurance Rate Maps. Zone AE areas are subject to inundation by the 1-percent-annual-chance flood event (commonly referred to as the 100-year storm). Zone VE areas are areas subject to inundation by the 1-percent-annual-chance flood event with additional hazards due to storm-induced velocity wave action.

Aggradation – Aggradation, also called accretion, is the increase in land elevation due to building up by natural (e.g., sediment deposition out of water onto a beach) or artificial (e.g., beach fill deposited mechanically on a beach) means.

Astronomical Tide – The astronomical tide is the rising and falling effect on sea levels caused by the gravitational effects of the Earth, sun, and moon, without accounting for atmospheric influences.

B

Barrier Island – Specific to Texas, a barrier island is a landform that is separated from the mainland by a bay (typically connected to the Gulf of Mexico by one or more inlets) or lagoon. Barrier islands are usually low lying, form parallel to the mainland, and contain a beach and dune system. They are subject to morphological changes on a geologic time scale, including erosion, overwash, and migration.

Bathymetry – Bathymetry is the terrain of the seafloor including rivers, bay, estuaries, and oceans. Bathymetric elevations should be referenced to vertical and horizontal datums; for example, the North American Vertical Datum of 1988 (NAVD88) and Texas State Plane coordinates.

Breakwater – A breakwater is a coastal structure generally aligned parallel to the shoreline, usually placed within a few hundred feet of the shoreline. Breakwaters are intended to reduce the amount of wave energy reaching the shoreline by reflecting or absorbing some of the wave energy, although some of the wave energy may still pass over or through the breakwater. Breakwaters can be built using various materials, the most typical of which is rock armor stone and/or riprap.

Breaking Wave – Breaking waves occur when a wave becomes overly steep and unstable based on the wave height compared to the wavelength. This occurs when waves reach shallower waters towards coastlines in an area known as the surf zone. Wave breaking dissipates wave kinetic energy by transferring some of that energy to turbulence. Wave breaking is a nearshore process.

Bulkhead – A bulkhead is a partition placed along the shoreline as a soil retaining structure. A bulkhead is typically designed to prevent land movement only and would not be designed to protect against wave loads. Therefore, a bulkhead will typically offer less protection than a seawall.

C

Coastal Processes – See ‘Nearshore Processes’

Coastal Zone – The coastal zone is the land area of the state located near the coast that may be impacted by coastal processes, extending offshore to the continental shelf and, for the purposes of this Manual, inland to the limits estimated by the FEMA-designated AE/VE zones, including a 1-mile buffer. The State of Texas (Texas General Land Office) also defines a coastal zone boundary for management purposes in the Texas Administrative Code Title 31 §503.1, which includes portions of the following Texas counties: Cameron, Willacy, Kenedy, Kleberg, Nueces, San Patricio, Aransas, Refugio, Calhoun, Victoria, Jackson, Matagorda, Brazoria, Galveston, Harris, Chambers, Jefferson, and Orange.

Continental Shelf – The continental shelf is the portion of a continent (i.e., North America) that is offshore and submerged, but significantly shallower than the open ocean. The continental shelf usually has a relatively mild slope throughout, followed by a steep drop-off to the open ocean. The limits of the outer continental shelf are approximately 200 nautical miles offshore.

Contraction Scour – Contraction scour may occur when water accelerates as it flows through a constricted area, where the downstream opening is narrower than the upstream channel or cross-sectional area. The increase in velocity can cause more sediment removal than under non-constricted conditions.

Cross-Shore – Perpendicular to the shoreline. Also called ‘shore-normal.’

Currents – Currents are the movement of water generally concentrated in a prevailing direction. Currents may be caused by the rise and fall of tides (‘periodic’ or ‘tidal’ currents), seasonal winds, or may be related to general ocean circulation.

D

Datum, Vertical – A vertical datum is a permanent elevation for a location in space used as an elevation to which other elevations are referred. Tidal datums are one type of vertical datum that relate elevations to the approximate land-water intersection at a particular location (such as a tide station). Orthometric datums are another type of vertical datum that use global mean sea level to approximate a land-based plane of zero elevation from which to reference other measured elevations.

Design Life – The forecast life expectancy of infrastructure based on its design.

Diurnal Tide – A tide cycle with one high tide and one low tide each day.

Diffraction, Wave – The process of waves changing direction (bending or wrapping) as they encounter an obstacle, structure, or narrow opening. This is a nearshore process.

E

Ebb Tide – The ebb is the portion of the tidal cycle when the tide is falling and water is moving out of bay systems towards (in the case of the Texas coast) the Gulf of Mexico.

Erosion – Erosion is the wearing away of land by natural forces, such as wave action or wind energy. Erosion may occur over a long period of time, such as in the case of shoreline change, or over the short-term, such as in the case of scour.

Estuary – An estuary is the transition zone between one or more rivers and the open sea. It includes the region of a river mouth in which the freshwater from the river intermixes with the salt water from the sea, creating brackish water.

Exceedance Probability – See 'Return Period'

F

Fetch – Fetch is the distance of open water that the wind acts upon to generate waves. Fetches may be a distance of a few feet to hundreds of miles. All else being equal, a longer fetch will allow larger waves to form.

Flood Tide – The flood is the portion of the tidal cycle when the tide is rising and water is moving from (in the case of the Texas coast) the Gulf of Mexico into the bay systems.

Flooding, Coastal – Coastal flooding occurs when low-lying land near the coastline is flooded by seawater.

Freeboard – Freeboard is the vertical distance between the water surface (or in some cases may be referenced from the wave crest) and the crest of the structure in consideration. For example, the freeboard of a breakwater is typically defined as the vertical distance between the structure crest and the water surface. Alternatively, the freeboard of a bridge span is the open distance between the lowest portion of the span and the water surface.

G

Groin – A groin is a coastal structure aligned perpendicular to the shoreline and intended to interrupt longshore sand transport along beaches. Accretion may occur on the updrift side of a groin, accompanied by erosion on the downdrift side.

H

Hindcast – A hindcast is a dataset used to forecast wave conditions based on a statistical analysis of measured wind information and past storm events. The data that is used to develop a hindcast is typically measured over a period of decades.

I

Inland Water Body – In most cases, this term describes a body of water located landward of the coastal zone and/or not directly connected to a bay, gulf, or ocean. Examples include lakes or rivers, even if the river eventually extends to the coast.

Inundation – See ‘Flooding, Coastal’

J

Jetty – A jetty is a structure aligned perpendicular to the shoreline intended to shelter navigation channel entrances from waves and reduce sediment deposition in the channel. Jetties are typically constructed where navigation channels pass through inlets connecting a smaller water body (such as a bay) to a larger water body (such as the Gulf of Mexico).

L

LiMWA – The Limit of Moderate Wave Action (LiMWA) is a FEMA-designated line that defines the inland limit of area expected to receive 1.5-foot or greater breaking waves during the 1-percent-annual-chance flood event. Waves of 1.5 feet or higher have been shown to cause significant damage to structures. A LiMWA is shown on some FEMA Flood Insurance Rate Maps for areas along coastlines.

Littoral Drift – Littoral drift is the process by which sediment is transported along the shoreline by waves and currents. Sediment movement is typically longshore but can include cross-shore components. Littoral drift occurs within the littoral zone.

Littoral Zone – The littoral zone is the area where sediment transport either onto or away from the beach occurs within the nearshore area. This zone typically extends from the depth of closure at a beach to the high-water mark at the shoreline. The depth of closure is the approximate location at which the beach profile taken over a series of months or years converge, indicating that the net sediment transport into or out of the littoral system is approximately zero. As sediment transport is caused by the nearshore process of wave breaking, the littoral zone encompasses and extends beyond the surf (breaker) zone.

Living Shoreline – A living shoreline is a shoreline stabilization technique that is used to protect shorelines using a combination of structural (e.g., breakwater) and non-structural, nature-based (e.g., vegetation, sand) components. In many cases, the structural solution will use natural materials such as rock.

Longshore – Parallel to the shoreline.

M

MHHW – Mean Higher High Water (MHHW) is a statistical tidal datum calculated using the highest water surface elevation of each day, typically over a 19-year recording period.

MHW – Mean High Water (MHW) is a statistical tidal datum calculated using the highest water surface elevation of each tidal period, typically over a 19-year recording period.

MLLW – Mean Lower Low Water (MLLW) is a statistical tidal datum calculated using the lowest water surface elevation of each day, typically over a 19-year recording period.

MLW – Mean Low Water (MLW) is a statistical tidal datum calculated using the lowest water surface elevation of each tidal period, typically over a 19-year recording period.

Morphology – The evolution of the form of a river, estuary, seafloor, or other landform over time.

MSL – Mean Sea Level (MSL) is a statistical tidal datum calculated using hourly water surface elevation values, typically over a 19-year recording period.

N

NTDE – National Tidal Datum Epoch (NTDE) is a 19-year averaging period to which all tidal datums are referenced. The current NTDE spans from 1983-2001 and is actively considered for revision every 20-25 years.

Neap Tide – The neap tide is the tide when there is the smallest difference between high and low tide in a day. The neap tide typically occurs two times each month during quarter moon phases.

Nearshore Processes – Nearshore processes, or ‘coastal processes’, are physical processes that occur in the nearshore zone due to wind, waves, currents, tides, and other forces impacting the land and seabed in coastal areas. Nearshore processes include, among many others: wave breaking, refraction, diffraction, reflection and shoaling; overwashing; and sediment transport. Nearshore processes occur in the nearshore zone, although their impacts may extend beyond this area.

Nearshore Zone – The nearshore zone is located along the coastline and is typically bounded by the low-tide shoreline on the landward side and the offshore zone on the seaward side (which typically begins at water depths of about 60-65 feet). The nearshore zone is roughly equivalent to the littoral zone, although it typically extends further seaward than the littoral zone.

Numerical Modeling – Numerical modeling is the application of computer software to perform calculations. Numerical models can range in complexity from simple, one-dimensional steady state calculations to large-scale, multi-dimensional systems.

O

Overtopping, Wave – Wave overtopping occurs when a wave transmits water over an existing structure such as a breakwater, seawall, or building.

Overwash – Overwash is the effect of waves overtopping a coastal structure, such as a seawall, without the water then directly flowing back to the sea or lake. Overwash often carries sediment landward, which is then lost to the beach system.

P

Period, Wave – The wave period is the amount of time required for the full length of wave to pass a given point in space.

Physical Modeling – Physical modeling can be applied to better understand anticipated hydrodynamic and wave processes. An example of a physical model is a scaled re-creation of a bridge pier placed in a wave tank.

R

Reflection, Wave – Reflection is the process by which wave energy is redirected seaward after impacting a structure or obstruction, such as a seawall or a steep beach face. Wave reflection can lead to complex wave interactions, increase turbulence, and generate scour.

Refraction, Wave – Refraction is the process by which waves tend to change direction and become more parallel to the shoreline as they approach the shore. This phenomenon occurs because the portion of the wave nearer to the shore that advances in shallow water moves more slowly than the portion of the wave farther from the shore in deeper water (due to bottom friction), causing the wave crests to bend.

Relative Sea Level Rise – See 'Sea Level Rise'

Return Period – The return period is a statistical value intended to provide design context on how often a specific value or event may occur. The inverse of the return period is the statistical percentage that the value or event will occur in any given year, also known as the exceedance probability. For example, a 100-year return period wind speed has a 1% chance of occurring within any given year.

Revetment – A revetment is a structure used to protect a shoreline or embankment from wave or current impacts. It is typically installed directly on grade and is composed of concrete, stone, or other armoring.

Rip Current – A powerful, swift current flowing seaward from the shore over a narrow portion of shoreline. It is typically formed due to an onshore-directed wave breaking over submerged nearshore sandbars and then flowing back out through a narrow opening between the bars. Rip currents are commonly called ‘rip tide’ and are particularly hazardous for swimmers.

Runup, Wave – Wave runup is the vertical distance that the water surface will temporarily increase as a wave propagates up a sloped surface. The runup is caused by the wave directly interacting with a structure or the shoreline and will be affected by surface roughness, porosity, and geometry.

S

Scour – Scour is the local erosion caused by swift-moving water at the interface between a structure and the sediment or substrate in which it is located. In the coastal zone, scour is of particular concern at bridge pilings or at the base of bulkheads, seawalls, and other manmade structures.

Sea Level Rise – Sea level rise is the long-term increase in mean sea level. It can be measured on a global scale, based on the global mean sea level, or at a local level. When measured locally, sea level rise is frequently computed in tandem with regional land elevation changes (such as subsidence). The long-term trend in sea level rise, accounting for regional land change, is known as relative sea level rise.

Seawall – A vertical structure built along a portion of coastline to prevent erosion and other damages due to wave impacts. A seawall typically offers more protection to an area than a bulkhead, and design will usually address storm surge, wave overtopping, toe scour, uplift caused by waves, and direct wave attack.

Sediment Budget – Sediment budget is the amount of budget added to and removed from the coastal system over an area of interest and time frame. The amount of sediment into the system less the amount of sediment out of the system yields a net sediment remaining within the system. A positive sediment budget indicates a sediment surplus and can lead to aggradation. A negative sediment budget indicates a sediment deficit and can lead to shoreline erosion.

Sediment Transport – Sediment transport is the movement of sediment within a nearshore system caused by wave breaking and the net direction of winds, waves, and currents. The sediment transport accounts for why an area may be accreting (aggrading) or eroding over time.

Semi-diurnal Tide – A semi-diurnal tide is a tide that has a cycle of approximately one-half of a tidal day (12.4 hours) and generates two high tides and two low tides each day. The predominating type of tide throughout the world is semi-diurnal.

Setup, Wave – Wave setup is the vertical increase in water surface elevation due to the presence of breaking waves. In contrast to individual wave crests which are easily observable, wave setup occurs as a relatively gradual increase in the water surface across a broad area and is typically indiscernible to the naked eye.

Setup, Wind – Wind setup is the vertical increase in water surface elevation due to the force of the wind piling up water along a shoreline. Wind setup can contribute to high tides and is a component of storm surge.

Shoaling, Wave – Shoaling refers to the phenomenon whereby wave heights increase as waves propagate towards the shoreline and enter shallower water. When waves reach a critical point where the ratio between wave height and water depth becomes too steep, the waves break.

Shoreline Change – Shoreline change is the gain or loss of land area along the coastline that classifies if a shoreline is accreting (aggrading) or eroding. Shoreline change can be measured over the long-term (decades), due to geological, morphological, or meteorological trends, or short-term, due to episodic events (e.g., tropical storms).

Spring Tide – The spring tide is the tide when there is the largest difference between high and low tide in a day. The spring tide typically occurs two times each month during new moon and full moon phases.

Stillwater Elevation – The stillwater elevation, also known as stillwater level (SWL), is the elevation of the top of water without including the crests of individual wave but including wave setup. When being used to describe storm conditions, SWL includes storm surge.

Storm Surge – Storm surge is the increase in water surface elevation due to a combination of reduced atmospheric pressure (typical before and during tropical systems) and wind and wave setup. Storm surge does not include the crests of individual waves.

Storm Tide – Storm tide is the elevation that combines the storm surge and the expected tidal elevation that would have occurred at that time without a storm.

Subsidence – Subsidence is the sinking of the ground due to many location-specific factors including sediment characteristics and reduction in amount of subsurface water.

T

Tide Range – The difference between the average high tide and low tide at a particular location.

Tide Station – A tide station is a station for which tide predictions are generated for a given location. Tide stations can be harmonic, for which tide predictions are generated directly from harmonic constants and have listed tidal datums, or subordinate, for which tide predictions are generated using high/low tide predictions from a designated harmonic tidal station and tidal datums are not generally calculated.

Transect – A transect is a straight line in the field or on a map along which data are recorded, measured, observed or calculated. Within the context of coastal engineering, transects are typically aligned perpendicular to the shoreline.

W

Wave Height – The wave height is the vertical distance between the wave crest and the wave trough.

Wave Height, Significant – The significant wave height is the statistical average of the largest one-third of the wave heights in a given time series, typically 17 to 20 minutes.

Wave Height, Maximum – The maximum wave height is the statistical largest wave height in a given time series.

Wavelength – The wavelength is the horizontal distance representing the length of wave. This distance is typically measured from wave crest to wave crest.

Section 2 — Introduction and Applicability

Purpose

This chapter describes the requirements for coastal hydraulic design studies performed for the Texas Department of Transportation (TxDOT). Its purpose is to guide Departmental staff and consultants performing design work in a coastal area and introduce the reader to general coastal engineering concepts. This chapter is not intended to be a detailed design guide but rather provides references for data sources and industry-standard coastal engineering guidance to assist design.

Applicable Projects

To determine whether the project needs to consider coastal engineering concepts, this section includes maps representing the five districts within TxDOT that are along the coast (Figure 15-1 through Figure 15-5). Each map shows an area defined as the Coastal Risk Area; the designer should consider the coastal forces and guidance presented in this chapter if a project is within this area. The extents of the Coastal Risk Area were derived using Federal Emergency Management Agency (FEMA) designated coastal VE and AE zones. The shown Coastal Risk Area serves only as initial guidance on applicability, and it is incumbent upon the project team to further assess and define the potential for coastal forces to impact any given project.

Additionally, some projects not included in the Coastal Risk Area still may need to consider this design guidance. Consider using this document if the project is:

- Located along inland lake shorelines where wave action and inland fetch can create wave forces that require consideration during design (further discussed below), or
- Located in a coastal county (Aransas, Brazoria, Calhoun, Cameron, Chambers, Galveston, Harris, Jackson, Jefferson, Kenedy, Kleberg, Matagorda, Nueces, Orange, Refugio, San Patricio, Victoria, and Willacy), or
- Known to have historic evidence of coastal influence (e.g. coastal flooding), including impacts from extreme weather events, such as hurricanes.

The Coastal Risk Area shown in the district maps below will change in extent and risk classification over time, as the coast experiences erosion, subsidence, and sea level changes. The maps include a one to two mile buffer inland of the FEMA designated coastal zones to partially represent these additional considerations. All these processes may expand the area of influence from coastal processes further inland than shown.

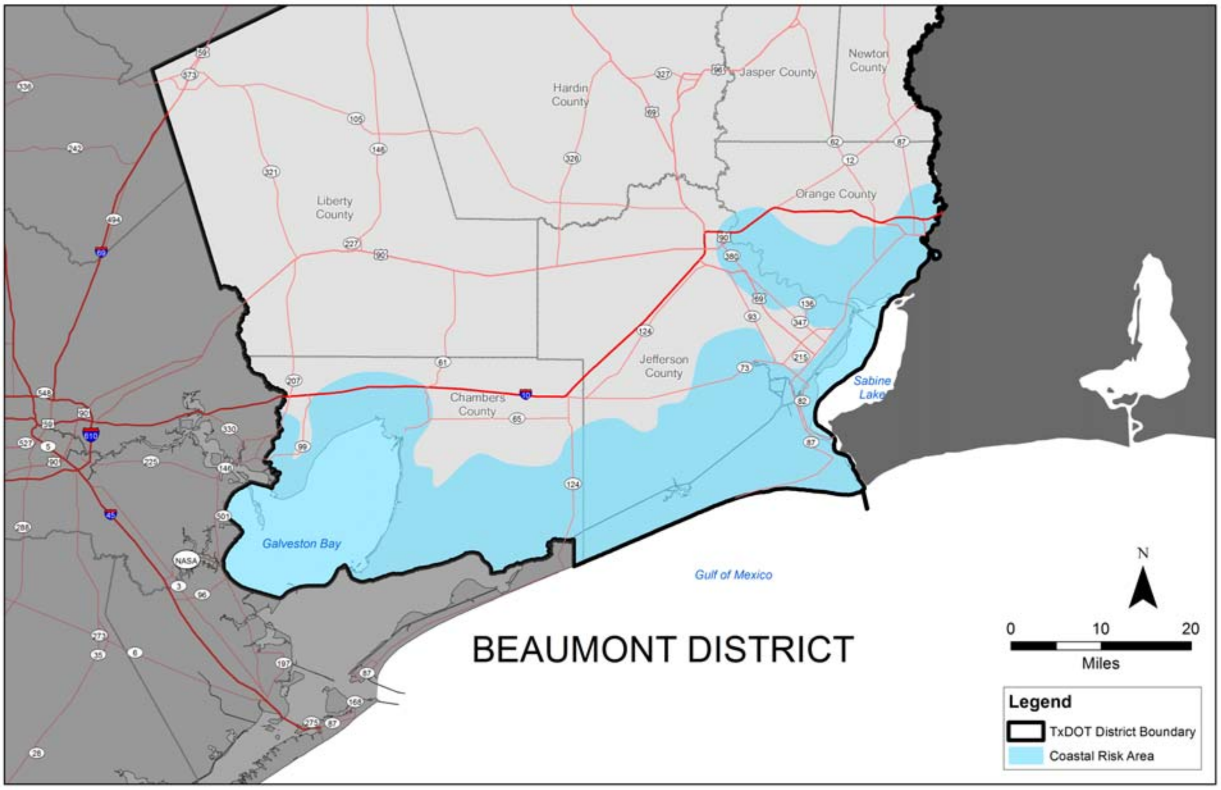


Figure 15-1. TxDOT Beaumont District Coastal Risk Area.

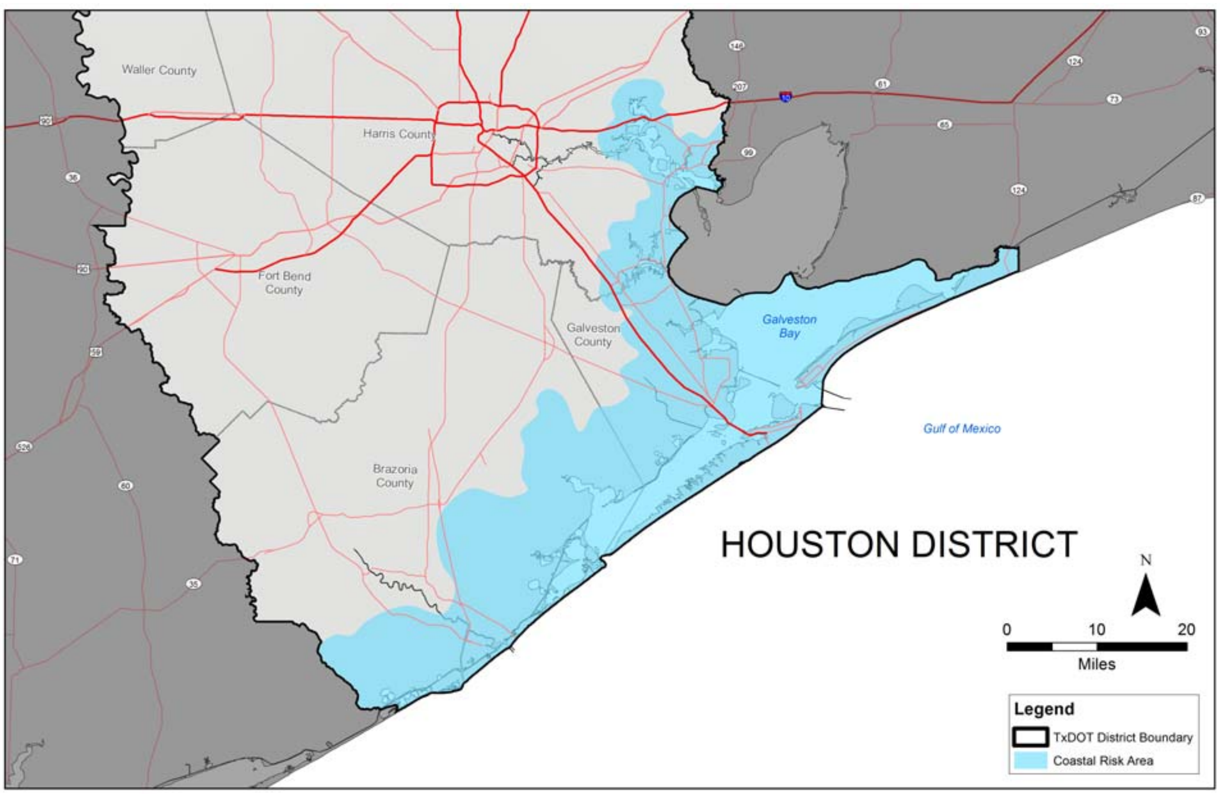


Figure 15-2. TxDOT Houston District Coastal Risk Area.

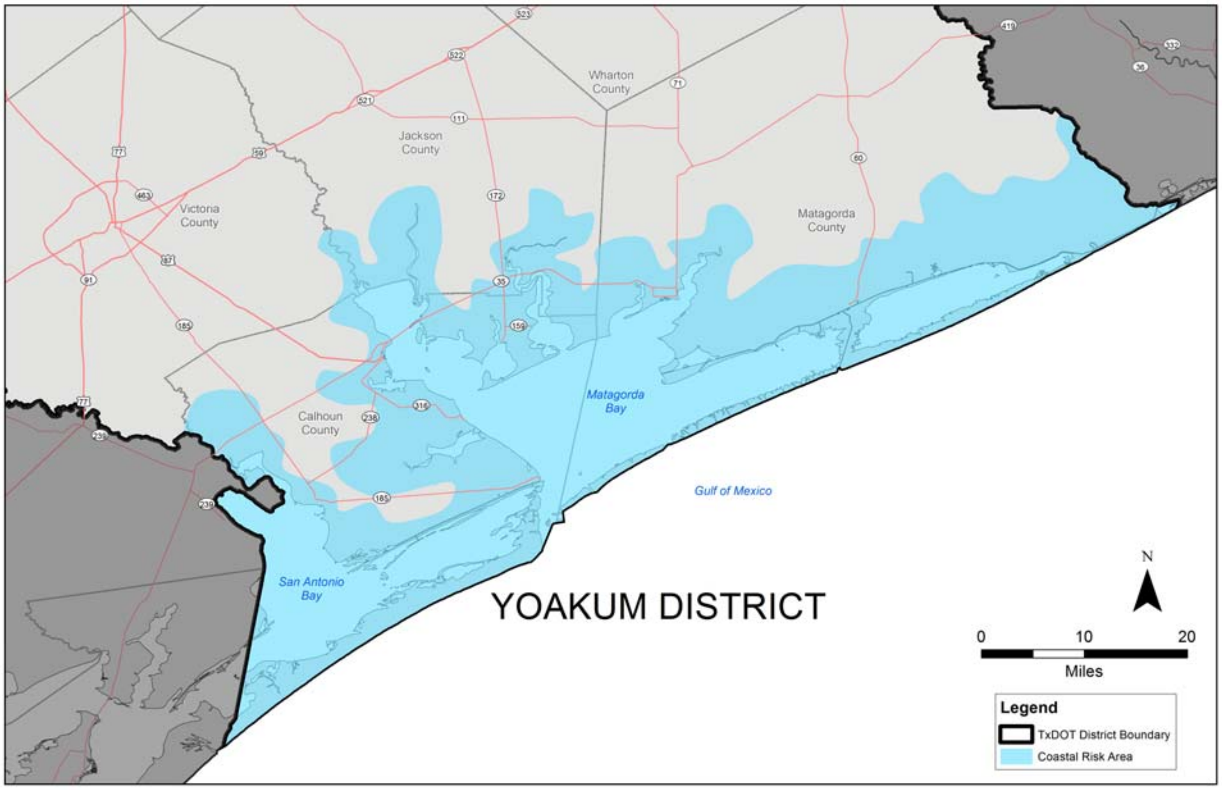


Figure 15-3. TxDOT Yoakum District Coastal Risk Area.

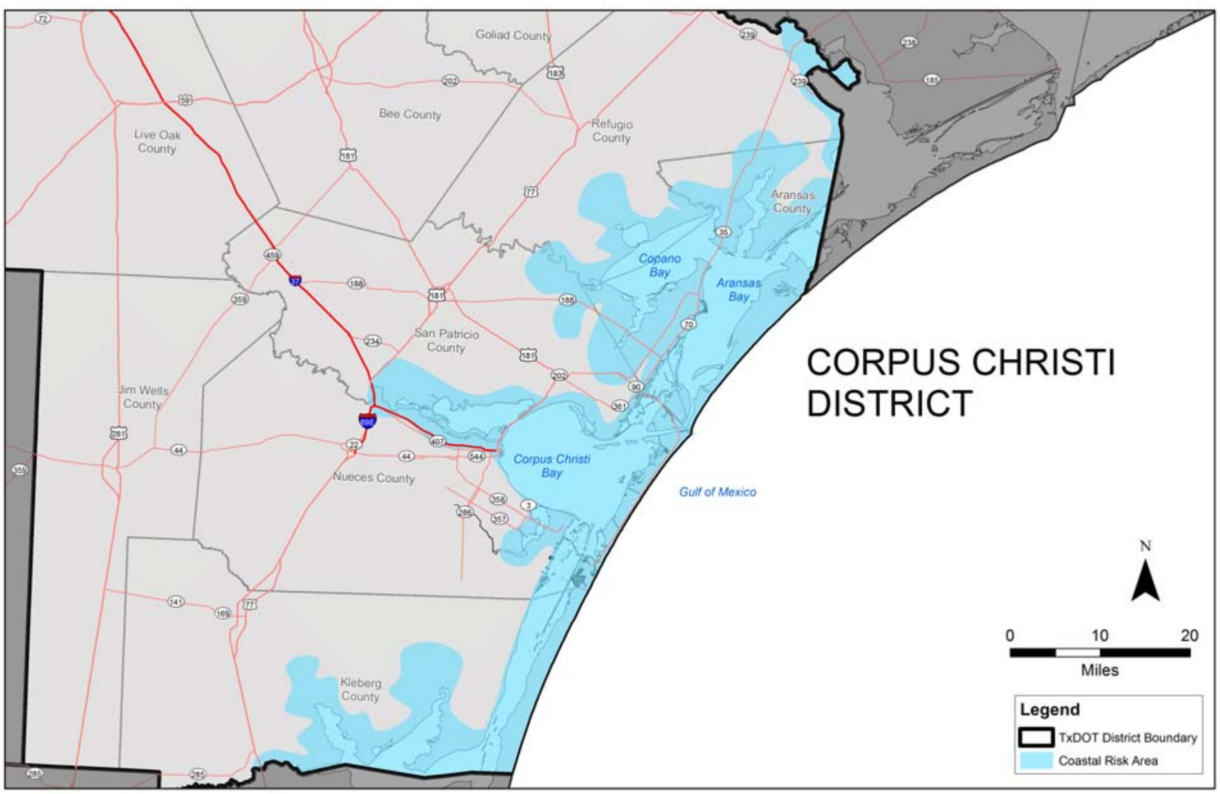


Figure 15-4. TxDOT Corpus Christi District Risk Area.

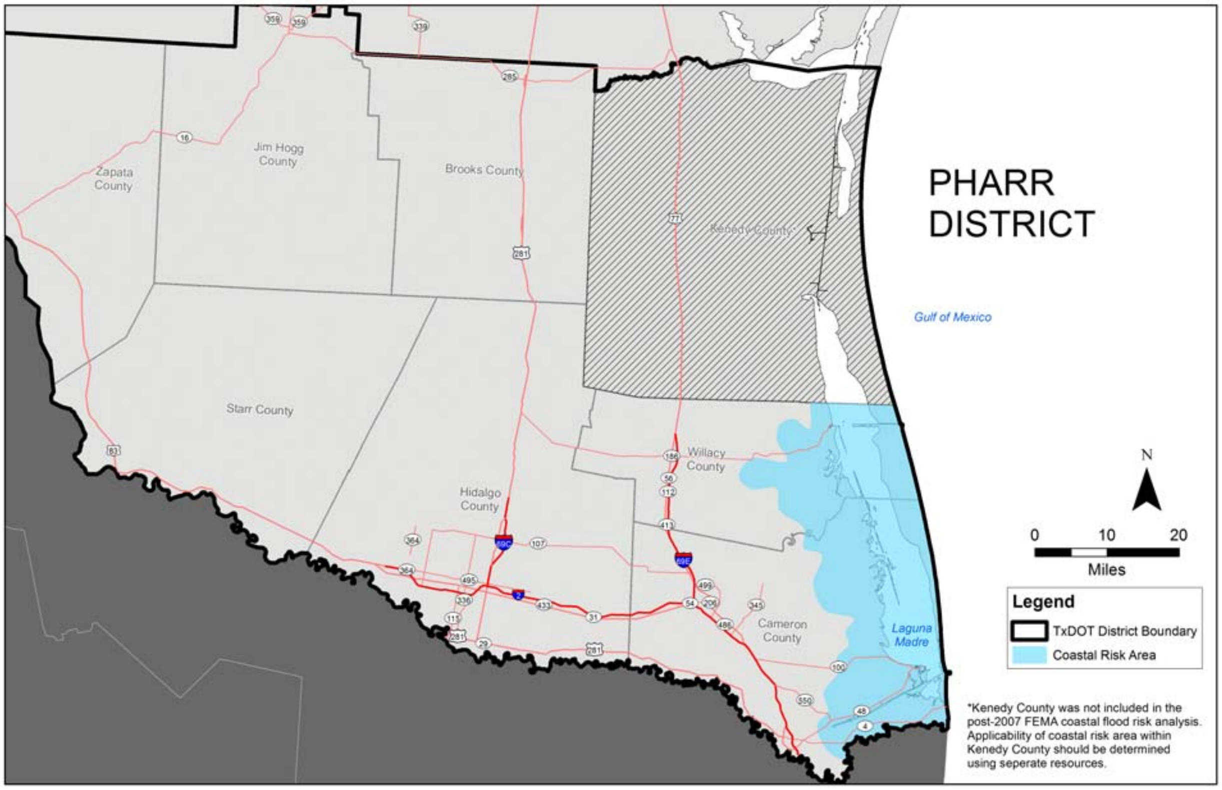


Figure 15-5. TxDOT Pharr District Risk Area.

Design for some projects located near inland water bodies should also consider coastal guidance, especially if surrounding areas may experience or have experienced erosion or other damage caused by wave conditions.

How to Use This Document

This document is intended to be a guide when designing or evaluating transportation facilities in coastal areas. Input from a TxDOT Precertified Coastal Engineer during the project's design phase will also help by providing a better understanding of the coastal processes influencing the study site.

Step 1: Determine whether the project is located within an area influenced by coastal processes.

Step 2: Determine the level of coastal analysis that the project requires, with this guidance following designations of Level 1, Level 2 or Level 3. As part of the level designation, the designer should use this document to evaluate what factors may impact the design. There are several factors that may influence design of coastal projects including: waves, wind, water depths and velocities, water surface elevations, tropical storms and hurricanes, and project location relative to the coast. Each of these factors is discussed within this chapter.

The procedure to determine the level of analysis is presented in the following section. It should be noted that the level of analysis can vary for a given project for different factors. For example, a project could have minimal risk of scour when evaluating a coastal roadway inland of the shoreline, justifying a Level 1 analysis for scour, but the presence of shoreline erosion and need for a higher roadway elevation may be of more concern, justifying a Level 2 analysis of those factors.

Level 1, 2, and 3 Analysis Discussion and Examples

To assist the reader, three categories of projects have been created, labeled as Level 1, Level 2, and Level 3. These represent an increase in complexity, vulnerability, and effort from Level 1 to Level 3. The purpose of these categories is to assist the designer when determining how to apply the elements described within this chapter.

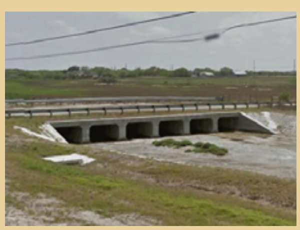
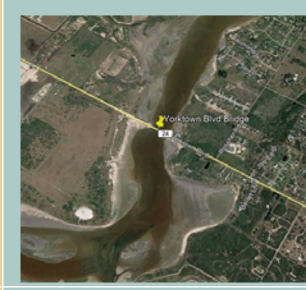
The most common level of analysis for TxDOT projects influenced by coastal processes is Level 1. Level 1 analyses are typically represented as having either a project that includes lower levels of investment by TxDOT or the factor has limited vulnerability to coastal forces. An example of this is a project located landward of the FEMA Limit of Moderate Wave Action line when considering wave analysis as the specific factor of interest. Coastal analysis for a Level 1 project will typically be based on solely evaluating existing coastal data.

Level 2 analyses are less frequent than Level 1, but more common the Level 3. These analyses may represent projects with moderate to high levels of investment by TxDOT and include factors that represent exposure to moderate coastal vulnerabilities. Coastal analysis for a Level 2 project will typically involve leveraging existing data for the more complex factors, such as stillwater level or scour, and in addition commonly require coastal models to develop additional necessary data for design, particularly with regard to wave analysis.

Level 3 analyses are rare within TxDOT. These are typified by high risk projects such as causeways or other significant coastal roadways with high levels of investment, often serving as major evacuation routes, with high vulnerability. Level 3 analyses require extensive data acquisition and modeling effort to develop project specific data for complex designs.

These three representative categories are introduced in Table 15-1 and will be referenced throughout this chapter. These levels are intentionally not prescriptively delineated as it is the responsibility of the District and engineer of record to review and determine the appropriate level of analysis for each project. This decision is similar to riverine modeling decisions when deciding the complexity of modeling that should be performed – approximate, detailed, or complex. Under this concept, one project may have multiple levels of analysis, such as a causeway project (Level 3) that also has several miles of approach road (Level 2). This decision will typically be made during scoping and documented in the DSR.

Table 15-1: Level of Analysis Required for Representative Coastal Infrastructure Projects

	Level 1 Analysis	**Level 2 Analysis**	**Level 3 Analysis**
Approximate Frequency	Most common	Less common	Infrequent
Road Type	Off-system and minor arterials in less critical areas	Various roadway types up to and including interstates in less/moderately critical areas–	Highly vulnerable routes, freeways/interstates in very critical areas
Bridge Type	Less critical bridges over a tidal creek in shallow estuary with minimal coastal scour risk	Less/moderately critical bridge that is well-protected with minimal coastal impacts; moderate coastal scour risk	Highly critical or major evacuation route bridges; severe or dynamic coastal scour risk
Vulnerability	Low to Moderate	Moderate to High	High
Road Type Example	Local or minor arterial, less critical roadway 	Principal arterial, but less critical road with armoring needs, located in FEMA Coastal AE/VE Zone 	Interstate with seawall located along major evacuation route 
Bridge Type Example	Culvert or small bridge in AE Zone, well-protected, local road 	Small bridge over protected bay in AE zone, local road 	Causeway connecting mainland and barrier island 
Other Considerations	Less complex analysis may require more conservative design assumptions, which can increase overall construction cost.	More complex analysis may be more time intensive and costly during the design phase, but can reduce overall construction cost for level of protection needed.	Complex geometries can cause waves to change direction or height; accounting for these complexities may not be feasible with simple analytical methods.

Chapter Overview

An overview of the elements to be reviewed in this chapter is shown in Figure 15-6, below.

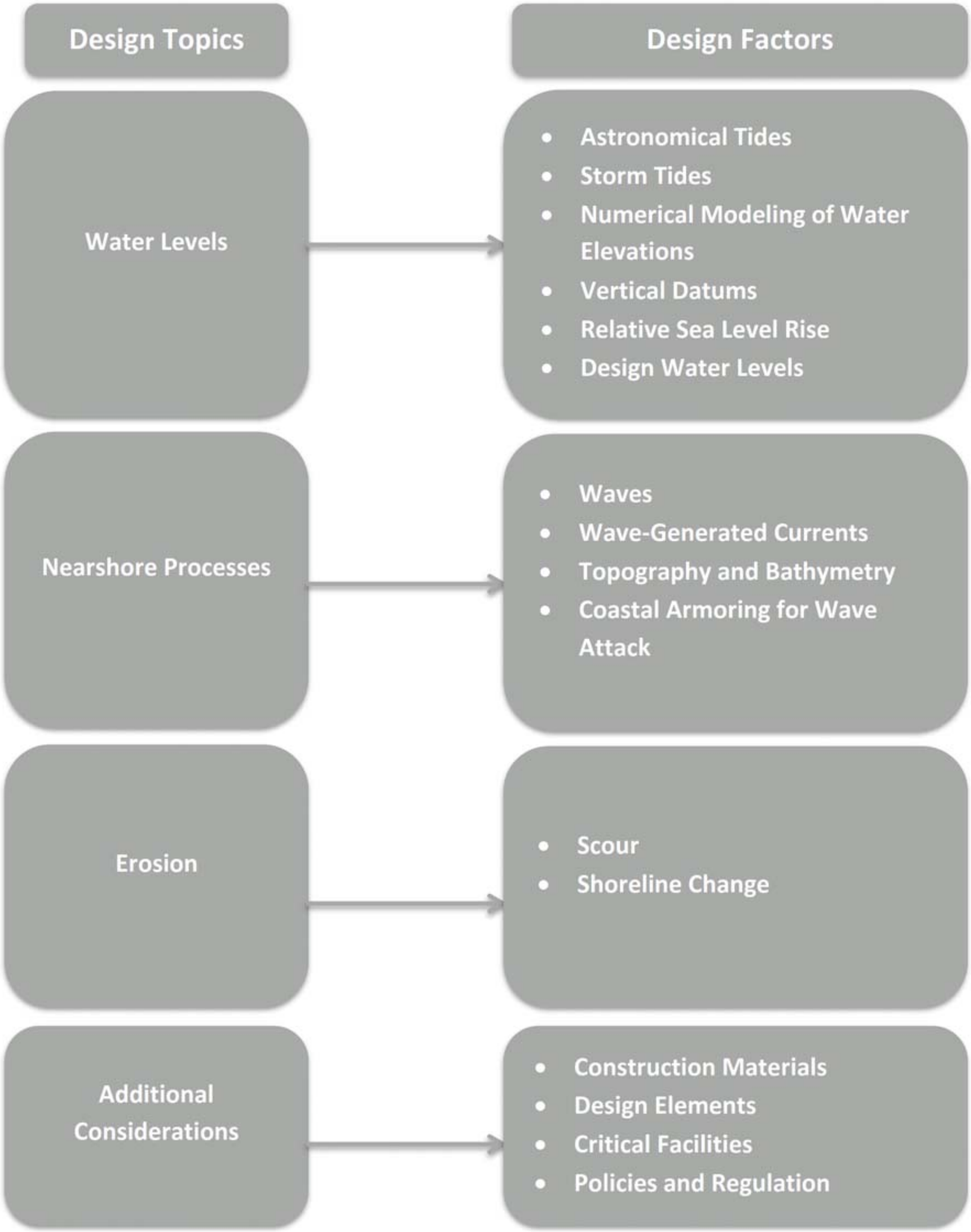


Figure 15-6. Design Topics and Design Factors.

Section 3 — Stillwater Levels

Consideration of Water Levels in Coastal Roadway Design

Stillwater level represent the water surface absent wave height and wave runup. Fluctuations in stillwater levels along the Texas coast are influenced by astronomical tides, storm surges, and long-term sea level changes. Tides and storm surge reflect dynamic processes that move water in a fairly short time period, but their effects are still incorporated into the overall term "Stillwater level." The stillwater level should be considered for all roadway projects located in coastal areas to determine the landward extent and elevation of inundation and wave forces.

Identification of a stillwater level is a critical element in the design of a roadway infrastructure located in coastal areas.

This section seeks to:

1. Synthesize – at a high level – the various factors that contribute to water level fluctuations along the Texas coastline; and
2. Provide guidance on the estimation of appropriate stillwater levels for design of coastal transportation infrastructure.

Astronomical Tides

Water levels fluctuate throughout the day, primarily due to astronomical tides caused by the gravitational pull of the moon and sun. In Texas, long-term water level stations are available online through the National Oceanic and Atmospheric Administration (NOAA) Tides and Currents: Center for Operational Oceanographic Projects and Services and the Texas Coastal Ocean Observation Network (Figure 15 7). The Texas Gulf Coast generally experiences a diurnal pattern of one high and one low tide each day; however, there are times each month where semi-diurnal tides occur (two high and two low tides each day). The average daily tide range (difference between the average high tide and low tide) along the Texas coast averages less than 2 feet. Table 15-2 shows the tide ranges at representative locations along the coastline.

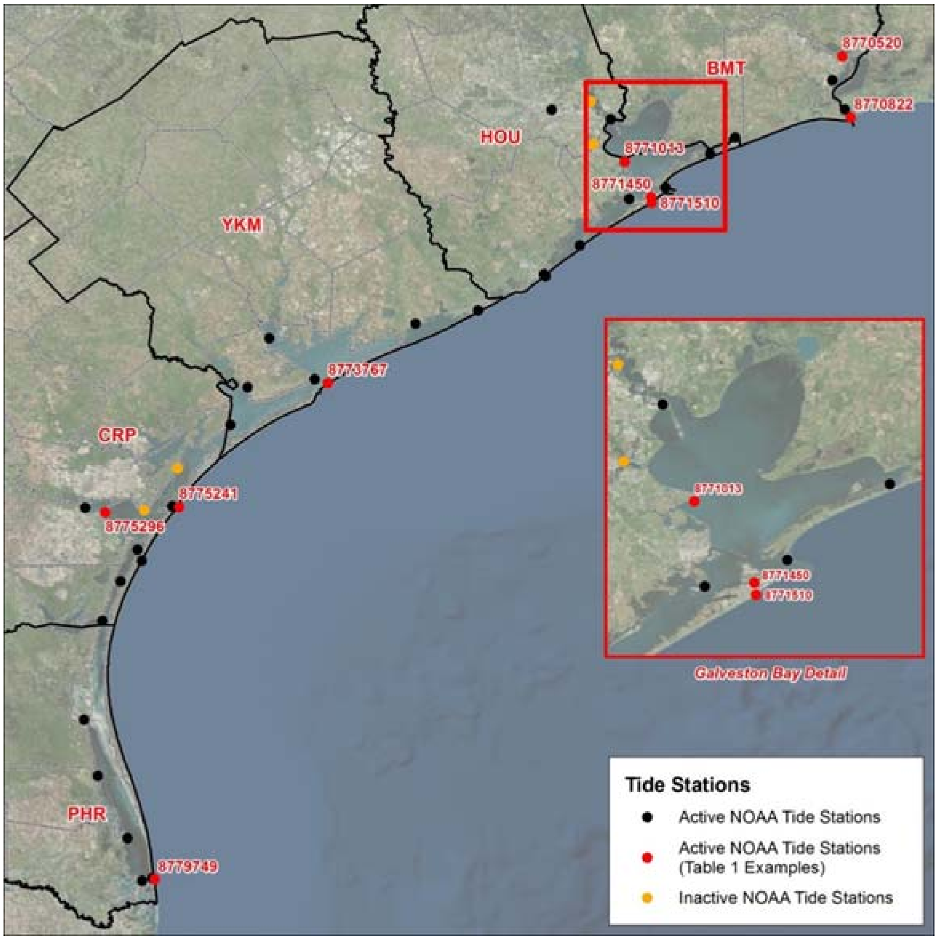


Figure 15-7. NOAA tide stations along the Texas Coast (USACE Galveston District Southwestern Division, 2018).

Table 15-2: Tidal Range at Representative Stations along the Texas Coast (feet)

NOAA Station Name	Station Number	Tidal Range (feet)
Rainbow Bridge, Port Arthur-Orange	8770520	1.1
Texas Point, Sabine Pass	8770822	2.0
Eagle Point, Galveston Bay	8771013	1.1
Galveston Pleasure Pier	8771510	2.0
Galveston Pier 21	8771450	1.4
Matagorda Bay Entrance Channel	8773767	1.2
Aransas Pass, San Patricio County	8775241	1.4
USS Lexington, Corpus Christi	8775296	0.6
South Padre Island, Brazos Santiago	8779749	1.4
NOTE: Tidal range reflects the great diurnal tidal range and is defined as the difference between Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW).		

Large differences in the tide range can occur at the same location throughout the month. During full and new moons, spring tides occur because the sun and moon are aligned with respect to the earth. Their combined gravity causes a larger than average tidal range: high tides are higher and low tides are lower. During quarter moons, neap tides occur when the gravity of the sun and moon are opposed, creating a smaller than average tidal range. More information on tides can be found the NOAA reference included in this section.

Tidal elevations also vary spatially and are influenced by local bathymetry, shoreline orientation, and wind/current patterns. Due to the complexities of the Texas shoreline, observed tide range varies locally, with some areas experiencing tidal amplification (e.g., Galveston Pleasure Pier), while others experience a dampening of the tidal signal (e.g., USS Lexington).

Annual High Tides

Annual high tides, also known as king tides, occur several days each year and produce ocean levels over a foot higher than average high tides along the Texas coastline.

Annual high tides often result in temporary flooding of low-lying coastlines, particularly if they coincide with a storm event or onshore wind that elevates tides above predicted levels. Because these tides can often cause nuisance flooding and reduce stormwater drainage efficiency, considering typical annual high tide elevations is especially important when designing low-lying roads and roadway drainage systems. In addition to temporary road closures, frequent flooding caused by these tides could lead to recurring damages to the roadway and increase the rate of scheduled maintenance.

Although most roadway infrastructure being constructed will be based on minimum design elevations that exceed the annual high tide, there may be cases where roadway assets remain vulnerable to flooding from annual high tide exposure (e.g., road repair or roadway culvert designs in rural areas). Therefore, it is recommended that the designer evaluates the design of the roadway project for potential exposure to annual high tide water level elevations and, where possible, construct roadway and drainage features at a higher relative elevation or design for the road base and support to account for frequent soil saturation. Care should also be taken to consider the hydraulic connectivity and ensure the proposed roadway does not obstruct the flow during or after high tide flooding events. The designer may consider incorporating some drainage features that will allow water to flow on either side of the asset.

Evaluating water levels collected at local tide stations during known annual high tide events provides a general range of expected water level events. Several approaches can be used to determine water elevations associated with an annual high tide event. Annual high tide events can be extracted using extreme value analysis of historical tide data observed at a local tide station.

Tidal Data Sources

- NOAA Center for Operational Oceanographic Products and Services (CO-OPS) — NOAA Tides and Currents is a collection of wind, water level, and current data from NOAA recording stations along U.S. coastlines. Water level observations and tidal datums are available at some of these recording stations.

- [Texas Coastal Ocean Observation Network (TCOON)] — TCOON is a network of wind and water level observation stations along the Texas coast, with data collected by a combination of federal, state, local, and academic institutions. Water level observations and tidal datums are available at most TCOON recording stations.

Storm Tides

Tropical and extratropical cyclones can be hazardous for projects located in coastal areas both along the shoreline and several miles inland. Storm tides, which represent the stillwater elevation inclusive of storm surge, can bring high water, strong currents, and large waves to locations that are not normally exposed to coastal forces. Storm tides are difficult to predict, as it is highly sensitive to small changes in storm track, size, approach speed and angle, and atmospheric pressure. Characteristics of the coastline, including the width and slope of the offshore continental shelf and the presence of inlets, such as bays and estuaries, also affect the elevation of a storm tide.

To account for the water elevations associated with storms during the infrastructure planning and design phase, storm surge is added to the astronomical tide (typically represented by the normal high tide) to derive a storm tide elevation (Figure 15-8). Storm tides are stillwater elevations, meaning they do not include wave effects, such as wave amplitude and wave setup. Storm tides are often expressed in terms of an exceedance probability, or the likelihood that the water level will surpass a given elevation based on a statistical analysis of historical observations or model simulations.

For example, a 100 year flood event identifies a stillwater elevation that has a 1% annual exceedance probability (AEP). It is commonly used as an indicator to inform assessments of flood risk or design criteria (FHWA, 2005). Refer to Chapter 4 (Hydrology), Section 2 (Probability of Exceedance) for commonly used terminology regarding exceedance probabilities. In this section, it is also referred to as a return period or return interval. For planning and design phases, care should be taken in understanding how the storm tide elevation was computed, based on the type of approach, modeling methodology, and models adopted, as it could include or not include wave setup and/or tides.

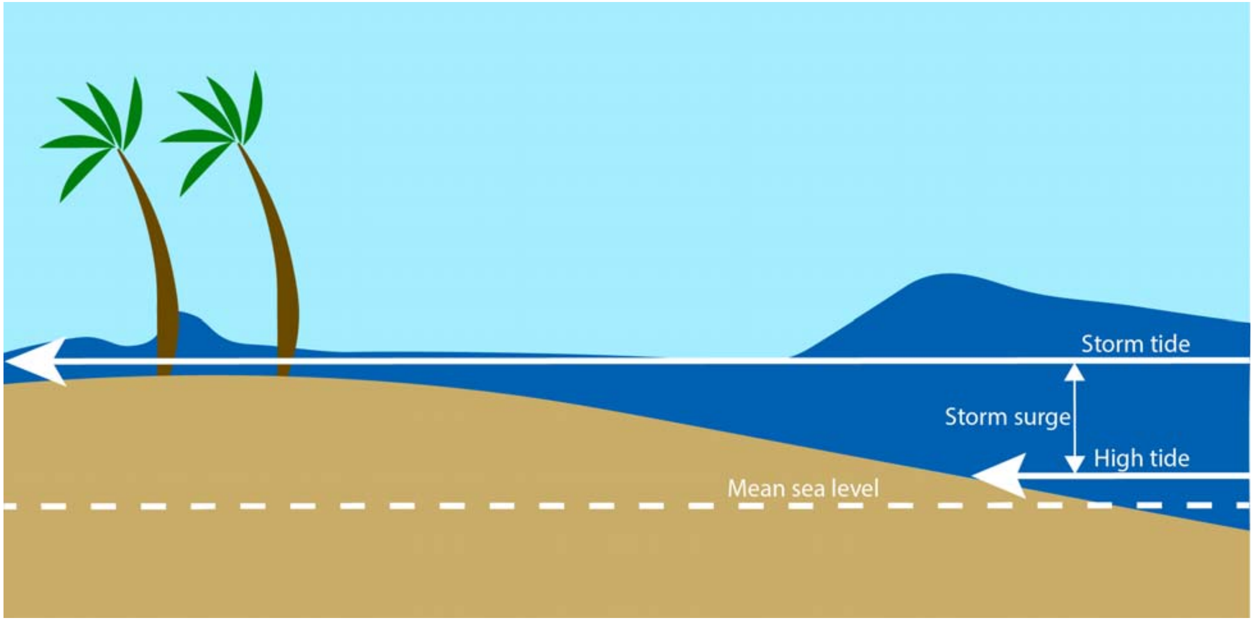


Figure 15-8. Storm Surge vs. Storm Tide (modified from NOAA Storm Surge Overview, https://www.nhc.noaa.gov/surge/)

Storm tides are an important consideration for transportation infrastructure located in the coastal environment because these events allow waves to reach farther inland, thus exposing areas at higher elevations to the risk of flooding and wave action. It can also influence the hydrodynamics along tidal inlets, increasing risk of scour around bridges and coastal structures.

Storm Tide Sources

The following list describes readily-available storm tide information that may be applicable for project design. The approach to derive storm tides varies in terms of the numerical equations used, assumed boundary conditions, quality of topographic and bathymetric data, and selection of statistical models used to estimate return intervals. Therefore, it is no surprise that the results between studies are likely to differ from each other, in some cases, significantly.

The storm tide sources described below, while useful for understanding relative extreme water elevations during storm events, will require review by a TxDOT Precertified Coastal Engineer as they may not be adequate for design of site-specific projects. For a Level 1 or 2 analysis, the designer may be able to obtain appropriate storm tide elevations from the sources listed below. These storm tide sources may also be applicable for a Level 3 analysis for initial project evaluation. Otherwise, numerical modeling (refer to the **Numerical Modeling of Water Elevations** subsection) may be required to calculate local storm tide conditions. FEMA flood insurance study data can also be useful for storm tide data, but the base flood elevation data they provide includes additional coastal factors such as wave and run-up data. Additional interpretation of this data would be necessary to determine stillwater levels.

- **U.S. Army Corps of Engineers (USACE)** — The USACE Galveston District is responsible for public navigation and engineering projects along the entire Texas coast. Prior USACE projects may have developed stillwater levels that could be used for transportation infrastructure design. For transportation projects planned near areas where USACE structures are located, it is recommended that the USACE District be contacted to inquire about any previous water level analyses that may have been performed for the area. Results will likely have been developed based on specific past project needs and should be carefully evaluated prior to use, but may be applicable for project design, particularly for a Level 1 analysis. https://www.swg.usace.army.mil/
- **NOAA Extreme Water Levels** — NOAA publishes storm tide estimates based on statistical analysis of historical tide data. Published values are often consistent with the stillwater level component of the FEMA base flood elevation. For NOAA-operated tide stations along the Texas coast, annual exceedance probability levels have been calculated for the 1-, 10-, 50, and 99-percent annual chance storm tides. https://tidesandcurrents.noaa.gov/est/

Extreme Storm History

In addition to obtaining numerical water level data, understanding the history of extreme storms and associated impacts at a project location is important for design. Although it is a qualitative assessment, reviewing prior storm history can provide valuable insight for project design based on observations of local impacts posed by past significant events.

For all analysis Levels, it is recommended that the local storm history be reviewed to consider local storm impacts. For Level 1, it may be reasonable to simply reference and compare high water marks or other recorded data to published flood frequency elevations. In particular, the designer should consider large storms that occurred after the FEMA maps were published. For example, Hurricane Harvey occurred in 2017. If the FEMA maps for the project area are dated 2010, then clearly the effects of Harvey potentially influencing the 100-year flood level estimates were not included. While a complete re-analysis may not be feasible for a given transportation project, multiple high stormwater events or even one very large storm after the FEMA elevations were published could indicate that the FEMA levels are underrepresenting the risk. Further consideration may be warranted and may elevate what was planned to be a Level 1 analysis into Level 2 or Level 2 to Level 3 analysis. For Levels 2 and 3, a more in-depth investigation of historical storm effects should be performed.

- NOAA Historical Hurricane Tracks — NOAA documents historical tropical system tracks based on user input. Options for input include location and radius of interest, timeframe, ocean basin, category, among others. The database includes hurricanes, tropical storms, tropical depressions, and extra-tropical storms. https://coast.noaa.gov/hurricanes
- NOAA Storm Events Database — NOAA maintains a database of recorded tropical and extra-tropical systems that can be searched by location, storm strength, and other factors. https://www.ncdc.noaa.gov/stormevents/

- Hurricane Reports — NOAA, USACE, the University of Texas–Bureau of Economic Geology, the Texas A&M University–Corpus Christi Harte Research Institute, and other public research institutes have published reports documenting the strength and impacts of many of the historical hurricanes that have impacted the Texas Gulf Coast. These reports can generally be obtained online and/or from library archives.
- Sea, Lake, and Overland Surges from Hurricanes (SLOSH) Model — NOAA's database compiles the results of the SLOSH numerical model developed by the National Weather Service. The SLOSH model provides estimates of storm surge heights for historical, hypothetical, and predicted hurricanes by modeling different storm tracks, approach angles and speeds, hurricane categories, and tide levels. Risk-based information (e.g., exceedance probability) and specific flood elevations are not provided, and location-specific storm surge elevations are provided as a function of storm intensity (e.g., Category 1, Category 2, etc.). The database can be viewed using the SLOSH Display Program (SDP). http://www.nhc.noaa.gov/surge/slosh.php.

  The SDP is intended for use by emergency managers to understand anticipated storm surge vulnerability. The results presented in the SDP can be used to understand worst possible surge levels. Although SLOSH model results are not likely to be used in roadway project design, they can be useful in the planning phase when assessing if the project site is vulnerable to hurricane storm surge exposure. SLOSH model results should be evaluated at each level of analysis for projects located in coastal areas to ascertain the risk from storm surge based on project location. If SLOSH results indicate that the site is exposed, it is recommended that additional freeboard or armoring be incorporated into the project design, particularly if it is a Level 2 or Level 3 analysis.

Numerical Modeling of Water Elevations

Where adequate long-term tide data are not available or when it is necessary to capture the dynamics of a complex shoreline or large-scale project (Level 3), numerical simulation of water levels can provide higher confidence in the selection of a design stillwater level. Numerical models account for site specific details and processes that give rise to complex interactions between water and the surrounding natural and built environments. The use of numerical models reduces the uncertainty associated with the representation of relevant coastal processes needed to design roads, bridges, and any other facilities TxDOT may develop. In general, a numeric modeling effort will need to be applied to appropriately capture the complexity of the study area. Table 15-3 describes the applicability of commonly used numerical models for roadway design.

Table 15-3: Applicability of Storm Tide Numerical Models Based on Level of Analysis

	Level 1 Analysis	Level 2 Analysis	Level 3 Analysis
Design or Modeling Inputs	FEMA flood map elevations, NOAA tide station data, USACE sea level maps, CHAMP outputs	FEMA flood map elevations, NOAA tide station data, USACE sea level maps, CHAMP outputs	2D and 3D hydrodynamic models including coupled wave, storm surge, and morphologic inputs (ADCIRC, Delft3D, MIKE21)

Additional information regarding model specifics (e.g., inputs, outputs) for each example can be found in the FHWA's *A Primer on Modeling in the Coastal Environment* or through consultation with a TxDOT Precertified Coastal Engineer.

Vertical Datums

Tide elevations are measured relative to a vertical datum, a reference system that allows one to locate a point on the Earth's surface. Without a common datum from which measurements are referenced, surveyors would calculate different elevation values for the same location. There are two main types of vertical datums for coastal applications: orthometric and tidal, defined below. Many cities also establish a city-specific vertical datum relative to a local point of reference (for example, high water line or mean sea level). Most coastal water level data is collected in reference to a local tidal datum, while most transportation projects are designed based on an orthometric datum. As a result, it is common to require conversion between datums to successfully evaluate coastal conditions.

Tidal Datums

Tidal datums are based on tidally-derived surfaces of high or low water elevations defined by phases of the tide (NOAA Vertical Datum Transformation). They are used to describe the average location where the water and land intersect for each major tidal phase. The hydrodynamics of tidal fluctuations are controlled by local processes; thus, it is important to remember that tidal datums provide a local tide characterization and will vary along the coastline.

Commonly Used Tidal Datums

- **Mean High Water (MHW):** Mean of all high-water elevations observed during NTDE
- **Mean Sea Level (MSL):** Mean of hourly water levels observed during NTDE
- **Mean Low Water (MLW):** Mean of all low water elevations observed during NTDE

In areas with mixed semi-diurnal tides, such as the Northern Texas coast, two additional datums are defined:

- **Mean Higher High Water (MHHW):** Mean of the higher of the two daily high-water elevations observed during the NTDE
- **Mean Lower Low Water (MLLW):** Mean of the lower of the two daily low water elevations observed during NTDE

All tidal datums are referenced to a 19-year averaging period known as the National Tidal Datum Epoch (NTDE). This 19-year period is significant, as it encompasses the length of time necessary for variations in lunar cycles (which influence tide levels) to occur. The current NTDE spans from 1983-2001 and is actively considered for revision every 20-25 years. Use of NTDE allows tidal datums throughout the U.S. to have a common reference. Tidal datums are commonly reported relative to the Mean Lower Low Water (MLLW), which is the lowest reported tidal datum. For example, when a report states the Mean High Water (MHW) at the Galveston Pier 21 station (NOAA #8771450) is 1.32ft, it means that, on average, over the 19-year period of 1983-2001, the average high tide was about 1.32ft above MLLW (NOAA Tides and Currents, 2019). However, tidal elevations can also be easily expressed relative to other datums reported at the same tide station. Refer to the **Relationships Among Datums** subsection for more information and common conversions.

Orthometric Datums

Orthometric datums use the Earth's gravity field to reference heights. The North American Vertical Datum of 1988 (NAVD88) is the current national standard vertical datum. It replaced the National Geodetic Vertical Datum of 1929 (NGVD29), which was the previous national standard for 60 years. Refer to NOAA for the most current orthometric datum information.

Relationships Among Datums

The relationship among datums often varies from one area of the shoreline to another. To make a comparison with land surveyed data, all data must be converted to a standard reference, such as NAVD88. As an example, Table 15-4 presents the relationship between tide heights referenced to NAVD88 and MLLW datums at the Galveston Pier 21 tide station.

To evaluate water surface elevations relative to the various datums reported for a tide station, visit the NOAA or TCOON tidal datum section of the tide station's website. A list of tidal datums, similar to those listed in Table 15-4 provided as an example for Galveston Pier 21 gauge, will be provided. Conversions between the datums can be calculated by making the datums relative to each other through subtraction. For example, to convert MHHW referenced to MLLW to be relative to NAVD88, subtract 0.14 feet from 1.41 feet in the right-hand column to obtain 1.27 feet in the left-hand column. This comparison indicates that MHHW is 1.27 feet above NAVD88 and 1.41 feet above the MLLW.

Table 15-4: Galveston Pier 21 (NOAA #8771450) Vertical Datums (NOAA Tides and Currents, 2019)

Datum	Relative To	
	NAVD88 (feet)	MLLW (feet)
MHHW	1.27	1.41
MHW	1.18	1.32
MSL	0.69	0.83
MLW	0.16	0.30
MLLW	-0.14	0.00
NAVD88	0.00	0.14

The relationship between NAVD88 and tidal datums has been calculated by NOAA for many of the tide gages along the Texas coastline. The conversion is different for every tide station. Therefore, investigating the relationship between tidal datums, orthometric datums, and site-specific upland surveys used for project design is very important. If a conversion for NAVD88 has not been calculated for a tide station of interest, refer to a professional surveyor or the USACE District office to obtain the necessary offset.

Tidal Datum Sources

- NOAA CO-OPS — Tidal datums are available at many of NOAA's tidal recording stations. https://tidesandcurrents.noaa.gov/
- TCOON — Tidal datums are available at many TCOON recording stations. http://cbi.tamucc.edu/TCOON/

Relative Sea Level Rise

Many of the roadway infrastructure networks located along the Texas shoreline have been in place since the early- to mid-1900s and were constructed with the underlying assumption that coastal water levels are stationary through time. Contemporary data trends in sea level rise call this assumption into question, resulting in uncertainty about the future performance of existing roadway infrastructure. Future sea level rise will increase the existing baseline elevations upon which daily tidal variations are measured and are therefore important to consider in project design.

Relative sea level rise reflects the combination of large-scale changes in global ocean levels (global mean sea level) and local changes in land elevations (for example, due to subsidence, or sinking, of land). Rates of relative sea level rise along the Texas coast are some of the highest in the nation: 4.0 mm/year (0.16 inches/year) at Port Isabel (1944-2018), 5.62 mm/year (0.22 inches/year) at Rockport (1948-2018), and 6.51 mm/year (0.24 inches/year) at Galveston Pleasure Pier (1908-2018) as shown in Figure 15-9 (NOAA Tides and Currents, 2019). These high rates of relative sea level rise are partially linked to subsidence exacerbated by the historical extraction of subsurface groundwater, oil, and gas.

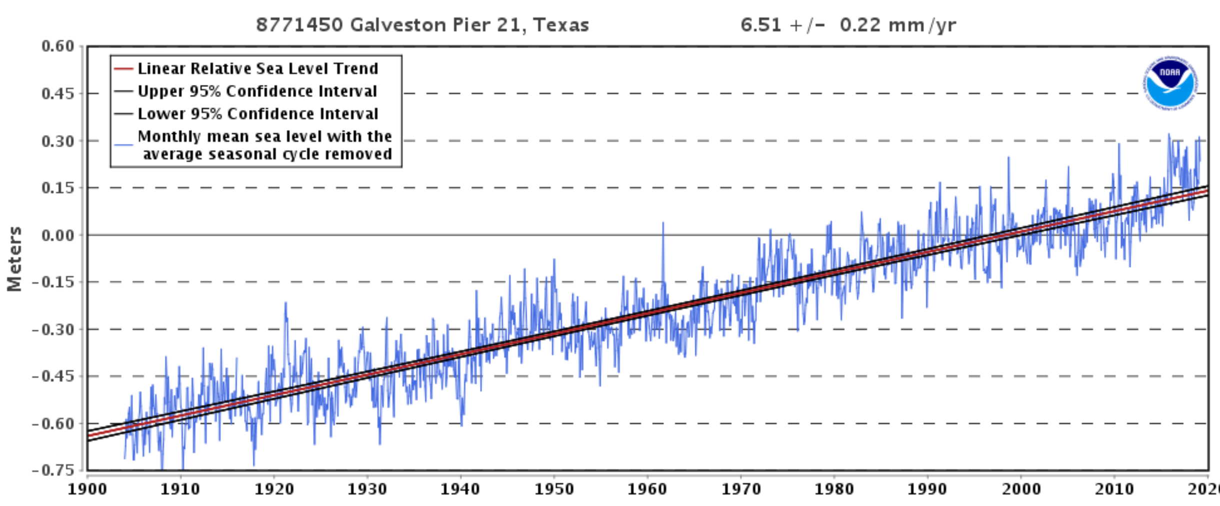


Figure 15-9. Historical Relative Sea Level Rise Rates at Galveston Pier 21 (NOAA Tides and Currents, 2019).

Figure 15-10 illustrates the projected sea level rise for points along the Texas coast in comparison to other national rates. Increases in relative sea level rise may have the following impacts on Texas transportation assets located in coastal areas:

- **Roadway flooding.** As coastal water levels increase, low-lying transportation routes may become exposed to more frequent and more intense flood events, interrupting roadway access and increasing maintenance costs.
- **Efficiency of stormwater drainage.** As low-lying stormwater outfalls become partially or completely inundated by rising coastal water levels, stormwater drainage may be impeded, resulting in roadway flooding. In addition, increases in coastal groundwater levels due to sea level rise may reduce the efficiency of existing stormwater drainage systems.
- **Erosion damage.** Elevated coastal water levels will allow wave action to reach higher elevations, which may cause erosion or scour near transportation features, such as unarmored embankments, causeways, and bridge pilings.

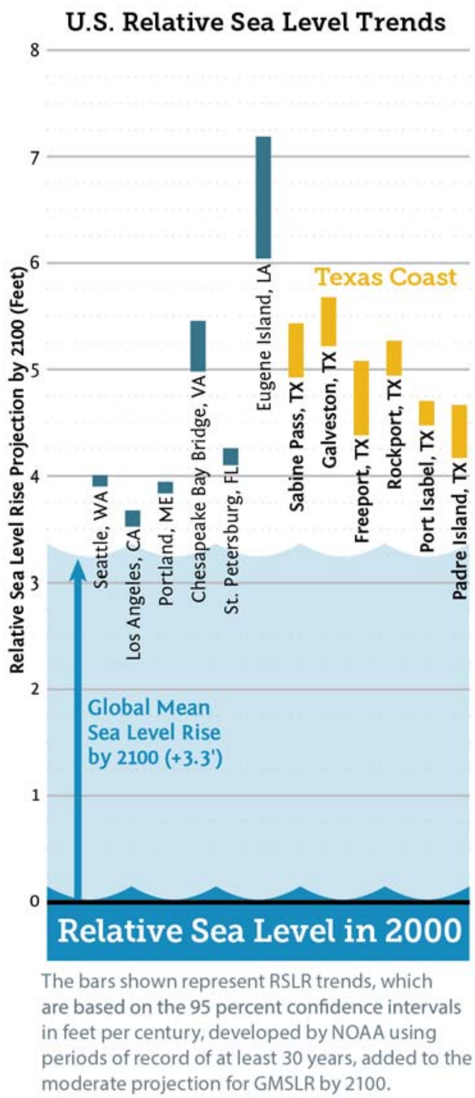


Figure 15-10. U.S. RSLR Trends

Due to the nature of transportation infrastructure (fixed alignment and elevation, linear features, long lifespan, etc.), assets are often limited in their ability to adapt to future sea level conditions. It is therefore important to incorporate future changes into project design whenever possible to cope with potential impacts of changing coastal conditions and minimize future cost and risk to these assets. Table 15-5 presents additional screening criteria that should be considered.

Table 15-5: Sea Level Rise Consideration Factors

	Factor to Consider	More Critical to Incorporate Sea Level Rise	Less Critical to Incorporate Sea Level Rise
1	Project Design Life	Long (20+ years)	Short (less than 20 years)
2	Redundancy/	No redundant/alternative route	Redundant/alternative route exists
3	Alternative Routes	Substantial delays	Minor or no delay
4	Anticipated Travel Delays Due to Sea Level Rise	Critical route for commercial goods movement	Non-critical route for commercial goods movement
5	Goods Movement	Vital for emergency evacuations	Minor or no delay in event of emergency
6	Evacuation/Emergency	Non-safety project	Safety project and delay would be substantial
7	Traveler Safety	Large investment	Small investment
8	(Delaying the Project to Incorporate Sea Level Rise would Lead to On-going or New Safety Concerns)	Minor or no effect – adjacent street and roads would not need to be modified	Substantial interconnectivity issues
9	Expenditure of Public Funds	Minor or no increase in project footprint in environmentally sensitive area	Substantial increase in project footprint in environmentally sensitive area
10	Interconnectivity Issues with Local Streets and Roads	Incorporating sea level rise may extend design into adjacent water bodies and/or properties or affect drainage of adjacent properties	Incorporating sea level rise will not have an impact on adjacent water bodies and/or properties

Selecting a Sea Level Rise Value for Design

Relative sea level rise should be incorporated in project design if project is characterized as having:
- design life greater than 20 years,
- large investment of public funds, or
- low risk tolerance.

When incorporating relative sea level rise into project design, it is important to select a future sea level rise projection that considers the project design life and risk tolerance to flooding. Climate change modeling and sea level rise projections have continued to evolve with significant advances in the understanding of global and regional factors that contribute to relative sea level rise. In 2019, the Texas General Land Office (GLO) published an updated Texas Coastal Resiliency Master Plan (GLO Plan), which includes detailed relative sea level rise scenarios for the state. The GLO Plan relies on the global sea level rise projections released in the 2017 NOAA report Global and Regional Sea Level Rise Scenarios for the United States, which reflects the latest published and peer-reviewed sea level science. The 2017 NOAA report includes six global sea level rise scenarios (low, intermediate low, intermediate, intermediate high, high, and extreme) to examine the full range of potential future water levels.

The 2017 NOAA projections have the added advantage of providing risk-based (probabilistic) planning capabilities, which were not previously available. The range of probabilities for each planning timeframe is dependent on modeled future climate conditions (referred to as Representative Concentration Pathways – or RCP), as described in the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment Report (AR5).

To address the impacts of relative sea level rise during the next 50 to 100 years, the GLO Plan adjusted NOAA's intermediate scenario of a 3.3-foot global mean sea level increase by 2100 to account for local effects (e.g., vertical land movement, tectonics, and sediment compaction) through statistical analysis of local tide stations along the Texas coast. This intermediate scenario has a 2 to 17 percent chance of being exceeded by future global sea levels by the year 2100. These local effects are what differentiate sea level rise from relative sea level rise and account for the relative portion of the rates.

Because of the diversity and expanse of the Texas coastline, the GLO Plan divides the coastal area into four regions to provide a more focused assessment within each region (Figure 15-11). The regions contain unique environmental characteristics, land use patterns, and vertical land movement that affect local water levels. Table 15-6 provides average relative sea level rise projections for each region. When considering future conditions for project design, a regional projection representative of the project location should be selected.

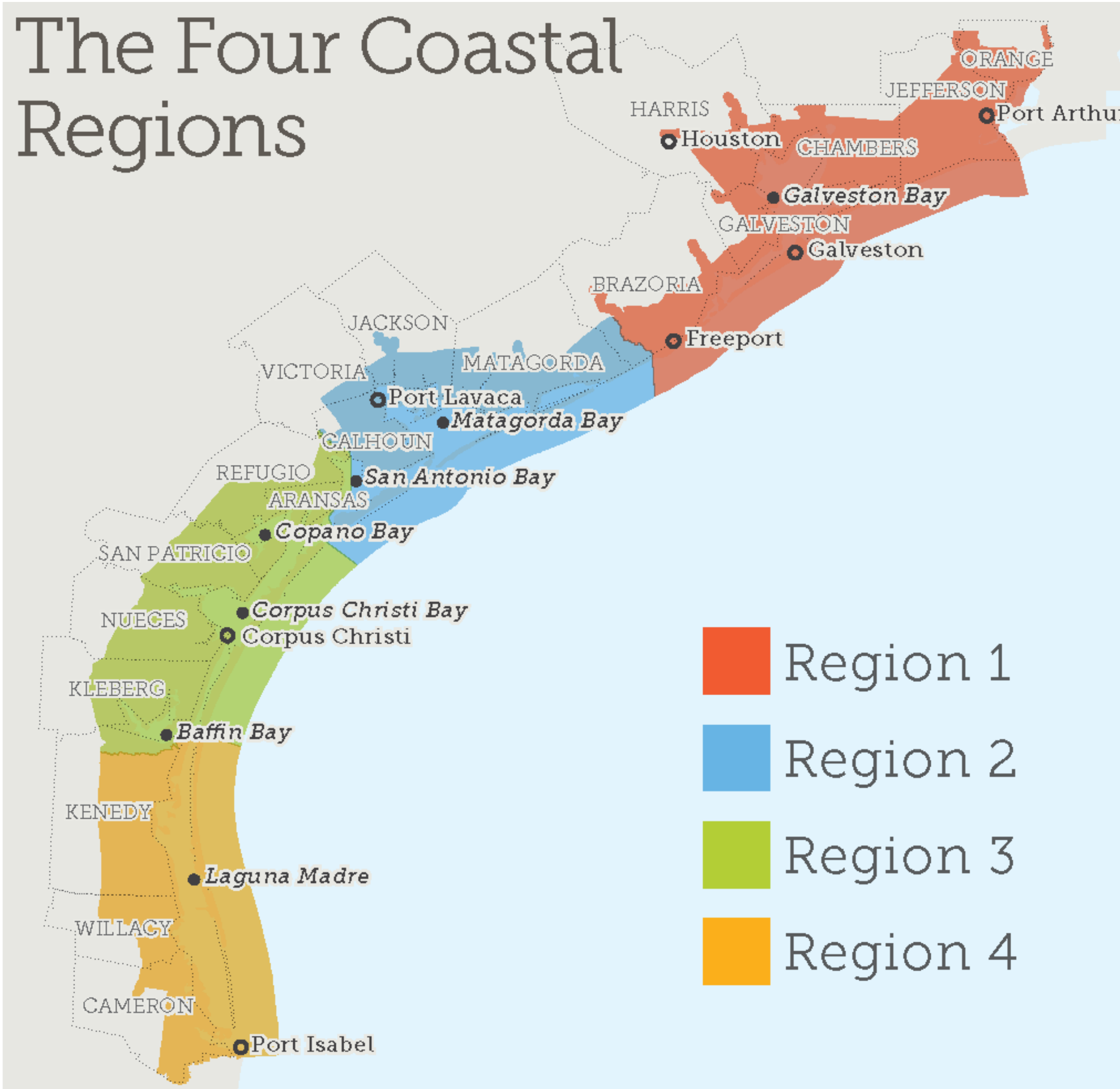


Figure 15-11. The GLO Texas Coastal Resiliency Master Plan's Four Coastal Regions (Texas General Land Office, 2019).

Table 15-6: Relative Sea Level Rise Projections (Texas General Land Office, 2019)

Planning Time Horizon (Year)	Relative Sea Level Rise (feet)			
	Region 1	Region 2	Region 3	Region 4
2020	0.8	0.8	0.7	0.6
2030	1.3	1.2	1.1	1.0
2040	1.7	1.6	1.5	1.3
2050	2.2	2.1	1.9	1.7
2060	2.8	2.6	2.4	2.2
2070	3.4	3.2	3.0	2.8
2080	4.1	3.9	3.6	3.3
2090	4.8	4.6	4.3	4.0
2100	5.5	5.2	5.0	4.6

Notes:

1. Projections are relative to 2000 and should be modified to a baseline for the estimated rise that has occurred between 2000 and the project start year to estimate the rise expected during the project life.
2. Projections are regional averages of projections calculated in the GLO Plan and include anticipated local subsidence.
3. Alternate RSLR scenarios and projections may be considered if it is determined that they are more appropriate for a given project's level of risk.

The design life of transportation infrastructure describes the time period the structure is expected to sustain usability under normal loads and conditions and varies depending on the particular asset type. TxDOT refers to the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications for bridge design life, and bridges are typically designed considering a 75-year period. Roadways are typically designed considering a 50-year period. In both instances, the relative sea level rise over the lifespan of the asset may be significant and should be considered in the design. Quantifying relative sea level rise into the project design will be dependent on the design life.

For example, if a project takes place in Region 1 and entails the installation of flexible pavement with a design life of 20 years starting in the year 2020, 0.9 feet of relative sea level rise is recommended in the design. This is calculated by subtracting 0.8 feet of relative sea level rise (year 2020) from 1.7 feet of relative sea level rise (year 2040) to account for the relative sea level rise that is expected to occur since the start of the project.

To consider a more complex example, consider a project in Region 2 starting in 2025 that will install concrete pavement roadway, which has a 30-year design life, connecting to a bridge, which has a 75-year design life. The relative sea level rise projection used for the road would be 1.4 feet (2.4 feet [year 2055] minus 1.0 feet [year 2025]). The relative sea level rise projection for the bridge would be 4.2 feet (5.2 feet [year 2100] minus 1.0 feet [year 2025]). The designer will need to consider whether elevating the roadway to the elevation of the bridge is feasible and appropriate for the project to conserve the life of the bridge asset.

To allow incremental adjustments to manage the impacts of relative sea level rise, the design of some transportation assets (e.g., causeway heights, pavement surfaces, facility protection design, and roadside vegetation) could also target a shorter intended design life. For example, a causeway could consider a 30-year design life rather than a 75-year design life so that future climate conditions are more moderate and achievable based on project cost restrictions and efficiencies. The **Consideration of Sea Level Rise in Project Design** section below describes additional criteria to consider when incorporating relative sea level rise into transportation projects.

In Summer 2019, FHWA is expected to release an update to their Hydraulic Engineering Circular 25 (HEC-25) Highways in the Coastal Environment: Assessing Extreme Events report. The updated edition will include recommendations for incorporating sea level rise into project design and should be reviewed prior to development of stillwater levels.

Considering Relative Sea Level Rise in Project Design

Once projections for relative sea level rise have been identified for the project, the following procedures should be followed:

1. Obtain elevation data for project site features (e.g., roadway, culvert, bridge). Project elevations can be obtained from as-built drawings for maintenance projects or from land surveys completed for planned projects.
2. Select a relative sea level rise projection from Table 15-6 based on the appropriate region to assess potential impacts.
3. Using the project survey elevation data collected in step 1 and relative sea level projections from step 2, assess the relative sea level rise over the project lifespan.
4. If applicable, identify the possible negative impacts of relative sea level rise on the project related to asset function or operation. Possible impacts include scour and/or erosion due to tidal action, reduced efficiency of drainage culverts due to higher tailwater conditions, and exposure to saltwater.
5. For identified impacts, assess if adaptive measures will be necessary. In many cases, the project footprint may be impacted, but no adaptive measures may be required. Impacts may also be temporary (e.g., wave splash during high tide events or during storms). Not all adaptive measures require physical alteration to the roadway design. Temporary impacts may be addressed through operational modifications, such as short-term road closures.
6. Identify the cost of potential relative sea level rise adaptive measures. Due to cost limitations, not all relative sea level rise adaptive measures may be included in the project design. For example, raising a roadway could cause a larger fill slope to encroach into an environmentally sensitive area. Assessments of potential adaptation measures and any limitations should be documented to indicate what can be achieved through evaluating the cost of adaptation vs. the cost of inaction. Costs should be considered in terms of economic, environmental, and social/human impacts.

7. Where feasible, adaptive measures for relative sea level rise (e.g., roads elevated on berms, bridge height and on ramp adjusted for future sea levels, long-term planned retreat, enhanced erosion protection along roadway) should be incorporated into project design, particularly where future impacts are anticipated.
8. Consider unintended hydraulic impacts when designing relative sea level rise adaptive measures. For example, elevating roadways may impede floodwater drainage or affect flooding of adjacent properties. To offset these impacts, incorporating additional drainage mechanisms into project design can alleviate flooding of low-lying areas located near the modified roadway.

Sea Level Rise Projection Sources

- **Texas General Land Office** — 2019 Texas Coastal Resiliency Master Plan, published February 28, 2019. http://coastalstudy.texas.gov/resources/files/2019-coastal-master-plan.pdf
- **FHWA** — Highways in the Coastal Environment: Volume 3, anticipated publish date in Summer 2019
- **NOAA** – Global and Regional Sea Level Rise Scenarios for the United States, published January 2017. e 2017 NOAA report Global and Regional Sea Level Rise Scenarios for the United States

Selecting Stillwater Levels for Project Design

Selecting appropriate stillwater levels for each project is critical for transportation infrastructure located in coastal areas. Stillwater levels are used as input for the design wave height (Section 3), which when combined, determine the design elevation. This cumulative elevation is discussed further in Section 4 of this chapter. The stillwater level at a project site is composed of a combination of the appropriate astronomical tide and storm surge, as previously discussed. If sea level rise is to be accounted for, it will be cumulatively combined with the tidal information.

Stillwater level is a combination of:
- Astronomical Tides
- Storm Surge
- Sea Level Rise

While the 1% AEP storm tide is the primary flood zone mapped by FEMA as either AE or VE zones, coastal transportation projects (e.g., local roadway) frequently justify a lower AEP (e.g., 50% to 10% AEP) and others (e.g., freeways or critical evacuation routes) that may justify a higher exceedance probability (e.g., 2% or 1% AEP). Knowing when, where, and how to appropriately select and apply a stillwater level comes from experience and sound judgement. Refer to Chapter 4 (Hydrology), Section 6 (Design Flood and Check Flood Standards) as a starting point for consideration by roadway classification and structure type. As demonstrated in this section, this decision is dependent on numerous factors and can be a subjective process unique to the coastal zone. Proficiencies in this topic typically reside with the Precertified Coastal Engineer; however, it is important that the designer demonstrate knowledge and competence assessing the project risk tolerance, budget restraints, and social and environmental impacts to select the most appropriate stillwater level for a safe, yet cost-effective project design.

For a project relying on a Level 1 or 2 analysis (e.g., local roads/street or other non-critical assets), it may be acceptable to develop a stillwater elevation by adding an appropriate sea level rise projection, as determined by the project's design life and risk tolerance, to a FEMA return period storm tide collected from the latest FIS for the project area (after excluding wave heights).

More complex projects that require a Level 3 analysis are likely to require additional effort. To obtain a site-specific water elevation needed for design, the designer may need to work with a Precertified Coastal Engineer to perform risk-based modeling of water level conditions that capture the local variability experienced at the project site. These models will simulate a number of possible independent storm scenarios that include a variety of storm parameters, including wind speed, pressure, and landfall angle, among others, combined with statistical analysis to determine the probability of the storm tide occurring at the project site. These models will also be capable of incorporating changes in relative sea level to evaluate how the return period design events chosen may evolve over time. Some descriptions of how these models are developed and utilized are described in subsequent sections.

Section 4 — Waves & Currents

Introduction

The dynamic and complex interaction of waves, currents, tides, and land in coastal areas are collectively known as nearshore processes. "Nearshore" generally refers to the coastal area where waves begin to break onshore to the landward limit of storm-induced wave action at the beach face.

It is very important that those responsible for design and maintenance of infrastructure, particularly roadways and bridges located in coastal areas, have accurate and detailed information regarding nearshore processes. Low-lying roadways and bridges are especially vulnerable to wave, current, and tide impacts during tropical storms and hurricanes, which are often associated with large storm surge, strong wave forces, and increased risk of erosion.

This section seeks to:

1. Synthesize – at a high level – the various factors that contribute to nearshore forces (waves and currents) along the Texas coastline; and
2. Provide guidance on estimating design wave heights and velocities necessary for engineering of coastal transportation infrastructure and coastal armoring for transportation asset protection.

Waves

Waves are caused by a disturbance of the water surface. The origin of the disturbance may be winds, boat or ship wakes, or other forces, such as underwater landslides due to earthquakes (tsunamis). Most waves observed at the shoreline are generated when wind blows over a vast expanse of the sea. Winds, especially during storm events, can impart a tremendous amount of energy into waves. After storm waves are formed, they may propagate across the surface of the sea for thousands of miles until their energy is dissipated, usually through wave breaking. Once waves arrive at the shoreline, their energy can be the primary cause of erosion or may generate nearshore currents and influence sediment transport patterns.

Several equations and theories have been developed to approximate wave characteristics, but the most basic approximation is to assume a regular sinusoidal wave. Linear wave theory applies numerous simplifying assumptions to wave calculations—for instance, that each wave does not interact with any other fluid motions (including other waves) and that waves maintain a sinusoidal shape. In reality, waves are typically much more complex and nonlinear, often propagating as a random mix of larger and smaller waves. For more complex cases, numerical models can be applied (refer to Numerical Modeling of Waves subsection). Numerical models include influences from bay and channel geometry, wind, bathymetry, and other factors. They are often computationally intensive and will likely require consultation with a coastal engineering specialist.

For the Texas coastline, wave impacts should be considered for transportation infrastructure along the open coast, as well as infrastructure that is subject to locally-generated waves within bays and inlets, and boat traffic-induced wakes acting on shorelines in navigation channels, such as the Gulf Intracoastal Waterway (GIWW).

Site-specific wave data can provide insight into the type and magnitude of wave forces at a structure. This information can be collected through direct measurements that are used as inputs in numerical or physical models. Direct measurements of waves in the vicinity of the project during a specific period of time can be collected via a wave gauge; however, it should be cautioned that gauges capture constantly changing wave condition requiring an analysis in order to obtain statistically representative wave data. Numerical or physical modeling may be conducted to approximate the design wave characteristics using inputs including wind, stillwater levels, and bathymetry, which are often readily available from monitoring stations. Additionally, or alternatively, the USACE Wave Information Studies (WIS) system provides wave hindcasts, or predictions based on past events and wind observations, that can be used to better understand the approximate wave characteristics. More information on WIS is discussed in the data sources below. Wave measurements and hindcasts tend to be publicly available for areas within the Gulf of Mexico but are generally not available for Texas bays and estuaries.

Wave Dynamics

Almost all waves in the Gulf of Mexico and Texas bays are generated by winds. The heights and periods of wind-generated waves, important parameters used in engineering calculations, are governed by wind velocity, the duration of time that the wind blows, and the fetch, or distance, over which the wind blows (Figure 15-12). Because the fetch distance restricts the time during which energy can be transferred from wind to the waves, longer fetches, such as the open coast, tend to generate larger waves than will be experienced in shorter fetch areas, such as a bay. Other factors, such as water depth and overall bathymetry, also affect wave generation, growth, and propagation.

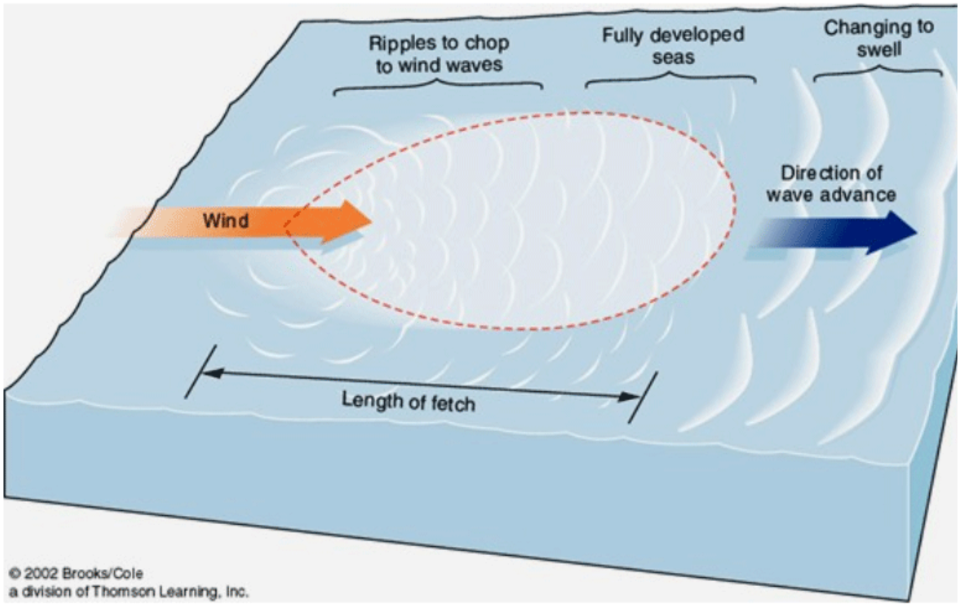


Figure 15-12. Conceptual Drawing of Length of Fetch (Brooks/Cole, 2002)

During a storm, wave patterns are highly complex and disorganized as the storm generates many different sets of waves with varying wave heights and little predictability over the length of fetch. As waves leave the storm area, they become more sorted (organized) based on their wave speeds and develop into a swell, having uniform wave heights and energies. The motion of a developed swell is periodic, or repetitive through fixed periods of time.

Figure 15-13 depicts basic parameter definitions for waves. The length dimensions used to define the wave are the wavelength (L), defined as the distance between wave crests, and the wave height (H), defined as the difference between the elevation of the crest and trough of an individual wave. The water depth (d) is defined as the distance from the bottom surface to the stillwater level (SWL), which is the level of water if waves were not present. The wave period (T) is the time required for a wave to travel one wavelength.

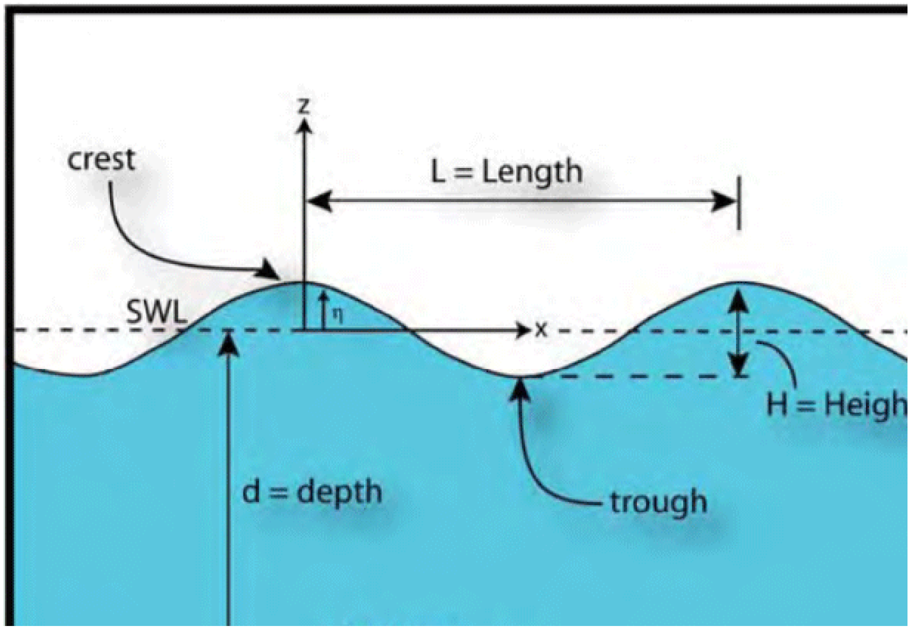


Figure 15-13. Wave Parameter Definitions (FHWA, 2008)

Design Wave Height

Designing to consider the maximum wave height is typically not warranted, because the maximum wave has the chance of being a statistical outlier or a measurement error and putting undue cost burden on project design and construction compared to the probability of occurrence. A commonly used way to apply wave height information in infrastructure design is through a parameter known as the significant wave height. This is a statistically-derived average of the highest one third of the waves measured over a period of time. The significant wave height is important to capture the potential impact of larger waves, which are more 'significant' than the smaller waves in terms of infrastructure impacts in coastal environments. The standard sampling time to measure the state of the sea is usually around 17 to 20 minutes, which represents the time interval needed to capture consistent conditions (Thompson, E.F. and C.L. Vincent, 1985). Sampling over shorter time periods may introduce too much variability and give incorrect results for the significant wave height.

Wave Breaking

Within the nearshore zone is a location where waves begin to enter shallow water, creating increases in wave height as waves move towards the shoreline. This process of wave transformation is also known as shoaling. When waves increase in height, they become steeper and begin to break in the relatively narrow area of the nearshore that contains wave breaking, called the surf zone. The impact of breaking waves on coastal infrastructure is an important design consideration.

Breaking waves create significant turbulence and energy transfer to the feature they impact. In shallow water, waves break when they reach a limiting shallow depth (depth-limited breaking).

Eventually, the waves become overly steep and break with the crest thrown forward as the wave dissipates near the shoreline. This phenomenon typically occurs as waves enter water that is approximately as deep as the waves are high. As waves travel landward, shoaling begins to occur and the ratio of wave height to water depth increases. After breaking, the wave may reform as a smaller wave and continue propagating towards the shoreline. This pattern may occur repeatedly until the wave becomes completely dissipated in shallow water.

The depth-limited breaking conditions in shallow water can be a very important consideration in the design of coastal revetments that protect highways and when determining coastal bridge elevations. A practical value for determining appropriate wave heights used in design for Texas, where there is a mild, sandy slope offshore of the roadway structure, is:

$$\left(\frac{H}{d}\right)_{max} = 0.8$$

Equation 15-1. (HEC-25)

Where:

d = local water depth (including tides and storm surge)

H = wave height

This relationship can be rearranged to describe the depth-limited wave height as:

$$H_{max} = 0.8d$$

Equation 15-2. (HEC-25)

This relationship is useful in selecting the upper limit for a design wave height for coastal structures in shallow water. Given an estimate of the water depth at the structure location, the maximum wave height (H_{max}) that can exist in the depth of water is known. Any larger waves would have broken farther offshore.

Wave Refraction

As waves enter shallow water, they are subject to a process called wave refraction, which is the directional change waves experience as they propagate over different depths. Wave refraction is important to consider in roadway and bridge design, as it can affect local wave height and influence the strength and speed of waves reaching structures. Along typical (relatively straight) coastlines, waves tend to become parallel with the shoreline as shown in Figure 15-14. This occurs due to the faster movement of waves in deeper water than shallower water, causing the wave to rotate. Irregular bottom topography or shorelines can cause complex wave refraction, producing significant variations and wave heights and energies along the coast (Figure 15-15). For instance, if waves encounter an obstruction, such as a reef, on the sea floor, waves will start to refract around it, converging and focusing their energy at the center of the obstruction. This creates higher, more powerful waves near the obstruction area. Conversely, if waves encounter a large, convex coastline, such as a shallow bay, waves start to bend outwards, diverging and dispersing energy away from the shoreline and reducing the wave height. Wave convergence or divergence due to refraction is an important consideration for sizing and placing coastal armoring for transportation infrastructure.

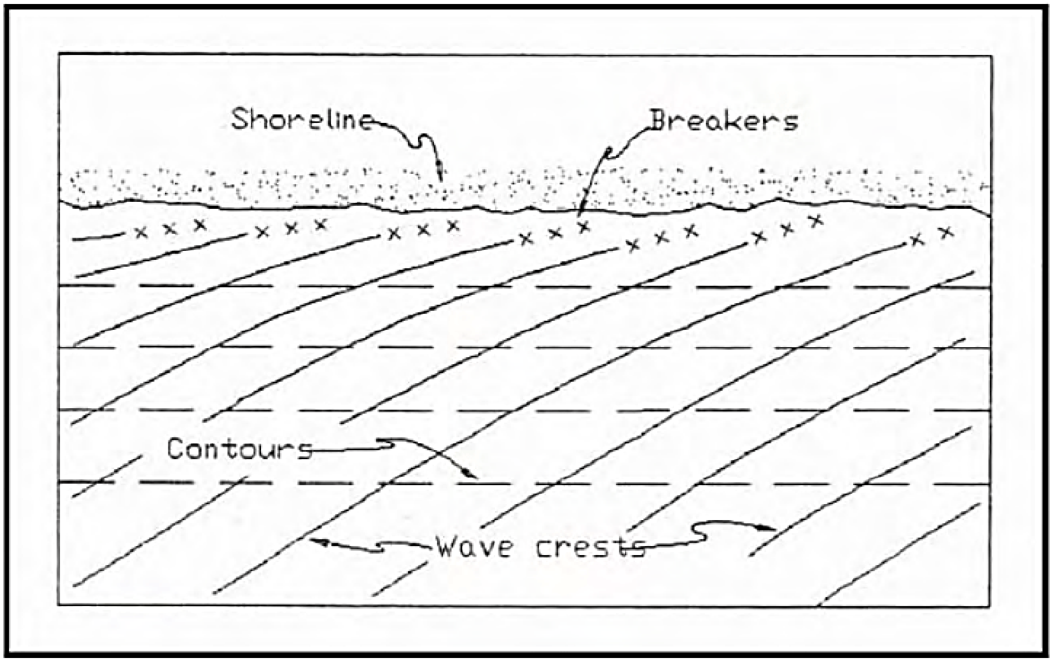


Figure 15-14. Example of Wave Refraction along a Straight Shoreline (USACE, 2002)

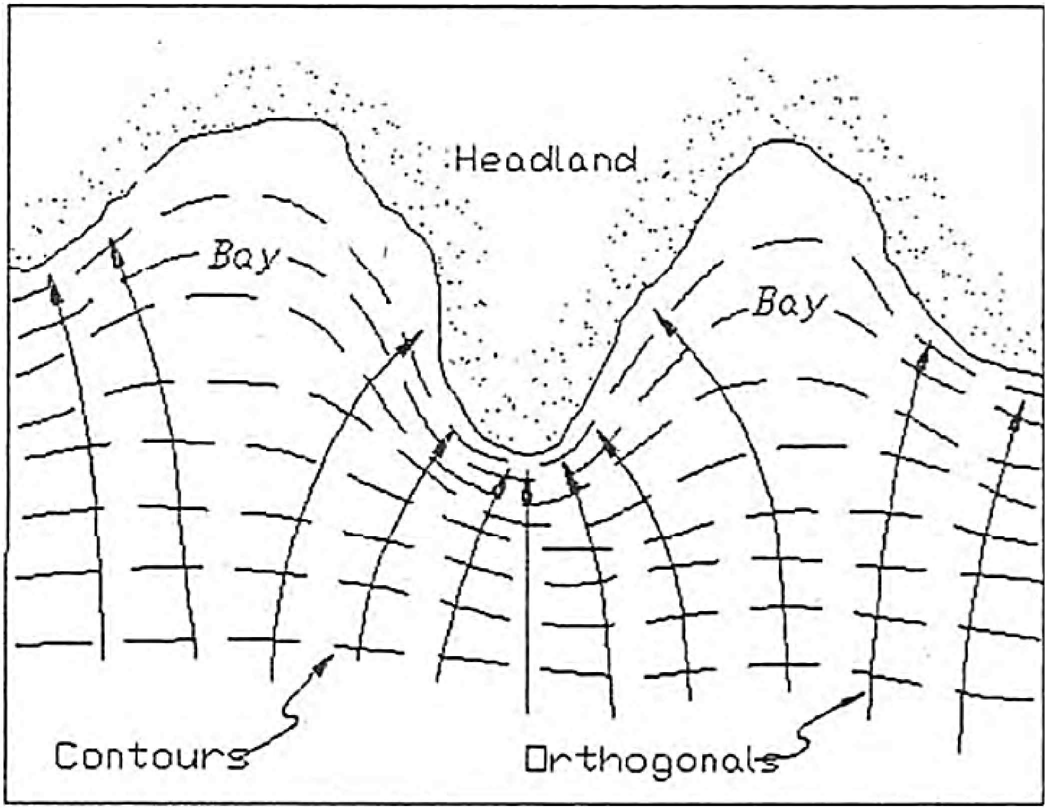


Figure 15-15. Example of Wave Refraction along a Complex Shoreline (USACE, 2002)

Wave Diffraction

Wave diffraction is another nearshore process related to wave refraction. The phenomenon occurs when waves propagate past a structure (e.g., jetty or breakwater), barrier island, or narrow inlet opening (Figure 15-16). The feature interrupts the wave energy and changes wave direction as waves wrap around the feature. Although wave refraction and wave diffraction both affect the direction of waves, refraction is affected by water depth while diffraction is not. The process of wave diffraction often creates a sheltered zone of low energy on the opposite side of the feature being impacted. It can, therefore, be an important consideration when placing coastal armoring, such as breakwaters, to attenuate wave energy and protect roadways vulnerable to erosion forces.

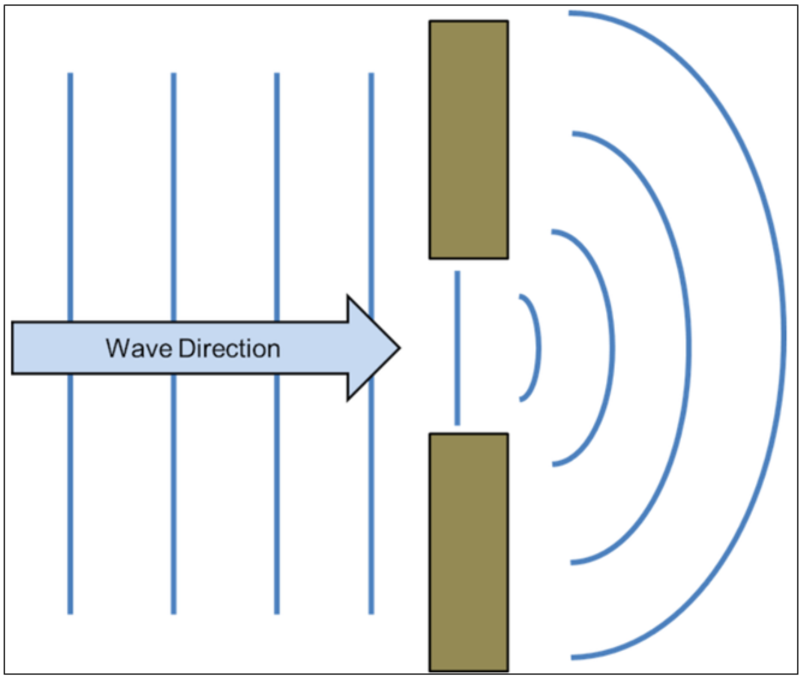


Figure 15-16. Example of Wave Diffraction (University of Hawaii, n.d. Wave-Coast Interactions)

Wave Reflection

A third way that waves can change is when waves encounter vertical structures (e.g., bulkhead or seawall). In these cases, the waves may reflect, or bounce back, from the shoreline. Depending on the amount of energy remaining in the wave, reflection may cause complex wave interactions, enhance local wave heights, and increase the likelihood for scour at the base of the vertical structure. Therefore, local wave patterns are important to understand prior to placing vertical armoring.

The Influence of Barrier Islands

The Texas shoreline is characterized by a chain of barrier islands that separate the bays from the Gulf of Mexico. These narrow islands provide the first line of defense for storm surge and reduce the wave energy that reaches the mainland. The low-wave energy area landward of the islands provides a sheltered environment for estuaries to form and helps create more stable shorelines on the mainland.

Barrier islands are highly dynamic landforms that evolve in shape and migrate due to changes in sea levels and wave forces. Over time, the islands often migrate toward the mainland, as waves wash over the islands during storms and transport sediment from the Gulf side to the inland side.

Built infrastructure, such as roadways, on barrier islands interrupts the islands' natural evolution. To protect established infrastructure, armoring (e.g., seawalls and groins) and beach nourishment practices are commonly used to maintain the existing barrier island position. Regardless of engineering attempts to preserve the islands, large hurricanes commonly overwash the features, damaging infrastructure, or breach the islands, creating new inlets.

Changes to the shape of the islands or their diminishing due to sea level rise or erosion may lead to increased exposure of infrastructure located on the barrier islands and the mainland.

Figure 15-17. Damages on Bolivar Peninsula following Hurricane Ike.

Wave Dynamics Sources

The following two sources can be used for wave dynamic sources data. However, due to the complex nature of waves, in many cases, wave generation, growth, and transformation are challenging to determine using empirical methods alone.

- **USACE Coastal Engineering Manual (CEM)** — Relatively simple empirical methods for wave calculation are presented in Part 2, Chapter 2 of the CEM. These can be useful references for estimating potential wave heights for fetch-limited conditions, breaking wave heights, and wave heights associated with a variety of hurricane strengths. https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/u43544q/636F617374616C20656E67696E656572696E67206D616E75616C/
- **HEC-25** — This can be a useful reference for empirical methods for wave calculation. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/07096/07096.pdf

Wave Impacts on Coastal Structures

Waves have the potential to create a variety of loading conditions on coastal bridges and scouring effects on roadways. Due to the dynamic nature of waves, they may create both lateral and uplift forces on bridges spanning waters subject to coastal storms. Guidance from AASHTO's (2008) Guide Specifications for Bridges Vulnerable to Coastal Storms recommends avoiding wave forces on bridge decks by designing to an elevation with a vertical clearance of at least one foot above the 100-year design wave crest elevation whenever possible. Raising bridge structures above the design wave crest elevation can prevent large wave forces from impacting critical bridge components but must be evaluated considering the overall associated increase in bridge cost and risk tolerance.

Design Wave Crest Elevation

Bridges and roadway structures should be designed using a design wave crest elevation that will help determine the elevation of the bridge deck or roadway. HEC-25 describes the detailed approach for calculating a design wave crest elevation.

- For a Level 1 analysis, waves can be assumed to be depth-limited and will break when their height is approximately 80 percent of the local water depth (including storm surge) as described in the Wave Breaking section, above. Depth-limited wave characteristics are also described in HEC-25 Chapter 4 (Waves).

  The elevation of the wave crests under depth-limited conditions can be approximated by adding 60 percent of the water depth to the design stillwater level, as shown in Equation 15-3:

  $$\textit{design wave crest elevation}_{max} = (\textit{design storm surge SWL}) + (0.6d_s)$$

  Equation 15-3. (HEC-25)

  Where

  d_s = local water depth, including tides and storm surge

  See HEC-25 Chapter 9 (Coastal Bridges) for more information about estimating wave crest elevations.

- For Level 2 or 3 Analysis, design wave crest elevations are modeled using nearshore/oceanographic numerical models (refer to **Numerical Modeling of Waves** subsection, below) that rely on local wind and water level conditions as inputs.

  If complete avoidance of wave loads is not feasible, the designer may choose to mitigate and/or accommodate the potential wave forces (refer to AASHTO Specifications, Sections 4.3 and 4.4). Selecting an appropriate level of analysis and design strategy to accommodate potential wave forces will depend on the importance/criticality of the bridge or roadway when considering the consequences of bridge and roadway damage caused by wave forces. For example, if a bridge is deemed "Extremely Critical," it will generally be designed to resist wave forces with little to no damage to the bridge, so that the structure could be in service immediately following the storm event. If a bridge is deemed "Critical," it will typically be designed to withstand the storm event but will likely require repairs to maintain service long-term. Bridges that are deemed “Non-Critical” will not require an evaluation of wave forces and may require major repairs or even replacement following a large storm event.

Using AASHTO Specifications, Section 5.1, a bridge should be classified as extremely critical/critical as follows:

- **Extremely Critical** — Bridges that are required to be open to all traffic after the design event and be useable by emergency vehicles and for security, defense, economical, or secondary life safety purposes immediately after the design event. Bridges that are formally designated as critical for a defined local emergency plan.
- **Critical** — Bridges that should, as a minimum, be open to emergency vehicles for security, defense, or economical purposes after the design event and open to all traffic within days after that event. Bridges will likely be formally designated as critical for a defined local emergency plan.

It is assumed that traffic will not be on bridges during the height of the storm, so the concept of immediate “life safety” is not an important consideration during design. Long bridges, in general, require a higher level of analysis than shorter bridges. It becomes more cost effective to consider a higher level of analysis as bridge length increases.

Details regarding the determination of the level of importance of a proposed bridge, associated performance objectives of the bridge, and appropriate level of analysis are described in the AASHTO Specifications. Refer to Table 15 3 for examples of bridge types related to their appropriate level of analysis. In general, the following synthesis may be helpful for determining if a greater level of analysis is required.

The following criteria will be required for consideration in the determinations (Florida Department of Transportation, 2009):

- Age and condition of existing bridge structure and the feasibility/cost of retrofitting to resist wave forces
- Proposed bridge location and elevation alternatives (elevation relative to the design wave crest)
- Estimated cost of elevating the structure above the wave crest clearance, and/or the justification of why it cannot be done
- Effect of varying wave loading on construction costs (due to location or height adjustments)
- Existing and projected traffic volumes
- Route impacts on local residents and businesses
- Availability and length of detours
- Evacuation/emergency response routes
- Duration/difficulty/cost of bridge damage repair or replacement
- Other safety and economic impacts due to the loss of the structure

In addition to coastal bridges, it is important to calculate wave forces on coastal armoring placed for protection of transportation assets. For guidance on calculating wave forces on vertical walls, concrete caps, convex and concave cornered structures, and submerged structures, see the USACE CEM (refer to Part VI, Chapter 5-4). Specific wave force calculations are complex and depend on factors that require inputs related both to the designed structure—shape, configuration, and elevation relative to the design water elevation—as well as the wave itself. These calculations, as defined in the USACE CEM, require defining not only wave magnitude in height and period, but the wave's shape, which impacts how air pockets are trapped at the location of wave impact and the associated force upon the structure. Further discussion on the effects of waves on scour and scour protection design are discussed in Section 5. Given the importance and complexity of these considerations to the integrity of bridges and roadways, involving a precertified coastal engineer in the project's design or review phases is highly recommended and should be considered requisite for projects expecting coastal waves exceeding 1.5 feet in height, in accordance with the current definition from FEMA designating that wave height as the Limit of Moderate Wave Action (LiMWA) in the coastal zone (FEMA, 2013). The LiMWA defines that waves of at least 1.5 feet in height are likely to cause structural damage upon impact.

Wave Generation on Lakes

Under the right set of conditions, wind can generate large or damaging waves on lakes. As discussed in the Wave Dynamics subsection, wave height is a function of fetch length, so that the longer (or larger) the lake, the higher the waves that may be generated. It is also important to note that even the persistent presence/effects of tiny waves may cause severe erosion on shorelines or embankments.

In fresh or brackish waters, establishing vegetation along the shoreline fronting low-lying coastal roadways can often provide effective erosion protection, similar to what is done for dunes facing the Gulf of Mexico shoreline. When considering vegetation as an erosion prevention measure, designers should not overlook the possibility of erosion occurring before the vegetation becomes established. Light armoring, such as rubble or geotextile fabric, should be applied in addition to vegetation during the transitional period.

Ship Wakes

Ship wakes may be the largest waves to occur in sheltered locations, such as Intracoastal Waterways, and can be applied to represent the design waves in equations used in designing coastal protection armoring or other transportation assets, including designs for TxDOT operated ferry facilities. The magnitude of the wake depends on the vessel size, hull shape, speed of vessel, and distance from shoreline. Large ships can generate wakes with wave heights exceeding 10 feet, while smaller vessels, such as tugboats and barges, may generate wakes with wave heights of around 5 feet. The maximum size and speeds of all possible vessels should be considered in the engineering design. The following studies present approaches for estimating ship wakes:

Large Vessels

- Weggel, J.R. and Sorensen, R.M., 1986. Ship Wake Prediction for Port and Channel Design. Proceedings from the Ports 1986 Conference. American Society of Civil Engineers. P. 794-814.
- Kriebel, D.L., Seelig, W., and Judge, C., 2003. A Unified Description of Ship-Generated Waves. Proceedings of the PIANC Passing Vessel Workshop. Portland, Oregon.

Small, Recreational Watercraft

- Bottin, R.R. Jr., McCormick, J.W., and Chasten, M.A., 1993. Maryland Guidebook for Marina Owners and Operators of Alternatives Available for the Protection of Small Craft Against Vessel-Generated Waves. Prepared for the Maryland Department of Natural Resources. Coastal Engineering Research Center. Vicksburg, Mississippi. 92pp.

Numerical Modeling of Waves

Where local wave data are not available or when it is necessary to capture the dynamics of a complex shoreline or large-scale project, numerical simulation of wave conditions may provide higher confidence in the selection of a design wave crest elevation. Numerical models account for site specific details and processes that give rise to complex interactions between water and the surrounding natural and built environments. Using numerical models reduces the uncertainty associated with representing relevant coastal processes needed for designing roads and bridges.

Table 15-3 provides general direction on selecting the level of analysis and potential models that may be applicable for each case.

Wave Data Sources

- **NOAA National Data Buoy Center (NDBC)** — The NDBC provides several offshore oceanographic stations that record wave data, including significant wave heights, in the Gulf of Mexico. http://www.ndbc.noaa.gov/
- **USACE Wave Information Studies** — WIS is a series of multi-decade wind and wave hindcasts along the U.S. coastlines. The long-term nature of this data provides coastal engineers with information to perform statistical calculations and a better understanding of local conditions. http://wis.usace.army.mil/
- **FEMA Flood Insurance Studies** — Some FEMA FISs contain wave height estimates considered in flood hazard mapping analyses. However, the FIS may include a wave height statistic rather than the significant wave height, which is commonly used in design. FEMA FIRMs and FIS data are applicable for a Level 1 analysis or to determine the exposure of infrastructure to coastal flooding. They are not recommended for Level 2 or Level 3 designs. https://msc.fema.gov/portal/search

Wave-Generated Currents

As wave energy enters the nearshore area, some of the energy is transformed into nearshore currents and is responsible for the movement of sediment. Wave-generated currents move both along the coast (longshore), parallel to the shoreline, and cross-shore, or perpendicular to the shoreline. Wave-generated currents in the nearshore environment are extremely complex and highly sensitive to wave direction and storm events. Currents are capable of moving large amounts of sediment over timescales ranging from hours (like during a storm event) to decades, making them important to consider in transportation infrastructure design. Understanding of the behavior of currents, including any associated local trends, is particularly important to determine necessary armoring needed to protect transportation assets from coastal erosion.

Longshore Currents

Longshore currents are generated by waves breaking at the shoreline (Figure 15-18). Longshore currents cause movement of sand along the beach, referred to as littoral drift or longshore sediment transport. The direction of longshore currents changes frequently in response to changes in wave approach angles but is generally associated with an average movement of sediment in one direction. This sediment movement contributes to the shoreline shape and evolution over time.

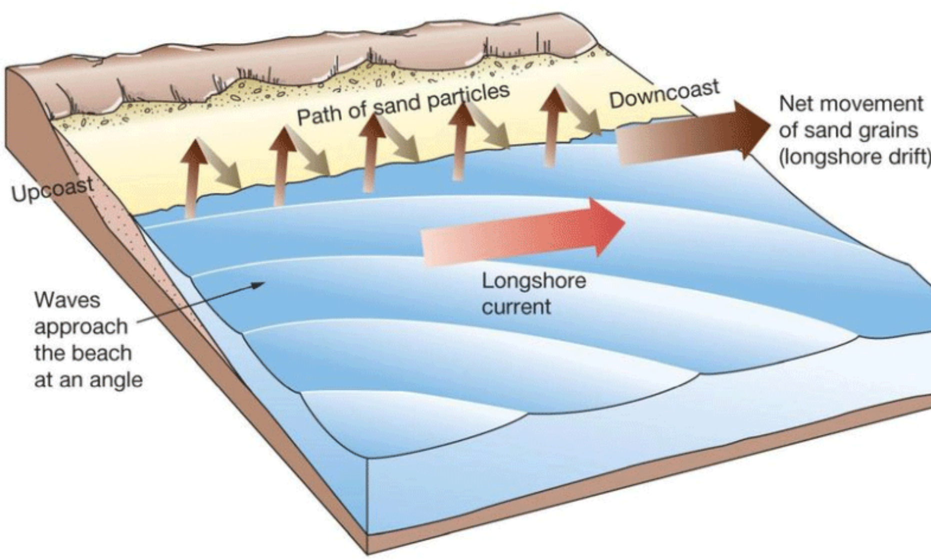


Figure 15-18. Diagram of Longshore Currents (Pearson Education, Inc., 2011)

During storm events, the rate of longshore sediment transport can be significant, causing erosion and scouring of engineered structures, including low-lying coastal roadways and bridge pilings. A single storm event can cause changes to the shoreline that take years to recover without intervention.

Longshore currents in the Gulf of Mexico impact the sediment movement along the Texas coast. The Texas coast experiences, in general, a longshore current from north to south for the upper

Texas coast and another longshore current from south to north for the lower Texas coast. The currents converge around Padre Island near the upper Laguna Madre in Kenedy County. This helps to explain why beaches in the Coastal Bend, or middle Texas coast, are generally wide and ‘sediment rich’, whereas beaches located on the upper and lower coasts are ‘sediment starved’ and require more frequent renourishment. Upstream impacts caused by engineered structures (e.g., jetties capturing sand that would otherwise move downstream) can also have a major impact on shoreline positions.

The most common equation for estimating longshore sand transport rate is the ‘CERC Equation’, also known as the energy-flux method. This equation estimates the sediment transport rate based on the longshore component of energy flux or wave power entering the surf zone. This longshore transport calculation is most useful in understanding short-term fluctuations and sediment budgets for a project site. To understand longshore transport over long-term periods, historical imagery comparison is the most widely available data source, making it the primary resource. Details regarding calculation of the wave-energy flux factor and how it relates to longshore sediment transport can be found in HEC-25.

Cross-Shore Currents

In addition to longshore currents, breaking waves generate cross-shore currents that provide an offshore-directed flow of water and sediment entrained in the water column. Cross-shore currents are responsible for beach width changes following storm events, when large amounts of sediment may get pushed into deeper waters and be stored in the nearshore environment until they are slowly carried back to the beach face over the weeks or months following the event.

Rip currents, a unique type of cross-shore current, form in places where water that is driven ashore by strong waves in the presence of shallow sandbars drains back out to sea, creating a strong current perpendicular to the shoreline (Figure 15-19). Rip currents pull sand offshore, leaving behind recessional areas that can become ‘erosional hot spots’ where the beach is narrower. Low-lying roadways beyond these recesses are more vulnerable to erosion by larger waves. Although predicting where rip currents will appear is challenging, as they are poorly understood, advanced numerical models (refer to **Numerical Modeling of Waves** section) may be able to evaluate the storm conditions preferential to rip current development and provide insight on the potential scale of erosional hot spots that could develop. Local knowledge is also a key source of information that can indicate if rip currents are known to exist in a particular area. This information may be useful in developing coastal armoring for low-lying roadways located directly along coastlines.

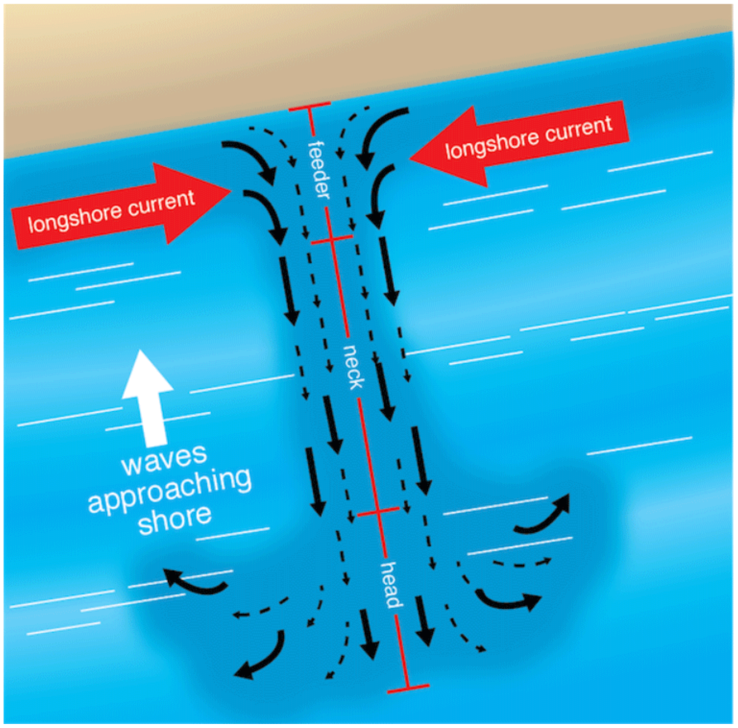


Figure 15-19. Diagram of Rip Currents ([University of Hawaii, n.d. Wave-Coast Interactions](#))

Impacts on Coastal Structures

Wave-generated currents can be a concern for structural assets along the coast, primarily due to the potential to cause erosion. Both local scour on a bridge and the larger erosion potential of a shoreline should be considered as risks from wave generated currents. Likewise, the impacts on engineered structures should be considered with regards to how the infrastructure might change the existing wave and current patterns. If longshore sand transport is interrupted by a ship channel or other engineering projects, such as a jetty system to stabilize an inlet for shipping or large bridge pilings located in an area of littoral drift, erosion can occur for many miles down shore of the feature. Beaches adjacent to and near tidal inlets are part of a dynamic system influenced by water flowing into and out of the inlet and, therefore, exhibit much more shoreline change than beaches farther from inlets.

Engineered changes to the Gulf of Mexico-inlet-bay system include stabilizing the inlet with jetties or dredging the inlet or bay for navigation. Less obvious changes include the impacts of engineered structures in the bay that affect the amount of water leaving and entering the bay with each change of the tide. This can include filling of wetlands for coastal construction or constructing causeways in bays. Additional discussion on erosion mechanisms and mitigations measures will be discussed later in Section 5.

Currents Data Sources

- NOAA Tides and Currents — NOAA Tides and Currents is a collection of NOAA wind, water level, and current data recording stations along U.S. coastlines. Current records are available at some of these recording stations, including locations in Galveston and near the Texas/ Louisiana border.

Section 5 — Design Elevation and Freeboard

In determining the design elevation for transportation infrastructure, the designer must consider multiple factors, including flood frequency, road classification, average daily traffic (ADT), site restrictions, wave height, sea level rise, and freeboard. This section describes the process in determining design elevation and considerations when evaluating appropriate freeboard requirements. This section builds off the concepts in Sections 2 and 3, which provided the technical background on coastal water levels and wave dynamics.

Establishing a Design Elevation

For existing infrastructure, begin by reviewing the bridge or roadway elevations from as-built drawings or surveys, as well as maintenance reports that would indicate how nearshore coastal processes have impacted the project area. For new coastal infrastructure projects, the historical coastal hazards at the site can be investigated through references such as site surveys, desktop analysis, historical imagery, interviews with residents and stakeholders, among others and can help inform the design elevation ultimately selected.

Bridges and roadways designed on the coast may be elevated above the design elevation, which includes the cumulative effects of stillwater levels above typical water levels from storm tides and relative sea level rise, as well as increases in elevation from waves and subsequent runup. This design elevation would be dictated by the appropriate return period as established in the previous sections. In many cases, however, it is not practical to elevate a coastal roadway above the design elevation. In these cases, understanding that a coastal roadway may be more readily influenced by tidal ranges and relative sea level rise will help determine likely drainage impacts and general accessibility limitations that should be communicated to appropriate authorities and stakeholders. In general, determining the primary drivers of infrastructure impacts, such as the nearshore coastal processes that are most likely to occur, will enable proper consideration of coastal conditions when elevating a structure or providing freeboard. The design elevation can be computed using the methods previously discussed in Sections 2 and 3.

The site conditions, such as the nearshore processes that appear to impact the site, will dictate the considerations to be made in water level data. Later in this section, three examples, one for each level of analysis, will illustrate how to determine a design elevation. In general, the designer should select a tide station close to the project site, convert the tidal data to the same datum as the project, and account for the stillwater level components that affect the project design, which can include: average tide range, annual high tides, storm tides, and future relative sea level rise projections. Upon determining the appropriate stillwater scenario, a subsequent wave analysis should be completed, resulting in an appropriate design elevation. Potential design elevation components are shown in Figure 15-20.

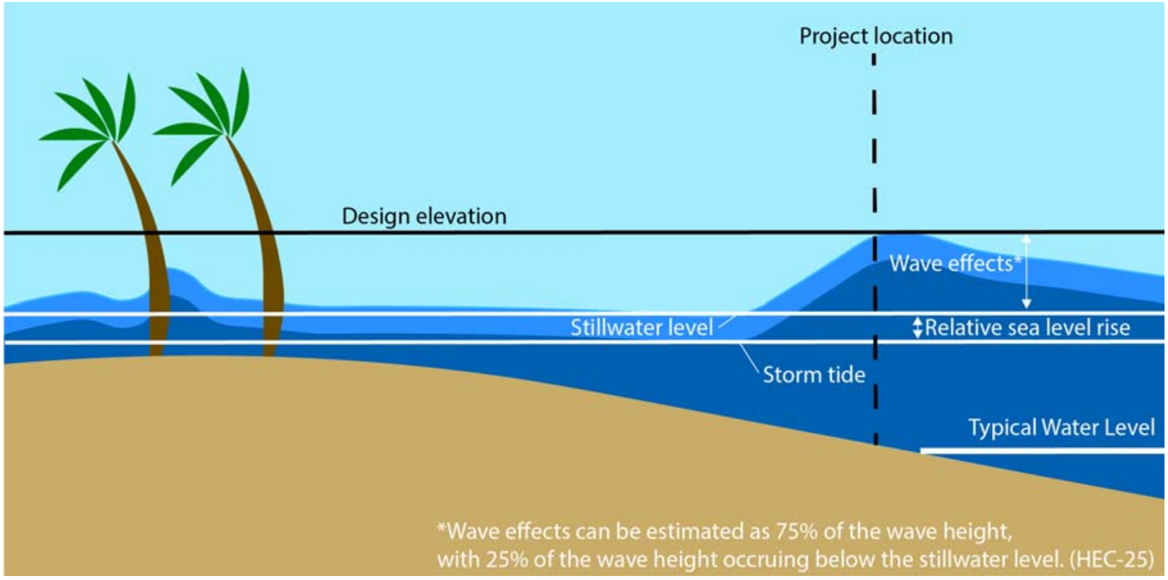


Figure 15-20. Design Elevation Diagram.

Freeboard Considerations

For coastal freeboard considerations, federal requirements require freeboard when practicable for bridges (23 CFR 650.115). TxDOT recommends one foot minimum of freeboard for coastal bridge applications, barring specific exemptions. TxDOT recommends two feet of freeboard for coastal bridge designs that include assumptions associated with Level 1 analyses and more approximate Level 2 analyses. For roadway applications, TxDOT does not have a specific recommendation for freeboard above the design storm, but for nuisance flooding concerns, TxDOT recommends utilizing 0.5-1.0ft of freeboard whenever possible above these more frequent annual tide events to minimize issues. Variations from these recommendations should be documented in the Design Summary Report (DSR), drainage report, and/or plans for each project. Freeboard, when included, is added to the maximum wave crest elevation to determine the design elevation.

Bridges

HDM Chapter 9 (Bridges), Section 3 (Bridge Hydraulic Considerations) indicates that navigational clearance and other reasons notwithstanding, the low chord elevation is established as the sum of the design elevation and freeboard.

For on-system bridges, the Department recommends a suitable freeboard based on the following criteria:

- Higher freeboards may be appropriate for bridges over waterways with debris potential or requiring clearance for the GIWW, as well as to accommodate other clearance needs.
- Lower freeboards may be desirable because of constraints such as approach geometry. However, the design elevation should not impinge on the low chord. While it is acknowledged that

certain coastal roadways cannot (or do not) meet this criteria, these exceptions must be clearly noted on both the plans and drainage reports.

Generally, for off-system bridge replacements, the low chord should approximate that of the structure to be replaced, unless the results of a risk assessment indicate that a different structure is the most beneficial option. The design freeboard, when considered in a coastal context, should also account for the uncertainty in high waves in the larger areas of open water, since the uncertainty in coastal storm surge and wave analysis can be just as great as uncertainty in riverine conditions.

Historically, in some cases, bridges have survived during storms because their elevations were lower than the full storm wave height. In those cases, it has been observed that wave loads may have been small enough that they did not generate critical damage to the bridge and that the weight of the bridge span was sufficient to overcome wave forces. However, a study of the I-10 Bridge over Mobile Bay, conducted by Auburn University, suggests that it is the structural design of a bridge in conjunction with the wave height, rather than wave height alone, that will determine the impacts of wave forces on the bridge structure. More research is needed to further detail the critical elevation for wave height on bridges.

Complete clearance over the wave height might not be needed or be achievable due to roadway and other design considerations. Likewise, it may not be economical within the roadway geometry design constraints. When complete clearance is desired but cannot be achieved, then the wave and storm tide impacts on the structures can be addressed through erosion and structural countermeasures, such as shoreline armoring or structural retrofits to counter wave forces. An advanced approach is to set the deck elevations based on modeled wave loads analyzed over the deck and pile cap design, to ensure that both can withstand wave loads.

Design Elevation and Freeboard Calculation Examples

Example design calculations for each level of analysis, including freeboard considerations, are shown in Table 15-7 through Table 15-9 for the process of determining of a final design elevation. Project design elevation should also consider whether the project is a repair, replacement, or new alignment. For repairs and replacements, elevation changes could be more constrained and storm return periods or individual considerations, such as relative sea level rise, should be evaluated for practicality.

Table 15-7: Level 1 Analysis Design Elevation and Freeboard Calculation Example

Level 1 Analysis Example		
Step	**Activity**	 
1	**Determine Storm Tide Elevation** *Section 2*	**Obtain appropriate FEMA flood map elevations for study area** FEMA FIS Report Summary of Stillwater Elevations of 10 ft for 100-year design storm tide and 8.4 for 50-year design storm tide. **Zone AE BFE shown on Flood Insurance Rate Map includes wave effects and runup and should not be used for stillwater elevation, reference the FIS for stillwater elevations.*
2	**Determine Relative Sea Level Rise Rate** *Section 2*	**Obtain RSLR rates for project life cycle** Located in Nueces County, Region 3. Assumed 50-year design life for roadway and bridge starting in 2020. Relative sea level for 2070 is 3.0 ft per Section 2 guidance.
3	**Interpolate Project Specific Relative Sea Level Rise** *Section 2*	The 3.0 ft for 2070 minus 0.7 ft from 2020. Sea level change of 2.3 ft for project life.
4	**Calculate Stillwater Elevation**	**Add the result of Step 3 (RSLR) to the elevations obtained in Step 1 (storm tide)** ◆ 10 ft plus 2.3 ft. gives a 100-year stillwater elevation of 12.3 ft. ◆ 8.4 ft plus 2.3 ft. gives a 50-year stillwater elevation of 10.7 ft.
5	**Obtain Ground Data** *Section 2*	**Obtain appropriate ground surface elevation maps for study area** LiDAR and bathymetry from TNRIS show an average channel elevation of 7 ft. **Ground elevation should be confirmed with plans, survey, or bathymetry.*
6	**Determine Stillwater Depth**	**Subtract ground elevations (Step 5) from values found in Step 4 to obtain flood depth** ◆ 100-year: 12.3 ft minus 7 ft equals a depth of 5.3 ft. ◆ 50-year: 10.7 ft minus 7 ft equals a depth of 3.7 ft.

Table 15-7: Level 1 Analysis Design Elevation and Freeboard Calculation Example

Level 1 Analysis Example		
7	**Determine Maximum Wave Height** *Section 3*	**Multiply stillwater depths (Step 6) by 0.8 to determine maximum wave height.** ◆ 100-year: 5.3 ft times 0.8 equals 4.24 ft wave height ◆ 50-year: 3.7 ft times 0.8 equals 2.96 ft wave height **If the project is located adjacent to a FEMA coastal transect, it is also possible to determine appropriate wave parameters from the Flood Insurance Study documentation. If wave heights significantly exceed the FEMA designated zone, a different method should be considered, such as Level 2 techniques. For example, if landward of the LiMWA, wave heights should likely not exceed approximately 1.5 feet, and if in an AE zone rather than a VE, they should not exceed 3 feet. Inclusion of RSLR could impact FEMA data applicability in some cases.*
8	**Determine Wave Crest Elevation (Design Elevation w/out Freeboard)**	**Multiply maximum wave height (Step 7) by 0.75, add to Step 4 for wave crest elevations. 75% of the wave height being above the stillwater elevation is a standard estimate for basic wave calculations (HEC-25).** ◆ 100-year: ● 4.24 ft times 0.75 equals 3.18 ft ● 12.3 ft plus 3.18 ft equals 15.48 ft ◆ 50-year: ● 2.96 ft times 0.75 equals 2.22 ft ● 10.7 ft plus 2.22 ft equals 12.92 ft **These values should be compared to historical storm observations, while considering adjustments for RSLR. If there are significant discrepancies, a Level 2 analysis may be warranted.*
9	**Determine Freeboard** *Section 4*	**Consider freeboard based on road classification and site conditions.** Local roads and streets, non-critical, do not require freeboard per AASHTO guidance. Per guidance in this chapter, bridged waterways analyzed with Level 1 methods recommend 2.0 ft freeboard over the design storm tide plus wave height. Both an example of bridge calculations and roadway calculation are shown in Steps 10 and 11.
10	**Determine Bridge Design Elevation (Low Chord)** *Section 4*	**The bridge low chord design elevation is determined using the freeboard from Step 9 and the 50-year design elevation from Step 8.** ◆ 12.92 ft plus 2.0 ft equals 14.92 ft **Application of wave effects, RSLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.*

Table 15-7: Level 1 Analysis Design Elevation and Freeboard Calculation Example

Level 1 Analysis Example		
11	**Determine Roadway Design Elevation** *Section 4*	The roadway design elevation is determined using the 50-year return period based on roadway classification in AASHTO guidance documents. Because there is no freeboard recommended, the design elevation is the same as the elevation determined in Step 8, 12.92 feet. **Application of wave effects, RSLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.*

Table 15-8: Level 2 Analysis Design Elevation and Freeboard Calculation Example

Level 2 Analysis Example		
Step	**Activity**	
1	**Determine Storm Tide Elevation** *Section 2*	**Obtain appropriate FEMA flood map elevations for study area** Multiple return periods could be required for additional calculations such as scour, but for simplicity, only the 50-year return period is shown in this example for establishing deck elevation. FEMA FIS Report Summary of Stillwater Elevations of 12.5 ft for 50-year design storm tide. **Zone AE BFE shown on Flood Insurance Rate Map includes wave effects and runup and should not be used for stillwater elevation, reference the FIS for stillwater elevations.*
2	**Determine Relative Sea Level Rise Rate** *Section 2*	**Obtain RSLR rates for project life cycle** Located in Orange County, Region 1. Assumed 75-year design life for bridge starting in 2020. Relative sea level for 2095 is 5.15 ft per Section 2 guidance.
3	**Interpolate Project Specific Relative Sea Level Rise** *Section 2*	The 5.15 ft for 2095 minus 0.8 ft from 2020. Sea level change of 4.35 ft for project life.
4	**Calculate Stillwater Elevation**	**Add the result of Step 3 (RSLR) to the elevations obtained in Step 1 (storm tide)** ◆ 12.5 ft plus 4.35 ft. gives a 50-year stillwater elevation of 16.85 ft.

Table 15-8: Level 2 Analysis Design Elevation and Freeboard Calculation Example

<table>
<tr><th colspan="3">Level 2 Analysis Example</th></tr>
<tr><td>5</td><td>Obtain Ground Data
Section 2</td><td>Obtain appropriate ground surface elevation maps for study area
LiDAR and bathymetry from TNRIS show an average elevation of 4 ft.
*Ground elevation should be confirmed with plans, survey, or bathymetry.</td></tr>
<tr><td>6</td><td>Determine Stillwater Depth</td><td>Subtract ground elevations (Step 5) from values found in Step 4 to obtain flood depth
◆ 50-year: 16.85 ft minus 4 ft equals a depth of 12.85 ft</td></tr>
<tr><td>7</td><td>Determine Maximum Wave Height
Section 3</td><td>Utilizing stillwater data obtained previously and local historical wind data from a nearby NOAA station, develop a wave analysis using a 1D wave model such as Coastal Engineering Design and Analysis System (CEDAS) or through spreadsheet-based calculations.
Model inputs include water depth, wind speed and duration, fetch length, typical temperature and project latitude. The resulting datasets should be statistically evaluated to determine a 50-year return period value for wave height and period. This example analysis yielded a 50-year wave height of 3.81 feet and a wave period of 3.82 seconds.</td></tr>
<tr><td>8</td><td>Determine Wave Crest Elevation (Design Elevation w/out Freeboard)</td><td>Multiply design wave height (Step 7) by 0.75, add to Step 4 for wave crest elevations. 75% of the wave height being above the stillwater elevation is a standard estimate for basic wave calculations (HEC-25).
◆ 50-year:
● 3.81 ft times 0.75 equals 2.86 ft
● 16.85 ft plus 2.86 ft equals 19.71 ft
*These values should be compared to historical storm observations, while considering adjustments for RSLR. If there are significant discrepancies, a Level 3 analysis may be warranted.</td></tr>
<tr><td>9</td><td>Determine Freeboard
Section 4</td><td>Consider freeboard based on road classification and site conditions.
Per guidance in this chapter, bridged waterways analyzed with more approximate Level 2 methods recommend 2.0 ft freeboard over the design storm tide plus wave height.</td></tr>
<tr><td>10</td><td>Determine Bridge Design Elevation (Low Chord)
Section 4</td><td>The bridge low chord design elevation is determined using the freeboard from Step 9 and the 50-year design elevation from Step 8.
◆ 19.71 ft plus 2.0 ft equals 21.71 ft
*Application of wave effects, RSLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.</td></tr>
</table>

Table 15-9: Level 3 Analysis Design Elevation and Freeboard Calculation Example

Level 3 Analysis Example		
Step	**Activity**	
1	**Determine Storm Tide Elevation** *Section 2*	**Select, develop, and prepare appropriate numerical modeling tools** 2D and 3D hydrodynamic, wave, storm surge, and morphologic models (e.g., ADCIRC+SWAN, Delft3D, MIKE3). Calculations required for design could determine whether a 2D or 3D model is required. If scour is expected to be critical, a 3D may be necessary to account for complex current patterns at the structure. Validate and/or calibrate the models through hindcast simulations and analysis. Review past storms for data to use in calibrating the chosen model. *Elevations will be determined concurrently with Step 8.*
2	**Determine Relative Sea Level Rise Rate** *Section 2*	**Obtain RSLR rates for project life cycle** Located in Galveston County, Region 1. Assumed 75-year design life for bridge starting in 2020. Relative sea level for 2095 is 5.15 ft per Section 2 guidance. *Review of current guidance or projections for RSLR should be considered and best available data should be utilized.*
3	**Interpolate Project Specific Relative Sea Level Rise** *Section 2*	The 5.15 ft for 2095 minus 0.8 ft from 2020. Sea level change of 4.35 ft for project life. Incorporate into water level inputs or similar model parameters.
4	**Calculate Stillwater Elevation**	*Step completed concurrently with Step 8.*
5	**Obtain Ground Data** *Section 2*	**Obtain appropriate ground surface elevation maps for study area and surrounding water body as needed.** An integrated LiDAR and bathymetry based digital elevation model will be necessary to utilize the model selected in Step 1. The extent of required data is dependent on the model mesh extents.
6	**Determine Stillwater Depth**	*Internal calculations within model.*
7	**Determine Maximum Wave Height** *Section 3*	*Step completed concurrently with Step 8.*
8	**Determine Wave Crest Elevation (Design Elevation w/out Freeboard)**	Using the hydrodynamic model, run the desired simulation for the design storms to get the information needed for design elevation: water levels, wave heights, and velocity. Derive probabilities associated with water levels, wave heights, velocity, etc. as appropriate for the selected model.

Table 15-9: Level 3 Analysis Design Elevation and Freeboard Calculation Example

Level 3 Analysis Example		
9	**Determine Freeboard** *Section 4*	**Consider freeboard based on road classification and site conditions.** Per guidance in this chapter, 1.0 ft minimum freeboard is recommended for bridged waterways. Based on project specific observations and concerns with regards to debris, 3.0 ft freeboard is selected.
10	**Determine Bridge Design Elevation (Low Chord)** *Section 4*	**The bridge low chord design elevation is determined using the freeboard from Step 9 and the 50-year design elevation from Step 8.** **Application of wave effects, RSLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.*

TxDOT design flood and check flood minimum standards can be found in Chapter 4 (Hydrology), Section 6 (Design Flood and Check Flood Standards). In some cases, it may be required to calculate additional design storms for scour computations per the *TxDOT Geotechnical Manual*, Chapter 5 (Foundation Design), Section 6 (Scour).

Section 6 — Erosion

Erosion in the coastal environment can take many forms, with the most common issues for coastal roadways resulting from scour and shoreline change. Scour is the local-scale removal of sediment at the base of a structure (or other stationary object) due to forces from moving water, the effects of which have the potential to expose or undermine the structural foundations or protective structures of a project. Although scour is a form of erosion, scour does not include the loss of sediment on a larger (often regional) scale, such as occurs with shoreline change. Large-scale changes in the shoreline can include both recession and accretion. In this section we will discuss the physical drivers of scour and shoreline change, impacts on coastal structures, mitigation measures, and data sources that provide additional guidance.

Scour

Scour is addressed elsewhere in this Manual for cases involving hydraulics for bridges (Chapter 9), reservoirs (Chapter 12), and storm water management (Chapter 13). Scour occurring in a coastal environment is fundamentally different than riverine scour. In the coastal environment, the primary driver of scour is from head differences across a structure driven by storm tide, which are propagating inland. Riverine scour is typically opposite in direction and is driven primarily by storm runoff, resulting in different mechanisms and directions for scour considerations. Many coastal structures require analyzing for both riverine and coastal scour, and some scenarios require evaluation of combined effects. HEC-25 classifies three levels of approach for coastal scour which will be discussed later. A Level 3 analysis should be evaluated by a precertified TxDOT engineer with experience in coastal processes.

This section focuses on scour due to coastal exposure including storm surge, waves, and tidal currents. Scour should be assessed or evaluated for most projects, especially in locations that may have significant water velocities at the large scale or localized levels. The potential for scour is influenced by a combination of factors including water velocity, structure configuration, bathymetry, bay/inlet configuration, sediment characteristics, and wave climate. Of particular interest for TxDOT projects is the potential for scour at bridges and roadway embankments. Scour can occur along roadway embankments from several scour mechanisms including wave action, weir-flow, and shoreline change. These mechanisms will be discussed in the next section.

All bridges need to be evaluated for scour (23 CFR 650.313(e) and FHWA, HEC-18, Evaluating Scour at Bridges). Especially susceptible locations are areas with abrupt changes in bathymetry, or tidal inlets that can cause flow constrictions or focus wave energy, and areas of active wave breaking. Other resources available to provide in-depth information, equations, and procedures to perform coastal scour include the First and Second volumes of HEC-25, Highways in the Coastal Environment, and HEC-18 (Sections 9.7 and 9.8).

Scour Mechanisms

Scour results from the erosive action of flowing water, which may remove and carry away material from the bed, banks of waterway, and from around coastal structures, such as piers and abutments. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are generally more scour-resistant.

Determining the magnitude of scour risk at structures in coastal waters is complex, as they are at the confluence of concurrent hydraulic forces (e.g., currents and breaking waves) that arise due to the interface of the land and water. Reviewing site conditions can lead to a better anticipation of the types of scour that may be present and should be evaluated. Once the types of existing structures or planned structures are known, the list of typical scour mechanisms can be reviewed for applicability. Table 15-10 lists common scour mechanisms. Examples of many of these mechanisms are also shown in Figure 15-21.

Table 15-10: Common Types of Scour Mechanisms

General Scour Mechanisms	Additional Coastal Scour Mechanisms
Scour around piers and abutments	Roadway damage by wave attack
Erosion along toe of highway embankment	
Erosion of embankment due to overtopping flow	
Long term vertical degradation of stream bed	Roadway and railway damage by coastal “weir-flow”
Horizontal migration of stream banks	
Debris impact on structure	Roadway damage by bluff erosion and shoreline recession
Clogging due to debris causing redirection of flow	

A - Partial embankment damage caused by wave attack

B - Example of the weir-flow damage mechanism as it occurs

C - Pavement damage due to waves and surge in an extreme event in Brazoria County after Hurricane Ike

D - Two bridges destroyed by wave loads in Hurricane Katrina

E - Wave scour hole formed by Hurricane Katrina

Figure 15-21. Photo Examples of Coastal Scour Mechanisms (FHWA, 2008)

Hydraulic analysis must consider the magnitude of design storms, characteristics and geometry of the tidal inlet, estuary, or bay, and the long-term effects due to placement of the bridge or other transportation asset. Structures located in coastal environments must also consider scour resulting from flow in two directions due to tidal fluctuations. In addition, the analysis must consider the long-term effects of normal tidal cycles. This can be achieved by modeling and reviewing the time-dependent tidal flows, velocities, and depth on the following processes:

- **Aggradation or degradation**. These processes are the long-term elevation changes, which can affect the reach of the waterway on which the bridge is located. Aggradation involves the deposition of eroded material from upstream of the bridge, while degradation is the lowering of the bed of the waterway due to a deficit in sediment supply upstream.
- **Contraction scour**. Contraction scour is the lowering of a waterway bed at the bridge location due to constriction of the flow. This differs from degradation in that contraction scour occurs in the vicinity of the structure. Worst-case conditions of contraction scour are typically the result of storm surges.
- **Local scour**. Local scour involves removal of material around engineered structures caused by acceleration of flow and vortices induced by obstructions. Worst-case conditions of local scour are typically the result of storm surges.
- **Channel instability**. Lateral migration of the waterway occurs naturally and may affect the stability of engineered structures or change scour patterns. Channel stability is affected by local geomorphology, flood characteristics, and bed/bank materials. HEC-20 and FHWA's Hydraulic Design Series Number 6 (HDS 6) provide guidance on incorporating channel stability into roadway design.

Geotechnical investigations are commonly used to evaluate scour mechanisms. Understanding the local soil and rock properties is important, as it provides a basis for describing common engineering properties of geomaterials and how different materials may behave under various conditions. Geotechnical investigation can include surface samples, borings, and SEDflume testing. Surface samples and boring can be evaluated via Unified Soil Classification System (USCS) classifications, gradation, plasticity index, and hydrometer testing.

Due to the dynamic nature of sediment transport in the coastal zone, most technical guidance is based on empirical equations, experience, and field observations. The type of sediment located throughout the study area and suspended in the water column should be considered and equations based on available data used as appropriate. One area of focus that can vary the level of analysis needed or types of equations used is the type of soil to be evaluated, such as cohesive, non-cohesive, coarse, or erodible rock. For non-cohesive material scour equations, it is important to know the D_{50} and D_{90} sediment diameters. For cohesive materials, when possible, it is important to know the erosion rate and critical velocity of sediment particles. These values can also be derived empirically if the D_{50} and classification are known.

Long-term sediment budgets are also important for understanding areas prone to scouring. The long-term sediment budget can be generally characterized by typical sediment movement within the littoral zone. The littoral zone is the zone from the shoreline to just beyond the breaking wave zone. Littoral drift is the movement of beach material in the littoral zone by waves and currents. An aggrading or stable waterway may exist if the sediment supply to the project area from the littoral drift is large. Such a situation may minimize the effect of contraction scour, and possibly local scour. Conversely, long-term degradation, contraction, and local scour can be exacerbated if the sediment from littoral drift is reduced. Important information for evaluating the effect of littoral drift includes historical information, future (dredging, installation of jetties, etc.), and sources of sediment. The **Shoreline Change** subsection provides a more in-depth discussion for long-term shoreline change and sediment budgets.

Once the project location is determined to be coastal (as described and shown in the maps in Section 1), the asset type to be designed or evaluated is chosen (such as a bridge), and the site's geotechnical background is developed, then the scour mechanism(s) that apply can be evaluated. Besides the list in Table 15-10, the USACE CEM provides examples of locations that have a high potential for coastal scour (Figure 15-22). Other methods of determining the observed or potential scour mechanism include reviewing maintenance reports, reports from community engagement, and photos after storms.

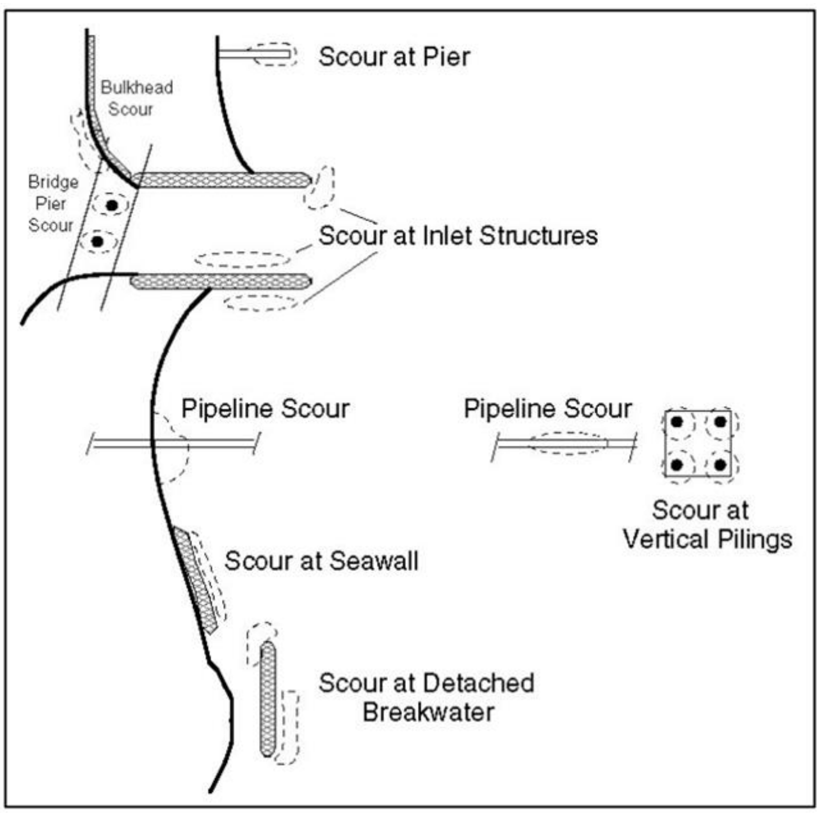


Figure 15-22. Locations with High Potential for Coastal Scour (USACE, 2002)

Guidance on methods for evaluating scour analysis in tidal waterways are outlined in HEC-18, HEC-20, HEC-23, HEC-25, and USACE CEM.

Understanding sediment budgets and geomorphic existing conditions provides a valuable tool for evaluating the potential for scour in tidal waterways. In most cases, this principle is not easy to quantify without direct measurements and hydraulic modeling. For the analysis of roadway structures located in tidal waterways, a three-level analysis approach, similar to the approach outlined in HEC-20, is recommended. Determining the appropriate level of analysis is dependent on-site conditions, level of approach, available data, and potential applicable scour mechanisms.

- **Level 1 Analysis** — Includes a qualitative evaluation of the stability of the inlet or estuary, estimating the magnitude of tides, storm conditions, flow, and long-term stability of conditions in the waterway. It is appropriate for planning phases and deliverables where uncertainty is acceptable. A Level 1 analysis is not appropriate for coastal bridge hydraulic design or scour estimates. However, a Level 1 approach can be useful in the asset management and planning phase, or when determining the potential level of effort required for a specific project.
- **Level 2 Analysis** — Represents the engineering assessment necessary to obtain the velocity, depths, and discharge for tidal waterways to be used in determining long-term aggradation, degradation, contraction scour, and local scour. A Level 2 analysis is suitable for a wide range of bridge sizes and types. A Level 2 analysis, when applied over the tidal prism (or volume of water contained in a tidal inlet between low and high tide levels), does include a level of uncertainty but can provide generally conservative estimates of potential scour. A Level 2 analysis is not recommended for bridges at inlets or causeway bridges.
- **Level 3 Analysis** — Is used for complex tidal situations that consider a larger area and a higher range of storm conditions. A Level 3 analysis can vary in complexity from one-dimensional unsteady model to advanced and physical modeling. In general, the more complex the flow parameters, the more advanced the model. HEC-25 provides more information regarding advanced tidal hydraulic modeling approaches.

Figure 15-23 summarizes the guidance on level of approach as it pertains to scour estimation methods.

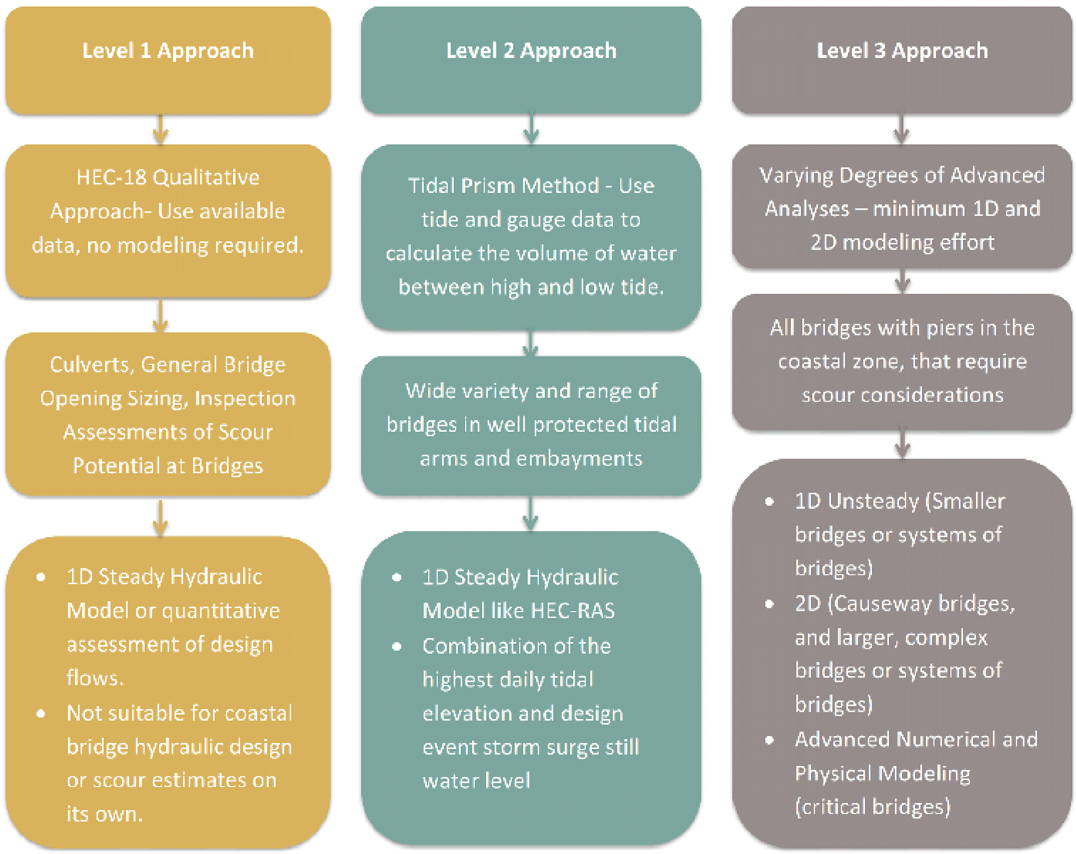


Figure 15-23. Scour Methods and Corresponding Level of Analysis.

Scour Mitigation Measures

There are several methods of infrastructure adaptation that can be incorporated during planning, designing, construction, operating, or maintaining infrastructure to mitigate for coastal scour. HEC-25 recommends a five-part approach: manage and maintain, increase redundancy, protect, accommodate, and relocate. Table 15-11 summarizes each of the five steps in the recommended approach.

Table 15-11: Summary of the HEC-25 Recommended Five-Part Approach for Scour Mitigation

<table>
<tr><td>Manage and Maintain
◆ Increase and target maintenance activities (e.g., culvert cleaning)
◆ Relocate movable assets prior to forecast storm events (e.g., moving maintenance vehicles and equipment to less vulnerable areas)
◆ Enhance and practice emergency procedures
◆ Stockpile and strategically place fuel, temporary bridges, and road construction materials for quick deployment</td><td>Increase Redundancy
◆ Identify and enhance, as appropriate, alternative roads, routes, or modes to serve transportation needs during times of compromised service (e.g., enhanced ferry service)
◆ Consider constructing or enhancing closely spaced roads perpendicular to the coastline versus one roadway parallel to coastline</td></tr>
<tr><td>Relocate
◆ Relocating infrastructure further inland away from the coastline
◆ Repurposing or reclassifying paved road to all-terrain vehicle road
◆ Reconditioning a damaged vehicular bridge to serve as a pedestrian bridge or fishing pier</td><td>Accommodate
◆ Increasing bridge deck elevations and strengthen bridge structures,
◆ Lowering roadway profiles to allow overwash without pavement damage during extreme events
◆ Raising tunnel portal walls to reduce likelihood of flooding</td></tr>
<tr><td colspan="2">Protection
◆ New or enlarged seawalls, bulkheads, revetments, etc. (hard structures to provide barriers or resistance to damaging forces)
◆ Retaining walls, mechanically stabilized earth (MSE), etc.
◆ Beach nourishment (soft strategy to move water away from infrastructure)
◆ Dune restoration and vegetation (soft strategies to provide barriers and resistance to damaging forces)
◆ Living shorelines along sheltered coasts (combination of hard and soft structures/strategies to increase resistance to damaging forces)</td></tr>
</table>

If an existing transportation asset is identified as at risk to erosion, then protection might be the first option to immediately mitigate risk. All options can be reviewed and incorporated for proposed transportation assets. For general scour conditions, such as local scour, contraction scour, stream instability, and overtopping flow, refer to Table 2.1 (Stream Instability and Bridge Scour Countermeasure Matrix) in HEC-23 for guidance on reviewing the hydraulic countermeasures, structural countermeasures, biotechnical countermeasures, and monitoring as part of a scour evaluation. Since those measures are applicable for the above conditions and more well-known in riverine analysis, the remainder of this section will focus on examples of coastal protection and countermeasures.

Roadway Mitigation

Many different natural processes and forces impact roads near the coast. This section will focus on the most common erosion issues (e.g., overwashing, coastal weir-flow damage, and wave action) for roadway infrastructure.

Overwashing:

For roadways in coastal environments, highway overwashing is a very common occurrence due to nearshore locations and low elevations. There are several mechanisms that damage pavements subject to overwash including:

- Direct wave attack on the seaward shoulder of the road;
- Flow across the road and down the landward shoulder or “weir-flow”; and
- Flow parallel to the road as water moves to “breaches” or lower spots in the road as the storm surge recedes.

Coastal weir-flow and wave action will be discussed as they relate to impacts from these mechanisms later in this section.

Strategies for minimizing damage during the overwashing condition include:

- Bayside Location — Relocating the road to a portion of the barrier island where sand will likely bury the road during overwash.
- Low Elevation — Lowering the elevation of the road to be at or below much of the existing grade to encourage burial by sand during overwash. This will protect the pavement from high velocities and flows that can break-up the pavement and damage the bayside embankment.
- Dune Construction — Constructing a sand dune seaward of the road to reduce the likelihood of overwashing and to provide a reservoir of sand to bury the pavement when overwashing occurs.
- Armoring — Armoring of the shoulders of the road to resist erosion during overwashing.

Figure 15-24, below, shows the first three strategies in a roadway cross section view.

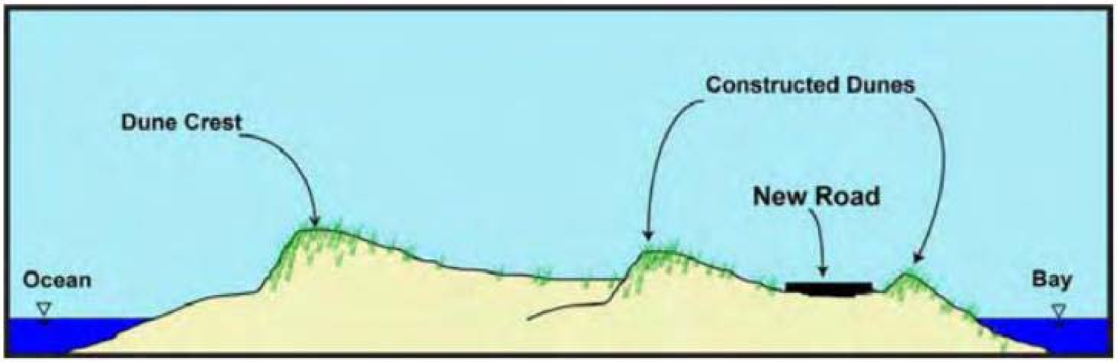


Figure 15-24. Schematic Summarizing Three Approaches (Bayside Location, Low Elevation, Dune Construction) to Minimize Damage to Roads that Overwash (FHWA, 2008)

Coastal Weir-Flow Damage:

One scour mechanism that damages pavements during overwashing is the coastal weir-flow damage mechanism. During this mechanism, water overtops the roadway embankment. The embankment acts as a broad-crested weir to the incoming storm surge. The water that flows over the embankment can become supercritical as it reaches the edge of pavement and have a free fall or a high tailwater condition. It is likely that waves will exacerbate the weir-flow damage mechanism and increase the scour risk. This happens when waves moving across the pavement with the storm surge increase the instantaneous flow velocities on the downstream shoulder. In addition to incoming storm impacts, the embankment can act as a broad-crested weir and experience similar damage mechanisms as storm surge recedes. There is also a concern of uplift on overtopped pavements due to a potential increase in underlying pore pressure in the sandy base materials.

Mitigation measures for weir-flow are similar to general overwashing. Lowering the roadway elevation to an elevation at or below adjacent ground elevations can prevent weir-flow from occurring, since the crest of the pavement is no longer the highest portion of the grade. In addition, considerations can be incorporated into the pavement design to mitigate pore pressure increases. Armoring the shoulder to withstand weir-flow mechanisms is recommended where lowering the roadway is not possible.

Wave Action:

Waves, as discussed in Section 3, cause some of the primary hydraulic forces on coastal roadway embankments. Some key variables that are often considered in revetment design to protect roadways include design wave height, potential for wave runup, and potential for overtopping splash. Where wave action and wave runup are a concern, revetment, seawalls, and bulkheads may be considered for shoreline or roadway embankment protection, as previously discussed. Relocation further inland can also reduce impacts of wave action.

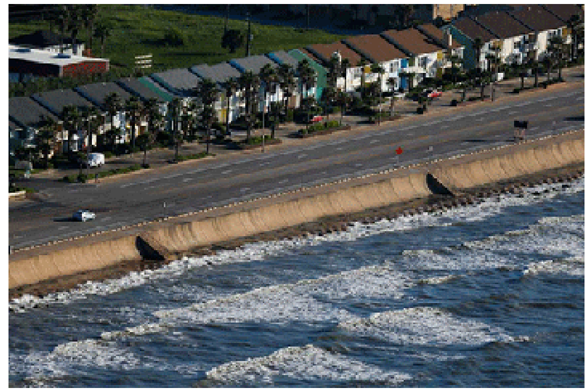
Coastal Armoring for Wave Attack

Revetments, seawalls, and bulkheads are all coastal protection features designed to protect roadways and other coastal assets from dynamic coastal processes as shown in the examples in Figure 15-25. The preferred use of each option is dependent on the local conditions. Revetments are layers of protection on top of a sloped surface to protect the underlying, elevated soil. Seawalls are free-standing structures that can be stable at greater heights than revetment and are designed to protect against large wave forces. Bulkheads are designed primarily to retain the soil behind a vertical wall and are primarily used in areas with limited wave action. The fetch is a length of ocean or lake over which the wind blows in a constant direction. Given the relationship between wave height and fetch, the flowing discussion distinguishes between the three types of coastal protection and their applicability:

- Bulkheads are most common where fetches and wave heights are small, because they have limited capacity to protect against large wave forces.
- Revetments are often found in intermediate situations, such as along bays or lakes. Most transportation projects throughout Texas will rely on revetment to protect low-lying roadways, since the wave action protection provided by revetment is needed, but the height of a seawall is not required.
- Seawalls are most common where fetches and wave heights are very large, because the free-standing structures can be designed high enough to withstand larger waves and be generally more stable than revetments.

Although these three structural alternatives are the traditional approaches to coastal armoring, combinations of them with softer methods, as later discussed, are becoming more viable and can often provide more dynamic long-term protection. It is important to note that the more robust the structural alternative, such as a seawall, the more likely for residual shoreline or structural erosion adjacent to the project site. Residual impacts should be considered when evaluating project solutions.

A - Revetment construction along CR257 in Brazoria County, Texas (Apollo Environmental)



B - Galveston Seawall (Houston Chronicle)

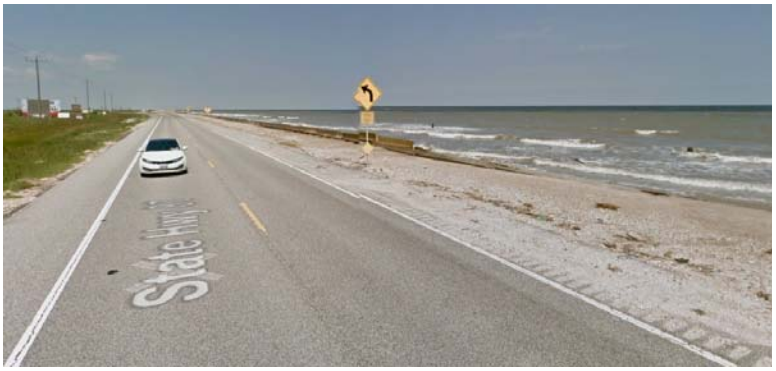
C - Bulkheads protecting roadways in Aransas County, Texas

D - Vertical walls provide roadway protection along this Ocean Drive crossing in Corpus Christi, Texas

Figure 15-25. Examples of Shoreline Protection.

A well-designed and constructed revetment can protect roadway embankments from waves. The rubble is efficient at absorbing wave energy and any damage that the revetment receives is often inexpensive to repair. HEC-23 and HEC-25 provide detailed procedures for the design of stone revetments and recommend Hudson's equation to estimate stone size for the outer layer of rock revetments subject to wave action. Hudson's equation accounts for the most important variables, including design wave height, structure slopes, and different stone densities and angularities. HEC-25 provides a more detailed description of calculations for the design wave heights for revetment design. HEC-25 also provides examples for the applicability of design alternatives depending on specific project site conditions and requirements.

Some revetment design failure mechanisms include: inadequate armor layer design for wave action, inadequate under layer, flanking, toe scour, and overtopping splash (Figure 15-26).



A - Toe Scour causing concrete median barriers to topple along the shoreline adjacent to SH87 near High Island, Texas

B - Inadequate Armor Layer - A revetment with rocks too small to withstand wave attack along SH316 in Calhoun County, Texas

Figure 15-26. Examples of Failed Shoreline Protection.

To prevent failure mechanisms, precaution should be taken when applying design equations, like from HEC-23 or HEC-25, to ensure values used represent the intent of the equations. Stone should be sized to ensure wave action is appropriately addressed. Toe design should have significant depth of stones to prevent undermining. The protection should tie back into natural structures or adjacent resistant structures to prevent flanking and should extend high enough to prevent overtopping splash damage. The biggest risk is exposure of the underlying soil. Concrete block and panel revetment have not had a good performance record in severe coastal environments; neither approach matches the performance and flexibility of stone revetment (FHWA, 2008).

New technologies are emerging for scour protection; some examples include scour control mats and geo-fabric stone bags. As case studies develop and results are reviewed, these technologies might serve as an additional option for shoreline and structure protection (Figure 15-27).

Figure 15-27. Cellular Concrete Mattresses Protecting a Coastal Highway in Calhoun County, Texas.

Other hard coastal structures that are successful in shoreline protection include groins, breakwaters, and hybrid structures (Figure 15-28). Groins are narrow, roughly shore-normal structures built to reduce longshore currents, and/or to trap and retain littoral material. Breakwaters are structures protecting a shore area, harbor, anchorage, or basin from waves. Hybrid structures are some functional combination of groins, breakwaters, or structural measures functioning in a complementary fashion. Further studies need to be conducted on all coastal protection measures, but redundancy in protection methods should be considered for adaptive designs in coastal environments.

Living shorelines, discussed more in Section 6, are techniques that involve combining ecologically-based, “soft” mitigation techniques, such as coastal vegetation, with hardened solutions, such as breakwaters. This is also a more adaptable system that can potentially provide improved long-term mitigation due to its more dynamic response to future coastal change (e.g., vegetated features can migrate with sea level rise). One example of a soft coastal protection method along roadways is beach nourishment, particularly on Gulf shorelines. Placing sand to widen a beach along with hard coastal structures can lead to a great balance between structural needs and environmental conditions to mitigate erosion.

A - Wetlands and submerged oyster reef barriers provide living shoreline protection for Broadway Street in Aransas County, Texas

B - Groin fields shelter local roadways in Calhoun County, Texas

Figure 15-28. Examples of Alternative Shoreline Protection Strategies.

Coastal Bridge Scour Mitigation

Engineering coastal bridges can be complex and requires consideration of forces and processes unique to the coastal environment, including tidal bridge scour and hydrodynamic loads from waves and tidal currents. The types of scour that occur at bridges in the coastal environment include the same general categories as found at riverine bridges. However, in the case of coastal bridges, scour can potentially be caused by more varied directions and magnitudes, thus increasing the complexity of analysis. Additionally, coastal bridges can experience scour as a result of wave action (wave scour) and localized areas of high velocity flows.

Methods of design equations to consider in coastal scour evaluation can be found in HEC-18, HEC-20, HEC-25, and the USACE CEM. Scour as defined for coastal bridges is broken into three approaches per HEC-25. For a level 3 approach as shown previously in Figure 15-23 where advanced modeling is needed because of the variety of nearshore processes, a precertified TxDOT coastal engineer should be involved. Each design storm or nearshore process scenario that is simulated should be considered for impacts on scour. Combinations of different physical processes that could increase the scour rate should be considered. For instance, if a bridge experiences tidal, wave, and storm surge conditions as part of the design storm scenarios, the hydrodynamic impacts of each condition above should be reviewed for variables like water elevation, velocity, depth, and other factors needed in scour evaluations.

Guidance documents for countermeasures included in HEC-23 apply for coastal areas as they do for riverine, and include: *Design Guideline 11 - Rock Riprap at Bridge Piers* and *Design Guideline 17 - Riprap Design for Wave Attack*. In Texas, riprap often refers to concrete; in FHWA guidance, riprap is typically equivalent to stone revetment. For the purpose of this chapter, riprap directly referencing a design guideline title means stone revetment.

Stone revetment is a common revetment countermeasure at bridge piers and abutments. Stone revetment should follow HEC-23 *Design Guideline 4 - Riprap Revetment* for guidance on sizing, gradation, placement methods, and failure mechanisms. Some failure mechanisms of stone revetment include: particle size being too small, channel changes like migration or channelization, improper gradation, improper placement, and lack of or inadequate filter. To avoid the failure mechanisms, attempts should be made to evaluate variables used for appropriateness when applying equations to size stone revetment, and any channel changes as evaluated per HEC-20 should be considered in the application for the design life of the bridge. Similar to riverine applications, HEC-23 for countermeasure design can be utilized for coastal applications methods, but all available coastal inputs should be considered and applied in the design (such as tides, wave, and storm surge).

Scour Data Sources

- **Federal Highway Administration**
  - **HEC-18 – Evaluating Scour at Bridges** — Chapter 9 of HEC-18 provides information on scour as a result of tidal currents and potential related considerations. The document outlines three levels of analysis ranging from a Level 1 qualitative analysis to a Level 3 quantitative analysis including physical models or sophisticated numerical models.
  - **HEC-20 – Stream Stability at Highway Structures** — Chapter 2 of HEC-20 discusses geomorphic factors and principles, and while geared toward riverine environments, the same considerations should be applied to evaluate coastal or lake shoreline stability. Chapter 4 discusses analysis procedures for stream instability, including a discussion on mathematical and physical model studies. This would be used during a scour evaluation required by 23 CFR 650.313(e) to evaluate stream stability. Chapter 5 discusses stream reconnaissance techniques which can apply to coastal conditions in the method of data collection and conducting a field visit.
  - **HEC-23 – Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance** — *Design Guideline 17* in HEC-23 provides guidance for stone revetment design for sites subject to wave action. Other Design Guidelines within HEC-23, can also be useful in coastal applications.

  - **HEC-25 – Highways in the Coastal Environment** — HEC-25 discusses scour at coastal bridges and provides descriptions of the multiple ways that a bridge can experience scour due to different geometries. Also discussed are the time-dependent nature of scour and the influence of longer storm events.
- **USACE Coastal Engineering Manual** — The "Scour and Scour Protection" chapter of the CEM provides background on the dynamic nature of coastal scour and presents several empirical methods to estimate potential scour. The chapter also discusses common structures and geometries that may induce scour.
- **TxDOT Geotechnical Manual** — Chapter 5 (Foundation Design), Section 6 (Scour) of the *TxDOT Geotechnical Manual* notes that the guidelines outlined in HEC-18 should be used. Section 6 (Scour) also provides minimum design flood frequencies to be used for scour design. Although guidance was not separately developed for coastal settings, the same theory may be applied once controlling conditions are determined. The primary difference in using these equations for coastal settings is the need to better define input velocities and wave conditions based on the two- and three-dimensional nature of coastal hydrodynamics (for example, tides cause ebb and flood flow in tidal channels, whereas riverine flow is almost always downstream).

Shoreline Change

Shoreline change can be long-term, short-term, or storm induced/episodic and is often expressed in terms of the average change in shoreline position with time (e.g., 1 foot per year). Shoreline change can also occur in several directions, including moving landward (recession) or seaward (accretion). Shoreline change differs from scour in that a scour analysis focuses on structural assets, like a roadway or bridge, while shoreline change is a large-scale view of a section or region of coastline. The difference between coastal scour and coastal shoreline change is similar in principle to riverine contraction and local scour of structural assets, as compared to the long-term aggradation and degradation of the stream bed and lateral stream migration in riverine systems.

Dynamics

Wind, waves, and currents are the primary forces that move sediment and larger stones. If a change occurs in the pattern of winds, waves, currents, or sediment supply, then shoreline change can occur. Another impact that can change the forces listed above is variation in the sea level. Changes in sea level, discussed in Section 2, can affect where the shoreline and nearshore processes occur and influence the shoreline elevation and width. The presence of shoreline features, such as tidal inlets and coastal engineering structures (e.g., jetties and breakwaters) can also affect shoreline change, as they can exacerbate or attenuate wave and current conditions, depending on their location.

Episodic or storm-induced shoreline change can cause significant retreat or impacts to a shoreline. An individual storm may cause significant erosion or even trigger the beginning of an erosional period (an erosional period is a short-term change that may not relate to or follow long-term shoreline trends). Specific to Gulf shorelines in Texas, large storms on low-lying barrier islands can cause island rollover and migration, or in some locations may remove large amounts of sand from the beach. Island rollover and migration occurs as storm surge moves sediment along the barrier islands landward. It can take years for a Gulf shoreline to naturally recover from a storm, although sometimes the culmination of lesser storm activity can result in the replacement of much of the sand that was lost. It should be noted that storm-induced erosion of bay shorelines is not typically followed by any shoreline recovery, as bay shorelines are commonly clay- and bluff-based shorelines that do not have the dynamic ability to recover as a Gulf sand-based shoreline does.

Beyond storm-induced changes, long-term changes along Texas coastlines are driven by sediment movement and the typical annual current and wave conditions, or even typical vessel wakes. These long-term trends are more frequently recognized at the regional level, particularly for Gulf shorelines. Most Texas Gulf shorelines are considered eroding over the long-term, with some net neutral or accreting locations near the central Texas coast. This is the result of a general trend for net positive longshore sediment movement towards the central Texas coast; however, there are localized exceptions resulting primarily from groin structures interrupting the natural occurring sediment movement.

Quantifying Shoreline Change Rates

Shoreline change rates vary along the Texas shoreline and through time. When evaluating shoreline change rates, it is important to consider all available observations and as long of a period of record as possible. Long-term shoreline change typically refers to historic time scales where measurements were recorded (e.g., 1800s to present) and is useful for understanding potential impacts to roadway infrastructure through the coming century. In contrast, short-term shoreline change is typically calculated using recent multi-decadal timescales (e.g., 1970s to present). Understanding short-term shoreline change is useful for evaluating impacts to a project that may occur at an earlier planning time frame, or if the project is in an area that has been largely influenced by human activity, such as engineering shoreline protection structures, in recent decades.

While there is no national or state standard for shoreline change analyses, there are common approaches to quantifying changes. Mapped historical shoreline positions are often used to interpret the movement of shorelines through time, and linear regression statistical approaches are useful to approximate change rates that vary along the shoreline (Figure 15-29). Observed rates are commonly extrapolated into the future to assess future long-term impacts at design sites. This approach is especially valuable when planning the future location of highways located in areas with receding shorelines.

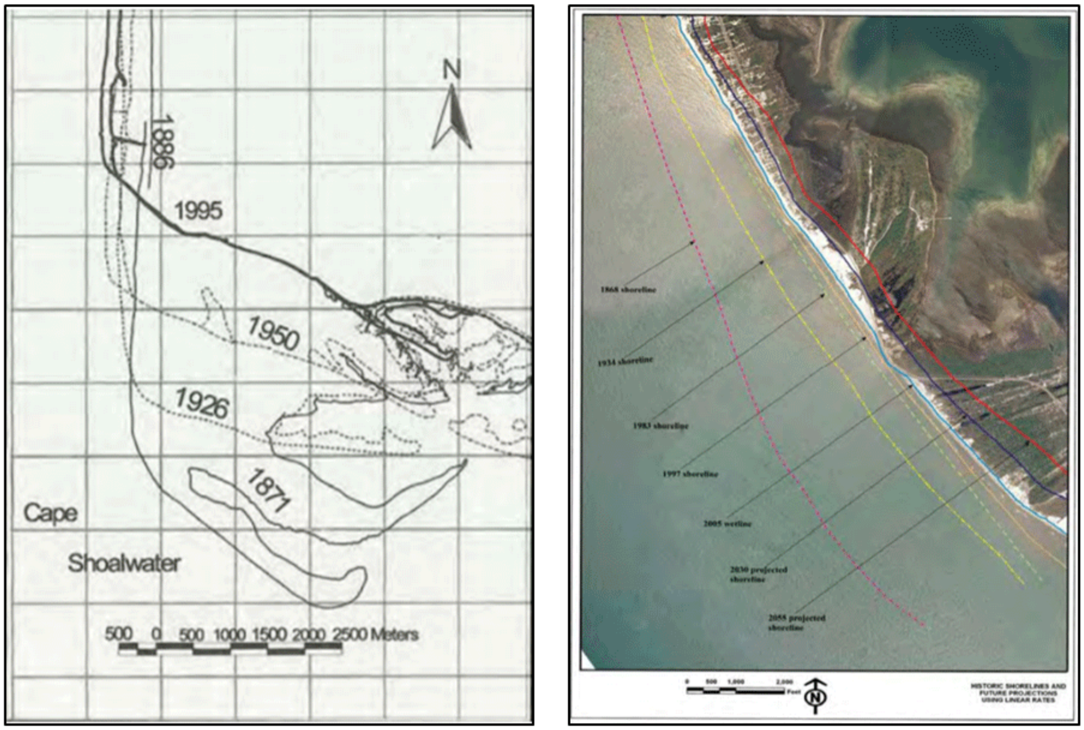


Figure 15-29. Example of Historic (left) and Projected Future (right) Shoreline Positions (FHWA, 2008)

Historical shoreline data for Texas Gulf shorelines is primarily documented through the [Texas Shoreline Change Project] managed by the Bureau of Economic Geology (BEG) at the University of Texas (Figure 15-30).

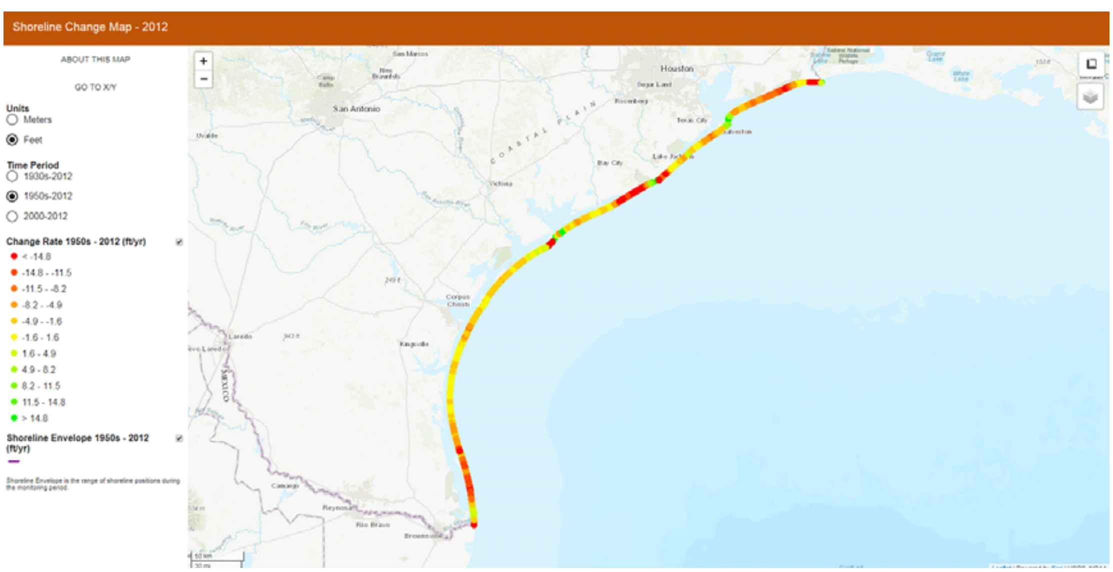


Figure 15-30. Shoreline Erosion Map Interface for BEG Gulf Shoreline Tool (as of 2012)

Bay shoreline data is not widely available within Texas for historical rates, and frequently the best alternative is to utilize historical imagery to identify trends, with Google Earth serving as the primary option.

Some limitations to using historic shoreline change rates to estimate future shoreline position include:

- Shoreline processes are often not linear with time;
- Human influence may have influenced historic shoreline changes;
- Human influence may influence future shoreline change; and
- Data collected is limited to human record timeframes.

Local sediment budgets play a large role in the dynamics of shoreline change. Sediment budgets describe the input, transport, storage, and export of sediment along the shoreline. Whether evaluated quantitatively or qualitatively, all sediment budgets are conceptually understood by a continuity equation stating that during a given time period, the amount of sediment coming into the area minus the amount leaving the area equals the change in the amount of stored sediment. Historical aerial imagery as well as survey can be used to evaluate long-term trends in the shoreline as mentioned above but review of the sediment budget can also be used to estimate future shoreline positions.

Sediment budgets typically require much more data and analysis than simple shoreline change extrapolation. Aerial photography and remote sensing can be used to quantify visual and volumetric changes in sediment along a shoreline. In addition, sediment sampling in the water column and along the seafloor can be combined to develop data needed for sediment transport models. In the mapping or quantifying of sediment transport, specific erosion problems can be identified or better understood in order to develop appropriate site-specific solutions. Figure 15-31 is an example of a sediment budget evaluation and map. The sediment budgets were determined through volumetric methods that evaluated the beach profile changes from 1973-1997.

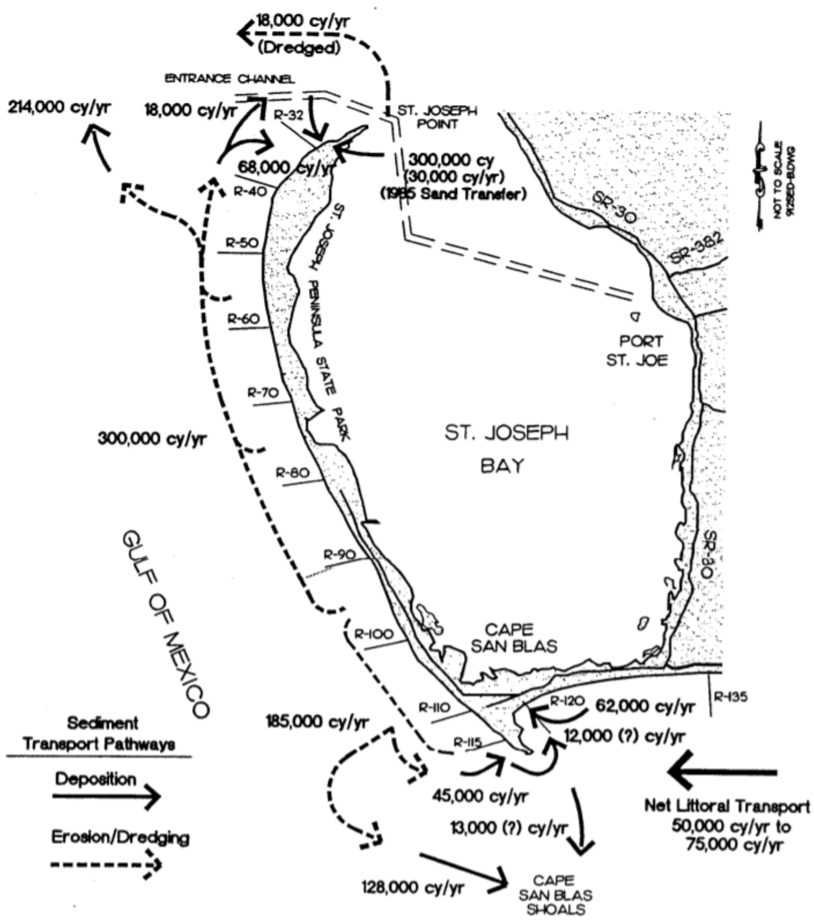


Figure 15-31. Example of a Coastal Sediment Budget (FHWA, 2008)

Numerical Modeling of Shoreline Change

In addition to prorating historical trends of shoreline change, numerical models have been developed to estimate future shoreline positions. Few numerical models can accommodate all of the design factors discussed in this chapter within the same model. Models also vary in their degree of complexity, ranging from 1D to 3D capabilities. Sediment transport models have more inherent uncertainty than hydrodynamic models, as they must consider additional factors such as sediment grain size and sources/sinks of sediment. Figure 15-32 shows inputs per level of analysis and available modeling for each level.

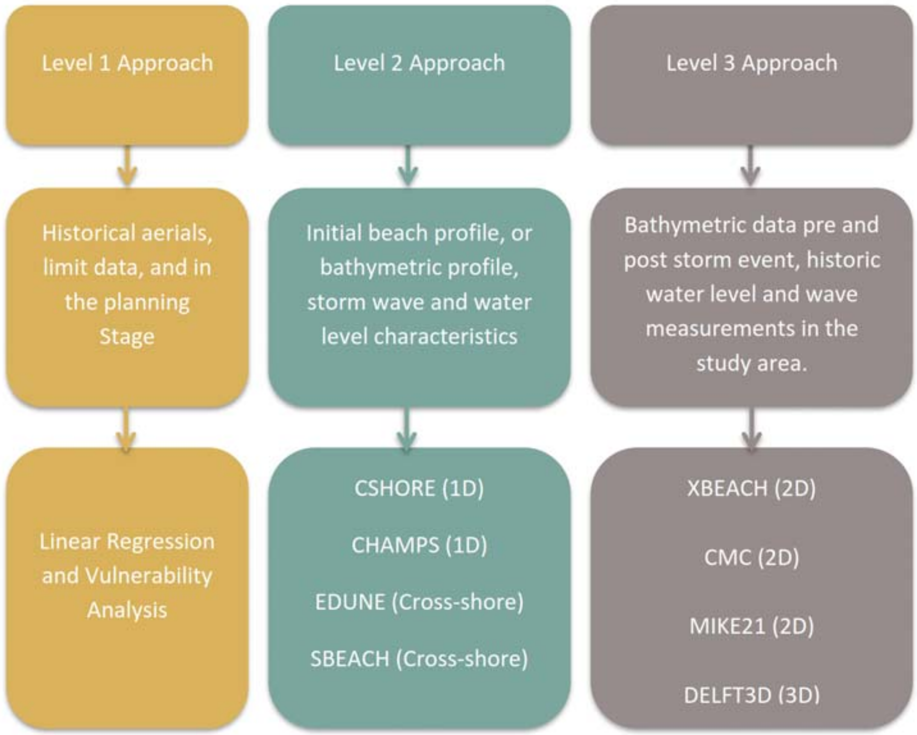


Figure 15-32. Shoreline Change Level of Analysis Inputs and Applicable Models.

The most common method for estimating future shoreline positions is direct extrapolation of historic shoreline change rates to the present shoreline from historic maps and aerials. Shortcomings in this method include the fact that shorelines change processes may not be linear with time; it is difficult to tie the shorelines to a specific datum, engineering may have impacted shoreline change in the past and may impact future change. If an area is rural and has experienced minimal engineering design along its coast, then historic maps and aerials can be appropriate for shoreline change. If shoreline change and modeling is required for a large study area with many engineered structures and recorded shoreline change, it is usually necessary to use a highly advanced model to achieve the sediment transport detail needed for design.

Dredging can play a large role in shoreline change and especially in sites adjacent to the GIWW. Sargent Bridge in Texas is an example of an abutment being designed for future dredging depth and impacts, rather than current conditions. This bridge site and the large area impacted by repeated or planned dredging would be a candidate for advanced modeling to evaluate shoreline change and sediment transport.

The type of project may dictate when to choose between a long-term and a short-term analysis. When evaluating short-term shoreline change due to extreme events, such as hurricanes, available data may limit the type of approach. The validation and calibration of such short-term shoreline change models is only possible when the available data, such as shoreline positions and elevations, are available both before and after the storm event. Modeling a short-term change is needed to calibrate the model to review long-term changes. Long-term change models can then include several probable storm events. Such data has become more common thanks to technological advancements in elevation data collection. Pre-storm and post-storm shoreline position and elevation data are available in limited locations for some recent storms in the form of coastal LiDAR data. Data can be obtained from the NOAA Coastal Services Center Digital Coast web portal.

Impacts on Coastal Structures

Roadways and other transportation assets along the coast are vulnerable to shoreline change (Figure 15-33). Once shoreline change has been identified as a design factor impacting a roadway asset, the vulnerability should be assessed. One method to determine vulnerability is to map the elevation and shoreline positions near structures versus the long-term recession rate (the long-term recession rate is the long-term shoreline change at an area that is determined to be recessing landward), using methods previously outlined. The less time a structure has until exposed by shoreline recession, the greater the potential vulnerability. This approach should also be combined with consideration of short-term or storm-induced erosion to help prioritize structures with a high potential for exposure to erosion. Early consideration is highly beneficial for planning repair, protection, or relocation strategies. If possible, transportation structures vulnerable to shoreline change should be considered for relocation to a landward position. However, if relocation is not an option due to adjacent private property, environmental concerns, or other issues, engineered shoreline stabilization combined with monitoring may provide long-term protection.

Relocation example in North Carolina (FHWA, 2008)

Road destroyed by shoreline recession: SH87 in Jefferson County, Texas was destroyed during Hurricane Ike. Due to shoreline retreat, it is not viable to be rebuilt in its current location.

Figure 15-33. Examples of Roadways affected by Shoreline Change.

As discussed in the *Scour Mitigation Measures* subsection, stabilization or remediation can be achieved through both hard engineering (e.g., seawalls, groins, breakwaters, and hybrid structures) or soft engineering measures (e.g., beach nourishment, dune regeneration, creating marshland, and vegetative plantings). A combination of techniques is often most effective in ensuring success. Mitigation measures for shoreline stabilization are similar to mitigation for scour but should be viewed on a broader scale in terms of sediment transport. Right-of-ways are often narrow, and joint efforts or partnerships may be required for roadway revetment and combination of hard and soft protection.

Shoreline Change Data Sources

- **U.S. Geological Survey** — USGS provides historical shoreline positions and calculated short-term and long-term shoreline change rates for many areas along the Gulf Coast. http://coastal.er.usgs.gov/shoreline-change/
- **NOAA Coastal Services Center Digital Coast** — Data is available through NOAA Digital Coast web portal. https://coast.noaa.gov/digitalcoast/
- **University of Texas Bureau of Economic Geology** — The Texas General Land Office, working with the University of Texas BEG, maintains and updates coast-wide erosion rates for Texas Gulf shorelines to estimate the areas of greatest long-term shoreline change in Texas. This information can be reviewed on their interactive map. http://www.beg.utexas.edu/research/programs/coastal/the-texas-shoreline-change-project.
- **Federal Highway Administration** — HEC-25 discusses quantifying shoreline change rates and provides descriptions of the multiple ways that a roadway and shoreline can experience shoreline changes. Also discussed are the modeling methods for analyzing shoreline change.

Section 7 — Additional Considerations

This section contains various topics that should be considered during transportation design in a coastal area. These topics include:

- Topography and bathymetry
- Construction materials
- Design elements

Topography and Bathymetry

Topographic and bathymetric information provide the basis for equation inputs, such as water depth, beach slopes, and land elevations. These data are also valuable when creating numerical modeling domains to determine various coastal processes at a particular location. The following discussion focuses on evaluating and acquiring existing data for these purposes, but there are locations or projects that may require new data collection. In general, the entire Texas coastal region has had LiDAR data collected since 2007 that provide topographic coverage of high resolution, that would only need to be supplemented by local survey for design work. Bathymetry is much less recently or accurately collected along the Texas coast. Readily available bathymetry datasets would be sufficient for Level 1 or possibly Level 2 analyses, but for Level 2 analyses where water depths at or near the project site are critical for coastal design calculations and any Level 3 analysis, local bathymetric survey would be required to supplement the regional or statewide datasets.

In many cases, existing high-resolution datasets, such as LiDAR terrain or bathymetry data, will be readily available. When used in combination with site specific field surveys necessary for project design, they can provide the information necessary to understand historical trends of elevation changes in the project area, including in the surrounding submerged environment when bathymetric field survey is collected. In locations where LiDAR or other high-resolution elevation/bathymetric datasets already exist, the designer must determine whether and how the datasets can be used in the project. The following conditions should be considered:

- **Date of Collection**. While there is not a specific age requirement for elevation data, coastal areas are dynamic, and preference should be given to more recent data. If a known area within an older dataset has experienced a terrain disturbance since the data collection (such as a hurricane that caused coastal erosion), new data may be collected within this smaller area and the older dataset may be used for the remainder of the area.
- **Metadata**. All terrain datasets should have accompanying metadata. This information should include collection and processing information, as well as the vertical and horizontal accuracy of the dataset. If metadata is missing, it may be necessary to contact the agency responsible for data collection to obtain the metadata or to perform horizontal or vertical accuracy tests on an area of known topography to determine validity.

- **Accuracy**. Generally, data with the highest vertical accuracy is most appropriate for use. However, other factors such as age, extent, resolution and availability of the dataset should also be considered. For example, if physical changes have occurred to an area during the intervening years within a dataset with very high vertical accuracy, the reported vertical accuracy may be rendered meaningless for the area. If possible, comparison of multiple datasets for the area will highlight the most accurate in terms of representing the terrain.

Topographic and Bathymetric Data Sources

- **Project Specific Surveys** — Depending on the availability and age of publicly available bathymetric and topographic data, project specific surveys may be necessary as noted at the beginning of this section. Site specific surveys are particularly important in areas with recent erosion or accretion that may not be apparent in older, publicly available data, but is identified by local stakeholders or TxDOT operations staff.
- **Texas Natural Resources Information System (TNRIS)** — TNRIS serves as the geospatial data clearinghouse for the state of Texas. They maintain publicly available datasets for Texas-based LiDAR as well as a statewide coastal bathymetry dataset. This is the primary topographic and bathymetry data source for TxDOT projects in the coastal zone. https://data.tnris.org/
- **USACE Hydrographic Surveys** — The USACE Galveston District performs routine channel-condition surveys along the federally-maintained ship channels and the Gulf Intracoastal Waterway. The age, resolution, and format of these data vary by location. http://www.swg.usace.army.mil/Missions/Navigation/HydrographicSurveys/
- **NOAA National Centers for Environmental Information (NCEI)** — Formerly the National Geophysical Data Center (NGDC). The NCEI provides a map interface that displays the location of publicly available bathymetric data. The age, resolution, and format of these data vary by data type, source and location and can be found at https://www.ngdc.noaa.gov/. Also see https://www.ngdc.noaa.gov/mgg/bathymetry/estuarine/
- **U.S. Geological Survey: The National Map** — USGS provides both existing and historical topographic information via The National Map. The age, resolution, and format of these data vary by source and location. USGS provides a national level resource to supplement TNRIS, if necessary. http://nationalmap.gov/

Construction Materials in Transportation Infrastructure

This section identifies best practices in materials selection that may be influenced by coastal processes. The material guidance included in this section would apply to various TxDOT infrastructure projects, including pavements, facilities, ferries, drainage structures, bridges, etc. Besides typical design factors such as traffic loading and subgrade soil characteristics, infrastructure located within coastal areas can be subject to additional effects such as saltwater intrusion, uplift from storm surges, and rapid drawdowns.

Saltwater Intrusion

Infrastructure's exposure to saltwater can lead to serious damage and drastically reduce the design life of infrastructure if not considered in the design. It is recommended the designer consult guidance on material selection for infrastructure that could encounter this environment.

Infrastructure

Infrastructure including, but not limited to, drainage structures, bridge piles, and guard rails may need to consider saltwater influence when choosing the most appropriate material. For example, for bridge piles located in saltwater environments, marine grade steel is recommended (USACE New Orleans District, 2012).

The U.S. Army Corps of Engineers provides guidance on materials selection for various infrastructure and coastal conditions.

Pavements

Similar to all typical pavements, coastal pavements can be categorized into rigid and flexible pavement types. Rigid pavements are composed of a stiff Portland Cement Concrete (PCC) surface, while flexible pavements consist of bendable surfaces, such as asphalt. Common to both pavement types are base courses and subbase courses, consisting typically of aggregate materials. All pavement sections are supported by a subgrade, which usually entails the existing ground formation. Pavement type, including future maintenance (of joints, materials, etc.), will primarily be determined by the transportation design (e.g., loading, traffic), but coastal impacts should also be considered.

The general impact of saltwater intrusion on pavements is the effect of corrosion, and pavement materials must be selected to counteract this factor. For PCC pavements, the primary element susceptible to corrosion is any steel reinforcement contained within, as well as any steel dowels used at the joints. One way to afford corrosion protection is by using corrosion-resistant materials for reinforcement and dowels. Alternatively, plastic fiber reinforcement, which is not subject to oxidation, may be used as a form of reinforcement. Improvements to the PCC mix design itself can also provide enhanced protection from saltwater, such as by using sulfate resistant cement and by reducing the permeability of concrete through the use pozzolans, including fly ash, slag cements, and silica fume. Another consideration during pavement design is increasing the concrete thickness over reinforcing bars to provide additional protection.

Asphalt surfaces are not significantly affected by saltwater corrosion. Generally, a dense-graded asphalt mixture with low porosity is recommended for pavement construction. A soft asphalt binder is preferred so that a longer service life to hardening under saline attack is obtained. Additionally, eliminating the use of moisture-susceptible aggregates and anti-stripping agents can minimize moisture damage of the asphalt mixture in general.

Construction Materials Data Sources

- **American Concrete Institute (ACI)** — ACI 357R-84, Guide for the Design and Construction of Fixed Offshore Concrete Structures (Chapter 2, Materials and Durability).
- **The Aberdeen Group** — Designing Concrete for Exposure to Seawater, Bruce A. Suprenant, Publication # C910873, 1991.
- **Federal Highway Administration (FHWA)** —
  - Techniques for Reducing Moisture Damage in Asphalt Mixtures, FHWA/TX-85/68+253-9F, November 1984.
  - Advanced High-Performance Materials for Highway Applications: A Report on the State of Technology, FHWA-HIF-10-002, October 2010.
- **U.S. Army Corps of Engineers New Orleans District** — Hurricane and Storm Damage Risk Reduction System Design Guidelines: Chapter 5.0 - Structures. https://www.mvn.usace.army.mil/Portals/56/docs/engineering/HurrGuide/Ch5.pdf

Storm Surge and Drawdown Protection

For protection of pavements from the effects of storm surge uplifts and rapid draw downs, the use of a treated base, such as cement-treated base or asphalt-stabilized base, is most effective in waterproofing PCC pavements. For flexible asphalt pavements, the critical factor is to utilize permeable aggregate materials in the base and subbase layers under the pavement that can tolerate and convey seawater surges. To accomplish this, the aggregate layers under the pavement must be of sufficient thickness (as well as be resistant to saltwater corrosion) to convey the surge through each layer and not cause the surge to back up and produce uplift in the impermeable pavement surface constructed overtop of it.

Alternatively, a permeable asphalt surface may be considered under load conditions that warrant its use. Typically, the asphalt layer lies on top of an aggregate drainage layer that drains into underdrains that lead to roadside ditches or are conveyed into an overall storm drainage system for the roadway.

Where coastal pavements are supported by subgrades of low strength, or for pavements that are subject to water level changes, the use of geotextile grids or fabrics to reinforce subgrades and bases is recommended. The major actions of geotextiles are separation of materials and reinforcement. Benefits afforded by geotextiles used in subgrade and base layers include base and subgrade restraint, lateral restraint, and membrane-type support of the pavement system.

Storm Surge and Drawdown Protection Data Sources

- **Federal Highway Administration** —
  - Hydraulic Engineering Circular No. 25, *Highways in the Coastal Environment*, 2nd Edition, FHWA NHI-07-096, June 2008 (update anticipated in Summer 2019).

  - Hydraulic Engineering Circular No. 25 – Volume 2, *Highways in the Coastal Environment: Assessing Extreme Events*, FHWA-NHI-14-006, October 2014.
  - Hydraulic Engineering Circular No. 17, Highways in the River Environment- Floodplains, Extreme Events, Risk, and Resilience, 2nd Edition, FHWA-HIF-16-018, June 2016.
- *Handbook of Geosynthetics*, Geosynthetic Materials Association (GMA), 2010.
- *The Handbook of Groundwater Engineering* (Chapter 37: Geosynthetics), 2nd Edition, Zornberg, J.G., and Christopher, B.R., 2007.

Design Elements

Coastal structures can include roadways and bridges, but also include structures that promote coastal defenses with the objective of minimizing shoreline erosion or protecting against nearshore processes. Some of these coastal structures were discussed in Section 5. The general types and functions of coastal structures and roadway design, bridge design, and site-specific design conditions are discussed below as part of planning a coastal design.

Depending on the site conditions and the nearshore processes at play, there are several types of coastal structures that can be designed to protect and mitigate concerns in coastal areas. Coastal structures shown in Figure 15-34 below can be considered in combination for a complementary design that works to best address multiple, site specific concerns.

Table VI-2-1
Types and Functions of Coastal Structures

Type of Structure	Objective	Principal Function
Sea dike	Prevent or alleviate flooding by the sea of low-lying land areas	Separation of shoreline from hinterland by a high impermeable structure
Seawall	Protect land and structures from flooding and overtopping	Reinforcement of some part of the beach profile
Revetment	Protect the shoreline against erosion	Reinforcement of some part of the beach profile
Bulkhead	Retain soil and prevent sliding of the land behind	Reinforcement of the soil bank
Groin	Prevent beach erosion	Reduction of longshore transport of sediment
Detached breakwater	Prevent beach erosion	Reduction of wave heights in the lee of the structure and reduction of longshore transport of sediment
Reef breakwater	Prevent beach erosion	Reduction of wave heights at the shore
Submerged sill	Prevent beach erosion	Retard offshore movement of sediment
Beach drain	Prevent beach erosion	Accumulation of beach material on the drained portion of beach
Beach nourishment and dune construction	Prevent beach erosion and protect against flooding	Artificial infill of beach and dune material to be eroded by waves and currents in lieu of natural supply
Breakwater	Shelter harbor basins, harbor entrances, and water intakes against waves and currents	Dissipation of wave energy and/or reflection of wave energy back into the sea
Floating breakwater	Shelter harbor basins and mooring areas against short-period waves	Reduction of wave heights by reflection and attenuation
Jetty	Stabilize navigation channels at river mouths and tidal inlets	Confine streams and tidal flow. Protect against storm water and crosscurrents
Training walls	Prevent unwanted sedimentation or erosion and protect moorings against currents	Direct natural or man-made current flow by forcing water movement along the structure
Storm surge barrier	Protect estuaries against storm surges	Separation of estuary from the sea by movable locks or gates
Pipeline outfall	Transport of fluids	Gravity-based stability
Pile structure	Provide deck space for traffic, pipelines, etc., and provide mooring facilities	Transfer of deck load forces to the seabed
Scour protection	Protect coastal structures against instability caused by seabed scour	Provide resistance to erosion caused by waves and current

Figure 15-34. Table VI-2-1 from Coastal Engineering Manual (USACE, 2002).

Coastal Roadway Design Considerations

Roadway design in coastal environments can begin similar to upland roadway design. The roadway geometry and layout should consider, when applicable, locations that have minimal impacts from nearshore processes. The roadway elevation should consider the design elevation (Section 4) to include any additional height requirements to avoid overtopping or wave splash impacts on the pavement structure. If raising the grade is not preferred or possible, other techniques mentioned in Section 5 for overwash should be applied to protect the roadway during overwash conditions. If the roadway is adjacent or close to areas where wave action and storm surge can impact the structure, protection of the embankment should be considered.

Hard structures often considered as a complement in roadway design from Figure 15-34 above include revetments and seawalls. Revetments and seawalls start with similar design considerations, primarily wave concerns. Revetments are onshore structures with the principal function of protecting the shoreline from erosion. Revetment sizing was discussed with regards to design wave height in Section 3 and Hudson's formula for particle sizing in Section 5. Seawalls, also discussed in Section 5, are used to protect promenades, roads, and buildings placed seaward side of the beach on the slope next to the top of the dune. The USACE CEM provides typical cross sections and layouts as well as example design problems for these structures, as seen in Figure 15-35 below.

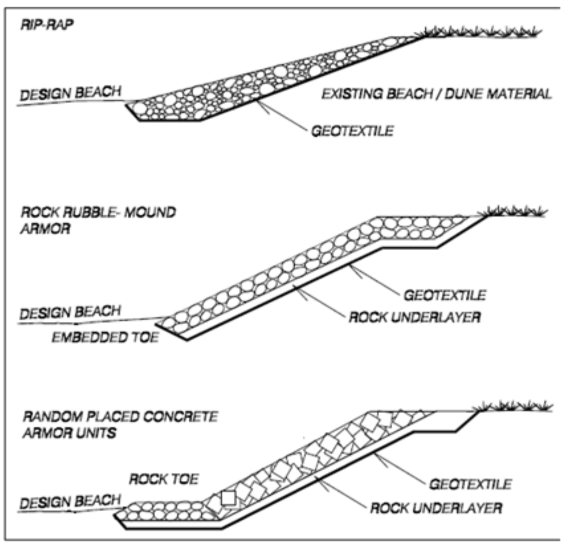


Figure VI-2-3. Examples of sloping front rubble-mound seawall/revetment structures

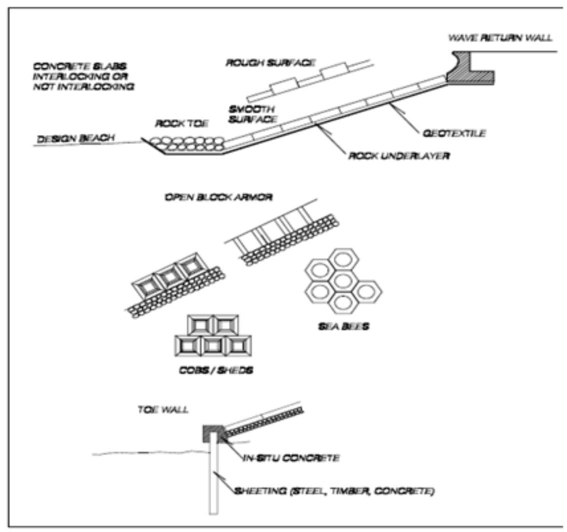


Figure VI-2-4. Examples of sloping-front seawalls/revetments with pattern-place concrete armor units

Figure 15-35. Seawall and Revetment Guidance from Coastal Engineering Manual (USACE, 2002)

Although there is always some risk associated with the design of coastal roadways, significant risks are those classified as having a probability of catastrophic failure with loss of life, or when a road is the only vehicular egress route available to a community. Roadways designated as evacuation routes or sole egress should be designed to a higher standard; for example, applying a higher freeboard than AASHTO requires would allow efficient and effective inland evacuation. For instance, it is common practice to design the roadway elevation of an evacuation corridor to be at or above a 100-year return period design elevation. Similarly, the evacuation road corridor may also be designed to survive overwashing during the storm event to minimize road damage and thus allow post-storm emergency first responders and supplies to quickly re-enter an affected area. Evacuation corridors are also commonly designed with additional capacity (including options for contraflow) to remove bottlenecks, allowing safe transportation of people and goods in times of excessive demand on the roadways (Figure 15-36).

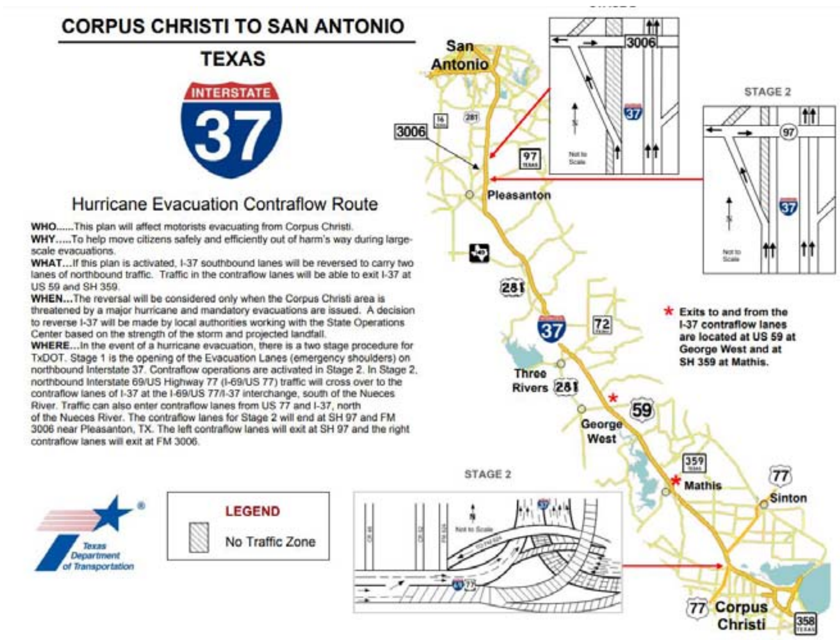


Figure 15-36. I-37 Contraflow Plan Corpus Christi.

Coastal Bridge Design Considerations

Hydraulic bridge design typically starts with an assessment of hydraulic capacity and evaluation of design elevation for the low chord. Other design considerations include bridge length, span arrangement, abutment location, and pier arrangement. The process for creating a design elevation is discussed in Section 4. Once the design elevation is determined, the proposed bridge design can be evaluated for environmental impacts, changes in nearshore processes, and can be structurally designed to withstand the various coastal loads. Hydraulic considerations vary between riverine and coastal scenarios, with many designs requiring consideration of both, or even cumulative effects, for proper assessments.

One type of hard coastal structures listed in the previous table and described in detail in the USACE CEM is pile structures. The most common pile structures in coastal engineering are bridge piers. Pile structures are designed based on the bearing capacity and settlement characteristics of the seabed. With coastal bridges, the substructure and how it interacts with the superstructure and deck can be quite complex. In addition to the bridge deck and piers carrying loads from traffic, the pile structures of the piers are exposed to loads from waves and currents that can result in scouring. Where scour analyses or geotechnical reports indicate potential instability at pile structures, scour protection can be placed. Scour protection often consists of rock bed on a stone or geotextile filter; however, specialty designed concrete block and mattress systems are also used. Scour protection was discussed in Section 5 with regards to methods to determine scour depth, along with recommended resources for determining countermeasures. In addition, as mentioned in Section 5, both FHWA HEC-25 and the USACE CEM discuss coastal scour and scour protection.

Many variables can be important in determining forces on piles subjected to wave action. Besides scour potential, wave and current load forces on the piles should be reviewed, if applicable, for structural instability. Although the analysis can be quite complex (refer to HEC-25 and the CEM for guidance), there are some factors that can be taken in to account during design that help reduce impacts. Some factors the designer can control include: pile diameter, pile shape, pile roughness, and pile orientation to the flow field. Factors that impact pile wave forces but are not controllable by the designer include: wave height, water depth, and wave period/wavelength. Figure 15-37 below shows the wave forces on a vertical cylinder.

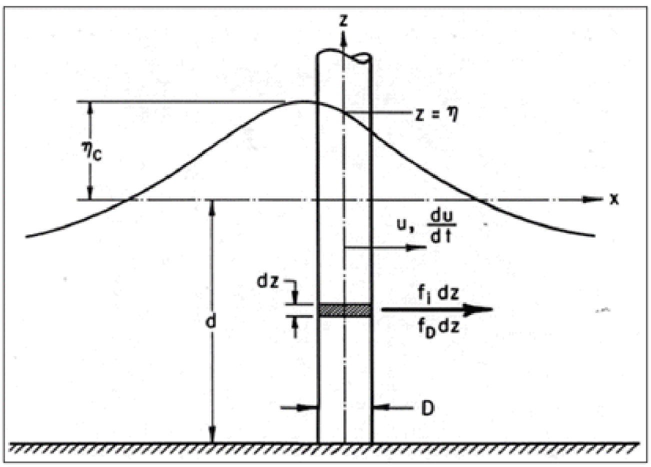


Figure VI-5-125. Definition sketch of wave forces on a vertical cylinder

Figure 15-37. Coastal Engineering Manual Vertical Wave Forces on a Vertical Cylinder (USACE, 2002)

When possible, or when determined as part of a project scope, the site-specific wave forces on the bridge pier design should be evaluated. When this is not possible or other standards are used for pier construction, there are recommendations that can help minimize wave forces. Rounded piles have beneficial structural properties and have less resistance to flow than rectangular piles. Spacing of piles along a bent can also impact the flow field. For multiple columns skewed to the flow direction, the scour depth can be significantly affected by the spacing between the columns. When possible, rounded, well-spaced piles aligned to the flow field can reduce risk of potential scour and limit the bridge backwater.

Additional Coastal Design Considerations

Additional design considerations include geotechnical, multimodal traffic, utilities, environmental, community access, and design aesthetics.

When designing a roadway, a geotechnical investigation is required to evaluate the anticipated pavement design and to consider potential failure mechanisms from nearshore processes, like overwashing and wave action. In both bridge and roadway design, the interaction between coastal infrastructure and underlying soil is a critical aspect of project performance. In addition, a bridge scour analysis could not be conducted without this investigation since the channel bed particle size and behavior is at the core of the scour equations.

Environmental regulations and identified environmental concerns should be reviewed for the project site prior to design. Addressing environmental concerns would involve a review of potential impacts that may result in ecological changes, even those changes that may be beneficial. Permits, consultations, and/or coordination with resource and regulatory agencies may be required for potential impacts to the water itself, navigability of the waterway, state and federally protected species, archeological and historic resources, and many other environmental resources. Permits, consultations, and coordination with regulatory and resource protection agencies can require increased environmental review of anticipated impacts from a proposed design and can be expected to increase the cost and timeframe required to deliver a project. Working closely with the Environmental Affairs Division and District environmental staff to identify environmental constraints and permitting pathways early in design will ultimately benefit project delivery.

When planning for new or improved infrastructure in a coastal area, especially a bridge, the volume of traffic is often a concern. Also, a potential concern is the ability for traffic, particularly aquatic vessel traffic, to pass under the bridge with sufficient vertical clearance in areas like the Gulf Intracoastal Waterway, which are travel ways for very large barges. There may also be existing or future utilities that need to be considered in the allowable vertical clearance for roadway and bridge traffic, in bridge design loads, or in placement of piers to avoid subsurface conflicts.

When designing new or improved coastal infrastructure, the existing condition should be evaluated for community needs. For instance, if a new or improved roadway is being designed and a new revetment is needed along the seaward side, the revetment may impact recreation or community access to the shoreline. Areas around coastlines are very often used for fishing, kayaking, swimming, and other activities. An evaluation should be performed to determine if the proposed infrastructure will impact the current community use of the coastline or region.

Proposed coastal infrastructure projects could have potentially significant impacts related to effects on a scenic vista or degradation of the existing visual character of a project area. Hard coastal structures are beneficial for erosion control but can often create aesthetic or shoreline access concerns for the community. Community engagement and feedback on the proposed design should be considered to ensure infrastructure projects can meet both design criteria and local stakeholder needs.

Living Shorelines

Transportation systems within the coastal zone are subject to highly dynamic systems and are susceptible to changing water levels, storm surge, currents, and wave attack. Because these forces are often very large and varied, identifying design techniques that can mitigate and adapt to meet future risks is valuable to extending project life cycles. Living shoreline techniques involve combining ecologically-based mitigation techniques, such as coastal vegetation, with hardened solutions, such as breakwaters or revetments, to create strong, but also more adaptable systems. Living shorelines can provide long-term risk mitigation due to their more dynamic nature, as they incorporate natural processes, such as sedimentation and vegetation growth, that can rise or adapt—to some extent--with long-term changes such as relative sea level rise.

Incorporating local natural elements, particularly vegetation, can lead to a more resilient transportation structure that includes nature-based buffers between the transportation system and the dynamic forces of coastal waters. In addition to coastal vegetation, these designs can include sediment capture or shoreline stabilization features along beaches or other shoreline systems. Such natural elements are more dynamic and adaptable to nearshore processes, which can be beneficial in adapting to changing shoreline conditions and water levels. For projects located near shorelines, it is recommended to evaluate living shoreline techniques for cost effective applicability to promote project longevity and risk mitigation over traditional hard armoring techniques. Examples of shoreline stabilization relevant to living shoreline techniques are shown in Figure 15-38 below. Marsh grasses and nearshore man-made oyster reefs submerged breakwaters, much like the example below, have been planted and constructed along Broadway Street in Little Bay, providing roadway infrastructure in Aransas County with a more resilient long-term solution to erosive coastal forces.

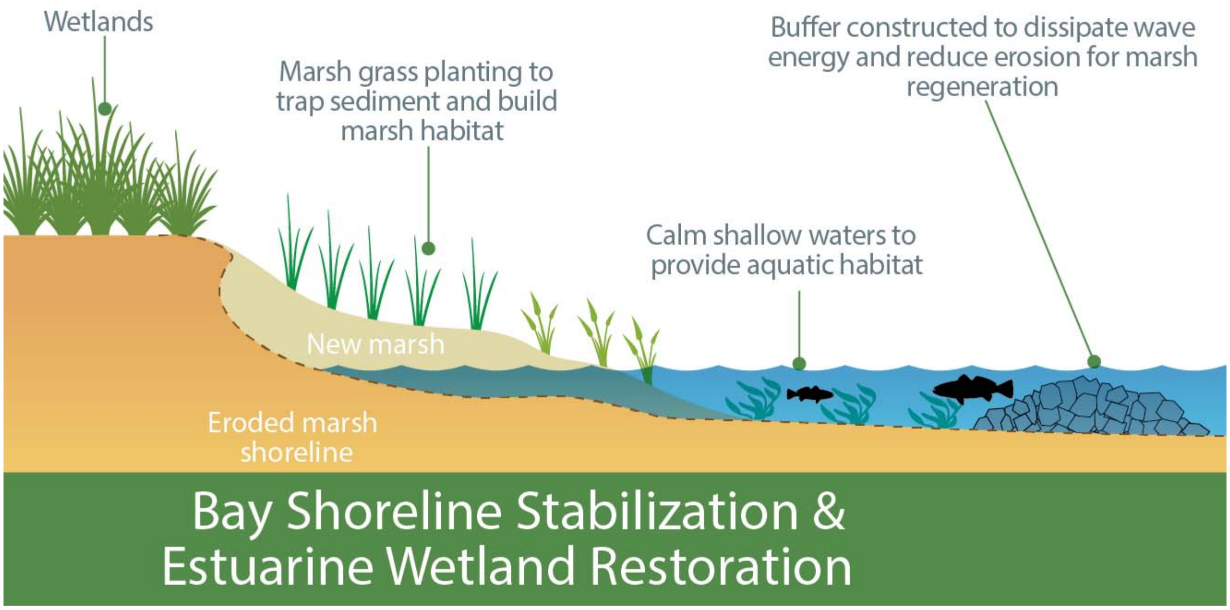


Figure 15-38. Living Shoreline Example Section.

Additional living shoreline references include NOAA Habitat Blueprint and the Texas Coastal Resiliency Master Plan.

Government Policies and Regulations Regarding Coastal Projects

Refer to Chapter 2 for general state and federal regulations and policies. Below are plans, policies, and regulations that apply to most projects occurring in coastal environments.

Federal

- **USACE.** The USACE issues permits and authorizations for discharges of dredged or fill material into waters of the U.S., which includes a variety of waters and wetlands. Typical “fill” impacts such as shoreline stabilization, drainage outfalls, bridge piers/bents, and other structures in jurisdictional waters require permitting, and most likely compensatory mitigation. In general, the greater the impacts, the longer, more complex, and more expensive the path may be to obtain USACE approval before construction can be initiated. USACE regulations require consideration of direct and indirect physical effects of future sea level change across the project life cycle for managing, planning, engineering, design, constructing, and maintaining USACE projects located in and near the coastline.

  The USACE also must grant permission for the alteration, occupation, or use of a USACE civil works project, for example to build a bridge or road that crosses a USACE-owned property or easement (levees, dams, reservoirs, flood mitigation areas, sea walls, bulkheads, etc.). The USACE supports levee certification decisions for the NFIP administered by FEMA. If USACE permission is required, it must be granted before the USACE can issue any other type of permit or authorization.

TxDOT maintains a funding agreement with the USACE Galveston District for expedited review and technical assistance. Contact the Environmental Affairs Division and District environmental staff to assist with USACE permits and authorizations.

- **USACE and U.S. Coast Guard.** Crossings of rivers, estuaries, and bays considered to be navigable waterways are regulated by the U.S. Coast Guard (USCG) and the USACE. For projects involving construction of bridges over navigable waterways, a permit or authorization may be required from the USCG and/or the USACE.

  The USCG reviews and permits bridges and causeways over navigable waters, authorizes associated lighting and signals, and imposes conditions relating to construction, maintenance, and operation in the interest of public navigation. The USCG requires plans suitable for general public use in addition to standard TxDOT plans. Refer to the *USGC Bridge Permit Application Guide* for permitting procedures and guidance. Environmental Affairs Division and District environmental staff can assist with USCG permits and authorizations, including those involving the sequential relationship with USACE authorizations and permits. Early coordination with the USCG is recommended; however, the final permit application may not be submitted until final design (i.e., 90 percent plans), and permitting may take six months or more.

- **U.S. Fish and Wildlife Service (USFWS) and National Marine Fisheries Service (NMFS).** Coastal highways traverse bays, estuaries, coastal wetlands, and beaches, which support a diversity of habitats for a variety of threatened or endangered plant and animal species. Each project must be assessed for its potential to impact federally protected species and essential fish habitat. Depending on anticipated impacts and the species involved, coordination or consultation with the USFWS and the NMFS may be required to permit or authorize the project. Contact the Environmental Affairs Division and District environmental staff to assist with protected species evaluations. All consultations with the USFWS and the NMFS are conducted through the Environmental Affairs Division.

  The USFWS also administers the Coastal Barrier Resources Act (CBRA), which discourages development in coastal areas that are vulnerable to hurricane damage and that host valuable natural resources by designating certain undeveloped coastal areas ineligible for most new federal expenditures and financial assistance. If the project includes any type or amount of federal funding and is located within designated areas, the project must be reviewed to determine if project activities qualify for an exception and if the project is consistent with the purposes of the CBRA. If the project qualifies for an exception, the determination must be approved though consultation with the USFWS. If the project does not qualify for an exemption, project activities are not eligible for federal funding under the CBRA. All CBRA consultations with the USFWS would be conducted through the Environmental Affairs Division.

State

- **Texas Parks & Wildlife Department (TPWD).** TxDOT and TPWD have a Memorandum of Understanding (MOU) that outlines responsibilities for the protection of the natural environment. Under the MOU, TxDOT is required to assess projects for their potential to impact state-protected species and other protected or imperiled resources, and to coordinate certain types of projects. Assessments and coordination with TPWD are conducted and managed by District environmental staff with assistance from the Environmental Affairs Division. Coordination is conducted under the MOU with transportation liaisons in the TPWD Wildlife Habitat Assessment Program. Coordination with TPWD should occur as early as possible to capture and implement recommendations to minimize habitat impacts.
- **Texas General Land Office (GLO), Texas Coastal Resiliency Master Plan.** The State of Texas has not developed statewide regulations related to sea level rise and coastal hazards. In March 2019, the GLO on behalf of the State prepared a Texas Coastal Resiliency Master Plan to reduce the vulnerability of the state's coastal infrastructure and resources. The plan provides resiliency goals for the state and a recommended set of prioritized projects to support enhanced protection for the state's coastline. During project planning, the Texas Coastal Resiliency Master Plan should be consulted to ensure the proposed roadway project is in alignment with overall state resiliency goals.

  The GLO also manages all state submerged lands (tidal and non-tidal). TxDOT is required to acquire easements from the GLO for all transportation projects that will acquire state-owned submerged lands. The identification of submerged lands and required easements is typically performed during project planning and is addressed during ROW acquisition.

Section 8 — References

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