Hydraulic Design Manual

Texas
Department
of Transportation

Revised March 2009

Manual Notice 2009-1

From: Mark A. Marek, P.E.

Manual: Hydraulic Design Manual

Effective Date: March 01, 2009

Purpose

This revision is intended to update the *Hydraulic Design Manual*, specifically to include TxDOT's Nondiscrimination Policy.

Contact

Address questions concerning the information contained in this manual to the Roadway Design Section in the Design Division.

Copyright Notice

Archives

Past manual notices are available in a PDF archive.

Table of Contents

Chapter 9 — Bridges

Chapter 10 — Storm Drains

Chapter 1
Manual Introduction

Contents:

Section 1
About This Manual

Non-discrimination

TxDOT policy is to ensure that no person in the United States of America shall on the grounds of race, color, national origin, sex, age or disability be excluded from the participation in, be denied the benefits of or otherwise be subjected to discrimination under any of our programs or activities.

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.

Manual Revision History

Version	Publication Date	Summary of Changes
2001-1	October 2001	New manual; replaced 1985 Bridge Division Hydraulic Manual.
2002-1	April 2002	Revision adding English measurement units, deleting unnecessary section on wave runup analysis, streamlining organization, and correcting minor errors.
2002-2	November 2002	Revision updating equations in Chapters 4, 5, and 8; providing new equations on pavement drainage ponding and curb inlets in sag configurations; updating the procedure for on-grade slotted drain inlets, and correcting minor errors.

Conventions and Assumptions

This manual provides information, where possible, in both English standard measurement units and in metric measurement units.

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 – Overview of the material covered in this manual.

- Chapter 2: Policy and Guidelines – Considerations regarding highway drainage design for TxDOT.
- Chapter 3: Documentation – Formal documentation required by highway drainage analysis and design.
- Chapter 4: Data Collection, Evaluation, and Documentation – Data sources and data management during highway drainage analysis and design.
- Chapter 5: Hydrology – Methods used by TxDOT for discharge determination or estimation, guidelines and examples for development of runoff hydrographs, and discussion of design frequency requirements and considerations.
- Chapter 6: Hydraulic Principles – Basic hydraulic concepts and equations for open channels, culverts, and storm drains.
- Chapter 7: Channels – Overview of channel design, methods, and guidelines governing location and need to subdivide cross sections.
- Chapter 8: Culverts – Discussion of culvert analysis and design procedures and concerns, equations for various culvert operating conditions, and appurtenances such as improved inlets and erosion velocity protection and control devices.
- Chapter 9: Bridges – Overview of stream-crossing design, bridge hydraulic considerations, bridge scour and channel degradation concerns, and design by risk assessment.
- Chapter 10: Storm Drains – Discussion of storm drain planning, components, calculation tools, and other guidelines.
- Chapter 11: Pump Stations – Discussion of the function of pump stations and flood routing approach.
- Chapter 12: Reservoirs – Overview of factors affecting highways either crossing or bordering reservoirs.
- Chapter 13: Storm Water Management – Guidance on storm water management practices, including erosion and sediment control, maintenance of erosion control measures, storm water runoff collection and disposal, and storm water pollution abatement.
- Chapter 14: Conduit Strength and Durability – Information on conduit durability, estimating service life, classes of bedding for reinforced concrete, RCP strength specifications, and jacked pipes.

Feedback

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

Section 2
Introduction to Hydraulic Design

Description

Hydraulic facilities include the following:

- open channels
- bridges
- culverts
- storm drains
- pump stations
- storm water quantity and quality control systems.

The hydraulic design or analysis of highway drainage facilities usually involves a general procedure that is essentially the same for each case. 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, 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
Policy and Guidelines

Contents:

Section 1

TxDOT Drainage-Related Policy

General Policy

This chapter uses “policy” as a general term. Federal and state regulations and rules have the force of law, and compliance is not at the discretion of TxDOT.

FHWA Policy

FHWA sets forth policy and guidance in the Federal Aid Policy Guide (FAPG). See the Project Development Policy Manual for more information on this guide. The primary policy for drainage is 23 CFR §650. The Hydraulic links page provides links to the FAPG relating to the location and hydraulic design of encroachments on floodplains and erosion and sediment control on highway construction projects.

Texas Administrative Code on Drainage

The five drainage-related items appearing in the Texas Administrative Code (TAC) -- 43 TAC §15.54(e)(1) through 45 TAC §15.54(e)(5) -- as local participation rules are described below:

- ◆ TxDOT is responsible for constructing drainage systems within state right of way.

  “In general, it shall be the duty and responsibility of the department to construct, at its expense, a drainage system within state highway right of way, including outfalls, to accommodate the storm water that originates within and reaches state highway right of way from naturally contributing drainage areas.”

NOTE: Outfalls are integral parts of highway drainage facility design. TxDOT is responsible for ensuring that natural runoff from a naturally contributing drainage area is transferred to the receiving waters without incurring significant impact to the receiving waters or adjacent property. The outfall should be extensive enough to create this condition, and adequate right-of-way should be acquired to ensure continued satisfactory operation of the outfall. This policy is not intended to preclude cost sharing with local agencies, nor is it intended to absolve local agencies and land developers of responsibility for increased runoff impacts due to development.

- ◆ TxDOT is responsible for adjusting or relocating any existing drainage channel when necessary.

  “Where a drainage channel, man-made, natural, or a combination of both, is in existence prior to the acquisition of highway right of way, including right of way for widening the highway, it shall be the duty and responsibility of the state to provide for the construction of the necessary structures and/or channels to adjust or relocate the existing drainage channel in such a manner that the operation of the drainage channel will not be injured. The construction expense required shall be considered a construction item. The acquisition of any land required to accomplish this work shall be considered a right of way item, with cost participation to be in accordance with §15.55 of this title (relating to construction cost participation)."

- TxDOT is responsible for adjusting the structure and channels to accommodate any approved drainage plan.

  "Where an existing highway crosses an existing drainage channel, and a political unit or subdivision with statutory responsibility for drainage develops a drainage channel to improve its operation, both upstream and downstream from the highway, and after the state establishes that the drainage plan is logical and beneficial to the state highway system, and there is no storm water being diverted to the highway location from an area that, prior to the drainage plan, did not contribute to the channel upstream of the highway, and after construction on the drainage channel has begun or there is sufficient evidence to insure that the drainage plan will be implemented, the department, at its expense, shall adjust the structure and/or channels within the existing highway right of way as necessary to accommodate the approved drainage plan."

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

- Others wishing to cross the highway where there is no drainage crossing must obtain approval from TxDOT and provide construction and maintenance costs.

  "Where a state highway is in existence, and there is a desire of others to cross the existing highway at a place where there is not an existing crossing for drainage, then those desiring to cross the highway must provide for the entire cost of the construction and maintenance of the facility that will serve their purpose while at the same time adequately serving the highway traffic. The design, construction, operation, and maintenance procedures for the facility within state highway right of way must be acceptable to the department."

- The local government wanting to join in diverting drainage must pay for collecting and carrying diverted water and contribute to its share of the system cost.

  "In the event the local government involved expresses a desire to join the department in the drainage system in order to divert drainage into the system, the local government shall pay for the entire cost of collecting and carrying the diverted water to the state's system and shall contribute its proportional share of the cost of the system and outfall based on the cubic meters per second of additional water diverted to it when compared to the total cubic meters per second of water to be carried by the system. The local government requesting the drainage diversion shall indemnify the state against or otherwise acknowledge its responsibility for damages or claims for damages resulting from such diversion."

Texas Administrative Code on Reservoirs

Directions on TxDOT facilities relating to reservoir construction are based on 43 TAC §15.54(f) through 45 TAC §15.54(g).

> "Where existing highways and roads provide a satisfactory traffic facility in the opinion of TxDOT 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 in accordance with the current design standards of the department, based upon the road classification and traffic needs."

> "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."

> "In soil conservation and flood control projects involving the construction of flood retarding structures where a highway or road operated by TxDOT will be inundated at less than calculated 50-year frequencies by the construction of a floodwater retarding structure, it will be expected that the NRCS or one of its cooperating agencies will provide funds as necessary to raise or relocate the road above the water surface elevation that might be expected at 50-year intervals."

> "In those cases where a highway or road operated by TxDOT will not be inundated by floods of less than 50-year calculated frequency, it will be the purpose of the department to underwrite this hazard for the general welfare of the State and continue to operate the road at its existing elevation until such time as interruption and inconvenience to highway travel may necessitate raising the grade."

Texas Administrative Code on Irrigation Facilities

The TAC establishes when construction or relocation is a highway right-of-way item and whether TxDOT is or is not responsible for costs. The following information is taken from 43 TAC §15.54(g)(1) and 43 TAC §15.54(e)(2).

1. Where an irrigation facility is in existence prior to the acquisition of highway right of way, including right of way for widening, and the highway project will interfere with such a facility, the following rules shall govern:
   a. If, at the place of interference the irrigation facility consists primarily of an irrigation canal that crosses the entire width of the proposed right of way, this shall be considered a crossing, and it shall be the duty and responsibility of the department to construct and maintain an adequate structure and to make the necessary adjustments or relocation of minor laterals and pumps, etc. associated with the crossing, in such a manner that the operation of the

irrigation facility will not be injured therefrom. The construction work at a crossing will be borne by the department. The acquisition of any land required to accomplish the adjustments and/or relocation shall be a right-of-way consideration.

b. Any irrigation facility encountered that does not cross the right of way and consists primarily of a longitudinal canal and/or associated irrigation appurtenances such as pumps, gates, etc., that must be removed and relocated shall be considered as a right-of-way item.

c. In those cases where both crossing and longitudinal adjustments or relocation of irrigation facilities are encountered, each segment shall be classified and placed in one of the above two categories.

2. Where a highway is in existence and there is a desire of others to cross the existing highway with an irrigation facility at a highway point where there is not an existing crossing facility, then those desiring to cross the highway must provide for the entire cost of the construction and maintenance of the irrigation facility that will serve their purpose while at the same time adequately serve the highway traffic. The design, construction, operation, and maintenance procedures for the facility within highway right-of-way must be acceptable to the department.

Section 2

Drainage Complaint Guidelines and Procedure

Complaints

The Bridge Division processes drainage complaints received by TxDOT. Usually the Bridge Division asks the District for an investigation and report.

TxDOT deals with drainage complaints promptly and in an unbiased manner. When the District is asked to investigate a complaint, it should determine the facts, analyze the facts, and make conclusions and recommendations, as follows:

1. Show on a map the location of the problem on which the complaint is based. Clearly determine the basis for the complaint, including extent of flooding, complainer's opinion of what caused the flooding, description of alleged damages, and dates and times, and duration of flooding. Relate history of other grievances. Briefly relate history of any other grievances expressed prior to the claim presently under investigation. Obtain pertinent dates. Identify approximate dates when those claiming damages acquired the damaged property or improvements. Collect the specific flood event facts involved. Document the facts regarding highway crossing involved. Consider possible effects by others, including the utilities such as pipelines, other highways or streets, railroads, dams, and any significant man-made changes to the stream or watershed that might affect the flooding.
2. Analyze the facts and decide what action to take to relieve the problem, regardless of who has responsibility for the remedy.
3. Make conclusions and recommendations. Describe the contributing factors leading to the alleged flood damage and specify feasible remedies.

Specific Flood Event Facts

When collecting specific flood event facts, include the following:

- Rainfall data, such as dates, amounts, time periods, and locations of gages.
- Observed high-water information at or in the vicinity of the claim. Locate high-water marks on a map and specify datum. Always try to obtain high-water marks both upstream and downstream of the highway and the time the elevations occurred.
- Duration of flooding at site of alleged damage.
- Direction of flood flow at damage site.
- Description of the stream condition before, after, and during flood(s), including density of vegetation, presence of debris and drift jams, flow conditions, and extent of erosion.
- The flood history at the site.

- The depth and velocity of flow over the road, if any.
- Narratives of any eyewitnesses to the flooding.
- Facts about the flood(s) from sources outside TxDOT, such as newspaper accounts, witnesses, measurements by other agencies (e.g., USGS, Corp of Engineers, NRCS) and individuals, maps, and Weather Bureau rainfall records.

Facts Regarding Highway Crossing Involved

When collecting facts about the highway crossing involved, include the following:

- Profile of highway across stream valley.
- Date of original highway construction.
- Dates of all subsequent alterations to the highway.
- Description of alterations.
- Description of what existed prior to the highway, such as a county road, a city street, an abandoned railroad embankment, etc.
- Description of the drainage facilities and drainage patterns prior to the highway.
- Description of existing drainage facilities.
- Original drainage design criteria or capacity and frequency of existing facility based upon current criteria.

Section 3

Authority over Waters of the United States

Introduction

A number of federal agencies have specific authority over United States waters. In addition to the following paragraphs, refer to the Interagency Coordination section of the *Environmental Procedures in Project Development Manual* for more information on their constitutional power and policy. Refer to the Environmental Affairs Division for details.

Constitutional Power

The Congress of the United States has constitutional power to regulate "commerce among the several states." Part of that power is the right to legislate on matters concerning the instruments of interstate commerce, such as navigable waters. The definition of navigable waters expands and contracts as required to adequately carry out the Federal purpose. The result is that Congress can properly assert regulatory authority over at least some aspects of waterways that are not in themselves subject to navigation.

Four federal agencies carry out existing federal regulations relating to navigable water

US Coast Guard (USCG). Under 23 CFR §650.807(a), the USCG is responsible for determining whether a USCG permit is required for the improvement or construction of a bridge over navigable waters, except for the exemption exercised by FHWA described below, and to approve bridge location, alignment, and appropriate navigational clearances in all bridge permit applications.

US Army Corps of Engineers (USACE). USACE has regulatory authority over the construction of dams, dikes, or other obstructions (that are not bridges and causeways) under Section 9 (33 U.S.C. 401). USACE also has authority to regulate Section 10 of the Rivers and Harbors Act of 1899 (33 U.S.C. 403), which prohibits the alteration or obstruction of any navigable waterway with the excavation or deposition of fill material in such waterway. Section 11 of the Rivers and Harbors Act of 1899 (33 U.S.C. 404) authorizes the Secretary of the Army to establish harbor lines. Work channelward of those lines requires separate approval of the Secretary of the Army, and work shoreward requires Section 10 permits.

Section 404 of the Clean Water Act (33 U.S.C. 1344), prohibits the unauthorized discharge of dredged or fill material into waters of the United States, including navigable waters. Such discharges require a permit. "Dredged material" is any material that is excavated or dredged from waters of the United States. "Discharges of fill material" refers to the addition of rock, sand, dirt, concrete, or other material into the waters of the United States incidental to any activity. USACE has granted a Nationwide General Permit for 43 categories of certain minor activities involving discharge of fill material.

Refer to the Environmental Affairs Division regarding relevant permit requirements and procedures.

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

Environmental Protection Agency (EPA). Under Section 404 (c), Clean Water Act (33 U.S.C. 1344), EPA is authorized to prohibit the use of any area as a disposal site if the discharge of materials at the site will have an unacceptable adverse effect on municipal water supplies, shellfish beds and fishery areas, wildlife, or recreational areas. EPA is also authorized under Section 402 of the Clean Water Act (33 U.S.C. 1342) to administer and issue a National Pollutant Discharge Elimination System (NPDES) permit for point source discharges, provided prescribed conditions are met.

NPDES is the regulatory permit program that controls the quality of treated sewage discharge from sewage treatment plants as established in 40 CFR §125 pursuant to the Clean Water Act, 33 U.S.C. 1342. In compliance with this regulation, TxDOT will need a permit for sewage treatment facilities for highway safety rest areas.

Permits are also required for non-point source pollutants associated with industrial activities and also Municipal Separate Storm Sewer Systems (MS4). Refer to the TxDOT publication "Storm Water Management Guidelines for Construction Activities" for requirements for conformance industrial activity permits. Refer to the Environmental Affairs Division for details regarding the status and provisions of MS4 permits for municipalities.

Section 4
Required Hydraulic Analysis

Function and Scope of Hydraulic Analysis

Flood frequency for design and checks must be considered for a new location, replacement, or modification of a facility. Hydrologic and hydraulic analyses are required to determine, justify, and document the need for and size of a hydraulic facility. Each District must maintain complete hydrologic and hydraulic design data for all waterway crossings. The same hydraulic analysis is required for new locations, proposed facility replacements, and widening of existing facilities.

The intent of a design flood is to establish conditions under which the highway facility will provide uninterrupted service with minimal damage to the highway. The design flood must not overtop the highway.

A check flood must be applied on proposed highway or stream crossing facilities to determine whether a proposed crossing will cause significant damage to the highway or to any other property, in excess of damage that is likely to occur without the proposed facility. For TxDOT design, the 100-year event is the primary check condition. The check flood may be conveyed both over the highway and through the hydraulic facilities. An additional check flood is the incipient highway overtopping condition.

Analysis should include a comparison of existing conditions with proposed conditions for interim and estimated future watershed characteristics. Its extent should correspond with the importance of the highway and its environment. The goal should be to achieve an adequate balance between incurred capital costs and potential risks.

Usually watershed characteristics will have changed since the placement of the existing facilities. Most often, runoff rates are higher due to increased impervious cover and more efficient drainage. In such cases, larger facilities may be needed to replace the existing ones.

Occasionally, flood control systems may have been constructed that significantly reduce runoff rates at the highway site. In such instances, verify the permanence and effect of the flood control facility, and consider the possibility of designing smaller hydraulic facilities than those to be replaced.

Widening Existing Facilities

If available, valid, hydraulic data exist, simplify the process. Changed watershed conditions or outdated hydrologic and hydraulic methods warrant reappraisal using updated methods and field information.

Expend additional effort to show any impact of the widening and to justify why replacement is not necessary or practicable for all bridge class structures and culverts. However, a hydraulic adequacy estimate based on past performance may be reasonable for culverts on existing rural locations where all of the following are true:

- Minor modifications only are planned (e.g., safety-end treatment and short extensions).
- Traffic volumes are low.
- Surrounding properties are not sensitive to damage due to backwater or high velocities.
- There is no adjustment of the roadway profile or addition of a roadway safety barrier.
- Sufficient information on past performance is available.

The determination of hydraulic adequacy refers to an estimate of design frequency based on a review and appraisal of historical high water, overtopping frequency, duration and depth, and maintenance history.

In addition to bridges and culverts, roadway widening often involves the relocation of inlets and extension of storm drain conduit. Design storm drain inlets and conduit systems in accordance with practices outlined in Chapter 10.

When an existing structure is discovered to be inadequate or oversized, either adjust the size of the facility as appropriate or assign a new capacity rating with a corresponding increase or decrease in the hydraulic standards that were previously established.

Because highway rehabilitation, modification, or maintenance work is not intended to include physical adjustment to hydraulic facilities, it does not preclude the need for considering the hydraulic-related impact. The following instances should include verification of continued adequate hydraulic performance:

- Roadway surface overlays or regrading projects that reduce the effective opening area or allowable ponding depth of storm drain inlets or reduce gutter capacities.
- Roadway widenings or addition of roadway barrier resulting in a higher overtopping elevation where the 100-year flood would previously have overtopped the roadway. If such modifications are made in a designated floodway, coordination with FEMA is required, as discussed in Section 5.
- Replacement of inlet grates with lower effective openings than existing.
- Removal of any hydraulic feature including flumes and energy dissipaters.

Section 5
FEMA Policy and Procedure

National Flood Insurance Program

The amended National Flood Insurance Act of 1968 (42 U.S.C. 4001 et seq.) established the National Flood Insurance Program (NFIP), which requires communities--whether city, county, or state--to adopt adequate land use and control measures to qualify for flood insurance in riverine flood-prone areas.

When the Administrator of the Federal Insurance Administration has identified the flood-prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided unless it is demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of a similar nature, will not increase the water surface elevation of the 100-year flood more than 1 ft. (0.3 m) at any point within the community.

After the floodplain area has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community may designate a floodway that will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 ft. (0.3 m) at any point. Also, the community must prohibit, within the designated floodway, fill, encroachments, and new construction and substantial improvements of existing structures that would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.

The participating cities or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance should require that development in the designated floodplain be consistent with the intent, standards and criteria set by the NFIP. Failure on their behalf to enforce basic requirements can result in losing their status in the program.

The highway designer needs to be familiar with FEMA NFIP requirements because meeting them may either control the design of a facility within a floodplain or, when encroachments (any physical object placed in a floodplain that hinders flow) are proposed, necessitate considerable analysis, coordination, and expense to acquire FEMA approval of the project. Incorporate considerations concerning FEMA rules and procedures early in the project planning stages. (See Task 2200and Task 5080 of the *Project Development Process Manual* for more information.)

Determining the status of a community’s participation in NFIP and reviewing applicable NFIP maps and ordinances are essential first steps in conducting location hydraulic studies and preparing environmental documents. Information of community participation in NFIP is provided in the

National Flood Insurance Program Status of Participating Counties, published semi-annually for each state, and available through Federal Emergency Management Agency (FEMA) headquarters and the Texas Natural Resources and Conservation Commission (TNRCC).

NFIP Maps

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

- Flood Hazard Boundary Map (FHBM) -- An FHBM does not generally originate from a detailed hydraulic study, and, therefore, the floodplain boundaries shown are approximate.
- Flood Boundary and Floodway Map (FBFM) -- An FBFM generally originates from a detailed hydraulic study. These hydraulic data are available through the FEMA regional office and should provide reasonably accurate information. This study is normally in the form of computer input data records or hand data for calculating water surface profiles.
- Flood Insurance Rate Map (FIRM) -- The FIRM identifies base flood elevations and rate zones for flood insurance and is generally produced at the same time as the FBFM using the same hydraulic model.

Flood Insurance Study

A Flood Insurance Study (FIS) documents methods and results of a detailed hydraulic study. The report includes the following information:

- name of community
- hydrologic analysis methods
- hydraulic analysis methods
- floodway data including areas, widths, average velocities, base flood elevations, and regulatory elevations
- water surface profile plots

NFIP Participation Phases

A community can be in the emergency program or the regular program, in the process of converting from the emergency program to the regular program, or not participating in NFIP. The emergency program is intended to provide a “first layer” amount of insurance on an emergency basis on all insurable structures before a risk study can be performed. Approximate flood boundaries are shown on a FHBM. The regular program provides a “second layer” coverage, which is offered only after the Floodplain Administrator has completed a risk study for the community. (The Floodplain Administrator is the mayor, county judge, or delegate responsible for the administration and enforcement of the floodplain management ordinances of a community participating in the NFIP.) A detailed hydraulic study has usually been performed and the results published in the FIS report, FIRM, and FBFM.

Regulated Floodplain Components

Figure 2-1 illustrates the basic components of an FEMA-regulated floodplain. The floodplain is established by the base flood, which is the extent of inundation resulting from flood flow having a one percent exceedance probability in any given year (100-year flood). The floodplain is divided into a regulatory floodway (RFW) and floodway fringes. Another component of the regulated floodplain is differences in projects.

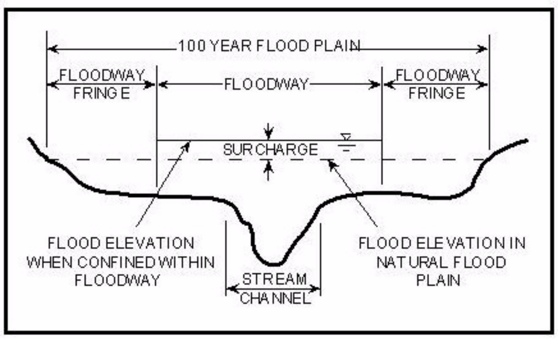

Figure 2-1. Basic Constituents of FEMA-NFIP-Regulated Floodplain

The regulatory floodway is the main stream channel and any floodplain areas that must be kept free of encroachment so that the base flood can be carried without a considerable increase in water surface elevations. The maximum increase above the base flood elevation (BFE) is usually 1 ft. (0.3 m). Existing insurable buildings, the potential for hazardous velocities, or other conditions may result in lower allowable increases.

The floodway fringe is the remaining area between the floodway and the floodplain boundary. Theoretically, the floodway fringe can be completely obstructed without increasing the water surface elevation of the base flood by more than 1 ft. (0.3 m) at any point.

Projects Requiring Coordination with FEMA

Several possible conditions may apply in a participating community and corresponding regulations apply to each, as shown in the "FEMA Requirements for Applicable Conditions" table below. You are responsible for determining the status of the waterway and taking the appropriate action.

FEMA Requirements for Applicable Conditions

Type of Map	WS Elev	RFW	Coastal Hazard Area	NFIP Section	Requirements
None	No	No	No	60.3(a)	◆ Permits to determine if flood prone
FHBM	No	No	No	60.3(b)	◆ Permits within Flood Hazard Areas ◆ Notify adjacent communities and FEMA before alteration or relocation of watercourse ◆ Assure capacity is maintained
FIRM	Yes	No	No	60.3(c)	◆ Permits within Flood Hazard Areas ◆ Notify adjacent communities and FEMA before alteration or relocation of watercourse ◆ Assure capacity is maintained ◆ No construction until RFW is designated unless WS will not increase over 1 ft (0.3 m) ◆ Amend FIRM when WS increases over 1 ft (0.3 m)
FIRM	Yes	Yes	No	60.3(d)	◆ Permits within Flood Hazard Areas ◆ Notify adjacent communities and FEMA before alteration or relocation of watercourse ◆ Assure capacity is maintained ◆ No construction until RFW is designated, unless WS won't increase over 1 ft (0.3 m) ◆ No encroachment within RFW unless WS will not increase over 1 ft (0.3 m) ◆ Amend FIRM and RFW when WS increases over 1 ft (0.3 m)
FIRM	Yes	Yes	Yes	60.3(e)	◆ Permits within Flood Hazard Areas ◆ Notify adjacent communities and FEMA before alteration or relocation of watercourse ◆ Assure capacity is maintained ◆ Amend FIRM and RFW when WS increases over 1 ft (0.3 m) ◆ No alteration of sand dunes or mangrove stands within coastal hazard areas that would increase potential flood damage
Note. FEMA criteria are designated in English units.					

TxDOT coordinates with FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances ordinarily requiring coordination with FEMA include the following:

- When a proposed crossing encroaches on a regulatory floodway and, as such, requires an amendment to the floodway map.
- When a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated and the maximum 1-ft. (0.3-m) increase in the base flood elevation would be exceeded.
- When a local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are underway.
- When a local community is participating in the emergency program and base FEMA flood elevation in the vicinity of insurable buildings is increased by more than 1 ft. (0.3 m). Where insurable buildings are not affected, simply notify FEMA of changes to base flood elevations as a result of highway construction.

In many situations, it is possible to design and construct cost-effective highways such that their components are excluded from the floodway. This is the simplest way to be consistent with the standards and should be the initial alternative evaluated. If a project element encroaches on the floodway but has a minor effect on the floodway water surface elevation (such as piers in the floodway) and hydraulic conditions can be improved so that no water surface elevation increase is reflected in the computer printout for the new conditions, then the project may normally be considered consistent with standards.

The draft Environmental Impact Statement or Environmental Assessment (EIS/EA) should indicate the NFIP status of affected communities, the encroachments anticipated, and the need for floodway or floodplain ordinance amendments. Coordination means furnishing to FEMA the draft EIS/EA and, upon selection of an alternative, furnishing to FEMA, through the community, a preliminary site plan and water surface elevation information and technical data in support of a floodway revision request as required. If a FEMA determination would influence the selection of an alternative, obtain a commitment from FEMA prior to the final Environmental Impact Statement (FEIS) or a finding of no significant impact (FONSI). Otherwise, this later coordination may be postponed until the design phase. Refer to the Environmental Affairs Division for more details.

Floodway Revisions and NFIP

Where it is not cost-effective to design a highway crossing to avoid encroachment on an established floodway, consider modifying the floodway itself. Often, the community is willing to accept an alternative floodway configuration to accommodate a proposed crossing, provided NFIP limitations on increases in the base flood elevation are not exceeded. This approach is useful where the highway crossing does not cause more than a 1-ft. (0.3-m) rise in the base flood elevation. In some cases, it may be possible to enlarge the floodway or otherwise increase conveyance in the floodway above and below the crossing in order to allow greater encroachment. Such planning is best accomplished when the floodway is first established. However, where the community is willing to amend an established floodway to support this option, the floodway may be revised.

The responsibility for demonstrating that an alternative floodway configuration meets NFIP requirements rests with the community. However, this responsibility may be borne by the agency proposing to construct the highway crossing. FEMA prefers that floodway revisions be based on the hydraulic model used to develop the currently effective floodway but updated to reflect existing encroachment conditions. This allows determining the increase in the base flood elevation caused by encroachments since the original floodway was established. You may then analyze alternate floodway configurations. Reference increases in base flood elevations to the profile obtained for existing conditions when the floodway was first established.

Allowable Floodway Encroachment

When it is inappropriate to design a highway crossing to avoid encroachment on the floodway and where the floodway cannot be modified to exclude the structure, FEMA will approve an alternate floodway with backwater in excess of the 1-ft. (0.3-m) maximum only when the following conditions have been met:

- A location hydraulic study has been performed in accordance with FHWA, “Location and Hydraulic Design of Encroachments on Floodplains” (23 CFR §650, Subpart A), and FHWA finds the encroachment is the only practicable alternative.
- TxDOT has made appropriate arrangements with affected property owners and the community to obtain flooding easements or otherwise compensate them for future flood losses due to the effects of backwater greater than 1 ft. (0.3 m).
- TxDOT has made appropriate arrangements to assure that the National Flood Insurance Program and Flood Insurance Fund will not incur any liability for additional future flood losses to existing structures that are insured under the program and grandfathered under the risk status existing prior to the construction of the structure.
- Prior to initiating construction, TxDOT provides FEMA with revised flood profiles, floodway and floodplain mapping, and background technical data necessary for FEMA to issue revised Flood Insurance Rate Maps and Flood Boundary and Floodway Maps for the affected area, upon completion of the structure.

For more information on floodplain encroachments, see the Federal Aid Policy Guide.

Floodplain with a Detailed Study (FIRM). In NFIP participating communities where a detailed flood insurance study has been performed but no regulatory floodway is designated, design the highway crossing to allow no more than a 1-ft. (0.3-m) increase in the base flood elevation based on technical data from the flood insurance study. Submit technical data supporting the increased flood elevation to the local community and, through them, to FEMA for their files.

Floodplain Indicated on a FHBM. In NFIP-participating communities where detailed flood insurance studies have not been performed, TxDOT must generate its own technical data to determine the base floodplain elevation and design encroachments in accordance with the Federal Aid Policy Guide. Base floodplain elevations shall be furnished to the community and coordination carried out with FEMA as outlined previously where the increase in base flood elevations in the vicinity of insurable buildings exceeds 1 ft. (0.3 m).

Unidentified Floodplains. Design encroachments outside of NFIP communities or NFIP-identified flood hazard areas in accordance with the FAPG and TxDOT guidance. (See FAPG.)

Replacing Existing Structures

If an existing structure is replaced in a floodplain of an NFIP-participating community, the replacement structure is considered consistent with the NFIP criteria if it is hydraulically equal to or better than the one it replaces. That is, the replacement structure does not increase the base flood elevations. Generally, this applies directly to crossings in which either the roadway profile is lowered or the replacement structure is the same as or larger than the existing structure. In such instances, the designer may base the design solely on normal TxDOT design procedures. However, many bridge replacements combine an increase in structure size with an increase in the roadway profile elevation or a deeper bridge deck. If such changes constitute additional obstruction in the floodway, FEMA coordination is required.

Applicability of NFIP Criteria to TxDOT

Consistency with NFIP criteria is mandated for all TxDOT projects involving encroachments in floodplains of communities participating in NFIP. 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 of roadway resulting in a higher profile.
- Highway crossing at a new location.
- Longitudinal encroachment of highway on floodplain (with or without crossing).
- Storage of materials in floodplain.
- TxDOT buildings in floodplain.

Some communities and regional councils have adopted floodplain ordinances that are more restrictive than basic FEMA criteria. Examples include the following:

- No increase ordinances that preclude any encroachment on the floodplain (i.e., no floodway).
- Design to accommodate ultimate watershed development.
- Roadway profiles to be set above 100-year flood elevation.

Generally FEMA condones stricter ordinances, but it does not require them. In fact, FEMA regulations specifically state that existing watershed conditions are to be the basis for establishing flood insurance rate zones, not future conditions. The implication of an ordinance with such stricter requirements is that highway crossings would have to span and clear the 100-year flood elevation. Neither FHWA nor FEMA require states to comply with stricter ordinances. On Federal-aid projects, FHWA will not fund costs in excess of those required for highways to meet basic FEMA criteria.

If the design is to accommodate such ordinances, TxDOT requires that any cost in excess of what would be required to accommodate either FEMA basic criteria or TxDOT criteria be borne by the community or regional council enforcing such an ordinance unless otherwise mandated by federal or state law or policy. This rationale is consistent with both the hierarchical structure of government and the fact that TxDOT is responsible for ensuring equitable use of highway funds. This philosophy may not always result in additional cost to the local entity; a risk assessment involving a range of design alternatives possibly may yield a least total cost option that accommodates the provisions of the stricter ordinance.

FEMA NFIP Map Revisions

Currently, FEMA publishes the following forms of map revision:

- Conditional Letter of Map Revision (CLOMR -- This letter from FEMA provides comments on a proposed project as to the need for a revised FIRM if the project is constructed. It indicates whether or not the project meets NFIP criteria.
- Letter of Map Revision (LOMR) -- Issued by FEMA with an accompanying copy of an annotated FIRM, this acknowledges changes in the base flood elevation, floodplain boundary, or floodway based on post-construction or revised conditions.
- Physical Map Revision -- This reprint of the FIRM reflects changes to the base flood elevations, floodplain boundary, or floodway based on revised conditions.

Normally, a TxDOT request for a CLOMR requires a follow up request for a LOMR after construction is complete unless the response to a request for a CLOMR indicates that a map revision is not required. FEMA determines the need for a physical map revision. The other map revision topics discussed below are the following:

- Typical conditions requiring FEMA map revision
- Hydrologic data for FEMA map revisions
- Hydraulic analyses for FEMA map revisions

- NFIP map revision request procedure
- FEMA's response
- FEMA fees

You may submit any proposed project with a request for a CLOMR. FEMA will then determine need for a map revision. However, an application for a CLOMR is necessary when any of the following conditions is true:

- Proposed construction encroaches in the floodway and there is any increase in the base flood elevation associated with the floodway encroachment.
- Construction in the floodplain (not just floodway) changes the base flood elevation more than 1 ft (0.3 m).
- A floodway revision is desired to ensure other development does not obstruct a proposed bridge opening.
- New hydrologic and hydraulic analyses demonstrate that the existing study is not accurate.

The same is true of LOMR's that apply to post-construction conditions. FEMA considers a LOMR to apply to any existing construction that may have occurred since the imposition of the floodway.

No map revisions are necessary under the following conditions:

- All proposed construction is outside the floodway boundary, and bridge lowchords are above the regulatory floodway elevation.
- Construction occurs within the floodway (e.g., piers), but the base flood elevations are the same or lower due to compensatory excavation or other improvement measures within the floodway, and the floodway does not need to be revised.

Hydrologic Data for FEMA Map Revisions

The hydrologic data used for the most current NFIP maps should be used in the hydraulic models for checking FEMA compliance and requesting map revisions. The only exception is when TxDOT is contesting the validity of the existing hydrologic data. FEMA will only consider new hydrologic data if it can be demonstrated to be more accurate than the existing data. The following methods acceptable to FEMA are shown in order of their preference:

1. Statistical analysis of peak annual gauged discharges
2. Regional regression equations
3. Rainfall-runoff modeling (e.g., NRCS methods).

When a request for a CLOMR or LOMR is necessary, under most circumstances, the designer needs to develop the following computer models, with exceptions as noted. All models must tie into the effective FIS profile upstream and downstream of the revised reach using sound hydraulic engineering practices to avoid discontinuities in the profile. The distance will vary depending on the magnitude of the requested floodway revision and the hydraulic characteristics of the stream.

- Duplicate effective model of the natural and floodway conditions. Rerun the original study model using the same computer program used for the original study to ensure that the base line is accurate. If the effective model is not available, an alternate model must be developed. The model should be run confining the effective flow area to the currently established floodway and calibrated to reproduce, within 0.10 ft. (0.03 m), the “with floodway” elevations provided in the Floodway Data Table for the current floodway. The alternate model should be based on floodplain geometry that existed when the original model was developed.
- Corrected effective model of the natural and floodway conditions. Many original studies may have technical errors, inaccuracies associated with not having enough cross-sections, or inaccurate cross-section data, or they did not include bridges or other structures that existed at the time of the original study. Also, an updated version of the computer program may provide more accurate bridge modeling. The newer version of the same computer program may be used to show how the results would have appeared at the time of the original study if the newer technology had been used. With adequate justification, FEMA may consider this as the base line by which to compare the impacts of any changes that have occurred since the original model was developed. If the designer considers no such changes to have occurred that may detrimentally affect the TxDOT design, this model will not be necessary. FEMA may accept an alternative computer model to the original one if the original model is unavailable, inappropriate, or the alternative model is justified as providing more accurate results.
- Updated effective model reflecting changes in the floodplain that may have occurred since the original model was established. It is not the charter of TxDOT to provide studies for map revisions for changes other than those proposed by TxDOT. Often, either the community may not have requested map revisions or non-permitted activities may have changed base flood elevations. TxDOT does not consider itself responsible for such changes unless they were the result of TxDOT construction. However, such changes may either adversely affect the design of the TxDOT project or it is possible that the TxDOT project will incur no additional increase in the base flood elevation when accounting for these changes. Therefore, the need for development and submission of a pre-project model is left to the discretion of the designer.
- Post-project model reflecting the changes to the floodplain and floodway conditions anticipated by the proposed construction. This determines the impact of the project. FEMA only requires the duplicate effective model and the post-project model. The additional models (corrected and pre-project models) may be necessary to prove to FEMA that the existing effective model is not accurate and a new model should be the basis for comparison.

NFIP Map Revision Request Procedure

Generally, for TxDOT projects, an application for a CLOMR or LOMR should be prepared by TxDOT and submitted to FEMA by the participating community, TxDOT having provided supporting documentation. The procedural outline below assumes that a CLOMR or LOMR is needed.

1. Contact the FEMA coordinator for the participating community to discuss the need for map revision, to identify any conflicts, and to establish areas of cooperation.
2. Obtain detailed data for the FIS from FEMA. This will include the hydrologic and hydraulic analyses, current mapping, and active CLOMRs and LOMRs. The community may have this information. However, the source for the most current data is FEMA's Technical Evaluation Contractor.
3. Acquire cross section survey data and establish existing field conditions in the floodplain at the proposed site.
4. Document the results of the hydraulic models.
5. Acquire and complete Form MT-2 "Application/Certification Forms for Conditional Letters of Map Revision, Letters of Map Revision, and Physical Map Revisions."
6. Provide the participating community with the application and supporting documentation. Send the application and supporting documentation to the participating community with a request to submit the package to FEMA. Request the community to confirm the submittal and notify TxDOT of FEMA's response.

FEMA response is usually a request for additional data, issuance of a map revision, or an indication that no map revision is required.

Fees associated with the application and review process are revised periodically. In 2001 these totaled about $5,400 for a CLOMR and follow-up LOMR and did not include the cost of retrieving the original FIS data. All associated fees for TxDOT projects should be assigned to engineering costs.

Chapter 3
Types of Documentation

Contents:

Section 1

Types of Documentation

Documentation Categories

TxDOT hydraulic facility analyses and design generally fall into the following basic categories:

- 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. The standards of design development are the design criteria. Examples include allowable headwater (for a culvert), allowable through-bridge velocity (for a bridge), and maximum water elevation in a pump station sump. Both design parameters and criteria are established from the unique characteristics of the design site and situation. These items should be fully documented for the design of TxDOT drainage facilities.
- Federal and state regulatory criteria -- (See Chapter 2.)
- TxDOT policy or coordination with other policy – TxDOT policy represents a significant basis of design of any drainage facility. Chapter 2 addresses TxDOT drainage-related policy. In cases where TxDOT policy conflicts or differs with policy of an outside entity, include qualifying statements and explanations in the documentation. Note that federal and state regulatory criteria have the force of law with which TxDOT must comply.
- Hydrology and hydraulic analyses -- Documentation of the hydrologic and hydraulic analyses includes the assumptions, judgments, decisions, computations, and plans, profiles, and details.

Carefully identify, consider, and evaluate data sources used during the design process. In a conflict among data from different sources, evaluate the conflicting data to determine its relevance and usefulness.

Make subjective selections, decisions, and assignments throughout TxDOT drainage design procedures as part of the engineering design process. When subjectivity is necessary, document the qualifying statements about the selection process. For example, the assignment of a runoff coefficient for use in the rational method of estimating peak discharge is a subjective assignment based upon current and projected watershed characteristics. You should systematically assess those characteristics. However, in order to assign an appropriate runoff coefficient, you must make judgments regarding future watershed characteristics. Include the design considerations leading to those judgments in the files as qualifying statements.

Section 2

Documentation Requirements and Guidelines

Documentation Requirements for Existing Locations

When replacing a structure, prepare a comparative 100-year hydraulic analysis between existing and proposed structures. When rebuilding, improving, or rehabilitating a roadway, consider associated drainage facilities such as improvements, extensions, paralleling, replacements, and leaving the facility unchanged.

Whatever action is taken, the plans and submission information should document sufficient data, along with basic pertinent hydrologic and hydraulic information.

- In most cases, refer to the Data Documentation Requirements table for items to be included in the plans.
- If verified, use data from the original documentation file.
- If previous hydraulic data are available and applicable, provide a note on the proposed plans referring to the Control Section and Job number (CSJ), and state verification of the data.

Documentation of experience with the past performance of a facility is also useful. 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.

Refer to the stipulations found in 23 CFR §650.117 for absolute requirements on federally funded projects. Pertinent information from this document is in the Federal Aid Policy Guide (FAPG) in Policy. The detail of design investigation and documentation should be proportional to the risk associated with encroachment and other economic, engineering, social, or environmental concerns. Consult resource agencies for assistance with documentation. See Resources for primary resource agencies.

Documentation Reference Table

The following tables indicate the required documentation of various facility types for preliminary review, PS&E (Plans, Specifications, and Estimates) 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:

Data Documentation Requirements

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

The following table shows the hydrology documentation requirements:

Hydrology Documentation Requirements

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

Hydrology Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
Runoff hydrographs for design and check floods (where applicable)	X	X	X	X	X

The following table shows the channel documentation requirements:

Channel Documentation Requirements

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

The following table shows the culvert documentation requirements:

Culvert Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
	Preliminary Review	PS&E Review	Field Changes	Construction Plans	Permanent File

Culvert Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
Culverts					
See Hydrology for discharge data	X	X	X	X	X
See Channels for tailwater data	X	X	X	X	X
Design criteria (Allowable headwater, outlet velocities, FEMA etc.)	X	X	X	X	X
Culvert hydraulic computations	X	X	X		X
Unconstricted and through-culvert velocities for design and check floods	X	X	X	X	X
Calculated headwater for design and check floods	X	X	X	X	X
Estimated distance upstream of backwater effect	X	X	X	X	X
Magnitude and frequency of overtopping flood	X	X	X	X	X

The following table shows the bridge documentation requirements:

Bridge Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
	Preliminary Review	PS&E Review	Field Changes	Construction Plans	Permanent File
Bridges					
See Hydrology for discharge data	X	X	X	X	X
See Channels for highwater data	X	X	X	X	X
Design criteria/parameters/ assumptions (velocities, backwater, FEMA, etc.)	X	X	X	X	X

Bridge Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
Bridge hydraulic computations	X	X	X		X
Unconstricted and through-bridge velocities for design and check floods	X	X	X	X	
Calculated maximum back-water for design and check floods	X	X	X	X	
Estimated distance upstream of backwater effect	X	X	X	X	
Magnitude and frequency of overtopping flood	X	X	X	X	X
Scour calculations	X				X
Estimated scour envelope	X	X	X	X	

The following table shows the storm drain documentation requirements:

Storm Drain Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
	Preliminary Review	PS&E Review	Field Changes	Construction Plans	Permanent File
Storm Drains					
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	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 pump station documentation requirements:

Pump Station Documentation Requirements

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

The following table shows the facility documentation requirements:

Other Facility Documentation Requirements

Documentation Item (by facility type)	**Stage**			**Location of Information**	
	Preliminary Review	PS&E Review	Field Changes	Construction Plans	Permanent File
Other Facilities					

Other Facility Documentation Requirements

Documentation Item (by facility type)	Stage			Location of Information	
Drainage area maps	X	X	X	X	X
Design criteria/parameters/ assumptions	X	X	X		X
Hydrologic computations	X	X	X	X	X
Hydraulic computations	X	X	X	X	X
Plan/profile and details	X	X	X	X	
Design and check flood before and after conditions (highwater, velocities, etc.)	X	X	X	X	X

TxDOT Recommended Guidelines

The following checklist files, intended for use by both designer and reviewers, aid in the documentation process:

- Culvert hydraulic documentation checklist (File 3c).
- Bridge hydraulic documentation checklist (File 3b).
- Storm drain hydraulic documentation checklist (File 3d).
- Pump station hydraulic documentation checklist (File 3e) and checklist for hydraulic design project references (File 3a).

TxDOT recommends the following approach to documenting hydrologic and hydraulic designs and analyses:

- Compile hydrologic and hydraulic data, preliminary calculations and analyses, estimates of structure size and location, and all related information used in developing conclusions and recommendations in a permanent documentation file.
- Document all design assumptions and selected criteria, including the decisions related to the design.
- Make the amount of documentation detail for each design or analysis proportional to the risk and the importance of the facility. Characteristics governing detail and resulting documentation include facility importance, traffic load, adjacent property, drainage complexities, and requirements of other agencies (e.g., FHWA, FEMA, and the Texas Natural Resource Conservation Commission).

- Organize documentation in a concise and complete manner to lead readers logically from past history through the problem background into the findings and through the performance. This ensures that future designers can understand the actions their predecessors took.
- Include all related references in the documentation files, such as published data and reports, memos and letters, and interviews.
- Include dates and signatures where appropriate.
- Include data and information from the conceptual development stage through service life of the project.
- Include an executive summary at the beginning of the documentation that provides an outline of the documentation file to assist users in finding detailed information.
- Document all data sources in the files.

Section 3

Documentation Review Stages

Review Data

Hydrologic and hydraulic data and documentation should accompany all drainage structure proposals that are submitted for preliminary review and approval prior to the actual detailed design of the structure. The documentation expected for review is usually a summary of the pertinent data. Permanent documentation should be much more extensive and should include considerations, criteria, judgments, background data, computations, and details.

Submitting data and documentation for preliminary review does not relieve submission requirements for final review of the PS&E package.

The final review involves presenting data and documentation in the plans submitted for PS&E processing.

Documents supporting field change requests must describe and justify the effects of the proposed change on the expected hydraulic performance of the facility. Generally, this involves modifying existing documentation to include the following: a description of the specific hydraulic effects resulting from the proposed facility change, revised plan sheets and hydraulic data sheets, the hydraulic data sheets from PS&E submission for some projects, and a note on the plans clearly explaining the changes made to the original design.

Permanent Documentation Retention

Permanent documentation includes the construction plans and design files maintained in the district. Usually, PS&E reviews require only the construction plans. However, if the project did not receive preliminary review or there are other concerns, it may also be necessary to submit the design file for review.

Retain hydrologic and hydraulic documentation in the project plans or another permanent location at least until a new drainage study requires the replacement or modification of the facility.

Prepare and maintain permanent files with as-built plans for every drainage structure to document subsurface foundation elements such as the following:

- footing types and elevations
- pile types
- finished top elevations

Other information that should be included may become evident as the design or investigation develops. Include this information at your discretion.

Chapter 4
Data Collection, Evaluation, and Documentation

Contents:

Section 1
Introduction

This chapter discusses general hydraulic data collection needs, data location, analysis, evaluation, and documentation.

The importance and extent of the project and facility determine the amount of effort needed for data collection and evaluation. A comprehensive, accurate, and economical highway drainage design requires reliable data for its success. Failure to base a design on adequate and appropriate data can lead to economic loss and interruption of the roadway function (see Figure 4-1).

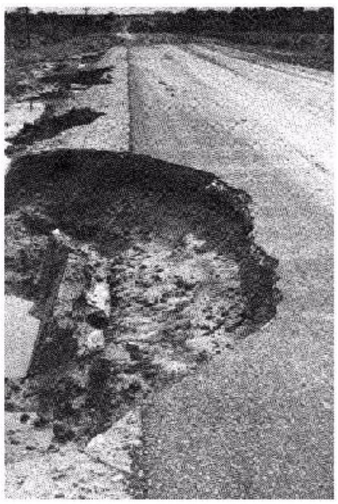

Figure 4-1. Roadway Base Failure

A systematic data collection program generally leads to a more orderly and effective analysis or design. The following table outlines the data collection process:

1. Identify data types: drainage area characteristics, land use, stream course data, facility site data, streamflow data, and climatological data.
2. Determine data sources: site investigation data and resource agencies.
3. Evaluate data.

Section 2

Site Investigation Data

Introduction

TxDOT policy requires a hydrologic and hydraulic analysis for projects that involve:

- new locations
- replacing facilities
- widening existing locations

Drainage Area Characteristics

Refer to linked "File 3a" for a Documentation Checklist for Hydraulic Design Project References based on the following paragraphs.

Size. Drainage area size is usually important for estimating runoff characteristics. Determine the size of the drainage by one of the following methods:

- Conduct direct field surveys with conventional surveying instruments.
- Use topographic maps together with field checks for artificial barriers such as terraces and ponds. (USGS topographic maps are available for many areas of the state through retail outlets for maps and surveying supplies. Many municipal and county entities as well as some developers have developed topographic maps of their own. Determine the suitability and usefulness of all these maps.)
- Use any other available resources.

Topography. Estimate relief and slope characteristics of the watershed by one or more of the methods listed above for drainage area sizes. Most hydrologic procedures used by TxDOT depend on watershed slopes and other physical characteristics.

Soil Type. Watershed soil type(s) and associated characteristics correlate with infiltration, interception, depression storage, and detention storage. Use Natural Resources Conservation Service publications, including maps, reports, and work plans, to identify and quantify soil parameters in the watershed. See U.S. Department of Agriculture for contact information.

Vegetation. Present and future vegetation characteristics influence the amount and rate of watershed runoff as well as the streamflow patterns expected in and around the drainage facility. Look at surveys or obtain data from a site visit.

Land Use

There are several forms of land use data and many sources from which to obtain them.

Development Prediction Source. Ordinarily, the drainage facility design includes a reasonable anticipation of service life. Because the facility must accommodate potential flows during that service life, consider possible future development of the watershed. Predicting future development of a watershed is difficult. However, you can estimate future development by interviewing landowners, developers, officials, planners, local and regional planning organizations, realtors, and local residents.

Watershed Characteristic Sources. Look at master plans for development from city planning departments. Land use data are available in different forms, including topographic maps, aerial photographs, zoning maps, satellite images, and geographic information systems. Municipalities have records and maps of storm drain systems and channel improvements.

Stream Course Data

Streams are classified as follows:

- rural, urban, or a mix
- unimproved to improved
- narrow to wide-wooded
- rapid flow to sluggish

Profile. Extend the stream profile sufficiently upstream and downstream of the facility to determine the average slope and to encompass any channel changes or aberrations. USGS recommends a minimum distance of 500 ft. (150 m) both upstream and downstream for a total of 1000 ft. (300 m) or a distance equal to twice the width of the floodplain, whichever is greater. Topographic maps published by USGS are useful in determining overall channel slopes.

Channel Location. Note the location of the main channel and any subchannels, creeks, and sloughs within the profile section.

Cross Sections. Cross sections must represent the stream geometry and contain the highest expected water-surface elevation to be considered. For hydraulic computations, use cross sections that are perpendicular or normal to the anticipated direction of flow. In some instances, particularly in wide floodplains where a single straight line across is not adequate, break the cross section into segments for a dogleg effect as shown in Figure 4-2. Adjacent cross sections should not cross each other.

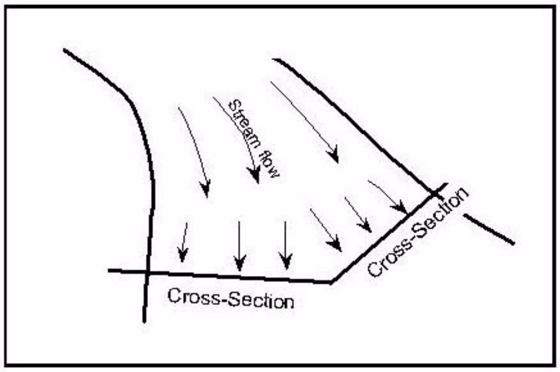

Figure 4-2. Dog-legged Cross Section

The minimum number of cross sections is four, located as follows:

- At the beginning of the profile stretch
- At the downstream face of the structure (or where the downstream face will be)
- At the upstream face of the structure (or where the upstream face will be)
- At the end of the profile stretch

Additional cross sections are necessary at each change in roughness, slope, shape, or floodplain width. Take enough cross sections to analyze fully the stream flow.

Do not leave the choice of the typical cross section entirely to the field survey party. Carefully consider the location and orientation of the cross section used in the channel analysis without regard to surveyor convenience or expedience.

Locate sections as follows:

- Sections along the right-of-way line can be misleading hydraulically because they may represent only local, cleared conditions that do not reflect the stream reach. For similar reasons, avoid cross sections along utility easements and other narrow cleared areas.
- Avoid local depressions or crests that are not typical of a whole stream reach.
- Generally try to space sections about 1.5 to 2 times the approximate floodplain width. A notable exception to this is at structures where more definition is needed.

Roughness Characteristics. The Manning's equation for uniform flow is the most commonly used conveyance relation in highway drainage design. Note and record the physical details of the streambed and floodplain; you will use them later to determine the Manning's roughness coefficients (n values). Details include vegetation type and density, material (rock type, clay soil, gravel), trash, streambed shape, cross section geometry, and any item that may affect streamflow during normal and flood conditions.

Flow Controls. Note anything upstream and downstream within the profile section, including the following:

- Any downstream confluences
- Significant choking sections
- Bridges and low water crossings
- Abrupt meanders
- Heavily vegetated areas
- Material borrow pits in the floodplain

Include all observations about size, type, location, and flow over or through. Bridge data should include span lengths and types and dimensions of piers.

Reservoirs. Note any reservoirs and ponds along with their spillway elevations and operations or other control operations. Dams with hydroelectric generators may raise water levels significantly during generator operations.

The following organizations may have complete reports concerning the operation, capacity, and design of proposed or existing conservation and flood-control reservoirs:

- Natural Resources Conservation Service (NRCS)
- Corps of Engineers (USACE)
- Bureau of Reclamation
- Texas Natural Resource Conservation Commission (TNRCC)
- Municipalities

Flood Stages. Obtain information on historic flood stages from TxDOT personnel, city and county officials, and local residents. If possible, observe the structure under flood conditions to learn about the stream behavior. When possible, take videos and photographs of the flood action at or near the structure for use in future studies. Determine the direction of stream lines with relation to the low flow channel, estimated velocity, estimated drifting material (amount and size), natural tendency for erosion in the channel, the drop in water surface elevation from the upstream side to the downstream side of the structure, and the highest stage with the date of occurrence.

Geotechnical Information

Soil Properties. A geotechnical report provides information about the soils in the area and soils used on highway projects. The detail of such reports can vary greatly but usually will include the following:

- Soil type, soil density (blow count), and depth for each soil type

- Soil properties such as acidity/alkalinity, resistivity, and other significant constituents
- Presence, depth, and type of bedrock
- Sieve analyses (D_{50} and D_{90} values)

Scour Observations. Note the presence of scour around pilings and abutments. Record size, depth, and location of each scour hole. Also record any deposition of material including type (rock, gravel, dirt, etc.), location, and depth.

Stream Stability. Erosion problems may occur in a stream system even without the presence of a bridge. Record the following data:

- Any occurrence or possibility of streambed degradation (head cutting). Head cutting may be caused by dredging or mining downstream or channel modifications such as straightening.
- Signs of bank slippage and erosion such as buildings located closer to the bank than seem reasonable, trees growing at odd angles from the bank, exposed tree roots, and trees with trunks curved near the ground.
- The location and likely direction of lateral migration (meanders).

For more information, see the discussion on stream stability in Chapter 7.

Adjacent Properties

Note the location of any driveways, utilities, and structures adjacent to the project site that will be affected by construction. Note the elevations of any improvements or insurable structures near the proposed site that may be affected by a rise in water surface elevations up through and including the 100-year event.

Section 3
Other Data Sources

Highway Stream Crossing Design Data Sources

Use a combination of the following sources to obtain data in the design of highway stream crossings, including the following:

- site investigations and field surveys
- files of federal agencies such as the National Weather Service, USGS (U.S. Geological Survey), and NRCS. (**Note**: NRCS was formerly the Soil Conservation Service.)
- files of state and local agencies such as TxDOT files, Texas water agencies, and various regional and municipal planning organizations
- other published reports and documents
- the Texas Natural Resource Information System (TNRIS)

Compile streamflow, land use, and other required data from the sources mentioned above. For a list of appropriate agency addresses, see References.

Streamflow Data

The primary source of streamflow information in Texas is USGS, the agency charged with collecting and disseminating this data. USGS collects data at stream-gauging stations statewide.

The USGS Internet site provides direct access to stream gauge data.

The Corps of Engineers (USACE) and the Bureau of Reclamation also collect streamflow data. Other sources of data include local utility companies, water-intensive industries, and academic or research institutions.

The International Boundary and Water Commission collects and compiles streamflow data along the Rio Grande and some tributaries.

Climatological Data

The National Weather Service (NWS) has a wealth of climatological data, specifically rainfall data. NWS issues periodic reports to the public and agencies such as TxDOT. NWS also publishes reports concerning reduced data that the designer can use as analytical tools.

Section 4

Data Evaluation and Documentation

Data Evaluation Procedure

Experience, knowledge, and judgment are important parts of data evaluation. After collecting data, use the following data evaluation procedure:

1. Compile and evaluate data into a usable format. Compile all collected information into a comprehensive and accurate representation of the hydrologic, hydraulic, and physical characteristics of a particular site.
2. Determine if the data contain inconsistencies or other unexplained anomalies that might lead to erroneous calculations, assumptions, or conclusions.
3. Separate reliable data from unreliable data.
4. Combine historical data with data obtained from measurements.
5. Evaluate data for consistency, and identify any changes from established patterns.
6. Review previous studies, old plans, or prior documentation for data types and sources, information on how the data were used, and indications of accuracy and reliability.
7. Carefully evaluate unpublished data for accuracy and reliability.

Review this historical data to determine whether significant changes occurred in the watershed and for usefulness of the data. TxDOT considers valid and accurate any data acquired from publications by established sources, such as the USGS.

Use the procedure to compare data for inconsistencies:

1. Evaluate basic data, such as streamflow data derived from non-published sources.
2. Summarize this data before use.
3. Compare the following data with each other and with the results of the field survey to resolve any inconsistencies: maps, aerial photographs, satellite images, videotapes, and land use studies.

Consult general references to help define the hydrologic character of the site or region under study and aid in the analysis and evaluation of data.

Data Documentation Items

Begin documenting obtained data as soon as you collect it. In design/analysis documentation, include types and identified sources, actual data items, evaluations, assumptions, and conclusions concerning the data.

Other Considerations for Drainage Facilities

Consider collecting descriptive data to address the following:

- coordination with other agencies
- compliance with TxDOT policy and administrative guidelines
- consideration of local ordinances and preferences
- careful coordination with affected property owners

Gain a thorough understanding of local, state, and federal requirements regarding the design of roadway drainage facilities.

Chapter 5
Hydrology

Contents:

Section 1
Introduction

Description

For the purpose of this manual, hydrology deals with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic feet per second (cfs) or cubic meters per second (m^3/s) and hydrographs as discharge per time. Use peak discharge to design facilities such as storm drain systems, culverts, and bridges.

For systems that are designed to control the volume of runoff, like detention storage facilities, or where flood routing through culverts is used, the entire discharge hydrograph will be of interest.

Fundamental to the design of drainage facilities are analyses of peak rate of runoff, volume of runoff, and time distribution of flow.

Errors in the estimates result in a structure that is either undersized, which could cause drainage problems, or oversized, which costs more than necessary. On the other hand, realize that 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 economic, engineering, social, and environmental factors.

Because hydrology is not an exact science, different hydrologic methods developed for determining flood runoff may produce different results for a particular situation. Therefore, exercise sound engineering judgment to select the proper method or methods to be applied. In some instances, certain federal or state agencies may require (or local agencies may recommend) a specific hydrologic method for computing the runoff.

While performing the hydrologic analysis and hydraulic design of highway drainage facilities, the hydraulic engineer should recognize and evaluate potential environmental problems that would impact the specific design of a structure early in the design process.

Most complaints relating to highway drainage facilities stem from the impact to existing hydrologic and hydraulic characteristics. In order to minimize the potential for valid complaints, gather complete data reflecting existing drainage characteristics during design.

Peak Discharge versus Frequency Relations

Highway drainage facilities are designed to convey predetermined discharges in order to avoid significant flood hazards. Provisions are also made to convey floods in excess of the predetermined discharges in a manner that minimizes the hazards. Flood discharges are often referred to as peak discharges as they occur at the peak of the stream's flood hydrograph (discharge over time). Peak discharge magnitudes are a function of their expected frequency of occurrence, which in turn relates to the magnitude of the potential damage and hazard. (All the methods described in this manual allow determination of peak discharge.)

The highway designer's chief interest in hydrology rests in estimating runoff and peak discharges for the design of highway drainage facilities. The highway drainage designer is particularly interested in development of a flood versus frequency relation, a tabulation of peak discharges versus the probability of occurrence or exceedance.

The flood frequency relation is usually represented by a flood frequency curve. A typical flood frequency curve is illustrated in Figure 5-1. In this example, the discharge is plotted on the ordinate on a logarithmic scale, and the probability of occurrence or exceedance is expressed in terms of return interval and plotted on a probability scale on the abscissa.

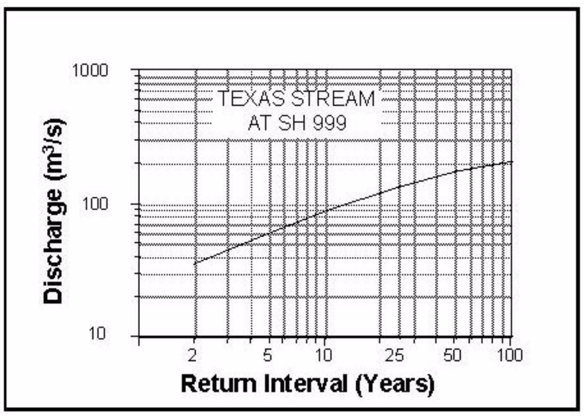

Figure 5-1. Typical Flood Frequency Curve

Also of interest is the performance of highway drainage facilities during the frequently occurring low flood flow periods. Because low flood flows do occur frequently, the potential exists for lesser amounts of flood damage to occur more frequently. It is entirely possible to design a drainage facility to convey a large, infrequently occurring flood with an acceptable amount of floodplain damage only to find that the accumulation of damage from frequently occurring floods is intolerable.

Flood Hydrographs

In addition to peak discharges, the hydraulics engineer is sometimes interested in the flood volume and time distribution of runoff. You can use flood hydrographs to route floods through culverts, flood storage structures, and other highway facilities.

By accounting for the stored flood volume, the hydraulics engineer can often expect lower flood peak discharges and smaller required drainage facilities than would be expected without considering storage volume. You can also use flood hydrographs for estimating inundation times of flow over roadways and pollutant and sediment transport analyses.

Unit Hydrograph

A unit hydrograph represents the response of a watershed to a rainfall excess of unit volume and specific duration. For department practice, the unit is 1 in. (1 mm) — that is, the volume associated with an excess rainfall of 1 in. (1 mm) distributed over the entire contributing area.

The response of a watershed to rainfall is considered to be a linear process. This has two implications that are useful to the designer: the concepts of proportionality and superposition. For example, the runoff hydrograph resulting from a two-unit pulse of rainfall of a specific duration would have ordinates that are twice as large as those resulting from a one-unit pulse of rainfall of the same duration. Also, the hydrograph resulting from the sequence of two one-unit pulses of rainfall can be found by the superposition of two one-unit hydrographs. Thus, by determining a unit hydrograph for a watershed, you can determine the flood hydrograph resulting from any measured or design rainfall using these two principles.

Interagency Coordination

Because many levels of government plan, design, and construct highway and water resource projects that might affect each other, interagency coordination is desirable and often necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis. (See the *Environmental Procedures in Project Development Manual* for more information on interagency coordination.)

Section 2

Factors Affecting Floods

Flood Factors

The following factors affect floods in the watershed: runoff, watershed area information, geographic location, land use, soil type, topography, vegetation, detention storage systems, flow diversions, channelization, and future conditions.

Runoff. Two main factors influence runoff from a watershed: precipitation and abstractions. Precipitation in the State of Texas is represented most significantly by rainfall, although snow, sleet, and hail can influence runoff. Rainfall rate distributions within a watershed vary both temporally and spatially. For most determinations of peak flow for use in department design and analysis efforts, assume rainfall rates not to vary within the watershed during the rainfall event.

Generally, the entire volume of rainfall occurring on a watershed does not appear as runoff. Losses, known as abstractions, tend to reduce the volume of water appearing as runoff. Abstractions of precipitation in its evolution into runoff are numerous. However, for the typical highway drainage design problem, only six abstractions are commonly considered. They are shown in the order of their significance to the runoff.

- Infiltration—The amount of the precipitation that percolates into the ground in the watershed. This abstraction is a function of soil type and characteristics, terrain slopes, and ground cover.
- Depression storage—The precipitation stored permanently in inescapable depressions within the watershed. It is a function of land use, ground cover, and general topography.
- Detention storage—The precipitation stored temporarily in the flow of streams, channels, and reservoirs in the watershed. It is a function of the general drainage network of streams, channels, ponds, etc. in the watershed.
- Interception—The precipitation that serves to first “wet” the physical features of the watershed (e.g., leaves, rooftops, pavements). It is a function of most watershed characteristics.
- Evaporation—The precipitation that returns to the atmosphere as water vapor by the process of evaporation from water concentrations. It is mostly a function of climate factors, but it is associated with exposed areas of water surface.
- Transpiration—The precipitation that returns to the atmosphere as water vapor and that is generated by a natural process of vegetation foliage. It is a function of ground cover and vegetation.

The specific consideration of each of these abstractions is not usually explicit in the many hydrologic methods available.

Watershed Area Information. Most runoff estimation techniques use the size of the contributing watershed as a principal factor. Generally, runoff rates and volumes increase with increasing drainage area. The size of a watershed will not usually change over the service life. However, agricultural activity and land development may cause the watershed area to change. Diversions and area changes due to urbanization and other development inevitably occur. Try to identify or otherwise anticipate such circumstances.

The watershed shape usually will affect runoff rates. For example, a long, narrow watershed is likely to experience lower runoff rates than a short, wide watershed of the same size and other characteristics. Some hydrologic methods accommodate watershed shape explicitly or implicitly; others may not. If a drainage area is unusually bulbous in shape or extremely narrow, the designer should consider using a hydrologic method that explicitly accommodates watershed shape.

The response of a watershed to runoff may vary with respect to the direction in which a storm event passes. Generally, for design purposes, the orientation of the watershed may be ignored because it is usual to assume uniform rainfall distribution over the watershed.

Geographic Location. The geographic location of the watershed within the State of Texas is a significant factor for the drainage designer. Rainfall intensities and distributions, empirical hydrologic relations, and hydrologic method applications vary on the basis of geographic location. You should use hydrologic methods and parameters that are appropriate for the specific location.

Land Use. Land use significantly affects the parameters of a runoff event. Land use and human activity within most watersheds vary with respect to time. For example, a rural watershed can be developed into a commercial area in a matter of weeks. Factors subject to change with general variations in land use include the following:

- permeable and impermeable areas
- vegetation
- minor topographic features
- drainage systems.

All of these factors usually affect the rate and volume of runoff that may be expected from a watershed. Therefore, carefully consider current land use and future potential land use in the development of the parameters of any runoff hydrograph.

Land Use Changes. Diversions and area changes due to urbanization and other development inevitably occur. Try to identify or otherwise anticipate such circumstances.

Soil Type. The soil type can have considerable effect on the discharge rates of the runoff hydrograph; the soil type directly affects the permeability of the soil and thus the rate of rainfall infiltration. The Natural Resources Conservation Service (NRCS) is an excellent repository for information about soils in Texas. The hydrologic procedure used may require specific data concerning the soil type.

Topography. Topography mostly affects the rate at which runoff occurs. The rate of runoff increases with increasing slope. Furthermore, rates of runoff decrease with increasing depression storage and detention storage volumes. Many methods incorporate a watershed slope factor, but fewer methods allow the designer to consider the effects of storage on runoff.

Vegetation. In general, runoff decreases with increasing density of vegetation; vegetation helps reduce antecedent soil moisture conditions and increases interception such as to increase initial rainfall abstractions. Vegetative characteristics can vary significantly with the land use; therefore, consider them in the assessment of potential future conditions of the watershed.

Detention Storage Systems. Detention storage systems are common in urban areas mostly due to governmental requirements aimed at controlling increased runoff from developed areas. The department designer should identify any detention storage systems that might exist within the subject watershed. A detention storage facility can attenuate the runoff hydrograph, thus reducing the peak discharge. The department may design facilities that involve detained storage to conform to federal and state environmental regulations, to cooperate with local ordinances or regulations, or where you deem flood attenuation necessary.

Flow Diversions. Flow diversions within a watershed can change the runoff travel times and subsequent peak discharge rates. They can decrease discharge at some locations and increase discharge elsewhere. Flow diversions may redirect flow away from a location during light rainfall but overflow during heavy rainfall. Make an assessment of the likely effect of diversions that exist within the watershed. Also, ensure that you minimize the potential impact of necessary diversions resulting from your highway project.

Channelization. Channelization in an urban area entails the following:

- improved open channels
- curb and gutter street sections
- inverted crown street sections
- storm drain systems.

Any of these channelization types serve to make drainage more efficient. This means that flows in areas with urban channelization can be greater, and peak discharges occur much more quickly than where no significant channelization exists.

Future Conditions. Changes in watershed characteristics and climate directly affect runoff. A reasonable service life of a designed facility is expected. Therefore, base the estimate of design flooding upon runoff influences within the time of the anticipated service life of the facility.

Prediction Information

In general, consider estimates for future land use and watershed character within some future range. It is difficult to predict the future, but you should make an effort at such a prediction, especially with regard to watershed characteristics. Landowners, developers, realtors, local and state and federal officials, and planners can often provide information on potential future characteristics of the watershed.

In estimating future characteristics of the watershed, consider changes in vegetative cover, surface permeability, and contrived drainage systems. Climatic changes usually occur over extremely long periods of time such that it is not usually reasonable to consider potential climatic changes during the anticipated life span of the facility.

Section 3
Design Frequency

Concept of Frequency

As with other natural phenomena, occurrence of flooding is governed by chance. The chance of flooding is described by a 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 a watershed, the designer must choose a design frequency appropriate for the structure.

The expected frequency for a given flood is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. For example, if a flood has a 20 percent chance of being equaled or exceeded each year, over a long period of time the flood will be equaled or exceeded on an average of once every five years. This is called the return period or recurrence interval (RI). Thus the exceedance probability equals 100/RI. The following table lists the probability of occurrence for the standard design frequencies.

Frequency versus Probability

Frequency (Years)	Probability (%)
2	50
5	20
10	10
25	4
50	2
100	1

The five-year flood is not one that will necessarily be equaled or exceeded every five years. There is a 20 percent chance that the flood will be equaled or exceeded in any year; therefore, the five-year flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods.

Frequency Determination

Derive the design frequency from the importance of the appropriate 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. Additional considerations include the design frequencies of other structures along the same highway corridor to ensure that the new structure is compatible with the rest of the roadway and the probability of any part of a link of roadway being cut off due to flooding. Address the list of considerations using either design by frequency selection or by examples for cost optimization or risk assessment.

Design by Frequency Selection

A traditional approach to establishing a frequency for design of a drainage facility is by use of reference tables in which specific ranges of design frequencies are established for different facility types. The following table presents recommended ranges for possible use on TxDOT projects. Inundation of the travelway dictates the level of traffic service provided by the facility. The travelway overtopping flood level identifies the limit of serviceability. This table relates desired minimum levels of protection from travelway inundation to functional classifications of roadways. For the selected design frequency, design the facility to avoid inundation of the roadway.

Recommended Design Frequencies (years)

	Design					**Check Flood**
Functional Classification and Structure Type	2	5	10	25	50	100
Freeways (main lanes):						
◆ culverts					X	X
◆ bridges					X	X
Principal arterials:						
◆ culverts			X	(X)	X	X
◆ small bridges			X	(X)	X	X
◆ major river crossings					(X)	X
Minor arterials and collectors (including frontage roads):						
◆ culverts		X	(X)	X		X
◆ small bridges			X	(X)	X	X
◆ major river crossings				X	(X)	X
Local roads and streets (off-system projects):						
◆ culverts	X	X	X			X
◆ small bridges	X	X	X			X
Storm drain systems on interstate and controlled access highways (main lanes):						
◆ inlets and drain pipe			X			X
◆ inlets for depressed roadways*					X	X
Storm drain systems on other highways and frontage:						

Recommended Design Frequencies (years)

	Design					Check Flood
◆ inlets and drain pipe	X	(X)				X
◆ inlets for depressed roadways*				(X)	X	X

Notes.
* A depressed roadway provides nowhere for water to drain even when the curb height is exceeded.
( ) Parentheses indicate desirable frequency.

In establishing a design frequency for a drainage facility, the designer takes the risk that a flood may occur that is too large for the structure to accommodate. This risk is necessary when limited public funding is available for the drainage facility. Using the “Recommended Design Frequencies” table only implies but does not quantify the level of risk. For many projects, you may determine the potential risks associated with design by frequency selection to be so small that you would need no further appraisal of risk. However, if contemplating deviation from the recommended design frequencies or the potential risks could be significant, perform a risk assessment. The extent of this assessment should be consistent with the value and importance of the facility.

NOTE: Federal law requires interstate highways to be provided with protection from the 50-year flood event, and facilities such as underpasses, depressed roadways, etc., where no overflow relief is available should be designed for the 50-year event.

Design by Cost Optimization or Risk Assessment

The objective of cost optimization is to choose a design frequency that results in a facility that satisfies all the design requirements with the lowest total cost. Structures with low design frequencies generally have lower capital costs but higher operational costs. In discussions of cost optimization, the following definitions apply:

- Capital costs are those associated with the direct construction of a facility that can be readily estimated. Generally, the higher the design frequency, the higher the capital cost.
- Operational costs are associated with maintenance and repair to the facility and costs of any damage incurred by the facility. For the hydraulic design of drainage structures, the primary concern is the potential for flood damage and risk to the traveling public.

A large structure with a high design frequency may have a much larger capital cost yet lower operational costs. The larger structure may last through several lifetimes of the smaller structure. In addition, potential costs of interruption to traffic and other damage may be higher for the smaller structure. Figure 5-2 shows a plot of the cost for design alternatives of varying design frequency. The optimal design is the one that balances capital costs with operational costs to produce the lowest total cost.

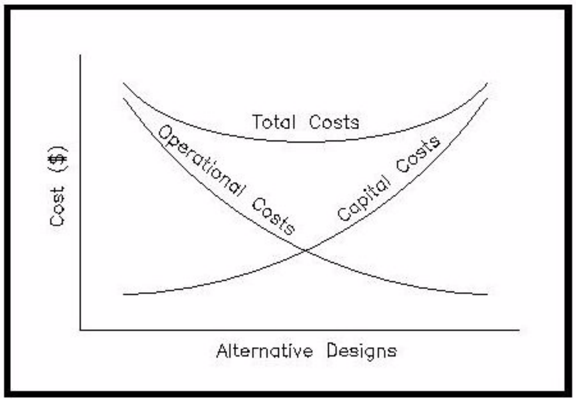

Figure 5-2. Lowest Total Expected Cost

Risk is defined as the consequences associated with the probability of flooding. For low frequency designs, the probability of flood-related damage is usually higher than that associated with higher frequency designs. A risk assessment involves appraising the levels of risk for selected design alternatives and is less extensive than a cost optimization approach.

FORMC1 provides examples of forms using risk assessment in bridge design. FORMC2 shows supplemental worksheets for summarizing economic risk and losses. The FHWA publication *Design of Encroachments on Flood Plains Using Risk Analysis,* Hydraulic Engineering Circular Number 17 (HEC #17), provides more extensive detail on risk assessment and cost optimization. Although the forms, worksheets, and the example in HEC # 17 refer to bridge design, risk assessment should not be limited to bridges. The same approach is valid for the design of most drainage facilities.

Design by cost optimization or risk assessment can be largely subjective, and data requirements often are much more extensive than design by frequency selection. The following examples illustrate situations in which either cost optimization or risk assessment might be appropriate:

- Replacement of off-system bridges where an existing facility has lower capacity than the recommended design frequency for given hydrologic conditions. Usually, off-system bridges are replaced for reasons other than hydraulic adequacy. A risk assessment would help to justify whether a structure larger than the existing structure is needed.
- Where there is a need to determine whether cost of exceeding 50-year design frequency for a floodplain crossing is justifiable.
- To justify any design that falls within the design frequencies recommended in the “Recommended Design Frequencies” table.
- A drainage facility type is not addressed in “Recommended Design Frequencies” table.
- Required roadway improvements where existing drainage facilities are in good condition but do not meet recommended design frequency. A risk assessment should be employed to determine if existing structures should be replaced.

- Any situation in which the potential risks of damage are high or questionable.

Check Flood Frequencies

Most flood events are of smaller magnitude than the design flood, but a few are of greater magnitude. From the standpoint of facility utilization, strive toward a facility that will operate in the following manner:

- efficiently for lesser floods
- adequately for the design flood
- acceptably for greater floods.

For these reasons, it is often important to consider floods of other magnitudes. To define the peak flows for frequencies other than the design frequency, use the approach of developing a general flood-frequency relation for the subject site.

For all drainage facilities, including storm drain systems, evaluate the impact of the 100-year flood event. In some cases, evaluate a flood event larger than the 100-year flood (super-flood) to ensure the safety of the drainage structure and downstream development. A 500-year flood analysis is required for checking the design of bridge foundations against potential scour failure.

If a catastrophic failure of a bridge or culvert can release a flood wave that would result in loss of life, disruption of essential services, or excessive economic damage, the bridge or culvert design should be evaluated in terms of a probable maximum flood or PMF. For example, a culvert under normal flood operation will act like a dam. PMF considers the conditions under which the culvert/dam may fail. The PMF is not related to an event frequency but is a specialized analysis. Consult the Bridge Division's Hydraulic Branch for assistance with the PMF determination.

Frequencies of Coincidental Occurrence

Where the outfall of a system enters as a tributary of a larger drainage basin, the stage-discharge characteristics of the outfall may operate independently of the main drainage basin. This is especially common in storm drain systems. For example, a small storm drain system designed for a five-year frequency discharge may outfall into a major channel associated with a much larger watershed. The two independent events affecting the design are the storm occurring on the small storm drain system and the storm contributing to discharge in the larger watershed.

The simultaneous occurrence of two independent events is defined as the product of the probability of the occurrence of each of the individual events. In other words, if the events are independent, the probability of five-year events occurring on the storm drain and the larger watershed simultaneously is $(0.2)^2$ or 0.04 or 4 percent. This is equivalent to a 25-year frequency.

In ordinary hydrologic circumstances, particularly with adjacent watersheds, flood events are not entirely independent. The "Frequencies for Coincidental Occurrence" table presents suggested frequency combinations for coincidental occurrence. Each design contains two combinations of frequencies; for instance, a five-year design with watersheds of 100 acres (or 1 km^2, that is, 1,000,000 m^2) and one acre (one hectare , that is, 10,000 m^2) that is, 100:1--can employ either of the following scenarios:

- a two-year design on the main stream and a five-year design on the tributary
- a five-year design on the main stream and a two-year design on the tributary.

The largest structure required to satisfy both frequency combinations is the five-year design.

Frequencies for Coincidental Occurrence

Area ratio	**2-year design**		**5-year design**	
	main stream	tributary	main stream	tributary
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
	10-year design		25-year design	
	main stream	tributary	main stream	tributary
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10

Frequencies for Coincidental Occurrence

Area ratio	**2-year design**		**5-year design**	
1:1	10	10	25	25
	10	10	25	25
	50-year design		100-year design	
	main stream	tributary	main stream	tributary
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

Rainfall versus Flood Frequency

Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input, with the basic assumption that the flood frequency and the rainfall frequency are the same. Depending on antecedent soil moisture conditions and other hydrologic parameters, this may not be true. For projects on small basins (under 10 sq. mi.) it is usually not practicable to distinguish between rainfall frequency and runoff frequency due to lack of available data.

Section 4
Hydrologic Method Selection

Method Selection

In general, follow these guidelines.

- Compare results from several methods.
- Use the discharge that appears to best reflect local project conditions. Averaging of results of several methods is not recommended.
- Document reasons supporting the selection of the results.

The peak discharge is adequate for design of conveyance systems such as storm drains, open channels, culverts, and bridges. However, if the design necessitates flood routing through areas such as storage basins, complex conveyance networks, and pump stations, a flood hydrograph is required.

Hydrologic Methods

Countless hydrologic methods are available for estimating peak discharges and runoff hydrographs. The omission of other methods from this manual does not necessarily preclude their use. Determine which method seems to be the most reasonable for the specific situation. Here are some of the most widely used methods:

- Rational Method
- NRCS Runoff Curve Number Methods
- Statistical analysis of stream data
- Regional regression equations.

Rational Method. The Rational Method provides estimates of peak runoff rates for small urban and rural watersheds of less than 200 acres (80 hectares) and in which natural or man-made storage is small. It is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes. See Section 6 for more information on The Rational Method.

NRCS Runoff Curve Number Methods. The Natural Resources Conservation Service (formerly Soil Conservation Service) developed the runoff curve number method as a means of estimating the amount of rainfall appearing as runoff. Technical Release 20 (TR 20) employs the Runoff Curve Number Method and a dimensionless unit hydrograph to provide estimation of peak discharges and runoff hydrographs from complex watersheds. The procedure allows you to estimate the effect of urbanization, channel storage, flood control storage, and multiple tributaries. Apply TR 20 to the design of culverts, bridges, detention ponds, channel modification, and analysis of flood control reservoirs. Technical Release 55 (TR 55) is a simplified form of TR 20 for use estimating peak discharges for small watersheds (urban and rural) whose time of concentration does not exceed 10 hours. TR 55 includes a hydrograph development procedure; however, where hydrograph determination is necessary, use TR 20 or another hydrograph procedure. See Section 7 for more information on the NRCS Runoff Curve Number Methods.

Statistical Analysis of Stream Gauge Data. Statistical analysis of stream gauge data provides peak discharge estimates using annual peak stream flow data. The method is particularly useful where long records (in excess of 25 years) of stream gauge data are available at or near to and on the same stream as the structure site. See Section 10 for more information on statistical analysis of stream gauge data.

Regional Regression Equations. Regional regression equations provide estimates of peak discharge for watersheds in specific geographic regions. See Section 11 for more information on regional regressional methods and equations.

Figure 5-3 provides a flowchart that you may use to help select an appropriate hydrologic method. You can use this at your discretion; however, you should ensure that the conditions in the watershed conform to the limitations of the selected hydrologic method.

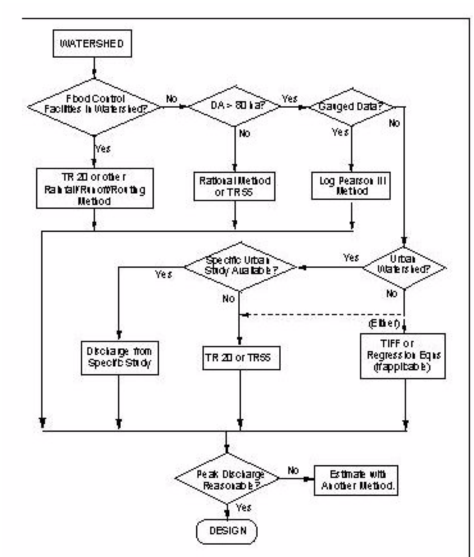

Figure 5-3. Hydrologic Method Selection Chart

Section 5

Time of Concentration

Description

Several common hydrologic methods require an estimation of the time of concentration. This section provides guidance on ways to estimate time of concentration and covers the following topics: description, flow components, and procedure to estimate time of concentration. For additional information on time of concentration, refer to the TR55. You may use other published methods at your discretion subject to the documented limitations of the methods.

Time of Concentration

The time of concentration (t_c) is the time at which the entire watershed begins to contribute to runoff; this is calculated as the time taken for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. Use of the rational formula requires the time of concentration for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). There may be a number of possible paths to consider in determining the longest travel time. Identify the flow path along which the longest travel time is likely to occur. This is a trial and error process.

Generally, it is reasonable to consider three following components of flow that can characterize the progression of runoff along a travel path: overland flow (sheet flow), shallow concentrated flow, and conduit and open channel flow (or concentrated channel flow).

One way to estimate the overland flow time is to use Figure 5-4 through Figure 5-7 to estimate overland flow velocity for a chosen path length. The path length divided by the velocity yields a travel time. For design conditions that do not involve complex drainage conditions, use Figure 5-4 and Figure 5-5. This method is most appropriate for distances of up to 525 ft. (160 m) over open paved and grassed areas such as parking lots, roadways, verges, and landscaped areas.

For each drainage area, determine the distance (L) from the outlet of the drainage area to the most remote point. Determine the average slope (S) for the same distance. Refer to Section 6 for discussion of the runoff coefficient(C).

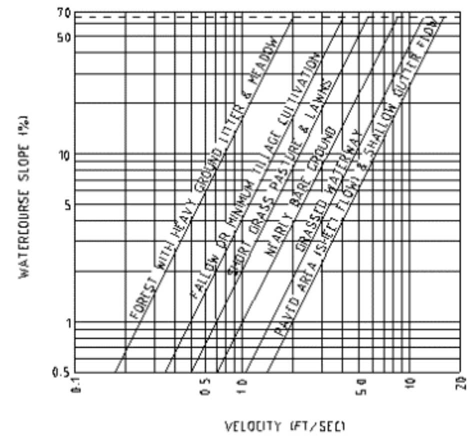

Figure 5-4. Velocities for Upland Method of Estimating Time of Concentration--English (Adapted from the National Engineering Handbook Volume 4)

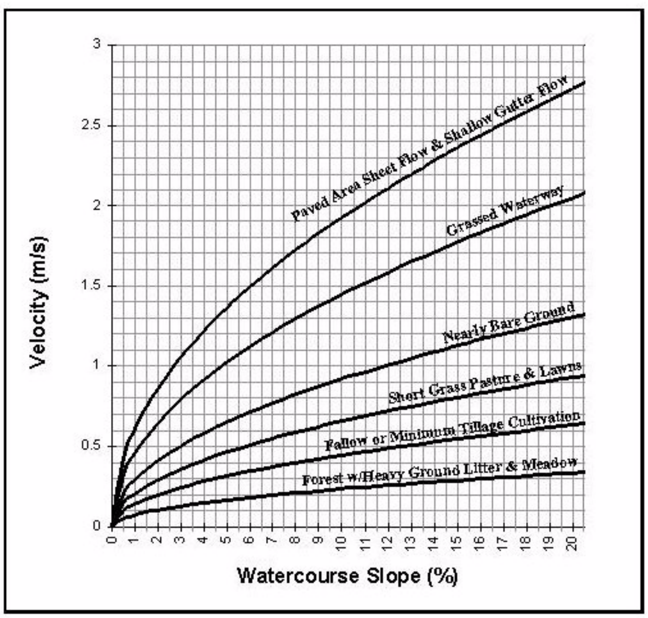

Figure 5-5. Velocities for Upland Method of Estimating Time of Concentration--Metric (Adapted from the National Engineering Handbook Volume 4)

For simplicity, you might employ Figure 5-4 and Figure 5-5 for shallow flow in gutters and swales. Alternatively, you might employ the method outlined in the following paragraphs.

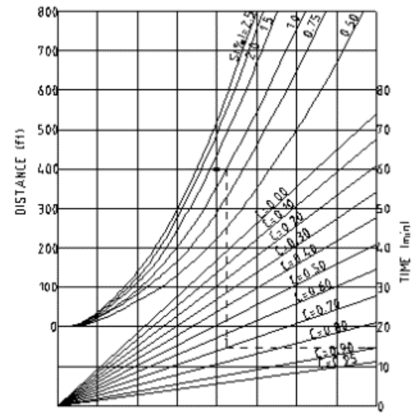

Figure 5-6. Overland Time of Flow--English (Adapted from Airport Drainage, Federal Aviation Administration, 1965)

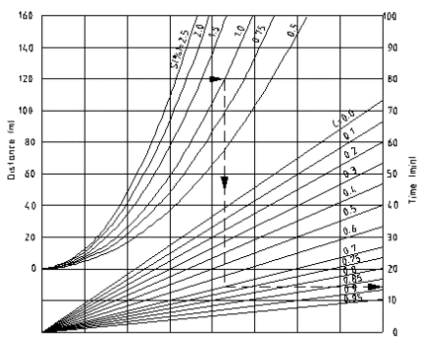

Figure 5-7. Overland Time of Flow--Metric (Adapted from Airport Drainage, Federal Aviation Administration, 1965)

You can estimate pipe or open channel flow time from the hydraulic properties of the conduit or channel. Generally, for department application, it is reasonable to assume uniform flow and employ Manning's Equation for Uniform Flow with the following. open channel and conduit flow considerations.

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 other methods that employ 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.

Procedure to Estimate Time of Concentration

Use the following procedure for estimating time of concentration:

1. Divide the flow path into reach lengths along which flow conditions remain reasonably consistent. Characterize the progression of runoff along a travel path as either overland (or sheet) flow, shallow concentrated flow, or concentrated channel or conduit using the table titled Characterizing Runoff Progression.
2. For each identified reach length, estimate the travel time using a method that is appropriate for the flow conditions. The Flow Conditions and Travel Time Methods table provides general guidance. Compute the time for each component reach using Equation 5-1.
3. Determine the total time. Add the individual travel times to determine the total time. The total time is given in Equation 5-2.
4. Choose an alternate flow path and repeat steps 1 and 2, as necessary.
5. Select the path that results in the longest time. This is the time of concentration (t_c), that is, $t_c =$ T (max), but TxDOT recommends a minimum time of concentration of 10 minutes. If t_c is less than 10 minutes, use 10; otherwise, use the actual t_c.

Characterizing Runoff Progression

Natural Drainage Areas	Flow Type
upper reaches	overland (or sheet) flow transitions to shallow concentrated
lower, larger reaches	concentrated flow in swales, ditches, creeks, and rivers

Flow Conditions and Travel Time Methods

If flow is:	...and the drainage conditions are:	...then the suggested method is:
overland	simple drainage conditions like open paved, grassed areas	Figure 5-4
shallow concentrated	gutters and swales	Velocities for Upland Method of Estimating Time of Concentration--Metric (Adapted from the National Engineering Handbook Volume 4) Figure 5-4
conduit and open channel	assume uniform flow	Manning's Equation with considerations

$$t_n = \frac{L_n}{60 v_n}$$

Equation 5-1.

where:

t_n = travel time over nth reach (min)

L_n = length of nth reach along flow path (ft. or m)

v_n = estimated flow velocity for nth reach (fps or m/s)

$$\mathbf{T} = \sum_{n=1}^{m} \boldsymbol{t}_n$$

Equation 5-2.

where:

T = total time along flow path (min)

M = number of reaches in flow path

n = reach number

Peak Discharge Adjustments

In some cases, runoff from a portion of the drainage area that is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, you can adjust the drainage area and time of concentration by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different contributing areas and associated times of concentration to determine the design flow that is critical for a particular application.

Overland Flow Path Selection

In drainage system design, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often, the land will be graded and swales and streets will intercept the flow that reduces the time of concentration. Exercise care in selecting overland flow paths in excess of 200 ft. (60 m) in urban areas and 400 ft. (120 m) in rural areas.

Section 6
The Rational Method

Introduction

The Rational Method was first introduced in 1889. Although it is often considered simplistic, it still is appropriate for estimating peak discharges for small drainage areas of up to about 200 acres (80 hectares) in which no significant flood storage appears.

Assumptions of the Rational Method

The rate of runoff resulting from any constant rainfall intensity is maximum when the duration of rainfall equals the time of concentration. That is, if the rainfall intensity is constant, the entire drainage area contributes to the peak discharge when the time of concentration has elapsed. This assumption becomes less valid as the drainage area increases. For large drainage areas, the time of concentration can be so large that the assumption of constant rainfall intensities for such long periods is not valid, and shorter more intense rainfalls can produce larger peak flows. Additionally, rainfall intensities usually vary during a storm. In semi-arid and arid regions, storm cells are relatively small with extreme intensity variations.

The frequency of peak discharge is the same as the frequency of the rainfall intensity for the given time of concentration. Frequencies of peak discharges depend on the following:

- rainfall frequencies
- antecedent moisture conditions in the watershed
- the response characteristics of the drainage system.

For small, mostly impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics are the primary influence on frequency. For drainage areas with few impervious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

The rainfall intensity is uniformly distributed over the entire drainage area. In reality, rainfall intensity varies spatially and temporally during a storm. For small areas, the assumption of uniform distribution is reasonable. However, as the drainage area increases, it becomes more likely that the rainfall intensity will vary significantly both in space and time.

The fraction (C) of rainfall that becomes runoff is independent of rainfall intensity or volume. The assumption is reasonable for impervious areas, such as streets, rooftops, and parking lots.

For pervious areas, the fraction of runoff varies with rainfall intensity, accumulated volume of rainfall, and antecedent moisture conditions. Thus, the art necessary for application of the Rational

Method involves the selection of a coefficient that is appropriate for storm, soil, and land use. By limiting the application of the Rational Method to 200 acres (80 hectares), these assumptions are more likely to be reasonable.

Applicability

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream and to provide storm water quality improvement. The Rational Method severely limits the evaluation of design alternatives available in urban and, in some instances, rural drainage design because of its inability to accommodate the presence of storage in the drainage area. When accommodation of any appreciable storage features in the drainage area is required, employ runoff hydrograph methods such as the NRCS Dimensionless Unit Hydrograph method.

The Rational Method Equation

The Rational formula estimates the peak rate of runoff at any 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 time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as Equation 5-3:

$$Q = \frac{CIA}{360}$$

Equation 5-3.

where:

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

C = runoff coefficient as outlined in Runoff Coefficient below

I = average rainfall intensity (in./hr. or mm/hr.) as outlined in Rainfall Intensity below

A = drainage area (ac. or ha)

360 = conversion factor for use only with metric measurements.

Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in in./hr (or mm/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 5-4, which is known as a rainfall intensity-duration-frequency (IDF) relationship.

$$\mathbf{I} = \frac{\boldsymbol{b}}{(\boldsymbol{t}_c + \boldsymbol{d})^e}$$

Equation 5-4.

where:

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

tc = time of concentration (min) as discussed in Section 5

e, b, d = coefficients for specific frequencies listed by county in the Rainfall Intensity-Duration-Frequency Coefficients. These are based on rainfall frequency-duration data contained in the National Weather Service Technical Paper 40 (TP 40).

The general shape of a rainfall intensity-duration-frequency curve is shown in Figure 5-8. As rainfall duration tends towards zero, the rainfall intensity tends towards infinity. Because the rainfall intensity/duration relationship is accessed 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 when using the coefficients presented in the Hydrology document. As the duration tends to infinity, the design rainfall tends towards zero. Usually, the area limitation of 200 acres (80 hectares) should result in design 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.

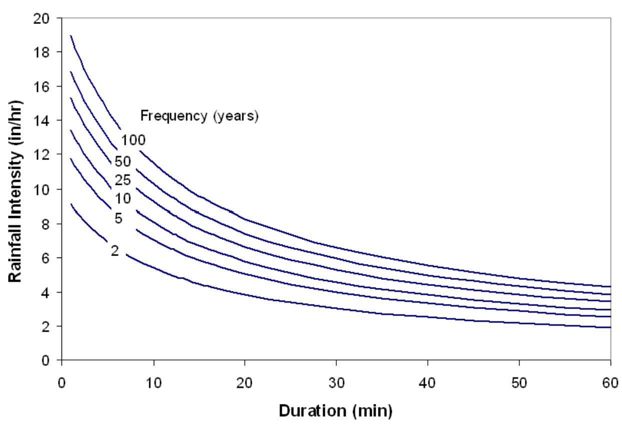

Figure 5-8. Typical Rainfall Intensity Duration Frequency Curve

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.

Runoff Coefficient

The assignment of the runoff coefficient (C) is somewhat subjective. At the time the rainfall producing runoff occurs, the coefficient varies with topography, land use, vegetal cover, soil type, and moisture content of the soil. In selecting the runoff coefficient, consider the future characteristics of the watershed. If land use varies within a watershed, you must consider watershed segments individually, and you can calculate a weighted runoff coefficient value.

The following table suggests ranges of C values for various categories of ground cover. This table is typical of design guides found in civil engineering texts dealing with hydrology. You must subjectively assign a C value based on what you see or anticipate in the watershed with reference to the table.

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

Runoff Coefficients for Urban Watersheds

Type of Drainage Area	Runoff Coefficient
◆ 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
◆ 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

The following table 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, make a systematic assignment of a runoff coefficient "component." Using Equation 5-5, add the four assigned components to form an overall runoff coefficient for the specific watershed segment.

$C = C_r + C_i + C_v + C_s$

Equation 5-5.

Runoff Coefficient for Rural Watersheds

	Extreme	High	Normal	Low
Relief - C_f	*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 negligble 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 plan 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 over 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 culitvated crops	0.04-0.06 good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover
Surface - C_s	0.10-0.12 negligible; surface depression 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 storage lakes and ponds and marshes	0.04-0.06 much surface storage, drainage system not sharply defined; large floodplain storage of large number of ponds or marshes
NOTE: The total runoff coefficient based on the four runoff components is $C = C_r + C_i + C_v + C_s$				

Runoff coefficients, listed in for urban and rural watersheds and others apply to storms of two-year, five-year, and 10-year frequencies. Higher frequency storms require modifying the runoff coefficient because infiltration and other abstractions have a proportionally smaller effect on runoff. Adjust the runoff coefficient by the factor C_f as indicated in the table titled Runoff Coefficient Adjustment Factors for Rational Method. The product of C and C_f should not exceed 1.0.

Runoff Coefficient Adjustment Factors for Rational Method

Recurrence Intervals (years)	C_f
25	1.1

Runoff Coefficient Adjustment Factors for Rational Method

Recurrence Intervals (years)	C_f
50	1.2
100	1.25

The Rational formula now becomes Equation 5-6.

$$Q = \frac{CC_f IA}{360}$$

Equation 5-6.

where:

360 = for metric calculations only

Rational Procedure

The following procedure outlines the Rational method for estimating peak discharge:

1. Determine the watershed area in acres (hectares).
2. Determine the time of concentration, with consideration for future characteristics of the watershed.
3. Assure consistency with the assumptions and limitations for application of the Rational Method.
4. Determine the rainfall IDF coefficients. Extract the Rainfall Intensity-Duration Frequency Coefficients e, b, and d values from the list in Hydrology according to the locality in Texas and the design frequency.
5. Use Equation 5-4 to calculate the rainfall intensity in in./hr (mm/hr).
6. Select or develop appropriate runoff coefficients for the watershed. Where the watershed comprises more than one characteristic, you must estimate C values for each area segment individually. You may then estimate a weighted C value using Equation 5-7. The runoff coefficient is dimensionless.

$$C = \frac{\sum_{n=1}^{m} C_n A_n}{\sum_{n=1}^{m} A_n}$$

Equation 5-7.

where:

C = weighted runoff coefficient

n = n^{th} subarea

m = number of subareas

Cn = runoff coefficient for n^{th} subarea

An = n^{th} subarea size (ha)

7. Calculate the peak discharge for the watershed for the desired frequency using Equation 5-6.

Section 7

NRCS Runoff Curve Number Methods

Introduction

The department has adopted the following two specific runoff determination techniques developed by the U.S. Department of Agriculture and Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS):

- graphical peak discharge (TR 55) procedure
- NRCS dimensionless unit hydrograph.

The procedures presented here are derived from the NRCS National Engineering Handbook, Section 4 and Hydrology for Small Urban Watersheds, TR55.

NRCS Runoff Curve Aspects

The techniques require basic data similar to that used in the Rational Method. However, the NRCS approach is more sophisticated in that it considers the following:

- time distribution of rainfall
- initial rainfall losses to interception and depression storage
- an infiltration rate that decreases during the course of a storm.

NRCS methods produce the direct runoff for a storm, either real or fabricated, by subtracting infiltration and other losses from the total rainfall using a method sometimes termed the Runoff Curve Number Method.

The primary input variables for the NRCS methods are as follows:

- drainage area size (A) in square miles (square kilometers)
- time of concentration (T_c) in hours
- weighted runoff curve number (RCN)
- rainfall distribution (NRCS Type II or III for Texas)
- total design rainfall (P) in inches (millimeters).

NRCS Rainfall-Runoff Equation. Equation 5-8 represents a relationship between accumulated rainfall and accumulated runoff. This was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. Data for land treatment measures, such as contouring and terracing, from experimental watersheds were included.

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

Equation 5-8.

where:

R = accumulated direct runoff (in. or mm)

P = accumulated rainfall (potential maximum runoff) (in. or mm)

Ia = initial abstraction including surface storage, interception, and infiltration prior to runoff (in. or mm)

S = potential maximum retention (in. or mm).

You may compute the potential maximum retention (S) using Equation 5-9:

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

Equation 5-9.

where:

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

RCN = runoff curve number described below.

Equation 5-9 is valid if 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 one-day storm rainfall. Generally, I_a may be estimated as the following:

$$I_a = 0.2S$$

Equation 5-10.

Substituting this in Equation 5-8 gives:

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

Equation 5-11.

Accumulated Rainfall (P)

For most highway drainage design purposes, you may abstract the accumulated rainfall from Technical Paper 40 (NWS, 1961) for a 24-hour duration storm for the relevant frequency. The data for

24-hour two, five, 10, 25, 50, and 100-year frequencies for Texas counties are presented in the 24-Hour Rainfall Depth versus Frequency Values for Texas Counties.

Rainfall Distribution

Figure 5-9 shows two design dimensionless rainfall distributions for Texas: Type II and Type III. Figure 5-10 shows the areas in Texas to which these distribution types apply. The distribution represents the fraction of accumulated rainfall (not runoff) accrued with respect to time. The differences between Type II and Type III are minimal. Additional information is provided in the NRCS 24 Hour Rainfall Distributions subsection of Section 8.

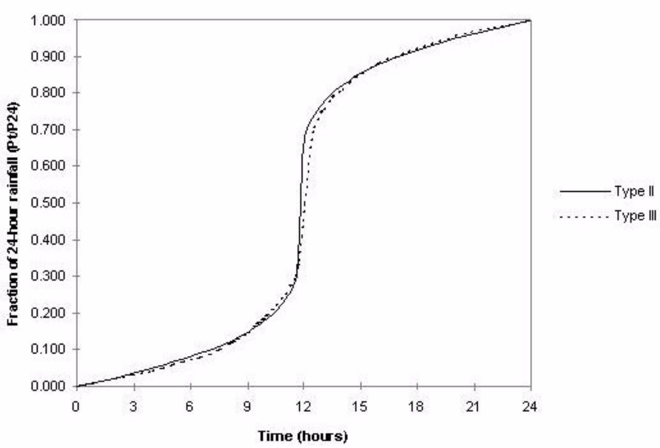

Figure 5-9. Soil Conservation Service 24-hour Rainfall Distributions - Adapted from TR55 (1986, pp. B-1)

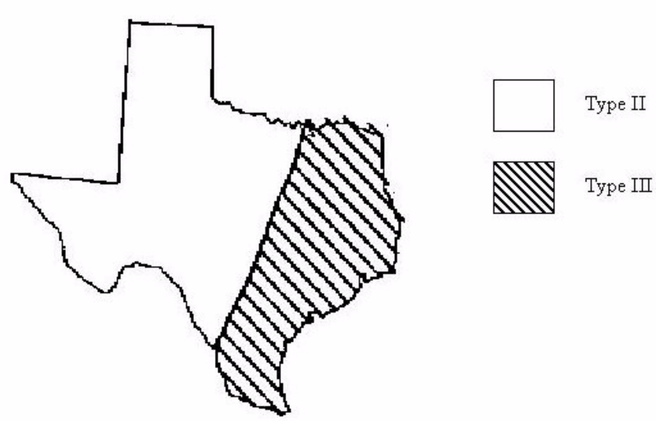

Figure 5-10. Rainfall Distribution Types in Texas

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). Remember to consider effects of urbanization on soil groups as well.

Group A. Group A soils have a low runoff potential due to high infiltration rates even when saturated (0.30 in/hr to 0.45 in/hr or 7.6 mm/hr to 11.4 mm/hr). These soils primarily consist of deep sands, deep loess, and aggregated silts.

Group B. Group B soils have a moderately low runoff potential due to moderate infiltration rates when saturated (0.15 in/hr to 0.30 in/hr or 3.8 mm/hr to 7.6 mm/hr). These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures (shallow loess, sandy loam).

Group C. Group C soils have a moderately high runoff potential due to slow infiltration rates (0.05 in/hr to 0.5 in/hr or 1.3 mm/hr to 3.8 mm/hr if saturated). These soils primarily consist of soils in which a layer near the surface impedes the downward movement of water or soils with moderately fine to fine texture such as clay loams, shallow sandy loams, soils low in organic content, and soils usually high in clay.

Group D. Group D soils have a high runoff potential due to very slow infiltration rates (less than 0.05 in./hr or 1.3 mm/hr if saturated). These soils primarily consist of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, shallow soils over nearly impervious parent material such as soils that swell significantly when wet or heavy plastic clays or certain saline soils.

Effects of Urbanization. Consider the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, you should make appropriate changes in the soil group selected.

Runoff Curve Number (RCN)

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 factors known as runoff curve numbers. These represent the runoff potential of an area when the soil is not frozen. The higher the RCN, the higher the runoff potential. The following tables provide an extensive list of suggested runoff curve numbers. The RCN values assume medium antecedent moisture conditions (RCN II).

If necessary, adjust the RCN for wet or dry antecedent moisture conditions. Use a five-day period as the minimum for estimating antecedent moisture conditions. 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. Equation 5-12 adjusts values for expected dry soil conditions (RCN I). Use Equation 5-13 to accommodate wet soils (RCN III). For help determining which moisture condition applies, see the table titled Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons.

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

Equation 5-12.

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

Equation 5-13.

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.)					
◆ 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-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
◆ Commercial and business	85	89	92	94	95

Runoff Curve Numbers for Urban Areas

Cover Type and Hydrologic Condition	Average Percent Impervious Area	A	B	C	D
◆ Industrial	72	81	88	91	93
Residential districts by average lot size:					
◆ 1/8 acre or less (town houses)	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 areas 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 RCNs.
Other assumptions are: impervious areas are directly connected to the drainage system, impervious areas have a RCN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

Runoff Curve Numbers for Cultivated Agricultural Land1

Cover Type	Treatment2	Hydrologic Condition3	A	B	C	D
Fallow	Bare soil		77	86	91	94
	Crop residue	Poor	76	85	90	93
	cover (CR)	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

Runoff Curve Numbers for Cultivated Agricultural Land1

Cover Type	Treatment2	Hydrologic Condition3	A	B	C	D
		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	SR	Poor	66	77	85	89
or broadcast		Good	58	72	81	85
Legumes or C		Poor	64	75	83	85
Rotation		Good	55	69	78	83
Meadow	C&T	Poor	63	73	80	83
		Good	51	67	76	80

Notes: 1 Values are for average runoff condition, and $I_a = 0.2S$.

[2] Crop residue cover applies only if residue is on at least 5 percent of the surface throughout the year.

[3] 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 percent), 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.

Runoff Curve Numbers for Other Agricultural Lands

Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 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 Fair Good	48 35 30	67 56 48	77 70 65	83 77 73
Woods – grass combination (orchard or tree farm)	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods	Poor Fair Good	45 36 30	66 60 55	77 73 70	83 79 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: RCNs shown were computed for areas with 50 percent grass (pasture) cover. Other combinations of conditions may be computed from RCNs for woods and pasture.

Woods: Poor is forest litter, small trees, and brush destroyed by heavy grazing or regular burning. Fair is woods grazed but not burned and with some forest litter covering the soil. Good is woods protected from grazing and with litter and brush adequately covering soil.

Runoff Curve Numbers for Arid and Semi Arid Rangelands

Cover Type	Hydrologic Condition	A	B	C	D
Herbaceous—mixture of grass,	Poor		80	87	93
weeds, and low-growing brush,	Fair		71	81	89
with brush the minor element	Good		62	74	85
Oak-aspen—mountain brush	Poor		66	74	79
mixture of oak brush, aspen,	Fair		48	57	63

Runoff Curve Numbers for Arid and Semi Arid Rangelands

Cover Type	Hydrologic Condition	A	B	C	D
mountain mahogany, bitter brush,	Good		30	41	48
maple, and other brush					
Pinyon-juniper—pinyon, juniper,	Poor		75	85	89
or both; grass understory	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-	Poor	63	77	85	88
bush, blackbrush, bursage, palo	Fair	55	72	81	86
verde, mesquite, and cactus	Good	49	68	79	84
Notes. Values are for average runoff condition, and $I_a = 0.2S$. Hydrologic Condition: Poor is <30% ground cover (litter, grass, and brush overstory), air is 30% to 70% ground cover, Good is >70% ground cover. Curve numbers for Group A have been developed only for desert shrub.					

Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons

Antecedent Condition	Description	Growing Season 5-Day Antecedent Rainfall	Dormant Season 5-Day Antecedent Rainfall
Dry AMC I	An optimum condition of watershed soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes pace	Less than 1.4 in. or 35 mm	Less than 0.05 in. or 12 mm
Average AMC II	The average case for annual floods	1.4 in. to 2 in. or 35 to 53 mm	0.5 to 1 in. or 12 to 28 mm
Wet AMC III	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2 in. or 53mm	Over 1 in. or 28 mm

Graphical Peak Discharge (TR 55) Procedure

You can use this method of peak discharge determination for relatively homogeneous watersheds with a maximum time of concentration of 10 hours (600 minutes). In a similar fashion to the Rational Method, if soils and land use vary, you should subdivide the watershed. Precipitation records published in TP-40 and an assumed rainfall distribution are used to construct a synthetic storm. You should not use the method for runoff amounts of less than 1.5 in. (38 mm) and runoff curve numbers of less than 60. Additionally, the range of curve numbers should be small (say 20 percent) to reasonably conform to the assumption of homogeneity. A detailed description of this method appears in *Urban Hydrology for Small Watersheds* (TR-55). See References for information on obtaining this document. Use the following procedure to determine the graphic peak discharge:

1. Determine the drainage area (A) in square miles (square kilometers).
2. Determine the soil classification based on runoff potential (Type A, B, C, or D) as described in the Soil Groups paragraphs. One approach for a general classification is to determine the soil name and type from NRCS soil maps or reports.
3. Determine the antecedent soil moisture conditions (AMC).
4. Classify the hydrologic condition of the soil cover. Classify the hydrologic condition of the soil cover as good, fair, or poor. For more information, refer to the footnotes on the tables for Urban Areas tables, Cultivated Agricultural Land, Other Agricultural Lands, and Arid and Semi Arid Rangelands.
5. Determine the RCN for the AMC II soil classification. Determine the runoff curve number (RCN) for the particular soil classification for an AMC II. If appropriate, adjust for AMC I or AMC III using Equation 5-12 and Equation 5-13, respectively. If necessary, determine a weighted value by dividing the sum of the products of the subarea sizes and RCNs by the total area. This process is similar to the weighting of runoff coefficients in the Rational Method. However, the runoff factors are not directly related.
6. Estimate the watershed time of concentration in hours (Tc).
7. Determine the potential maximum storage (S). Use Equation 5-9 to calculate the potential maximum storage.
8. Determine the initial abstraction (Ia). These are the losses that occur before runoff begins and include depression storage, interception, and infiltration. Use Equation 5-10 to calculate Ia. If Ia is greater than P, it is possible that the rainfall event would not produce runoff (which would be unusual for design frequencies). The abstraction equation may need modification, or an alternate means of estimating this value may be necessary, although no specific research has been performed to determine such adjustments.
9. Determine the rainfall distribution type based on the location of the watershed. Use Figure 5-10 to determine the rainfall distribution type based on the watershed location (Type II or Type III for Texas).

10. Determine the total rainfall (P) for watershed location. Based on the design frequency and the 24-Hour Rainfall Depth versus Frequency Values for Texas Counties to determine P for the watershed location.
11. Determine the accumulated direct runoff. Use Equation 5-11 to compute R. This value, when multiplied by the watershed area, will indicate the total volume of the rainfall that appears as runoff.
12. Determine the unit peak discharge. Refer to Equation 5-14 and the table that follows it with the relevant distribution type from step 10 to determine the unit peak discharge (q_u) using time of concentration (T_c) and the ratio I_a/P. If I_a/P is outside the bounds of the tables, use a more precise method that emulates the NRCS method, such as TR 20 or HEC-HMS.

$$q_u = \left(10^{C_0 - 3.36609}\right) \cdot \left(T_c^{\,C_1 + C_2 \log T_c}\right)$$

Equation 5-14.

where:

q_u = unit peak discharge (cfs/sq.mi./in. or $m^3/s/km^2/mm$)

T_c = time of concentration (hours)

Coefficients for Equation 5-14

Rainfall Type	**I_a/P**	**C_0**	**C_1**	**C_2**
II	0.1	2.5532	-0.6151	-0.164
	0.3	2.4653	-0.6226	-0.1166
	0.35	2.419	-0.6159	-0.0882
	0.4	2.3641	-0.5986	-0.0562
	0.45	2.2924	-0.5701	-0.0228
	0.5	2.2028	-0.516	-0.0126
III	0.1	2.4732	-0.5185	-0.1708
	0.3	2.3963	-0.512	-0.1325
	0.35	2.3548	-0.4974	-0.1199
	0.4	2.3073	-0.4654	-0.1109
	0.45	2.2488	-0.4131	-0.1159
	0.5	2.1777	-0.368	-0.0953

13. Determine the pond adjustment factor (F). Use the following table to determine F. This adjustment is to account for pond or swamp areas within the watershed that do not interfere with the time of concentration flow path.

Ponding Adjustment Factor

% Ponded/Swamp Area	Factor (F)
0	1
0.2	0.97
1	0.87
3	0.75
5	0.72
NOTE: This factor is not intended to replace a hydrograph routing technique where considerable detention storage is present (typically, with surface area of ponding in excess of 5 percent of the watershed area).	

14. Compute the peak discharge (Q). Use Equation 5-15 to compute Q:

$$\mathbf{Q = q_u\, ARF}$$

Equation 5-15.

where:

Q = peak discharge (cfs or m^3/s)

q_u = unit peak discharge (cfs/sq.mi./in. or $m^3/s/km^2/mm$) from step 12

A = drainage area (sq.mi. or km^2) from step 1

R = runoff volume (in. or mm) from step 11

F = ponding factor from step 13

NRCS Dimensionless Unit Hydrograph

In many instances for highway drainage design, peak discharge methods will suffice for runoff estimation. However, the estimation of runoff hydrographs may be necessary for situations such as detention pond design, reservoir routing, or channel routing, especially for larger areas and those in which watershed conditions cannot be considered homogeneous. Many hydrograph methods are available and not specifically excluded for use by the department. However, the NRCS Dimensionless Hydrograph Method is incorporated here due to its relative ease of use.

A unit hydrograph represents the time distribution of flow resulting from one in. (mm) of direct runoff occurring over the watershed in a specified time. You plot the NRCS dimensionless unit hydrograph in terms of the ratio of time over time to peak. A curvilinear dimensionless unit hydrograph is shown in Figure 5-11.

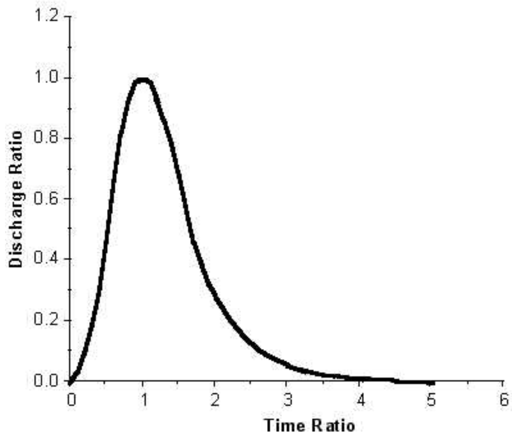

Figure 5-11. NRCS Dimensionless Curvilinear Unit Hydrograph

For hand computations, a triangular hydrograph is reasonable, as shown in Figure 5-12.

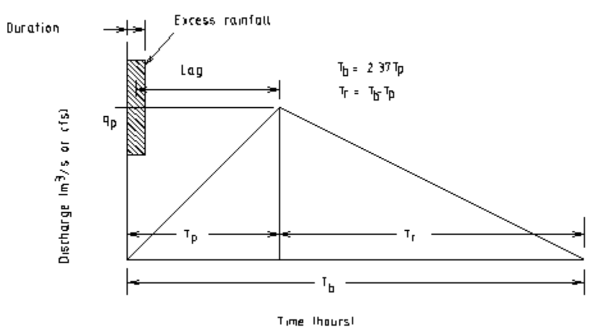

Figure 5-12. Triangular Unit Hydrograph

Triangular Hydrograph. The triangular hydrograph is a practical representation of excess runoff with one rise, one peak, and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the processes of estimating discharge rates. NRCS developed Equation 5-16 to estimate the peak rate of discharge for an increment of runoff.

$$\mathbf{q}_p = \frac{\mathbf{0.208Aq}}{\mathbf{T}_p}$$

Equation 5-16.

where:

q_p = peak rate of discharge (cfs or m^3/s)

0.208 = peak rate factor

A = area (sq. mi. or km^2)

q = storm runoff during time interval (in. or mm) = 1 in. or mm for unit hydrograph

T_p = time to peak runoff (hours), which is estimated using Equation 5-17

$$\mathbf{T_p = \frac{d}{2} + 0.6\,T_c}$$

Equation 5-17.

where:

T_P = time of concentration (hours)

d = duration of unit excess rainfall (hours)

Equation 5-18 provides an estimate of the duration of unit excess rainfall (d).

$$\mathbf{d = 0.133\ T_c}$$

Equation 5-18.

You can use Equation 5-16 to estimate the peak discharge for the unit hydrograph. You can then estimate the shape of the unit hydrograph derived with reference to Figure 5-9 or Figure 5-10. The peak rate factor of 0.208 is valid for the NRCS dimensionless unit hydrograph. Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and, therefore, a change in the peak rate factor. This constant has been known to vary from about 0.258 in steep terrain to 0.129 in very flat, swampy country.

More detail on the NRCS dimensionless hydrograph method is provided in the NRCS National Engineering Handbook, Volume 4. See U.S. Department of Agriculture for information on obtaining this document.

Dimensionless Unit Hydrograph Characteristics. Dimensionless unit hydrograph characteristics vary with the size, shape, and slope of the tributary drainage area.

Lag Time and Peak Characteristics. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge (q_p) for a given rainfall (see Figure 5-12). Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak.

Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high. Flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

Flood Hydrograph Determination Procedure

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

Use the following procedure for determining a flood hydrograph from a dimensionless unit hydrograph:

1. Determine the following parameters for each unit hydrograph: drainage area or subarea size, A, (sq. mi. or km^2), time of concentration (T_c--hrs), weighted runoff curve number (RCN), rainfall distribution type using Figure 5-10, and accumulated rainfall--P--(in. or mm) for design and check flood frequencies.
2. Determine the unit hydrograph variables: Determine the duration of excess rainfall (runoff) using Equation 5-18. For convenience, round d such that the actual duration of precipitation is a whole number times d. For example, if d is 0.332 hours for a 24 hour precipitation, then 24/ 0.332 = 72.29; use 72 in which case, d = 24/72 = 0.333 hours (20 min). Then calculate the time to peak of unit hydrograph (U.H.), T_p, using Equation 5-17. Then compute the peak runoff ordinate, q_p, for the unit hydrograph using Equation 5-16 and = 1 in. (1 mm). Finally, develop a table of the unit hydrograph ordinates using time step increments (d):
   - At each time n * d, where n is the time step, determine the time ratio (t/T_p):

$$\left(\frac{t}{T_p}\right) = \frac{nd}{T_p}$$

Equation 5-19.

   - Use the dimensionless hydrograph (curvilinear as appears in Figure 5-11, or triangular as appears in Figure 5-12) to find the discharge ratio (q/Q_p) at this time ratio.
   - Calculate the discharge at this time step using Equation 5-20.

$$q = q_p \left(\frac{q}{Q_p}\right)$$

Equation 5-20.

   - Repeat a, b, and c for each time step. The resulting table represents the runoff from 1.0 in. (1.0 mm) of rainfall excess occurring during a time of d hours.
   - Use Equation 5-21 to check the volume under the resulting hydrograph. The result should be 1.0, reflecting the 1 in. (1 mm) of runoff from the entire drainage area. Rounding of the unit duration, d, and the likelihood that T_p will not be an integer multiple of d will often result in a volume slightly higher or lower than 1. If so, adjust all the ordinates proportionally until the resulting volume is 1.

$$VOL = \frac{3.6d \times \sum(\text{hydrograph ordinates})}{A}$$

Equation 5-21.

3. Develop a runoff (excess rainfall) table. Referring to Figure 5-9 or Figure 5-10, develop a table of accumulated rainfall, P, for the appropriate distribution type and use a time increment of d hours. Determine the fraction of total rainfall. Use Figure 5-9 to determine the fraction of total rainfall. Multiply the total rainfall by the 24-hour precipitation. Calculate the accumulated runoff (R). On the same table, calculate R, using the estimated RCN, and Equation 5-9 and Equation 5-11. If, for any time interval, $P - 0.2S < 0$, then $R = 0$. Calculate the incremental runoff for each time step. Make the calculation as the difference between the current accumulated runoff and the accumulated runoff from the previous time step.
4. Compute the hydrographs resulting from each increment of runoff. For each incremental runoff, multiply the ordinates of the unit hydrograph by the increment of runoff using the same time step, d. This will result in as many hydrographs as there are increments of runoff, each of which should be displaced by the duration time (d) from the previous hydrograph.
5. At each time step, sum all the runoff values to yield the composite runoff hydrograph. This step is often termed convolution. The resulting hydrograph for the watershed or subarea may serve as an inflow hydrograph for channel or storage routing procedures.

Complex Watersheds

For complex watersheds, subdivide the area, develop runoff hydrographs for each sub-area, and perform combinations of flood routing and channel routing.

If appraisal of the effect of storage is required, such as for detention pond design, you may apply the resulting hydrographs to flood-routing techniques such as appear in Section 9, Flood Hydrograph Routing Methods.

You may use other superposition or tabular methods for the convolution process. However, you are expected to use computer spreadsheets or programs for large computations, and the basic theory remains the same.

Section 8
Design Rainfall Hyetograph Methods

Use of the Rainfall Hyetograph

A rainfall hyetograph is a graphical representation of the variation of rainfall depth or intensity with time. Rainfall-runoff hydrograph methods require a description of this variation. It is possible to use actual rain gauge data in rainfall-runoff models if the data are recorded using a small enough time period (such as 15-minute increments). Often such data are not readily available.

For design, the use of a single measured rainfall event without consideration of other events is not practical because storms vary considerably from event to event with no probability of occurrence established.

Storm Distributions

In the Rational method the intensity is considered to be uniform over the storm period. Unit hydrograph techniques, however, can account for variability of the intensity throughout a storm although the overall depth for a storm will be the same for a given duration for each method. Therefore, when using unit hydrograph techniques, determine a rainfall hyetograph or distribution. The NRCS Type II and II distributions are examples of standardized distributions that are available for use. These two distributions are typically described in either an incremental or accumulative rainfall format, usually in 15-minute increments. In addition, they are also considered to be dimensionless. That is, they represent a distribution of one inch of rainfall over a 24-hour period to which a design (frequency) 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 table below represents the NRCS 24-hour Type II and III distributions.

NRCS 24-Hour Rainfall Distributions

Time, t (hours)	Fraction of 24-hour Rainfall	
	Type II	Type III
0	0.000	0.000
2	0.022	0.020
4	0.048	0.043
6	0.080	0.072
7	0.098	0.089
8	0.120	0.115

NRCS 24-Hour Rainfall Distributions

Time, t (hours)	Fraction of 24-hour Rainfall	
8.5	0.133	0.130
9	0.147	0.148
9.5	0.163	0.167
9.75	0.172	0.178
10	0.181	0.189
10.5	0.204	0.216
11	0.235	0.250
11.5	0.283	0.298
11.75	0.357	0.339
12	0.663	0.500
12.5	0.735	0.702
13	0.772	0.751
13.5	0.799	0.785
14	0.820	0.811
16	0.880	0.886
20	0.952	0.957
24	1.000	1.000

The duration and temporal arrangement of the NRCS 24-hour Type II and III distributions may not always be statistically appropriate for some local conditions or basin sizes, in which case a site-specific rainfall distribution and duration may be necessary. For some sites it may also be necessary to relocate the maximum period of rainfall intensity within the distribution to reflect local conditions such as orographic effects.

Storm Duration

Selecting storm duration is the first step in storm modeling. The determination of appropriate rainfall duration depends on several factors. The first consideration is technical. Except for historical analysis, the minimum required storm duration for a basin model must be equal to or greater than the time of concentration of the total (undivided) watershed. (This is the fundamental basis of the Rational method.) This requirement is necessary to assure that a full runoff response from the basin is achieved.

The second consideration is statistical. Although more formal research is required, shorter duration rainfalls are generally more appropriate for application with smaller basins than longer duration storms. Some correlation may exist between storm duration and standard frequencies--that is, short storms may be responsible for producing the runoff for 2- and 5-year events, mid-length storms for the 10- and 25-year events, and longer storms for the 50- and 100-year flood events.

A third consideration is related to standard practice and regulatory preference. A local entity, for example, may prefer the use of specific storm duration based on local experience or a purely arbitrary duration that typically covers all the basin sizes in their jurisdiction. Likewise, a design office may simply prefer a standardized storm for simplicity. The NRCS 24-hour Type II and III distributions generally fall into this category.

A fourth consideration is based on engineering judgment relative to the critical nature of the project and the consequences of failure. Due to the consequences of failure, dams are typically designed to withstand relatively extreme conditions. Therefore, twenty-four hour storm duration is a more appropriate design consideration for a high hazard dam than a three-hour duration event that meets the minimum technical requirement based on the time of concentration.

For TxDOT use the NRCS 24-hour storm is a starting point for analysis. However, if the analysis results appear inconsistent with expectations, site performance, or experience, consider an alternative storm duration.

Consult the Bridge Division Hydraulics Branch for advice.

Depth-Duration-Frequency

The primary and current sources for rainfall depth-duration-frequency (DDF) relationships are:

- Technical Paper No. 40, Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 hours and Return Periods from 1 to 100 Years, U.S. Weather Bureau, 1961.
- NOAA Technical Memorandum NWS HYDRO-35, Five to 60 minute Precipitation Frequency for the Eastern and Central United States, NWS, 1977.
- Technical Paper No. 49, U.S. Weather Bureau, 1964.

Intensity-Duration-Frequency

If only intensity information is available, you can determine the IDF relationships either from IDF curves or by equations typically taking the form of Equation 5-4, where the storm duration (T_d) in minutes is used in place of T_c.

The rainfall depth for the selected intensity and duration is simply:

$D = I\, T_d / 60$

Equation 5-22.

where:

D = rainfall depth (in.)

I = design rainfall intensity (in./hr)

t_d = storm duration (min.)

Example: Determine the 3-hour, 2-year rainfall depth for Coleman County.

From the Rainfall Intensity-Duration-Frequency-Coefficients: e = 0.767, b = 40, and d = 7.6 for the 2-year frequency; T_d = 3 hours or 180 minutes and Equation 5-4.

Therefore: $I = 40/ (180 + 7.6)^{0.767}$

= 0.72 in/hr

…and from Equation 5-22, D = 0.72 x 180 / 60

= 2.16 inches

Standardized Rainfall Hyetograph Development Procedure

Use the following steps to develop a rainfall hyetograph:

1. Determine the rainfall depth (P_d) for the desired design frequency, location, and storm duration.
2. Determine the distribution type. Use Figure 5-8 to determine the distribution type.
3. Select a time increment that divides equally into an hour. Use the same time increment as that used for hydrograph generation. For storm durations of 1 to 24 hours, the increment should not exceed 15 minutes. The storm duration for most TxDOT projects will not exceed 24 hours.
4. Create a table of time and the fraction of rainfall to total t_d rainfall. Interpolate the Rainfall Distributions table for the appropriate distribution type.
5. Calculate the cumulative depth. Multiply the cumulative fractions by the total rainfall depth (from step 1) to get the cumulative depth.
6. Determine the incremental rainfall for each time period by subtracting the cumulative rainfall at the previous time step from the current time step.

A plot of the resulting incremental rainfall versus times represents the rainfall hyetograph.

Standardized Rainfall Hyetograph Example

The following is an example of a rainfall hyetograph for a 25-year, 24-hour storm duration in Harris County using a one-hour time increment for demonstration only.

From the 24-Hour Rainfall Depth Versus Frequency Values: e = 0.724, b = 81, d = 7.7

From Equation 5-4: $I = 81 / (1440 + 7.7)^{0.724} = 0.417$ in./hr

From Equation 5-22: rainfall depth = 0.417 in./hr. x 1440 min. / 60 min./hr. = 10.01 in.

Distribution type (from Figure 5-10) = III

For time = 1 hour:

1. Determine the cumulative fraction by interpolating the NRCS 24-Hour Rainfall Distributions table: $P_1/P_{24} = 0 +(0.02 - 0) * (1 - 0)/(2-0) = 0.01$.
2. The cumulative rainfall is the product of the cumulative fraction and the total 24-hour rainfall: $P_1 = 0.01 * 10.01 = 0.10$ in.
3. The incremental rainfall is the difference between the current and preceding cumulative rainfall values: 0.10 – 0 = 0.10 in.

Repeating the procedure for each time period yields the complete hyetograph ordinates.

The following table presents the calculations. Figure 5-13 shows the resulting hyetograph.

Example of Incremental Rainfall Tabulation (English)

Time	Cum. Fraction	Cum. Rain	Incr. Rain
(hours)	P_t/P_{24}	P_t (in)	(in)
0	0	0	0
1	0.01000	0.10	0.10
2	0.02000	0.20	0.10
3	0.03150	0.32	0.12
4	0.04300	0.43	0.12
5	0.05750	0.58	0.15
6	0.07200	0.72	0.15
7	0.08900	0.89	0.17
8	0.11500	1.15	0.26
9	0.14800	1.48	0.33
10	0.18900	1.89	0.41

Example of Incremental Rainfall Tabulation (English)

Time	Cum. Fraction	Cum. Rain	Incr. Rain
11	0.25000	2.50	0.61
12	0.50000	5.01	2.50
13	0.75100	7.52	2.51
14	0.81100	8.12	0.60
15	0.84850	8.49	0.38
16	0.88600	8.87	0.38
17	0.90375	9.05	0.18
18	0.92150	9.22	0.18
19	0.93925	9.40	0.18
20	0.95700	9.58	0.18
21	0.96775	9.69	0.11
22	0.97850	9.79	0.11
23	0.98925	9.90	0.11
24	1.00000	10.01	0.11

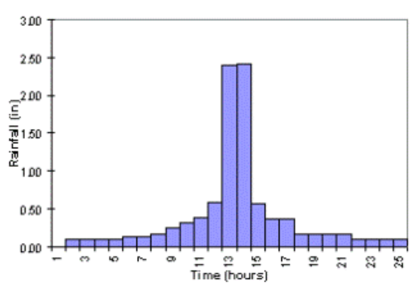

Figure 5-13. Example of Rainfall Hyetograph

Balanced Storm Method for Developing Hyetographs

The Balanced Storm Method (also called alternating block) is a straightforward way of developing hyetographs, especially for rainfall duration of less than 24 hours. You can use the method for the design of storm water detention and retention facilities as well as to investigate the effects of development on runoff volumes and discharges for different scenarios. The method employs the department's intensity-duration-frequency relationship (Equation 5-4).

1. Determine the rainfall intensity coefficients (e, b, and d) for the desired frequency.
2. Establish the desired rainfall duration.

3. Establish a duration interval that divides equally into an hour.
4. Tabulate the duration in increasing values of the interval.
5. Use Equation 5-4 to calculate and tabulate the rainfall intensity.
6. Calculate the cumulative depth for each duration. Multiply the rainfall intensity by the duration.
7. Calculate the incremental rainfall depth for each time period by subtracting the cumulative rainfall at the previous time step from the current time step.
8. Distribute the incremental depth values. Use time blocks that correlate with the duration intervals. Assign the highest incremental depth to the central time block, and arrange the remaining incremental depth blocks in descending order, alternating between the upper and lower time blocks away from the central time block. This is demonstrated in the example that follows.

You may then use the resulting ordinates of the hyetograph as a design rainfall in rainfall-runoff models such as the NRCS dimensionless unit hydrograph method covered earlier in this section. For an example of this distribution method, see Hyetograph Using the Balanced Storm Method.

Section 9
Flood Hydrograph Routing Methods

Introduction

This section presents two ways of routing flood hydrographs: storage (or reservoir) routing and channel routing:

- Use storage routing to account for inflow and outflow rates and significant water storage characteristics associated with reservoirs and detention.
- Use channel routing when known hydrographic data are located somewhere other than the point of interest or the channel profile or plan is changed to alter the natural velocity or channel storage characteristics.

Storage Routing

As a flood hydrograph approaches and passes through a reservoir or detention facility, the characteristics of unsteady flow become significant. You must make an accounting of inflow and outflow rates and water storage characteristics by routing a flood hydrograph through the storage facility.

Reservoir or detention pond storage routing also applies when outflow depends only upon the volume of flood storage. Use storage routing techniques to do the following:

- determine peak discharges from watersheds containing reservoir flood water detention basins and other flow retardation structures
- analyze pump station performance
- specify overtopping flood magnitudes
- evaluate traffic interruption due to roadway overtopping and the associated economic losses

Hydrograph Storage Routing Method Components

Several analytical and graphical methods route flood hydrographs through reservoirs or other detention facilities. All of the methods require reliable descriptions of the following three items:

- an inflow runoff hydrograph for the subject flood
- the storage capacity versus water elevation within the facility
- the performance characteristics of outlet facilities associated with the operation of the facility

By definition, when inflow and outflow from a reservoir (or any type of storage facility) are equal, a steady-state condition exists. If the inflow exceeds the outflow, the additional discharge is stored in the system. Conversely, when the outflow exceeds the inflow, water is taken from storage.

The basic reservoir routing equation is as follows:

In numerical form, this statement of flow continuity can be written in the form of Equation 5-23.

$$\frac{\mathbf{I}_t + \mathbf{I}_{t+1}}{\mathbf{2}} - \frac{\mathbf{O}_t + \mathbf{O}_{t+1}}{\mathbf{2}} = \frac{\mathbf{S}_{t+1} - \mathbf{S}_t}{\Delta \mathrm{T}}$$

Equation 5-23.

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

t = time step number

Various routing methods are useful in specific instances. Some of the more prominent and effective methods are storage-indication, ripple mass curve, and Sorenson graphical.

Storage Indication Routing Method

Of the many methods for routing floods through reservoirs, the Storage-Indication Method is a relatively simple procedure suitable for most highway drainage applications. Since the outflow discharge (O) is a function of storage alone, it is convenient to rewrite the routing equation as Equation 5-24.

$$\frac{\mathbf{2S}_{t+1}}{\Delta \mathrm{T}} + \mathbf{O}_{t+1} = \mathbf{I}_t + \mathbf{I}_{t+1} + \frac{\mathbf{2S}_t}{\Delta \mathrm{T}} - \mathbf{O}_t$$

Equation 5-24.

Relationship Determination

The use of the Storage-Indication Method requires that you determine the relationships among stage, storage, and discharge. This information is in addition to a description of the inflow hydrograph.

The stage-storage relation is simply the volume of water held by the reservoir or storage facility as a function of the water surface elevation or depth. This information is often available from the reservoir sponsor or owner. Where the stage-storage relation is not available, you may need to develop one by successive calculations of storage vs. associated stages in the storage facility.

The stage-outflow relation is based on the association of the reservoir stage (head) and the resulting outflow from the storage facility. This description of performance characteristics may be the following:

- 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

The stage-outflow relation of the outlet works of a reservoir often is available through the reservoir sponsor or owner. In some cases, the highway designer may have developed it.

With stage-storage and stage-outflow relations established, storage and outflow can be related at each stage. The relationship is described in the form of

$$O \text{ vs } \left(\frac{2S}{\Delta T}\right) + O$$

You can plot this relation over the range of anticipated stages. Figures 5-12 (English measurement) and 5-13 (metric) illustrate sample relationships.

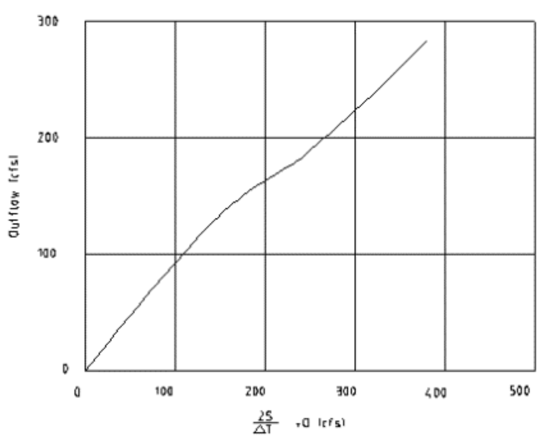

Figure 5-14. Storage Outflow Relation (English)

The form of Equation 5-24 is especially useful because the terms on the left side of the equation are known. With the relation between the outflow and storage determined (Figure 5-14), the ordinates on the outflow hydrograph can be determined directly.

Storage-Indication 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.
2. Acquire or develop a stage-storage relation.
3. Acquire or develop a stage-outflow relationship.
4. Develop a storage-outflow relation curve.
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 5-24.
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 identify success of the process. Use Equation 5-25 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 5-25.

where:

S_r = volume of runoff completely retained (cu. ft. or m^3)

ΣI_t = sum of inflow hydrograph ordinates (cfs or m^3/s)

ΣO_t = sum of outflow hydrograph ordinates (cfs or m^3/s)

There will be no retention volume if the outflow structure is at the flow line of the pond. You can expect a degree of imbalance due to the discretization process. If the difference is large yet the calculations are correct, reduce the time increment (ΔT); determine the inflow hydrograph values for the new time steps, and repeat the routing process.

Channel Routing

Routing of flood hydrographs by means of channel routing procedures is useful in instances where known hydrographic data are at a point other than the point 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 analysis 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).

Total Storage Equation. The Muskingum Method combines a prism component of storage, KO, and a wedge component, KX(I-O), to describe the total storage in the reach as Equation 5-26:

$$S = K\,[XI + (1-X)\,O]$$

Equation 5-26.

where:

S = total storage (cu. ft. or m^3)

K = a proportionality constant representing the time of travel of a flood wave to traverse the reach (s). Oftentimes, 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.

Time Rate of Change Equation. Equation 5-27 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 5-27.

where:

ΔT = time interval usually ranging from 0.3·K to K

t = time step number

Flow-Routing Equation. Applying continuity to Equation 5-28 produces the Muskingum flow routing equation as follows:

$$O_{t+1} = C_1 \cdot I_{t+1} + C_2 \cdot I_t + C_3 \cdot O$$

Equation 5-28.

where:

$$C_1 = \frac{\Delta T - 2KX}{2K(1 - X) + \Delta T}$$

Equation 5-29.

$$C_2 = \frac{\Delta T + 2KX}{2K(1 - X) + \Delta T}$$

Equation 5-30.

$$C_3 = \frac{2K(1 - X) - \Delta T}{2K(1 - X) + \Delta T}$$

Equation 5-31.

By definition, the sum of C_1, C_2, and C_3 should be 1. If measured inflow and outflow hydrographs are available, you may approximate K and X using Equation 5-33. 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.

$$K = \frac{0.5\Delta T[(I_{t+1} + I_t) - (O_{t+1} + O_t)]}{X \cdot (I_{t+1} - I_t) + (1 - X)(O_{t+1} - O_t)}$$

Equation 5-32.

You may also approximate K and X using the Muskingum-Cunge Method described in Chow, 1988; or Fread, 1993.

Section 10

Statistical Analysis of Stream Gauge Data

Stream Gauge Data

Some sites exist where a series of stream flow observations have been made and stream gauge data obtained. You may use these data, with certain qualifications, to develop a peak discharge versus frequency relation for peak runoff from the watershed.

Peak Stream Flow Frequency Relation. Stream gauging stations recording annual peak discharges have been established at 936 stream flow-gauging stations around Texas. If the gauging record covers a sufficient period of time, it is possible to develop a peak stream-flow frequency relation by statistical analysis of the series of recorded annual maximum flows. You can then use such relationships productively in several different ways:

- If the facility site is near the gauging station on the same stream and watershed, you can use the discharge directly for a specific frequency (T-year discharge) from the peak stream flow frequency relationship.
- If the facility site is within the same basin but not proximate to the gauging station, transposition of gauge analysis results is possible.
- If the facility site is not within a gauged basin, you can develop the peak-flow flood-frequency from data from a group of several gauging stations based on either a hydrologic region (e.g., regional regression equations), or similar hydrologic characteristics (e.g., Texas Interactive Flood Frequency Method.)

Curve Development Stipulations. It is possible to develop a peak stream flow versus frequency curve for a site by statistical means provided you meet the following stipulations:

- Sufficient peak discharge sample -- A sufficient statistical sample of annual peak discharges must be available. This usually means a minimum of eight years of data. Some statisticians prefer a sample of 20 or more years. However, 20 years usually is not realistic for available observation periods, and fewer observations are often used as a basis for an analysis.
- No significant change in channel/basin -- No significant changes in the channel or basin should have taken place during the period of record. If significant changes did occur, the resulting peak-stream flow frequency relation could be flawed. The urbanization character of the watershed must not be likely to change enough to affect significantly the characteristics of peak flows within the total time of observed annual peaks and anticipated service life of the highway drainage facility. No means of accommodating future changed characteristics of a watershed within the statistical methods are used in highway hydrology.

- No physical flow regulations existing -- A series of observed data from a watershed within which there have been, are, or will be physical flow regulations is not a sound basis for a hydrologic analysis.
- Data representative of watershed -- The measured data must be representative of the subject watershed, either directly or by inference.

Stream Gauge Record Sources. Generally, for department application, the designer will need to acquire a record of the annual peak flows for the appropriate gauging station. The following sources provide stream gauge records:

- U.S Department of the Interior, United States Geological Survey Water Resources Data—Texas, Surface Water. These are prepared annually and contain records for one water year per publication. As a result, abstracting annual peaks for a long record is time-consuming
- International Boundary and Water Commission water bulletins
- the USGS web site

Applicability and Limitations. For highway drainage purposes, a statistical analysis of stream gauge data is typically applied only in those instances where there is adequate data from stream gauging stations. The definition of adequate data comes from U.S. Geological Survey (USGS) practice and is illustrated in the table below.

Recommended Minimum Stream Gauge Record Lengths

Desired Frequency (Years)	Minimum Record Length (Years)
10	8
25	10
50	15
100	20

If adequate data are not available, base the design peak discharge on analyses of data from several stream flow-gauging stations.

In some cases, a site needing a design peak discharge is on the same stream and near an active or discontinued stream flow-gauging station with an adequate length of record (see the “Recommended Minimum Stream Gauge Record Lengths” table). Currently, the active and discontinued gauging station records for Texas are available for access on the USGS web site for Texas. See U.S. Geological Survey for more information.

Having determined that a suitable stream gauge record exists, you need to determine if any structures or urbanization may be affecting the peak discharges at the design site. Consider the following guidelines:

- Period of record similar to design site -- The period of record for the gauging station's annual peak discharges should represent the same or similar basin conditions as that of the design site. Therefore, you should exclude from the analysis any gauged peak discharges not representing the basin conditions for the design site.
- Factors affecting peak discharge -- The most typical factors affecting peak discharges are regulation by urbanization and reservoirs. Densities of impervious cover less than 10 percent of the watershed area generally do not affect peak discharges. The existence in the watershed of a major reservoir or many smaller reservoirs or flood control structures can greatly affect the runoff characteristics.
- Length of record -- You should adjust the length of record to include only those records that have been collected subsequent to the impoundment of water by reservoirs and subsequent to any major urbanization. If the resulting records then become too short, do not use the procedures in this section.

Log Pearson Type III Distribution and Procedure

- Numerous statistical distribution methods establish peak discharge versus frequency relations. The Log Pearson Type III statistical distribution method has gained the most widespread acceptance and is recommended by the US Water Resources Council Bulletin #17B. An outline of this method follows; however, you are not limited to using only this method, especially if the resulting discharge frequency relation does not seem to fit the data.

The Log-Pearson Type III method for the statistical analysis of gauged flood data applies to just about any series of natural floods. Three statistical moments are involved in the analysis.

- The mean is approximately equal to the logarithm of the two-year peak discharge. (See Equation 5-33)
- The standard deviation can be compared to the slope of the plotted curve. (Although, with the consideration of the third moment, skew, there is no single slope to the curve. See Equation 5-34.)
- The skew represents the form of curvature to the plotted curve.

For a negative skew, the flood-frequency curve is concave (downward), and for a positive skew, the curve is convex (upward). If the skew is zero, the following occurs:

- the plotted relation forms a straight line
- the distribution is defined as normal
- the standard deviation becomes the slope of that straight line

The significance of the skew becomes especially important in the estimation of floods based upon extrapolated curves.

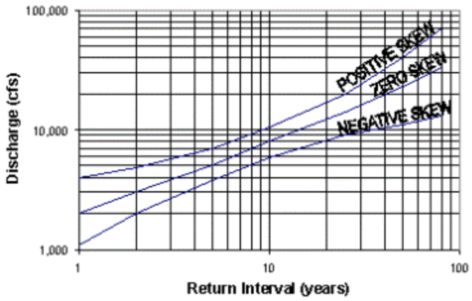

Figure 5-15. Skew of Discharge versus Frequency Plots

Flooding is often erratic in Texas such that 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 you should possibly exclude from the set of data to be analyzed. Additionally, you can make adjustments to incorporate historical data.

The following steps outline the Log-Pearson type III analysis procedure:

1. Acquire and assess the annual peak discharge record. The record should comprise only one discharge (maximum) per year. Note that the USGS water year is October to September.
2. Calculate the logarithm of each discharge value.
3. Use Equation 5-33, Equation 5-34, and Equation 5-35 to calculate the statistics.
4. Use Equation 5-36 to calculate the logarithm of the discharge for each frequency.
5. Plot discharge versus frequency on standard log probability paper.
6. Consider adjusting the calculations to accommodate a weighted skew (see skew, below) and accommodating outliers in the data.

$$\overline{Q}_L = \frac{\sum X}{N}$$

Equation 5-33.

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

Equation 5-34.

$$G_S = \frac{N^2\left(\sum X^3\right) - 3N\left(\sum X\right)\left(\sum X^2\right) + 2\left(\sum X\right)^3}{N(N-1)(N-2)S_L^{\,3}}$$

Equation 5-35.

where:

N = number of observations

X = logarithm of the annual peak discharge

SL = standard deviation of the logarithms of the annual peak discharge

GS = coefficient of skew of log values (station skew).

$$\mathbf{\log Q = \overline{Q}_L + KS_L}$$

Equation 5-36.

where:

-Q_L = mean of the logarithms of the annual peak discharges

Q = flood magnitude (cfs or m^3/s)

K = a frequency factor for a particular return period and coefficient of skew (values of K for different coefficients of skew, G, and return periods are given in Hydrology).

Skew

The three methods for determining the value of the skew coefficient for the Log Pearson Type III curve fit are as follows:

- Gauge data -- Calculate the station skew directly from the gauge data using Equation 5-35. This value may not well represent the skew of the data if the period of record is short or if there are extreme events in the period of record.
- Frequency factor -- Figure 5-16 shows the value of generalized skew coefficients across Texas that you may use to determine the frequency factor (K) in place of the station skew.
- Weighted skew -- You may compute a weighted skew. Refer to Bulletin 17B for the method to compute a weighted skew.

NOTE: The mean square error for the generalized skew is 0.35, which replaces the value of 0.55 presented in Bulletin 17B.

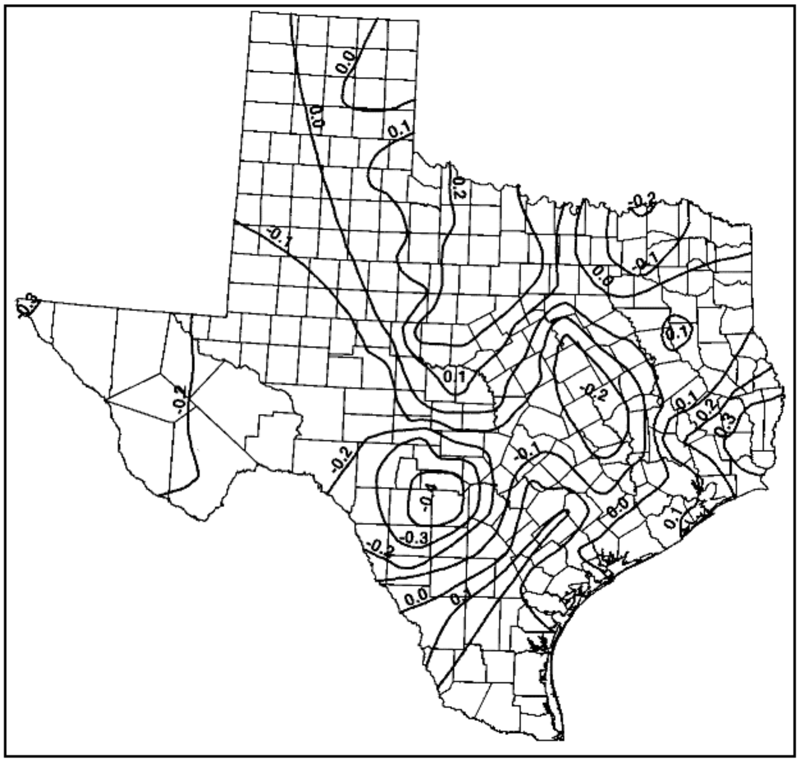

Figure 5-16. Generalized Skew Values for Texas

Accommodating Outliers in the Data

Frequency Curve Shape. The distribution of all the annual and historical peak discharges determines the shape of the 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.

Shape Based on Larger Peaks. Most peak stream flow frequency analyses require the larger recurrence-interval peaks more often than those do for the lower recurrence intervals. Most design peaks, for example, are based on 50-year or 100-year recurrence intervals rather than two-year or five-year intervals. Therefore, it is more desirable to base the shape of the frequency curve on the distribution of the larger peaks. You accomplish this by eliminating from the analyses peak discharges that are 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. You can subjectively choose this value by reviewing the sequentially ranked values for all of the peak discharges used in the analysis.

Example of Low Outliers. For example, the lowest sequentially ranked peak discharges for a station, in cubic feet per second (cfs) or cubic meters per second (m^3/s), are as follows: 0, 10, 25, 90, 450, 495, 630, 800, 1050. The largest difference between sequential values for these discharges is

360 cfs or m^3/s, which is the difference between 90 and 450 cfs or m^3/s. Therefore, the distribution of the peak discharges substantially changes below the value of 450 cfs or m^3/s, which could be used as the low value threshold.

Low-Outlier Threshold Identification. Equation 5-37 provides a means of identifying the low outlier threshold for a set of data as follows:

$$\mathrm{LOT} = 10^{(a\overline{Q}_L + bS_L + cG + d)}$$

Equation 5-37.

where:

LOT = estimated low-outlier threshold (cfs)

Q_L = mean of the logarithms of the annual peak discharge (see Equation 5-33)

S_L = standard deviation of the logarithms of the annual peak discharge (see Equation 5-34)

G = coefficient of skew of log values (station skew, see Equation 5-35).

a = 1.09

b = -0.584

c = 0.140

d = -0.799

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

High-Outlier Threshold Description. High outlier thresholds represent extremely high peak discharges—those with a recurrence interval larger than indicated by the period of record for a station. For example, a 100-year peak discharge could be gauged during a 10-year period of record. The frequency curve thus would be unduly shaped by the 100-year peak.

High-Outlier Identification. The USGS has made efforts to identify high outliers, referred to as historical peaks, by identifying and interviewing long-term 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.

Use of Peak Discharge Table. All known historical peak discharges and their associated gauge heights and dates appear in Hydrology and on the USGS web site.

- You should use the lowest peak discharge identified on this table for each station as the value for the high-outlier threshold.
- You should use the number of years from the highest since (or highest since at least) date to the last year of gauged record as the length of the historical record.
- For some stations, however, a historical-peak discharge may have been gauged without knowledge of its historical significance. When this is suspected for a station, you should review and compare the dates for historical peaks from nearby stations to dates of floods for the suspect station. These dates and historical periods may apply to stations where this information is absent.

Recomputation of Statistics. Having identified appropriate outliers, you should re-compute the statistics (Equation 5-33 through Equation 5-37) using a data set that excludes values beyond the established outlier thresholds.

Transposition of Data

You may estimate peak discharge for sites near gauged sites by transposition of stream gauge data by scaling the discharge by a ratio of the drainage areas raised to an exponent of 0.7. You can best use this method as a check of other methods rather than the primary means of estimating design discharge. Additionally, you can repeat this procedure for each available nearby watershed and average the results. The following presents an example using the results from three sites, as shown in the following table:

Example of Transposition

Watershed	Q_{25} (cfs)	Area (sq. mi.)
Gauged watershed A	62000	737
Gauged watershed B	38000	734
Gauged watershed C	45000	971
Ungauged watershed D	?	450

Example of Transposition

Watershed	**Q_{25} (cfs)**	**Area (sq. mi.)**
Notes: Because Texas gauges use English measurement units, the following examples are offered in English only: Gauged watershed A: $62{,}000(450/737)^{0.7} = 43{,}895 cfs$ Gauged watershed B: $38{,}000(450/734)^{0.7} = 26{,}980 cfs$ Gauged watershed C: $45{,}000(450/971)^{0.7} = 26{,}266 cfs$ Gauged watershed D: (43,895 + 26,980 + 26,266) / 3 = 32,380 cfs.		

Section 11

Regional Regression Methods and Equations

Introduction

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed.

Regression Methods and Equations

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas.

You may have difficulty choosing the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies.

Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

You must exercise care using regression equations in these instances:

- Conduct a field visit to assess the watershed characteristics for comparison with other watersheds.
- Collect all available historical flood data.
- Use the gathered data to interpret any discharge values.

Additional specific regional studies are under development, which may provide lower standard errors. When they are published, use such studies at your discretion.

Regional Regression Equations for Natural Basins

The following equation applies to rural, uncontrolled watersheds. The following figure presents the geographic extents of each region. Four tables, two with English measurement units and two with metric units, present the coefficients and limits of applicability by hydrologic region number, Regions 1-6 and Regions 7-11. Generally, use this equation to compare with the results of other methods, check existing structures, or where it is not practicable to use any other method, keeping in mind the importance of the facility being designed.

$$\mathbf{Q_T = aA^b SH^c SL^d}$$

Equation 5-38.

where:

Q_T = T-year discharge (cfs or m^3/s)

A = contributing drainage area (sq. mi. or km^2)

SH = basin-shape factor defined as the ratio of main channel length squared to contributing drainage area (sq. mi./sq. mi. or km^2/km^2)

SL = mean channel slope defined as the ratio of headwater elevation of longest channel minus main channel elevation at site to main channel length (ft./mi. or m/m). Note: This differs from previous rural regression equations in which slope was defined between points 10 and 85 percent of the distance along the main channel from the outfall to the basin divide.

a, b, c, d = multiple linear regression coefficients dependent on region number and frequency.

Regions 3, 4, 5, 7 and 10 have two sets of coefficients. For these regions, if the drainage area is between 10 and 100 sq. mi. (25 and 250 km^2), determine a weighted discharge (Q_w) as shown in the following equation.

$$Q_w = (2 - \log(A/z))Q_1 + (\log(A/z) - 1)Q_2$$

Equation 5-39.

where:

Q_w = weighted discharge (cfs or m^3/s)

A = contributing drainage area (sq. mi. or km^2)

z = 1.0 for English measurements units, or 2.56 for metric

Q_1 = discharge based on regression coefficients for A < 32 sq. mi. (cfs) or 83 km^2 (m^3/s)

Q_2 = discharge based on regression coefficients for A 32 sq. mi. (cfs) or 83 km^2 (m^3/s).

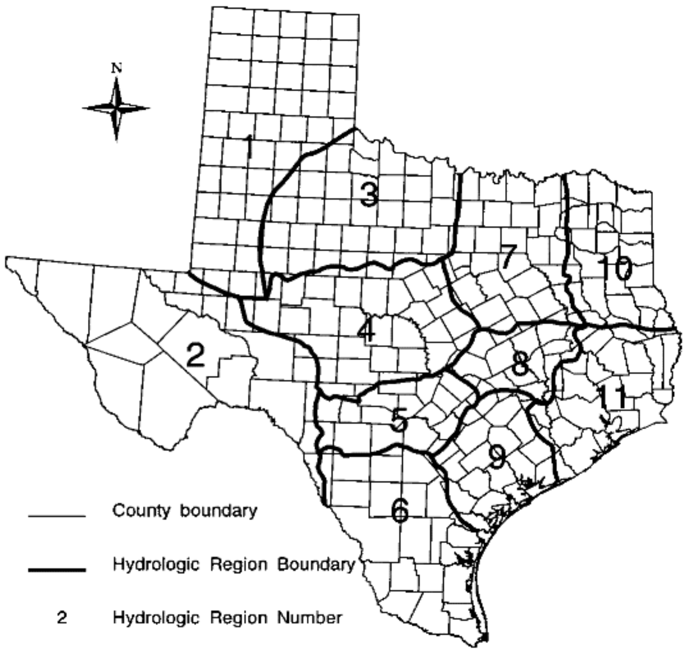

Figure 5-17. Hydrologic Regions for Statewide Rural Regression Equations

Regression Coefficients and Limits for Hydrologic Regions 1-6 (English)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
1	2	16.1	1.04	-0.537	0	A lower:	1.15	160
	5	53.2	0.958	-0.444	0	A upper:	2956	111
	10	96	0.921	-0.4	0	SH lower:	0.11	103
	25	178	0.885	-0.356	0	SH upper:	80.90	103
	50	263	0.864	-0.33	0	SL lower:	2.49	111
	100	371	0.847	-0.307	0	SL upper:	132	120
2	2	826	0.376	0.869	-0.689	A lower:	0.32	120
	5	6500	0.372	0.738	-0.933	A upper:	4305	92
	10	18100	0.369	0.673	-1.05	SH lower:	0.51	88
	25	55300	0.366	0.604	-1.19	SH upper:	14.8	92
	50	108000	0.363	0.566	-1.27	SL lower:	9.67	99
	100	199000	0.361	0.531	-1.34	SL upper:	130	107
3	2	119	0.592	0	0	A lower:	0.1	75
A<32 sq. mi.	5	252	0.629	0	0	A upper:	97.0	78
	10	373	0.652	0	0	SH lower:	0.16	88

Regression Coefficients and Limits for Hydrologic Regions 1-6 (English)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
	25	566	0.679	0	0	SH upper:	9.32	103
	50	743	0.698	0	0	SL lower:	10.7	120
	100	948	0.715	0	0	SL upper:	105	134
3	2	8.05	0.668	0.189	0.659	A lower:	11.8	60
A>=32 sq.mi.	5	42.0	0.626	0	0.574	A upper:	14635.0	57
	10	91.9	0.579	0	0.537	SH lower:	1.71	60
	25	233	0.523	0	0.476	SH upper:	75.00	66
	50	448	0.484	0	0.425	SL lower:	4.81	72
	100	835	0.447	0	0.372	SL upper:	36.3	92
4	2	97.1	0.626	0	0	A lower:	0.19	134
A<32 sq. mi.	5	196	0.65	0.257	0	A upper:	81.1	96
	10	293	0.697	0.281	0	SH lower:	0.05	92
	25	455	0.741	0.311	0	SH upper:	6.52	99
	50	53	0.927	0.333	0.558	SL lower:	13.5	107
	100	51	0.968	0.353	0.627	SL upper:	226	120
4	2	0.0066	1.29	0	2.09	A lower:	12.0	72
A>=32 sq.mi.	5	0.0212	1.24	0	2.18	A upper:	19819	51
	10	0.0467	1.2	0	2.18	SH lower:	0.49	49
	25	0.1020	1.16	0	2.18	SH upper:	19.7	54
	50	0.1660	1.13	0	2.19	SL upper:	3.52	60
	100	0.2520	1.11	0	2.19	SL upper:	36.1	69
5	2	159	0.68	0	0	A lower:	0.18	75
A<32 sq. mi.	5	396	0.773	0	0	A upper:	22.30	63
	10	624	0.82	0	0	SH lower:	0.50	66
	25	997	0.866	0	0	SH upper:	84.90	69
	50	278	0.973	0	0.36	SL lower:	20.9	72
	100	295	1.01	0	0.405	SL upper:	224	78
5	2	377	0.498	0	0	A lower:	45.0	43

Regression Coefficients and Limits for Hydrologic Regions 1-6 (English)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
A>=32 sq.mi.	5	1270	0.534	-0.145	0	A upper:	1861	28
	10	2310	0.552	-0.221	0	SH lower:	3.140	28
	25	4330	0.531	-0.307	0	SH upper:	20.800	31
	50	6450	0.583	-0.366	0	SL lower:	9.86	36
	100	9180	0.594	-0.42	0	SL upper:	48.8	41
6	2	66.2	0.63	-0.423	0	A lower:	0.36	96
	5	931	0.424	0	-0.41	A upper:	15428	60
	10	1720	0.41	0	-0.419	SH lower:	0.011	49
	25	3290	0.398	0	-0.428	SH upper:	10.9	51
	50	4970	0.391	0	-0.434	SL lower:	6.88	63
	100	1780	0.44	0	0	SL upper:	98.9	75

Regression Coefficients and Limits for Hydrologic Regions 7-11 (English)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
7	2	832	0.568	0	-0.285	A lower:	0.2	57
A<32 sq. mi.	5	584	0.61	0	0	A upper:	78.7	46
	10	831	0.592	0	0	SH lower:	0.037	43
	25	1196	0.576	0	0	SH upper:	36.6	46
	50	1505	0.566	0	0	SL lower:	7.25	51
	100	1842	0.558	0	0	SL upper:	116	57
7	2	129	0.578	0	0.364	A lower:	13	66
A>=32 sq.mi.	5	133	0.605	0	0.578	A upper:	2615	54
	10	178	0.644	-0.239	0.699	SH lower:	1.66	51
	25	219	0.651	-0.267	0.776	SH upper:	36.6	51
	50	261	0.653	-0.291	0.817	SL lower:	3.85	54
	100	313	0.654	-0.316	0.849	SL upper:	31.9	60
8	2	30.7	0.672	0	0.652	A lower:	0.75	51

Regression Coefficients and Limits for Hydrologic Regions 7-11 (English)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
	5	87.6	0.668	0	0.520	A upper:	7065	43
	10	134	0.675	0	0.475	SH lower:	1.94	43
	25	191	0.690	0	0.444	SH upper:	24.8	46
	50	229	0.703	0	0.433	SL lower:	3.83	49
	100	261	0.718	0	0.429	SL upper:	39.5	51
9	2	278	0.526	0	0	A lower:	0.24	54
	5	329	0.645	-0.246	0.220	A upper:	5198	49
	10	350	0.691	-0.321	0.343	SH lower:	0.091	46
	25	382	0.743	-0.413	0.466	SH upper:	30.1	49
	50	409	0.778	-0.477	0.541	SL lower:	2.77	49
	100	438	0.811	-0.539	0.607	SL upper:	70	54
10	2	54.9	0.788	0	0.279	A lower:	0.21	54
A<32 sq. mi.	5	80.7	0.835	0	0.330	A upper:	100	40
	10	98.2	0.860	0	0.359	SH lower:	0.008	38
	25	122	0.887	0	0.390	SH upper:	1.050	38
	50	141	0.904	0	0.408	SL lower:	2.0	41
	100	159	0.920	0	0.426	SL upper:	138	43
10	2	16.9	0.798	0	0.777	A lower:	23.4	63
A<32 sq. mi.	5	33.0	0.790	0	0.795	A upper:	6507.0	51
	10	51.3	0.775	0	0.785	SH lower:	1.77	43
	25	87.9	0.752	0	0.760	SH upper:	16.90	38
	50	129	0.733	0	0.735	SL lower:	1.48	36
	100	187	0.713	0	0.708	SL upper:	24.5	36
11	2	159	0.669	-0.262	0	A lower:	0.13	43
	5	191	0.696	-0.186	0.13	A upper:	3636	43
	10	199	0.718	-0.151	0.221	SH lower:	0.082	49
	25	201	0.713	0	0.313	SL lower:	18.8	54
	50	207	0.735	0	0.380	SL lower:	0.38	60
	100	213	0.755	0	0.442	SL upper:	169	66

Regression Coefficients and Limits for Hydrologic Regions 1-6 (Metric)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
1	2	0.1694	1.04	-0.537	0	A lower:	3.0	160
	5	0.6054	0.958	-0.444	0	A upper:	7656.0	111
	10	1.1315	0.921	-0.4	0	SH lower:	0.11	103
	25	2.1712	0.885	-0.356	0	SH upper:	80.90	103
	50	3.2727	0.864	-0.33	0	SL lower:	0.0005	111
	100	4.6920	0.847	-0.307	0	SL upper:	0.0250	120
2	2	0.0445	0.376	0.869	-0.689	A lower:	0.8	120
	5	0.0435	0.372	0.738	-0.933	A upper:	11149.9	92
	10	0.0445	0.369	0.673	-1.05	SH lower	0.51	88
	25	0.0411	0.366	0.604	-1.19	SH upper:	14.80	92
	50	0.0405	0.363	0.566	-1.27	SL lower:	0.0018	99
	100	0.0411	0.361	0.531	-1.34	SL upper:	0.0246	107
3	2	1.9183	0.592	0	0	A lower:	0.3	75
A<83sq.km	5	3.9218	0.629	0	0	A upper:	251	78
	10	5.6791	0.652	0	0	SH lower	0.16	88
	25	8.3991	0.679	0	0	SH upper:	9.32	103
	50	10.8281	0.698	0	0	SL lower:	0.0020	120
	100	13.5939	0.715	0	0	SL upper:	0.0199	134
3	2	34.2746	0.668	0.189	0.659	A lower:	31	60
A>=83sq.km	5	89.8174	0.626	0	0.574	A upper:	37904.6	57
	10	149.6630	0.579	0	0.537	SH lower:	1.71	60
	25	237.2570	0.523	0	0.476	SH upper:	75.00	66
	50	305.7774	0.484	0	0.425	SL lower:	0.0009	72
	100	374.8059	0.447	0	0.372	SL upper:	0.0069	92
4	2	1.5154	0.626	0	0	A lower:	0.5	134
A<83sq.km	5	2.9899	0.65	0.257	0	A upper:	210	96
	10	4.2741	0.697	0.281	0	SH lower:	0.05	92
	25	6.3650	0.741	0.311	0	SH upper:	6.52	99
	50	74.2031	0.927	0.333	0.558	SL lower:	0.0026	107
	100	124.0603	0.968	0.353	0.627	SL upper:	0.0428	120
4	2	3301.6992	1.29	0	2.09	A lower:	31	72
A>=83sq.km	5	24056.4457	1.24	0	2.18	A upper:	51331.2	51
	10	55048.3692	1.2	0	2.18	SH lower:	0.49	49
	25	124899.2199	1.16	0	2.18	SH upper:	19.70	54
	50	227873.0495	1.13	0	2.19	SL lower:	0.0007	60
	100	352574.9115	1.11	0	2.19	SL upper:	0.0068	69

Regression Coefficients and Limits for Hydrologic Regions 1-6 (Metric)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
5	2	2.3572	0.683	0	0	A lower:	0.47	75
A<83sq.km	5	5.3735	0.779	0	0	A upper:	58	63
	10	8.0970	0.829	0	0	SH lower:	0.50	66
	25	12.3829	0.88	0	0	SH upper:	84.90	69
	50	68.2492	0.993	0	0.37	SL lower:	0.0040	72
	100	102.8256	1.03	0	0.4170	SL upper:	0.0424	78
5	2	6.6460	0.498	0	0	A lower:	116.5	43
A>=83sq.km	5	21.6345	0.534	-0.145	0	A upper:	4820.0	28
	10	38.6826	0.552	-0.221	0	SH lower:	3.140	28
	25	71.2097	0.571	-0.307	0	SH upper:	20.800	31
	50	71.2097	0.583	-0.366	0	SL lower:	0.0019	36
	100	104.8700	0.594	-0.42	0	SL upper:	0.0092	41
		147.7025						
6	2	1.0293	0.63	-0.423	0	A lower:	0.9	96
	5	0.5242	0.424	0	-0.41	A upper:	39958.5	60
	10	0.9085	0.41	0	-0.419	SH lower:	0.011	49
	25	1.6273	0.398	0	-0.428	SH upper:	10.900	51
	50	2.3506	0.391	0	-0.434	SL lower:	0.0013	63
	100	33.1599	0.44	0	0	SL upper:	0.0187	75

Regression Coefficients and Limits for Hydrologic Regions 7-11 (Metric)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
7	2	1.1925	0.568	0	-0.285	A lower:	0.5	57
A<83sq.km	5	9.2543	0.61	0	0	A upper:	204	46
	10	13.3959	0.592	0	0	SH lower:	0.037	43
	25	19.5756	0.576	0	0	SH upper:	36.600	46
	50	24.8688	0.566	0	0	SL lower:	0.0014	51
	100	30.6700	0.558	0	0	SL upper:	0.0220	57
7	2	47.7294	0.578	0	0.364	A lower:	33.7	66
A>=83sq.km	5	300.2844	0.605	0	0.578	A upper:	6772.8	54
	10	1092.4994	0.644	-0.239	0.699	SH lower:	1.66	51
	25	2583.4202	0.651	-0.267	0.776	SH upper:	36.60	51
	50	4367.0970	0.653	-0.291	0.817	SL lower:	0.0007	54
	100	6883.4736	0.654	-0.316	0.849	SL upper:	0.0060	60

Regression Coefficients and Limits for Hydrologic Regions 7-11 (Metric)

Region	Freq. (yrs)	a	b	c	d	Limits		Wt % Error
8	2	122.6319	0.672	0	0.652	A lower:	1.9	51
	5	113.3012	0.668	0	0.52	A upper:	18298.3	43
	10	117.0641	0.675	0	0.475	SH lower:	1.94	43
	25	126.1105	0.69	0	0.444	SH upper:	24.80	46
	50	135.9037	0.703	0	0.433	SL lower:	0.0007	49
	100	147.5524	0.718	0	0.429	SL upper:	0.0075	51
9	2	4.7719	0.526	0	0	A lower:	0.6	54
	5	33.2386	0.645	-0.246	0.22	A upper:	13462.8	49
	10	97.1374	0.691	-0.321	0.343	SH lower:	0.091	46
	25	289.5838	0.743	-0.413	0.466	SH upper:	30.100	49
	50	570.3826	0.778	-0.477	0.541	SL lower:	0.0005	49
	100	1042.2502	0.811	-0.539	0.607	SL upper:	0.0133	54
10 A<83sq.km	2	8.0270	0.788	0	0.279	A lower:	0.5	54
	5	17.4696	0.835	0	0.33	A upper:	259	40
	10	26.6161	0.86	0	0.359	SH lower:	0.008	38
	25	42.0374	0.887	0	0.39	SH upper:	1.050	38
	50	55.7797	0.904	0	0.408	SL lower:	0.0004	41
	100	72.2851	0.92	0	0.426	SL upper:	0.0261	43
10 A>=83sq.km	2	174.8257	0.798	0	0.777	A lower:	60.6	63
	5	401.3713	0.79	0	0.795	A upper:	16853.1	51
	10	580.9288	0.775	0	0.785	SH lower:	1.77	43
	25	821.1726	0.752	0	0.76	SH upper:	16.90	38
	50	990.4262	0.733	0	0.735	SL lower:	0.0003	36
	100	1160.9910	0.713	0	0.708	SL upper:	0.0046	36
11	2	2.3820	0.669	-0.262	0	A lower:	0.3	43
	5	8.4989	0.696	-0.186	0.13	A upper:	9417.2	43
	10	18.9170	0.718	-0.151	0.221	SH lower:	0.082	49
	25	42.2410	0.713	0	0.313	SH upper:	18.800	49
	50	75.6556	0.735	0	0.38	SL lower:	0.0001	54
	100	129.9534	0.755	0	0.442	SL upper:	0.0320	60
								66

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:

$$v = \frac{z}{n}R^{\frac{2}{3}}S^{\frac{1}{2}}$$

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.

$$\mathbf{Q = \frac{z}{n} A\, R^{2/3}\, S^{1/2}}$$

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)

$\frac{v^2}{2\,g}$ = 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 (a). 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).

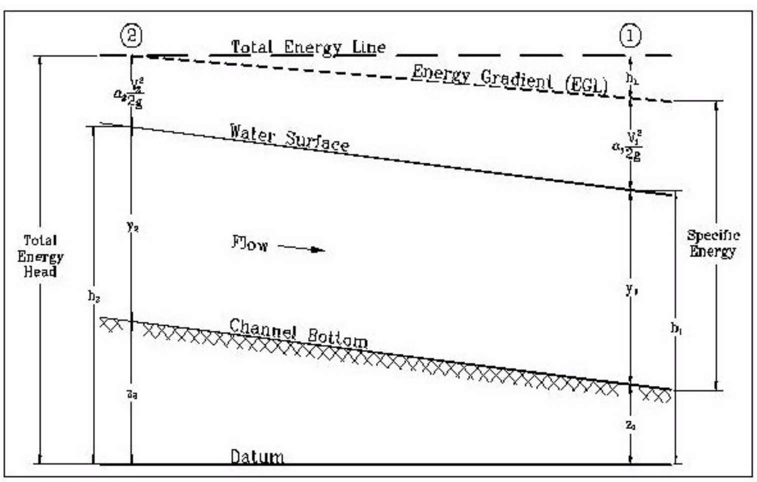

Figure 6-1. EGL for Water Surface Profile

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.

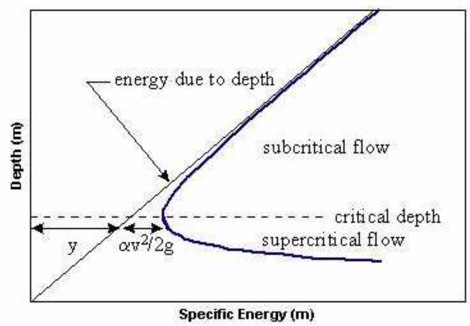

Figure 6-2. Typical Specific Energy Diagram

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.

$$F_r = \frac{v}{\sqrt{g\,d_m}}$$

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 subcritical 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.

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.

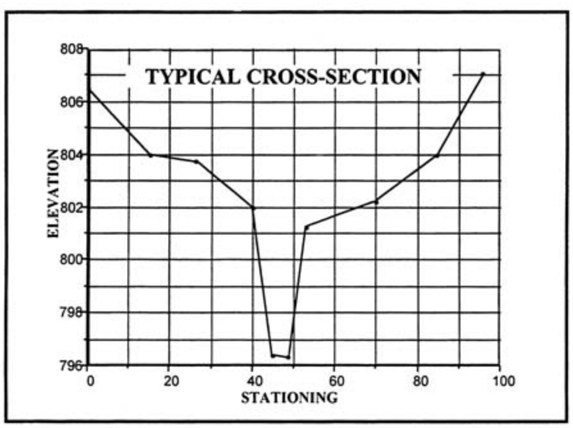

Figure 6-3. Plotted Cross Section

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*. (See Federal Highway Administration for information on obtaining this document.)

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:

$$n_w = \frac{\sum \left(n\ WP\right)}{\sum WP}$$

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.

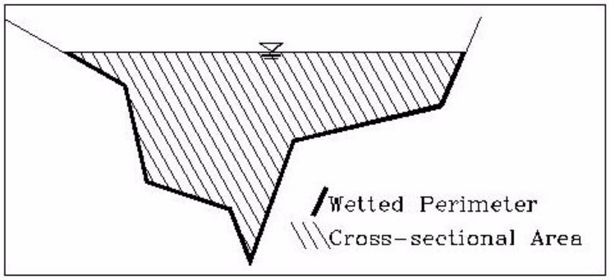

Figure 6-4. Cross Section Area and Wetted Perimeter

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. In other words, the wetted perimeter is considered only along the solid boundaries of the cross section (not along the water interface 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.

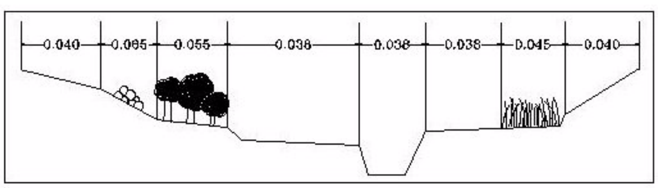

Figure 6-5. Subdividing With Respect to Geometry and Roughness

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.

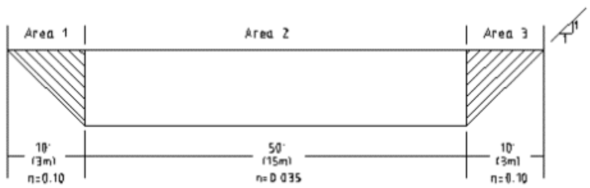

Figure 6-6. Subdivision of a Trapezoidal Cross Section

The conveyance for each subarea is calculated as follows:

$A_1 = A_3 = 50\ ft^2$	$A_1 = A_3 = 4.5\ m^2$
$P_1 = P_3 = 14.14$ 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.

$A_c = A_1 + A_2 + A_3 = 600\ ft^2$	$A_c = A_1 + A_2 + A_3 = 54\ m^2$
$P_c = P_1 + P_2 + P_3 = 78.28$ 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	$K_T = K_1 + K_2 + K_3 = 2768\ m^3/s$
$n = 1.486A_cR_c^{2/3}/K_T = 0.034$	$n = A_cR_c^{2/3}/K_T = 0.034$

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:

$A_1 = 195\ ft^2$	$A_1 = 20\ m^2$
$P_1 = 68\ ft$	$P_1 = 21\ m$
$R_1 = A_1/P_1 = 2.87\ ft$	$R_1 = A_1/P_1 = 0.95\ m$
$K_1 = 1.486A_1R_1^{2/3}/n = 14622.1\ cfs$	$K_1 = A_1R_1^{2/3}/n = 484.0\ m^3/s$
$A_2 = 814.5\ ft^2$	$A_2 = 75.5\ m^2$
$P_2 = 82.5\ ft$	$P_2 = 24.9\ m$
$R_2 = A_2/P_2 = 9.87\ ft$	$R_2 = A_2/P_2 = 3.03\ m$
$K_2 = 1.486A_2R_2^{2/3}/n = 139226.2\ cfs$	$K_2 = A_2R_2^{2/3}/n = 3954.2\ m^3/s$

The effective n-value calculations for the combined subareas are as follows:

$A_c = A_1 + A_2 = 1009.5\ ft^2$	$A_c = A_1 + A_2 = 95.5\ m^2$
$P_c = P_1 + P_2 = 150.5\ ft$	$P_c = P_1 + P_2 = 45.9\ m$
$R_c = A_c/P_c = 6.71\ ft$	$R_c = A_c/P_c = 2.08\ m$
$K_T = K_1 + K_2 = 153848.3\ cfs$	$K_T = K_1 + K_2 = 4438.2\ m^3/s$
$n = 1.486A_cR_c^{2/3}/K_T = 0.035$	$n = A_cR_c^{2/3}/K_T = 0.035$

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.

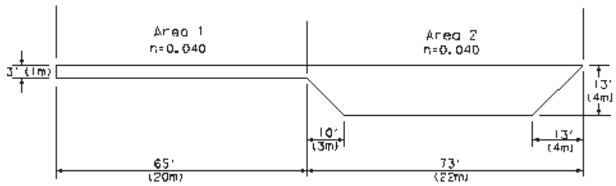

Figure 6-7. Subdividing a "Panhandle" Cross Section

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.

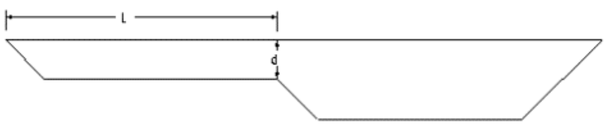

Figure 6-8. Bench Panhandle Cross Section

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.

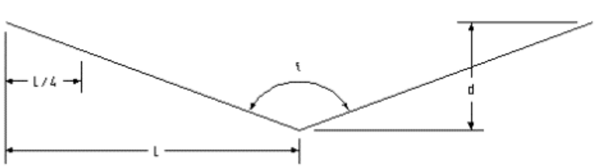

Figure 6-9. Triangular Cross Section

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).

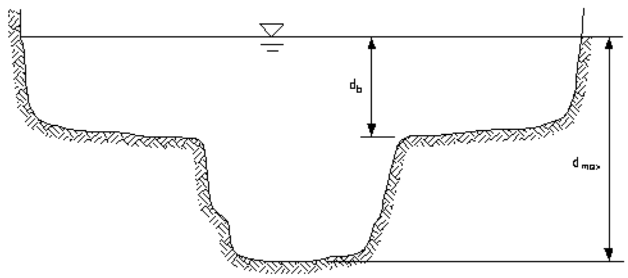

Figure 6-10. Problematic Cross Section

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 wetted perimeter is computed using Equation 6-19. For other shapers, acquire or derive ther 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, hf, as the product of friction slope and length of conduit.
- Consider the kinetic energy coefficient (a) 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.

$$h_v = \left[\frac{v^2}{2g}\right]$$

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 (dc) is higher than uniform depth (du), 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 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 10-11). 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.

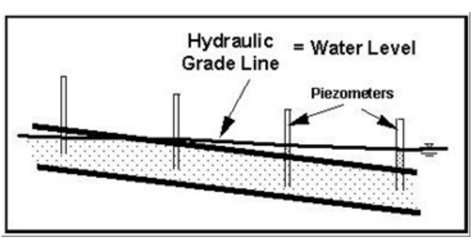

Figure 6-11. Hydraulic Grade Line

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. The Frequencies for Coincidental Occurrence provides a means for selecting an appropriate frequency for the tailwater elevation versus frequency for the storm drain system. This is based on the assumption that the storm causing the tailwater and the storm causing the flow through the storm drain may be neither completely dependent nor completely independent: the larger the ratio of areas contributing to the storm drain and outfall, the less likely the storm events are dependent on each other. Consider the worst conditions resulting from the possible combinations of tailwater frequency and storm drain frequency for design. Alternatively, you may choose a higher tailwater level (frequency) if you think it reasonable. 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 HGL calculation.
2. Compute the friction loss for each segment of the conduit system, beginning with the most downstream run. The friction loss (hf) 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 HGL elevation as a base, add the computed friction loss hf. This will be the tentative elevation of the HGL at the upstream end of the conduit segment.
2. Compare the tentative elevation of the HGL 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 4 above will be the controlling HGL elevation (HGLus)at the upstream node of the conduit run. (If you perform backwater calculations, the computed elevation at the upstream end becomes the HGL 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 (hm) on the HGL.

$$HGl_i = HGL_o + \frac{v_o^2}{2g} + h_m - \frac{v_i^2}{2g}$$

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 HGL for the next conduit run (HGLds) to be the higher of critical depth and the HGL from step 5 (or 6 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 HGL 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 HGL level exceeds a critical elevation, you must adjust the system so that a revised HGL 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 HGL 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 HGL through the system until the HGL level is no higher than the soffit of the conduit. At this point, continuance of the HGL 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 *Environmental Procedures in Project Development Manual* for environmental concerns. (See Texas Parks and Wildlife Department (TPWD), Clean Water Act (CWA), in the Environmental Procedures in Project Development 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 the *Environmental Procedures in Project Development Manual* 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 the *Environmental Procedures in Project Development 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 *Environmental Procedures in Project Development Manual* for more information.) 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.

Stream Channel Criteria

Stream channel criteria include the following:

- Evaluate the hydraulic effects of floodplain encroachments for the peak discharges of the design frequency and the 100-year frequency on any major highway facility.
- Avoid relocation or realignment of a stream channel wherever practicable.
- If you deem relocation necessary, the cross-sectional shape, plan-view, roughness, sediment transport, and slope should conform to the original conditions insofar as practicable.
- You may need 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.

Federal Emergency Management Agency (FEMA) Requirements

Consider FEMA rules and procedures early in the project planning stages.

Program application for permits and approvals by Federal and State agencies having regulatory authority over streams early in the project development process. (See the Project Development Process Manual for more details.) An increasing number of federal and state permits are required for construction activities that may involve navigation and water quality. Authorization of structures for work in navigable waters of the United States is required by Sections 9, 10, and 11 of the River and Harbor Act of 1899 (30 Stat. 1151, 33 U.S.C. 401, 403, and 404), and Section 404 of the Clean Water Act of 1977 (33 U.S.C. 1344). (See the Project Development Process Manual for more details.)

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 Clean Water Act (33 U.S.C. 1341) as amended. (See the Project Development Process Manual for more details.)

Many federal and state agencies have statutory authority to issue permits or approve construction plans for the purposes of erosion and sedimentation control, floodplain management, utilization of natural resources, environmental protection, and coastal zone management.

Assure compliance with our agreements with FHWA, USFWS, Texas Natural Resource Conservation Commission (TNRCC), and Texas Parks and Wildlife Department under the FWCA and each respective Memorandum of Understanding (MOU). Check on jurisdictional status with USACE and USCG. (See References for more information on contacting these agencies and the Project Development Process Manual for more information on policies and coordinating with these agencies.)

Section 3
Roadside Channel Design

Roadside Channels

To a large extent, the geometric safety standards of the project usually constrain the alignment, cross section, and grade of roadside channels. These channels should accommodate the design runoff in a manner that assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects

The roadway cross section usually defines roadside channel side slopes in conformance with the Roadway Design Manual. These considerations and procedures generally apply to median ditches and other small, excavated channels. Roadside channels should accommodate the design discharge in a safe, functional, and economical manner. Their alignment, cross section, and grade are usually constrained by the geometric and safety standards that apply to the highway project.

Local soil conditions, flow depths, and velocities within the channel are usually the primary considerations in channel design; however, terrain and safety considerations have significant influence. Design considerations should include channel safety, shape, and channel linings.

Channels that are safe for vehicles accidentally leaving the traveled way are generally hydraulically efficient. Channel shape is the primary safety criterion. It is recommended that, where terrain permits, roadside drainage channels built in earth should have flattened side slopes and a rounded bottom. A parabolic channel most nearly meets the requirements for safety in roadside ditches. Channels shaped in accordance with this recommendation tend to approach the circular shape, which is known to be the most hydraulically efficient shape for channels.

Channel shapes are generally determined for a particular location by the following considerations of terrain, flow regime, and quantity of flow to be conveyed. Channels located adjacent to roadways (roadside and median ditches) should conform to recommended shapes that will minimize the shock of impact by errant vehicles and provide a traversable section.

A trapezoidal shape is the most economical shape from a material cost standpoint. The use of this shape, however, is normally confined to areas with limited right-of-way. Erosion and silting usually produce an unlined trapezoidal shape. This shape, depending on the dimensions of side slopes and the bottom width, is easily constructed by machinery. Unlined channels are seldom constructed with side slopes steeper than 1-vertical to 3-horizontal. Where local conditions dictate the use of some type of rigid lining, the use of steeper side slopes may be more economical.

Limit V-shaped channels to those that are lined due to the propensity to erode when unprotected.

Channel Linings

Wherever possible, highway drainage channel design should make use of native, natural materials such as grass, crushed rock, and earth. It is often necessary, however, to use other types of materials for reasons of hydraulics, economics, safety, aesthetics, and environment.

Channel lining may be desirable or necessary to minimize maintenance, to resist the erosive forces of flowing water, to increase the velocity and conveyance to improve hydraulic efficiency, and to limit the channel size for right-of-way or safety considerations. Highway drainage channel linings vary in cost, durability, hydraulic roughness, and appearance.

Refer to the FHWA Hydraulic Engineering Circular No. 15 (HEC-15)for comprehensive descriptions, advantages, and disadvantages of different types of channel linings. (See References for information on contacting this agency to obtain this document.)

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.
- They inhibit natural infiltration in locations where infiltration is desirable or permissible. Smooth linings usually cause high flow velocities with scour occurring at the terminus of these sections unless controlled with riprap or other energy dissipating devices
- In areas where water quality considerations are of major concern, contaminants may be transported to the receiving waters where 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

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} (1.486)$$

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)

1.486 = conversion factor for use with English units, only

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.

$$\tau_d = 62.4RS$$

Equation 7-2. (English)

where:

τ_d = maximum shear stress at normal depth (lb./sq.ft.)

R = hydraulic radius (ft.) at ym

S = channel slope (ft./ft.)

$$\tau_d = 9810\ RS$$

Equation 7-3. (Metric)

where:

t_d = maximum shear stress at normal depth (N/m^2)

R = hydraulic radius (m)

S = channel slope (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)
	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)

Retardation Class for Lining Materials

Retardance Class	Cover	Condition
	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.)	tp (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
Gravel, D_{50} = 1 in. or 25 mm	0.40	19
Gravel, D_{50} = 2 in. or 50 mm	0.80	38

Permissible Shear Stresses for Various Linings

Protective Cover	(lb./sq.ft.)	tp (N/m^2)
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, you generally need to 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.

Identify these characteristics and understand the relationship of the actions and reactions of forces tending to effect change. This knowledge enables you 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. 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 you may perform a hydraulic analysis utilizing rigid boundary theory.

Stream Classification

Figure 7-1 illustrates the three main natural channel patterns: straight, braided, and meandering streams.

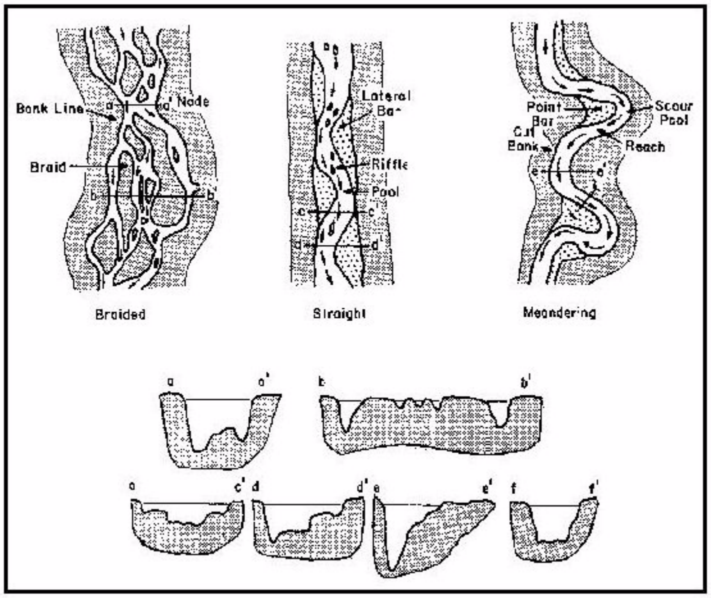

Figure 7-1. Natural Stream Patterns

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.

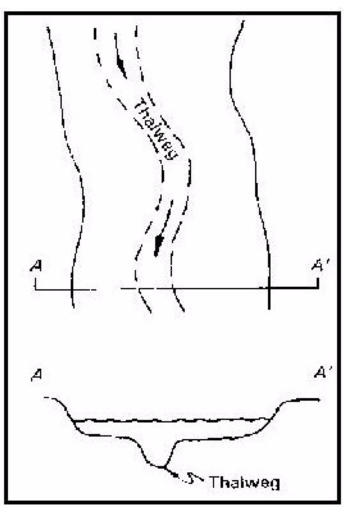

Figure 7-2. Thalweg Location in Plan View and Cross Section

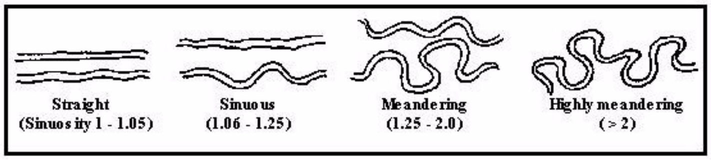

Figure 7-3. Various Degrees of Sinuosity

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 (Figure 7-1).

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.

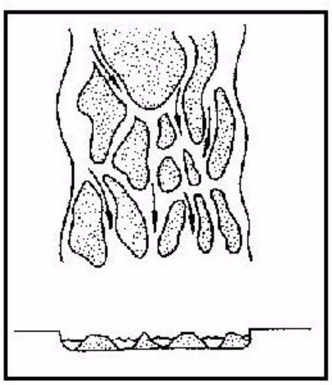

Figure 7-4. Plan View and Cross Section of a Braided Stream

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).

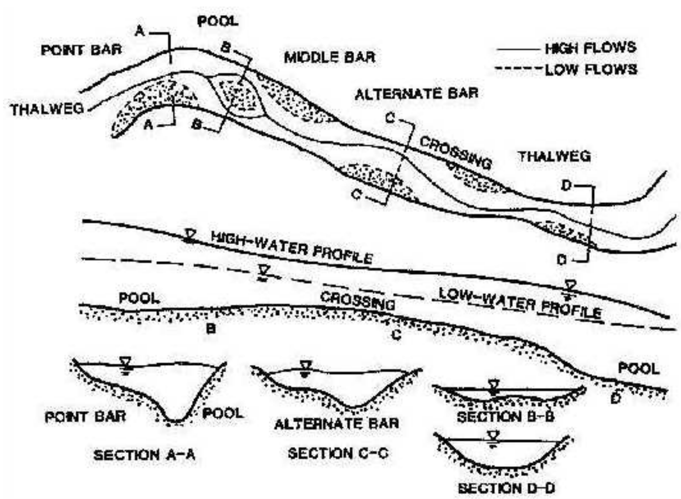

Figure 7-5. Plan View and Cross Section of a Meandering Stream

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).

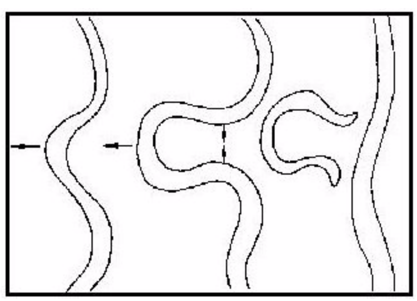

Figure 7-6. Migration Leading to Formation of Oxbow Lake

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.

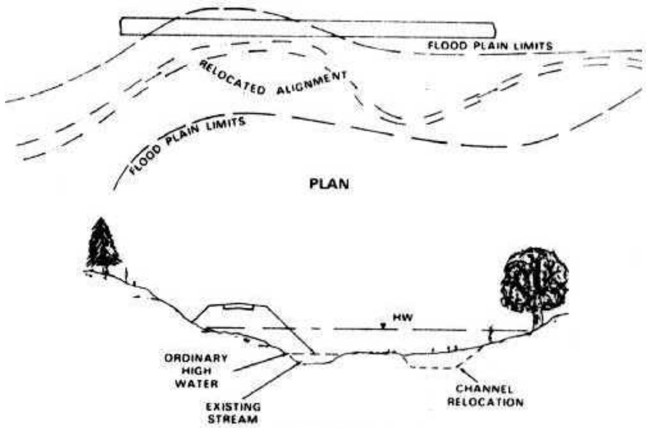

Figure 7-7. Highway Encroachment on Natural Streams and Stream Relocation

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 Bridge 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.

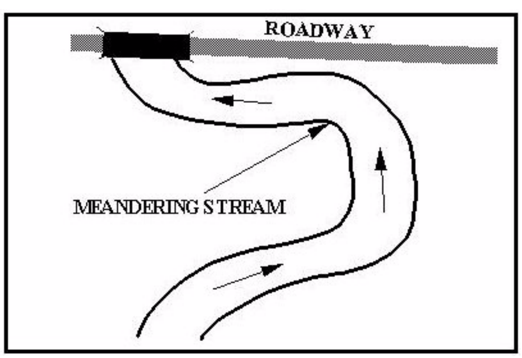

Figure 7-8. Meandering Stream Threatening Bridge and Approach Roadway

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.

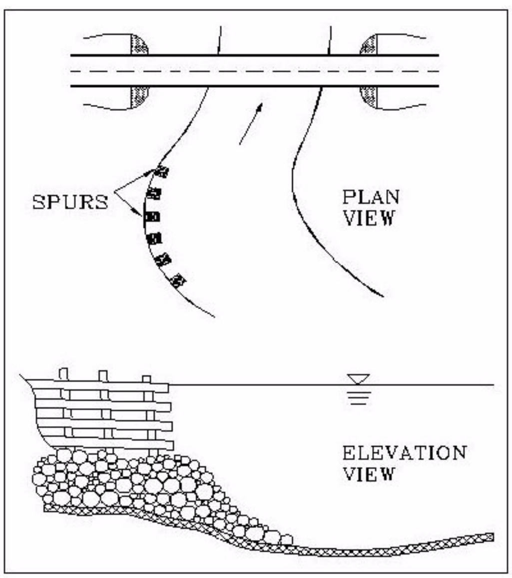

Figure 7-9. Permeable Fence Spurs as Meander Migration Countermeasures

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.

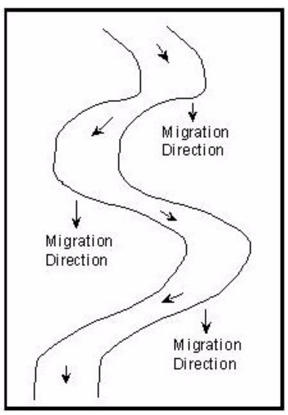

Figure 7-10. Meander Migration in a Natural Stream

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.

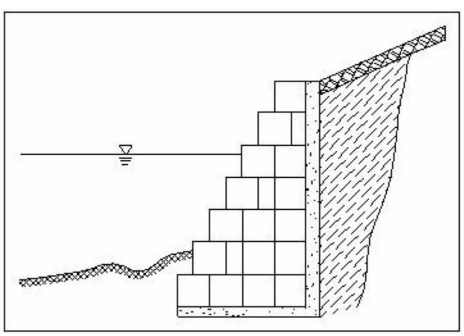

Figure 7-11. Gabions Used as Revetment

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.

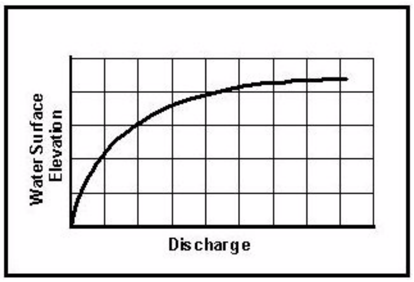

Figure 7-12. Typical Stage Discharge Curve

(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.

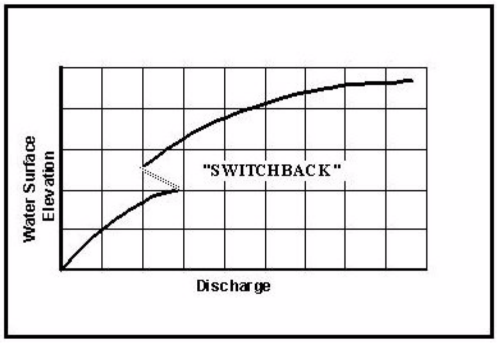

Figure 7-13. Switchback in Stage Discharge Curve

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 and Equation 6-3). 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

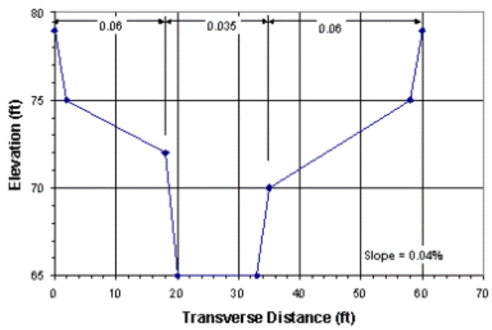

Figure 7-14. Slope Conveyance Cross Section – English Example

Water Ssurface Elevation of 67 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	0	27.37	0	27.37
Wetted Perimeter (ft)	0	17.23	0	
Hydraulic Radius (ft)		1.59		
n	0.060	0.035	0.060	
Q (cfs)		31.64		31.64
Velocity (fps)		1.16		1.16

Water Surface Elevation of 68 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	0	42.09	0	42.09
Wetted Perimeter (ft)	0	19.35	0	
Hydraulic Radius (ft)		2.17		
n	0.060	0.035	0.060	
Q (cfs)		59.99		59.99
Velocity (fps)		1.43		1.43

Water Surface Elevation of 69 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	0	57.49	0	57.49

Water Surface Elevation of 69 ft.

Subsection ID	L	C	R	Totals/Averages
Wetted Perimeter (ft)	0	21.47	0	
Hydraulic Radius (ft)		2.68		
n	0.060	0.035	0.060	
Q (cfs)		94.13		94.13
Velocity (fps)		1.64		1.64

Water Surface Elevation of 70 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	0	73.57	0	73.57
Wetted Perimeter (ft)	0	23.59	0	
Hydraulic Radius (ft)		3.12		
n	0.060	0.035	0.056	
Q (cfs)		133.37		133.37
Velocity (fps)		1.81		1.81

Water Surface Elevation of 71 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	0	90.14	2.30	92.44
Wetted Perimeter (ft)	0	24.63	4.71	
Hydraulic Radius (ft)		3.66	0.49	
n	0.060	0.035	0.060	
Q (cfs)		181.81	0.71	182.51
Velocity (fps)		2.02	0.31	1.97

Water Surface Elevation of 72 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	0	107.00	9.20	116.20

Water Surface Elevation of 72 ft.

Subsection ID	L	C	R	Totals/Averages
Wetted Perimeter (ft)	0	25.67	9.41	
Hydraulic Radius (ft)		4.17	0.98	
n	0.060	0.035	0.060	
Q (cfs)		235.35	4.49	239.84
Velocity (fps)		2.20	0.49	2.06

Water Surface Elevation of 73 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	2.67	124.00	20.70	147.37
Wetted Perimeter (ft)	5.43	25.67	14.12	
Hydraulic Radius (ft)	0.49	4.83	1.47	
n	0.060	0.035	0.060	
Q (cfs)	0.82	300.92	13.23	314.97
Velocity (fps)	0.31	2.43	0.64	2.14

Water Surface Elevation of 74 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	10.67	141.00	36.80	188.47
Wetted Perimeter (ft)	10.85	25.67	18.83	
Hydraulic Radius (ft)	0.98	5.49	1.95	
n	0.060	0.035	0.060	
Q (cfs)	5.22	372.78	28.49	406.49
Velocity (fps)	0.49	2.64	0.77	2.16

Water Surface Elevation of 75 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	24.00	158.00	57.50	239.50

Water Surface Elevation of 75 ft.

Subsection ID	L	C	R	Totals/Averages
Wetted Perimeter (ft)	16.28	25.67	23.54	
Hydraulic Radius (ft)	1.47	6.16	2.44	
n	0.060	0.035	0.060	
Q (cfs)	15.40	450.66	51.66	517.72
Velocity (fps)	0.64	2.85	0.90	2.16

Water Surface Elevation of 76 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft^2)	40.25	175.00	80.75	296.00
Wetted Perimeter (ft)	17.40	25.67	24.66	
Hydraulic Radius (ft)	2.31	6.82	3.28	
n	0.060	0.035	0.060	
Q (cfs)	34.88	534.33	88.21	657.42
Velocity (fps)	0.87	3.05	1.09	2.22

Water Surface Elevation of 77 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	57.00	192.00	104.50	353.50
Wetted Perimeter (ft)	18.51	25.67	25.77	
Hydraulic Radius (ft)	3.08	7.48	4.05	
n	0.060	0.035	0.060	
Q (cfs)	59.75	623.62	131.62	814.99
Velocity (fps)	1.05	3.25	1.26	2.31

Water Surface Elevation of 78 ft.

Subsection ID	L	C	R	Totals/Averages
Area (ft2)	74.25	209.00	128.75	412.00

Water Surface Elevation of 78 ft.

Subsection ID	L	C	R	Totals/Averages
Wetted Perimeter (ft)	19.63	25.67	26.89	
Hydraulic Radius (ft)	3.78	8.14	4.79	
n	0.060	0.035	0.060	
Q (cfs)	89.28	718.34	181.16	988.77
Velocity (fps)	1.20	3.44	1.41	2.40

Water Surface Elevation of 79 ft.

Subsection ID	L	C	R	Totals/Averages
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

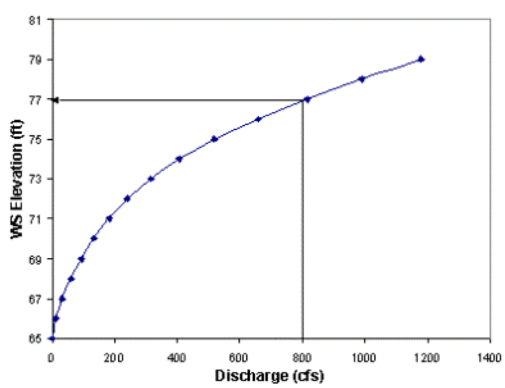

Figure 7-15. Stage Discharge Curve for Slope Conveyance—English Example

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 computer programs such as the following:

- Department of Army, Corps of Engineers HEC-2 and HEC-RAS
- Department of Agriculture, Natural Resources Conservation Service WSP-2
- Federal Highway Administration, U.S. Geological Survey WSPRO

See References for more contact information for these agencies. 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:
   - z1 = flowline elevation at section 1
   - y1 = 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, A1. Then use Equation 6-1 to calculate the velocity, v1, for the velocity head at A1. The next station upstream is usually section 2. Assume a depth y2 at section 2, and use y2 to calculate z2 and A2. Calculate, also, the velocity head at A2.
4. Calculate the friction slope (s_f) between the two sections using Equation 7-5 and Equation 7-6:

$$s_f = \left(\frac{Q}{K} \right)^2$$

Equation 7-5.

where:

$$K_{ave} = \frac{K_1 + K_2}{2} = 0.5\frac{A_1 R_1^{2/3}}{n_1} + \frac{A_2 R_2^{2/3}}{n_2}$$

Equation 7-6.

5. Calculate the friction head losses (h_f) between the two sections using

$$h_f = s_{ave} L$$

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 (8)
- 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 (7) as the downstream elevation (Section 1) and the next section upstream as Section 2, and repeat Steps (2) through (7).
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.

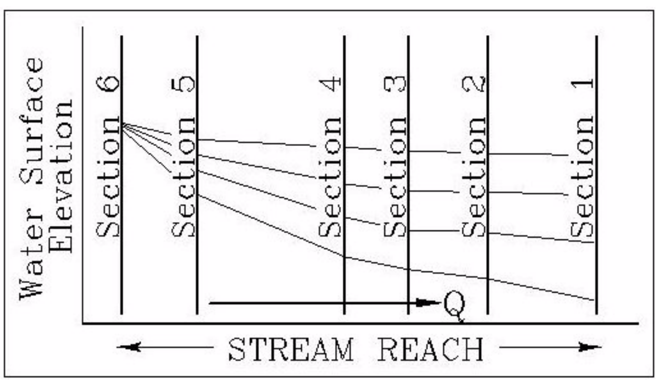

Figure 7-16. Water Surface Profile Convergence

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.

Example of the Standard Step Method

The Standard Step procedure is illustrated in the following example using a discharge of 2578 cfs. Four cross sections along a reach are shown in Figure 7-17 through Figure 7-24. Each cross section is separated by 500 ft., and is subdivided according to geometry and roughness. The calculations shown in Standard Step Calculations represent one set of water-surface calculations. An explanation of the calculations follows the table. The calculations represent the results of iterations at each section.

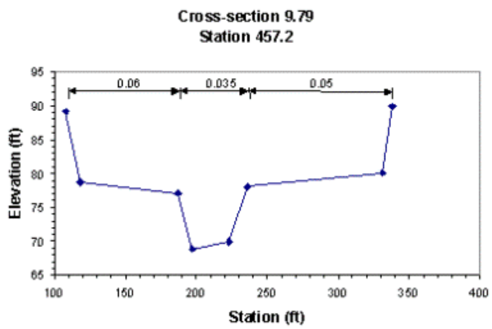

Figure 7-17. Cross Section 9.79 (farthest upstream) -- English

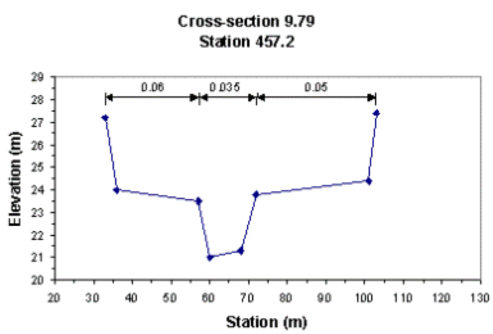

Figure 7-18. Cross Section 9.79 (farthest upstream) -- Metric

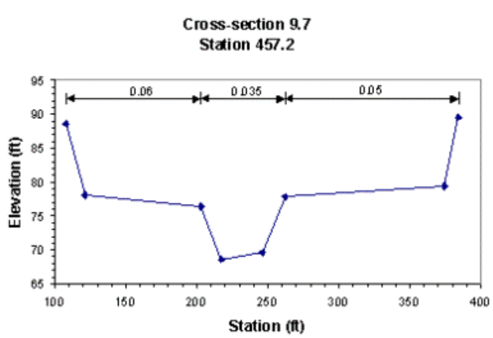

Figure 7-19. Cross Section at Station 9.7 -- English

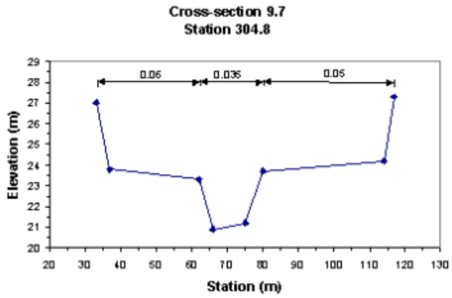

Figure 7-20. Cross Section at Station 9.7 -- Metric

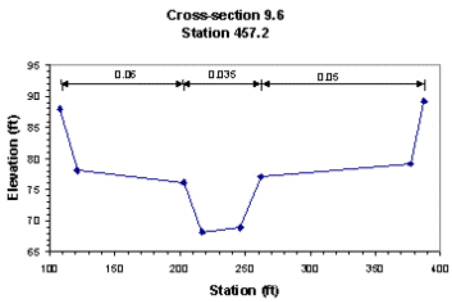

Figure 7-21. Cross Section at Station 9.6 -- English

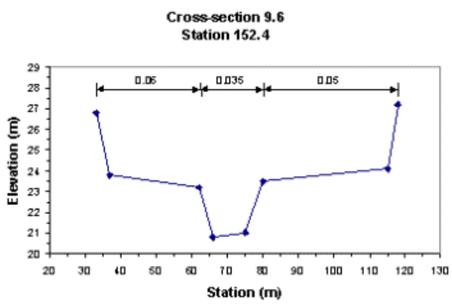

Figure 7-22. Cross Section at Station 9.6 -- Metric

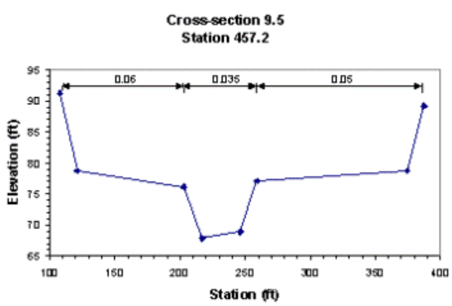

Figure 7-23. Cross Section at Station 9.5 (farthest downstream) -- English

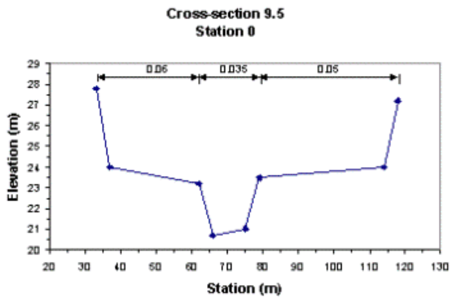

Figure 7-24. Cross Section at Station 9.5 (farthest downstream) – Metric

Standard Step Calculations (English)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Cross-Section ID	Cross-Section Station	Assumed WS (ft)	Computed WS (ft)	Area (ft2)	Wetted Perimeter (ft)	Hydraulic Radius (m)	n	K	Kavg	Sf (ft/ft)	Avg. Sf (ft/ft)	L (ft)	hf (ft)	[Ki(Ki/Ai)2]		V (ft/s)	V2/2g	(V2/2g)	ho(ft)	WS (ft)
9.5	0		79.30																	
				191	84.81	2.246	0.06	8093.6						14606344						
				506	61.78	8.189	0.035	87265						2596482371						
				146	114.5	1.279	0.05	5125.9						6284943						
				843				100485		0.0004		0	0	2617373659	1.83	2.39	0.162	-0	0	0
9.6	500	79.49	79.50																	
				191	84.81	2.231	0.06	8056.4						14405685						
				506	61.78	8.202	0.035	87358						2604775629						
				146	114.5	01.28	0.05	5127.7						6291507						
				843				100542	100513	0.0004	0.0004	500	0.2	2625472821	1.83	2.39	0.163	0	0	0.20

Standard Step Calculations (English)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
9.7	1000	80.22	79.71																	
	(Trial 1)			227	85.43	2.657	0.06	10792						24366622						
				531	61.75	8.596	0.035	94542						3000833825						
				196	115.1	1.706	0.05	8312.8						14968030						
				954				113647	107094	0.0003	0.000035	500	0.18	3040168476	1.88	2.1	0.129	0.034	0.003	0.21
	1000	79.71	79.91																	
	(Trial 2)			182	84.68	2.148	0.06	7500.9						12753428						
				501	61.78	8.102	0.035	85723						2514494345			-			
				136	114.3	1.186	0.05	4516.7						5009480						
				818				97741	105694	0.0004	0.000041	500	0.21	2532257252	1.81	2.46	0.171	-0.008	0.002	00.20
9.79	1500	79.89	80.10																	
				130	70.31	1.87	0.6	4896.2						6919551						
				403	52.85	7.612	0.035	66136						1785007382						
				84	89.5	0.951	0.05	2413.8						1995157						
				617				73447	85594	0.0008	0.000055	500	0.28	1793922090	1.72	3.26	0.285	-0.114	0.034	0.20

Standard Step Calculations (English)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21

Notes.

- Column 1: Column 1 contains the cross-section identification name.
- Column 2: This column contains the stream cross-section station number.
- Column 3: The assumed water surface elevation must agree with the resulting computed water surface elevation within ± 0.05 (or some other allowable tolerance) for trial calculations to be successful.
- Column 4: Column 4 is the stage-discharge (rating) curve value for the first section; thereafter, it is the value calculated by adding ΔWS (Column 21) to the computed water surface elevation for the previous cross section.
- Column 5: A is the cross-sectional area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel, and right overbank), then use one line of the form for each subsection and add to get the total area of cross section (At).
- Column 6: This column contains the wetted perimeter. If the section is subdivided, then one line will be used for each subsection wetted perimeter.
- Column 7: R is the hydraulic radius. Use the same procedure as for Column 5 if the section is complex, but do not add subsection values.
- Column 8: n is Manning's coefficient of channel roughness.
- Column 9: K is the conveyance and is determined with Equation 6-4. This column contains the total conveyance for the cross section. If the cross section is complex, add subsection K values to get the total conveyance (Kt).
- Column 10: Kave, the average conveyance for the reach, is computed with Equation 7-6.
- Column 11: This column contains the friction slope at the current section and is computed using Equation 7-5..
- Column 12: The average friction slope is determined using Kave in Equation 7-5.
- Column 13: L is the distance between cross sections.
- Column 14: The energy loss due to friction (hf) through the reach is calculated using Equation 7-7 through Equation 7-9.
- Column 15: This column contains part of the expression relating distributed flow velocities to an average value (see Column 16). If the section is complex, one of these values should be calculated for each subsection, and all subsection values should be added to get a total. If one subsection is used, Column 15 is not needed and the kinetic energy coefficient (Column 16) equals 1.0.
- Column 16: The kinetic energy coefficient (α) is calculated with Equation 6-10, if necessary.
- Column 17: The average velocity (V) for the cross section is calculated with the continuity equation (see Equation 6-1).
- Column 18: This column contains the average velocity head, corrected for flow distribution.
- Column 19: This column contains the difference between the downstream and upstream velocity heads. A positive value indicates velocity is increasing; therefore, use a contraction coefficient to account for "other losses." A negative value indicates the expansion coefficient should be used in calculating "other losses."
- Column 20: The "other losses" are calculated by multiplying either the expansion coefficient (Ke) or contraction coefficient (Kc) by the absolute value of Column 19. That is, for expansion, the change in velocity head will be negative, but the head loss must be positive.
- Column 21: ΔWS is the change in water surface elevation from the previous cross section. It is the algebraic sum of Columns 14, 19, and 20.

Standard Step Calculations (Metric)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Cross-Section ID	Cross-Section Station	Assumed WS (m)	Computed WS (m)	Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	n	K	Kavg	Sf (m/m)	Avg. Sf (m/m)	L (m)	hf (m)	S [Ki(Ki/Ai)2]	a	V (m/s)	$aV_2/2g$	D(aV2/2g)	ho(m)	D WS (m)
9.5	0		24.17																	
				17.7	25.85	0.68	0.06	229						38354						
				47	18.83	2.5	0.035	2472.7												
				13.6	34.89	0.39	0.05	145.7												
				78.3				2847.3	2847.5	0.0004		-0	0	6893221	1.83	0.73	0.049	-0	0	0
9.6	152.4	24.23	24.23																	
				17.7	25.85	0.68	0.06	229.2						38419						
				47	18.83	2.5	0.035	2472.7						6838227						
				13.6	34.89	0.39	0.05	145.7						16641						
		v		78.4				2847.5	2847.4	0.0004	0.0004	152.4	0.06	6893286	1.83	0.73	0.049	0	0	0.061
9.7	304.8	24.45	24.3																	
	(Trial 1)			21.1	26.04	0.81	0.06	304.9												
				49.3	18.82	2.62	0.035	2677.6												
				18.2	35.08	0.52	0.05	235.2												

Standard Step Calculations (Metric)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
				88.6				3217.8	3032.6	0.0003	0.0004	152.4	0.05	7998300	1.88	0.64	0.04	0.01	0.001	0.064
	304.8	24.29	24.29																	
	(Trial 2)			16.9	25.81	0.66	0.06	212.8												
				46.5	18.83	2.47	0.035	2425.5												
				12.6	34.85	0.36	0.05	127.4												
				76				2765.7	2806.6	0.0004	0.0004	152.4	0.06	6652069	1.81	0.75	0-.052	-0.003	-0.001	0.059
9.79	457.2	24.35	24.33																	
				12.1	21.43	0.57	0.6	138.1												
				37.4	16.11	2.32	0.035	1871.8												
				7.8	27.28	0.29	0.05	68.1												
				57.3				2078.1	2421.9	0.0008	0.0006	152.4	0.08	4716892	1.73	0.99	0.087	-0.035	-0.01	0.039

Standard Step Calculations (Metric)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21

Notes.

- Column 1: Column 1 contains the cross-section identification name.
- Column 2: This column contains the stream cross-section station number.
- Column 3: The assumed water surface elevation must agree with the resulting computed water surface elevation within ± 0.05 (or some other allowable tolerance) for trial calculations to be successful.
- Column 4: Column 4 is the stage-discharge (rating) curve value for the first section; thereafter, it is the value calculated by adding ΔWS (Column 21) to the computed water surface elevation for the previous cross section.
- Column 5: A is the cross-sectional area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel, and right overbank), then use one line of the form for each subsection and add to get the total area of cross section (A_t).
- Column 6: This column contains the wetted perimeter. If the section is subdivided, then one line will be used for each subsection wetted perimeter.
- Column 7: R is the hydraulic radius. Use the same procedure as for Column 5 if the section is complex, but do not add subsection values.
- Column 8: n is Manning's coefficient of channel roughness.
- Column 9: K is the conveyance and is determined with Equation 6-4. This column contains the total conveyance for the cross section. If the cross section is complex, add subsection K values to get the total conveyance (K_t).
- Column 10: Kave, the average conveyance for the reach, is computed with Eqution 7-6.
- Column 11: This column contains the friction slope at the current section and is computed using Equation 7-5..
- Column 12: The average friction slope is determined using Kave in Equation 7-5..
- Column 13: L is the distance between cross sections.
- Column 14: The energy loss due to friction (hf) through the reach is calculated using Equation 7-7 through 7-9.
- Column 15: This column contains part of the expression relating distributed flow velocities to an average value (see Column 16). If the section is complex, one of these values should be calculated for each subsection, and all subsection values should be added to get a total. If one subsection is used, Column 15 is not needed and the kinetic energy coefficient (Column 16) equals 1.0.
- Column 16: The kinetic energy coefficient (α) is calculated with Equation 6-10, if necessary.
- Column 17: The average velocity (V) for the cross section is calculated with the continuity equation (see Equation 6-1).
- Column 18: This column contains the average velocity head, corrected for flow distribution.
- Column 19: This column contains the difference between the downstream and upstream velocity heads. A positive value indicates velocity is increasing; therefore, use a contraction coefficient to account for "other losses." A negative value indicates the expansion coefficient should be used in calculating "other losses."
- Column 20: The "other losses" are calculated by multiplying either the expansion coefficient (K_e) or contraction coefficient (K_c) by the absolute value of Column 19. That is, for expansion, the change in velocity head will be negative, but the head loss must be positive.

Column 21: ΔWS is the change in water surface elevation from the previous cross section. It is the algebraic sum of Columns 14, 19, and 20.

Chapter 8
Culverts

Contents:

Section 1
Introduction

Culvert Design

A culvert conveys surface water through a roadway embankment or away from the highway right-of-way. In addition to this hydraulic function, it must also carry 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.

A culvert is any structure under the roadway, usually for drainage, with a clear opening of 20 ft. (6 m) or less measured along the center of the roadway between inside of end walls. 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. 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. 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 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. You can achieve improved traffic service and reduced cost by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

Before starting culvert design, consider site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

Construction

Culverts are constructed from a variety of materials and are available in many different shapes and configurations. When selecting a culvert, consider:

- 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. Base shape selection 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. Consider the following groups of variables:

- 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 has the lowest total annual cost over the design life of the project. Do not base culvert material selection 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, consider bidding the materials as alternates, allowing the contractor to make the most economical material selection.

Inlets

A multitude of different inlet configurations is 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, consider structural stability, aesthetics, erosion control, and fill retention.

You may improve culvert hydraulic capacity 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

The hydraulic design of a culvert always includes an economic evaluation. 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, conduct an economic analysis of the trade-offs with varying degrees of effort and thoroughness, depending on the need.

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 you consider the cost of emergency vehicle response time as well as the detour distance and cost of operation per vehicle mile, 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.

The purpose of a highway culvert is to convey water through a roadway embankment. The major benefits of the culvert are decreased traffic interruption time due to roadway flooding and increased driving safety. The major costs are associated with the construction of the roadway embankment and the culvert itself. Factor maintenance of the facility and flood damage potential into the cost analysis.

For minor stream crossings, you may preclude the need for a detailed economic analysis by using the Design by Frequency Selection (see Chapter 5), considering the importance of the highway. You may need a more rigorous investigation, such as a risk analysis for large culvert installations.

An infinite variety of combinations of circumstances make it unreasonable to specify an absolute design frequency for all highway projects. However, recommended guidelines are offered in the Design Procedure for Culverts section of this chapter. Use the Frequencies for Coincidental Occurrence in the Chapter 5 judiciously, based on factors such as the opinions of experienced designers, political requirements or preferences, and outside funding requirements

Refer to Chapter 5 for discussion on the possible need for design by risk assessment.. Conduct a risk assessment when deviations from recommended design frequencies are indicated.

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 4 for information on site surveys and Chapter 5 for information on 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 and requires little structural excavation or channel work.

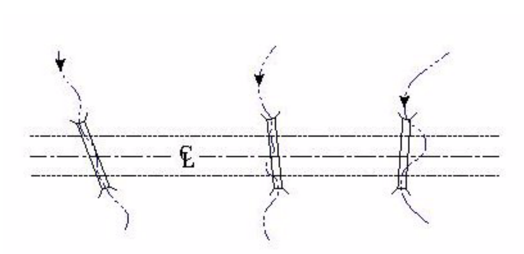

Figure 8-1. Culvert Placement Locations

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, you may need to lower or raise the upstream flowline. 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.

Avoid placing the downstream flowline of the culvert at a level higher than the roadway embankment toe of slope. 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, you can design a broken back configuration to reduce the outlet velocity by introducing a hydraulic jump inside the culvert.

Waterway Data

The installation of a culvert through a highway embankment may significantly constrict the floodplain. Therefore, collect pre-construction data to predict the consequences of this alteration. Refer to Chapters 4, 5, and 7 for information on site surveys and data collection, hydrology, and channel properties.

Determine the longitudinal slope of the existing channel in the vicinity of the proposed culvert in order to establish culvert vertical profile and to define flow characteristics in the natural stream. Often, you can position the proposed culvert at the same longitudinal slope as the streambed.

Evaluate the hydraulic resistance of the natural channel in order to calculate pre-project flow conditions. An average Manning's "n" value usually represents this resistance. Various methods are available to evaluate resistance coefficients for natural streams including comparing photographs of streams with known resistance values or tabular methods based on stream characteristics. Refer to the Roughness Coefficients information in Chapter 6.

Tailwater may affect culvert capacity under outlet control conditions. An obstruction in the downstream channel or by the hydraulic resistance of the channel may cause tailwater. In either case, you can perform backwater calculations from the downstream control point to estimate tailwater. (See the Standard Step Backwater Method in Chapter 7 for more information.) When hydraulic resistance of the channel controls the flow depth, use normal depth approximations instead of backwater calculations. (See the Slope Conveyance Method in Chapter 7 for more information.)

The storage capacity upstream from a culvert may have an impact upon its design. Follow these steps to determine the upstream storage capacity:

1. You can approximate upstream storage capacity from contour maps of the upstream area. However, it is preferable to obtain a number of cross sections upstream of the proposed culvert.
2. Reference these sections horizontally as well as vertically. The length of upstream reach required depends on the expected headwater and the stream slope.
3. Use the cross sections to develop contour maps or the cross sectional areas to compute storage.

The topographic information should extend upward from the channel bed to an elevation equal to at least the design headwater elevation in the area upstream of the culvert.

Roadway Data

The proposed or existing roadway affects culvert cost, hydraulic efficiency, and alignment.

Obtain information from the roadway profile and the roadway cross section from preliminary roadway drawings or from standard details on roadway sections. (See below for more details.) When the culvert must be sized prior to the development of preliminary plans, you can use a best estimate of the roadway section, but you must check the culvert design after the roadway plans are completed.

The roadway cross section normal to the centerline is typically available from highway plans. However, you need the cross section at the stream crossing. This section 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.

Necessary dimensions and features of the culvert become evident when you evaluate or establish the desired roadway cross section. Obtain the dimensions by superimposing the estimated culvert barrel on the roadway cross section and the streambed profile. This establishes the inlet and outlet invert elevations. These elevations and the resulting culvert length are approximate since the final culvert barrel size must still be determined.

The roadway embankment represents obstruction encountered by 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, design the location of overtopping to conform as closely as possible to the location of the majority of flood flow under existing conditions.

The profile contained in highway plans generally represents the roadway centerline profile. These elevations may not represent the high point in the highway cross section. Determine the profile that establishes roadway flooding and roadway overflow elevations. The low point of the profile is critical because this is the point at which roadway overtopping first occurs.

Allowable Headwater

Energy is required to force flow through a culvert. This 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 a culvert subtends is a function of several parameters, including the culvert geometric configuration. Base the culvert geometric configuration primarily on the allowable headwater. This geometric configuration consists of the number of barrels, barrel dimensions, length, slope, entrance characteristics, and barrel roughness characteristics.

Base the design headwater and the selection of design flood on 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. If roadway embankments are low, flooding of the roadway and delay to traffic are usually of primary concern, especially on highly traveled routes. Possible critical elevations on the highway itself that affect allowable headwater include edge of pavement, sub-grade crown, and top of headwall. If the roadway is encroaching on a FEMA-designated floodplain, ensure that the design meets NFIP criteria. Refer to FEMA Policy and Procedure in Chapter 2, and the Policy Manual for more information on FEMA and NFIP criteria. In any event, the design discharge must not inundate the travel way. Additionally, where practicable for the 100-year event, limit the net increase in water surface at the upstream face of the culvert to 1.0 ft (0.3 meters).

Culvert installations under high fills may present an opportunity to use a high headwater or ponding to attenuate flood peaks. If you consider deep ponding, investigate the possibility of catastrophic failure because a breach in the highway fill could be quite similar to a dam failure:

1. Evaluate culvert design in terms of a probable maximum flood or PMF.
2. Consult the Bridge Division's Hydraulic Branch for assistance with PMF determination.

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, you may use culverts of lesser depths or earthen training dikes, in some instances, to avoid diversion across watershed divides. In flat terrain, watershed divides are often undefined or nonexistent. Locate and design culverts for the least disruption of the existing flow distribution.

Outlet Velocity

Because a culvert usually constricts the available channel area, flow velocities in the culvert are likely to be higher than in the channel. These increased velocities can cause streambed scour and bank erosion in the vicinity of the culvert outlet. You can occasionally avoid minor problems by increasing the barrel roughness. The culvert sometimes requires energy dissipators and velocity protection devices to avoid excessive scour at the outlet. When a culvert is operating under inlet control and the culvert barrel is not operating at capacity, it is often beneficial to flatten the barrel slope or add a roughened section to reduce outlet velocities.

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 is the governing criterion in the selection and application of various downstream fixtures and appurtenances.

The types and characteristics of soil can vary considerably from site to site. The presence of culvert appurtenances in the downstream vicinity of the culvert also influences the allowable outlet velocity. Velocities at which soils become erosive may vary widely. Attempt to estimate the threshold of erosive velocity for each culvert location. You may reach this estimation by observing storm flows on various soil types and estimating those velocities at which erosion is occurring. Channels with rock or shale bottoms typically tolerate high velocities (15 to 20 fps or 4.5 to 6.0 meters per second). On the other hand, channels with silt or sand bottoms may erode at low velocities. Refer to *Hydraulic Design of Energy Dissipators for Culverts and Channels*, (FHWA - HEC14) Chapter 5, for more information on estimating scour at culvert outlets.

Exercise extreme caution when considering culvert designs with outlet velocities of greater than 15 fps (4.5 m per second). Refer to Channel Lining Design Procedure in Chapter 7 for a way to determine if velocities are excessive for various channel conditions. Provide riprap or control devices in situations where outlet velocity poses potential erosion problems. Section 6 of this chapter describes different velocity protection and control devices. If the culvert has been sized properly according to allowable headwater criteria, it is almost always more economical to protect against excessive outlet velocity with riprap and velocity protection or control devices than to try to adjust the culvert size to reduce the excessive outlet velocity.

Velocities of less than about 2 fps (0.5 m per second) usually foster deposition of sediments. Therefore, 2fps (0.5 m per second) 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 piping (flow through the embankment outside of the culvert) prevention.

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.

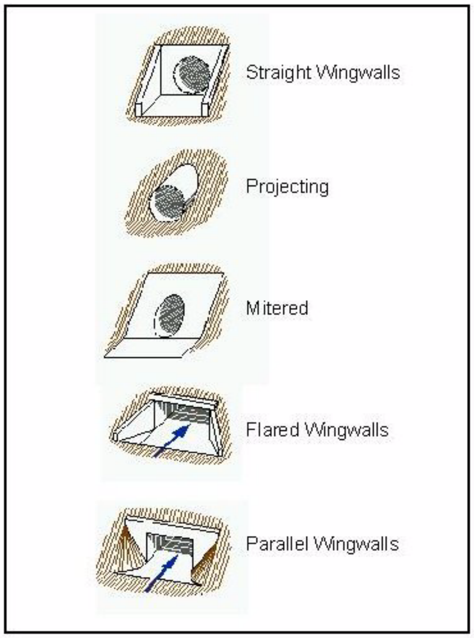

Figure 8-2. Typical Culvert End Treatments

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.

Safety treatment of culvert ends is a smooth, clean way of mitigating unsafe conditions, but it also represents a significant interference with the original purpose of the drainage structure. A safety end treatment has a tendency to accumulate trash and flood debris, thus blocking flow into and out of the culvert.

Use mitered end sections carefully for several reasons. First, mitered end sections may increase hydraulic head losses. Additionally, a non-reinforced mitered end may affect the structural integrity of the culvert. With the use of mitered end sections, where practicable, incorporate the safety end treatment standards issued by the Bridge Division to minimize potential interference to floodwater flow, particularly where such floodwater may be laden with debris. The simple step of removing the headwall and applying a mitered end section alone (see Figure 8-3) offers relatively little obstacle for passage of drift or debris.

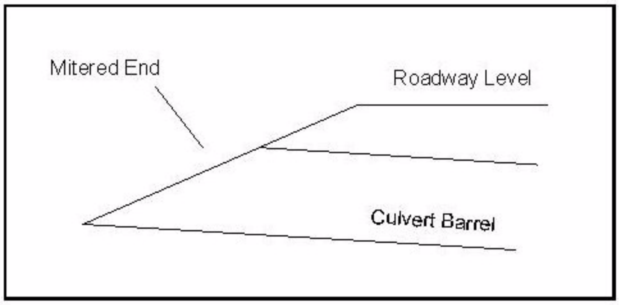

Figure 8-3. Mitered End Treatment for Safety

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 selection. Generally, the primary factors affecting culvert type selection in Texas are economics, hydraulic properties, durability, and strength.

The following factors influence culvert type selection, regardless of state:

- 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. The process for selecting material for culvert construction is as follows:

1. 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.
2. Choose culvert components that are readily available to construction contractors usually 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 and usually available from local suppliers at a lower cost than other shapes. 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, carefully evaluate scour potential and the

structural stability of the streambed. Structural plate metal arches are limited to use in low cover situations but have the advantage of rapid construction and low transportation and handling costs. This is especially advantageous in remote areas and in rugged terrain.

The terrain often dictates the need for a low profile due to limited fill height or potential debris clogging.

Multiple Barrel Boxes (or Multiple 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, if there will be more than one box in the culvert, use shapes of uniform geometry and roughness characteristics. The flow distribution for uniform multiple boxes is then a simple equal distribution of flow through each box.

In the case of box culverts, multiple boxes are usually more economical than a wide single span. On the other hand, multiple boxes tend to catch debris that clogs the waterway. They are also susceptible to silting and ice jams. Alignment of the culvert face normal to the approach flow and installation of debris control structures can help to alleviate these problems.

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.

Analysis versus Design

Culvert analysis involves hydraulic computations using known culvert geometry (shape, profile and dimensions) and roughness. Culvert design employs the same hydraulic calculations; however, you must chose the shape and profile and establish the dimensions by iterative application of the computation procedures using varied dimensions until you are satisfied that headwater is reasonable.

Culvert Design Process

The culvert design process includes the following basic stages:

1. Define the location, 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}), define an overall culvert configuration to be analyzed (as part of the design process of trial and error)-- culvert hydraulic length (L), entrance conditions, and conduit shape and material.
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.
6. Treat any excessive outlet velocity separately from headwater.

Design Guidelines and Procedure for Culverts

The flow charts of Figure 8-4 and Figure 8-5 guide you in computing for the vast majority of culvert design situations.

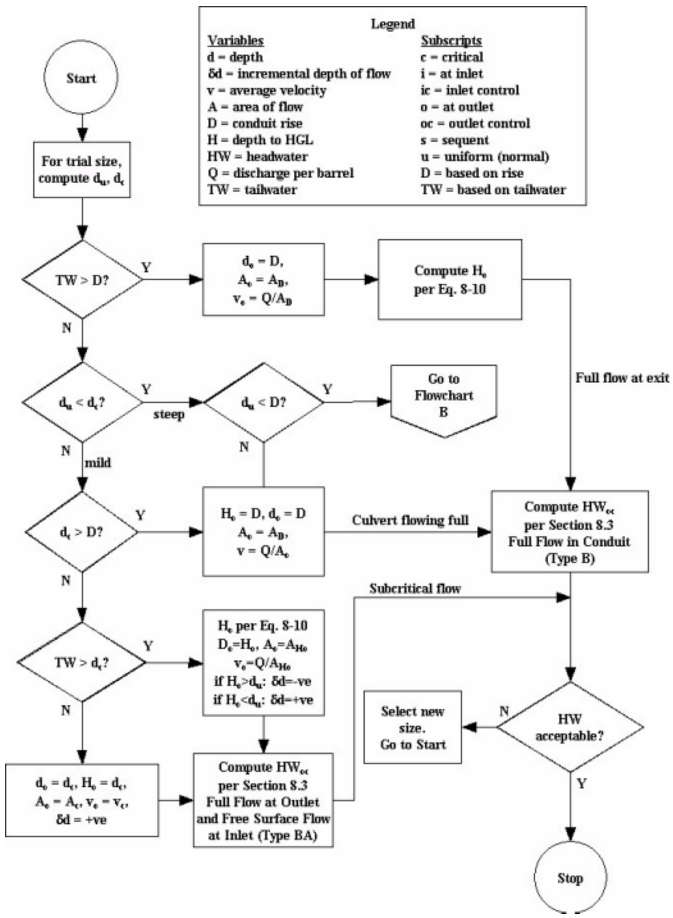

Figure 8-4. Flow Chart A - Culvert Design Procedure

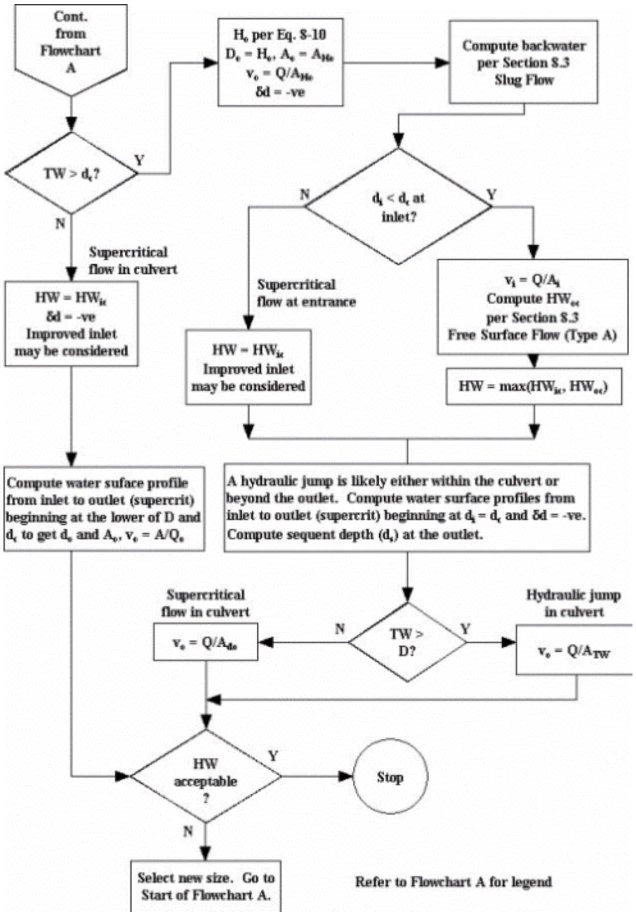

Figure 8-5. Flow Chart B - Culvert Design Procedure (cont.)

While you do consider allowable outlet velocity, it has little or no influence on the culvert barrel configuration in this design process. Treat any problem of excessive outlet velocity separately in most cases.

The following is a step-by-step culvert design procedure for a standard culvert configuration -- straight in profile and if multiple barrels are used, with barrels that are parallel and of equal size. Any of the configurations considered in the iterative process of design 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. You can pick the trial size at random or judiciously, based on experience. However, one expedient is to assume inlet control 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. 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.

$$h = HW_{max} - \frac{D_{max}}{2}$$

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).

$$A = 0.45\frac{Q}{h^{0.5}}$$

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:

- 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.
- For a circular pipe culvert, determine the ratio of area required to maximum barrel area according to 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.

$$\frac{4A}{\pi D_{max}^{2}} \le N$$

- 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, employ a computer program or spreadsheet. The department recommends that nomographs and simplified hand methods 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 8-4. Flow Chart A references Flow Chart B, which is shown in Figure 8-5.

4. Evaluate trial design. At this step in the design process, you have calculated a headwater and outlet velocity for the design discharge passage through a trial culvert configuration.
   - 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; go back to 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. Regardless of the changes made here, repeat the calculations. Go back to 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.
   - The culvert for which the calculated headwater is satisfactory may have an excessive outlet velocity. The definition of an "excessive" outlet velocity is normally an engineering judgment based on local conditions.
   - In comparison to adjusting the culvert barrel configuration, it is usually more economical to provide riprap, sills, or a stilling basin at the outlet end to control any excessive velocity.

   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.

5. 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 a wide range of discharges. Use the performance curve to check the 100-year discharge.
   - For any culvert design, the minimum additional analysis required is the application of the 100-year discharge to the culvert. Consider the design complete 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 Process Manual for more details.)

- However, if any of the hydraulic characteristics are unacceptable, some adjustment to the facility design may be in order. This is an analysis technique to define risks of greater floods to help judge whether or not to accept the risks involved.

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

Each culvert shape has distinct hydraulic properties, and each material has an associated wall roughness. Both factors influence hydraulic operation. As well, combinations of conditions can affect hydraulic operation. Consider a succession of parameters to reach the appropriate calculations. Variations of the identified conditions affecting culvert operation should not alter the result appreciably. The hydraulic operation and performance of a culvert involve the 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, though you may use hand methods and nomographs 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 free surface flow is in the barrel, 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 are the major flow controls. Inlet geometry includes barrel shape, cross-sectional area, and inlet edge.

Use a fifth-degree polynomial equation based on regression analysis to model the inlet control headwater for a given flow. The regression equations are for the range of inlet heads from one-half to three times the culvert rise. 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. For $0.5 \le HW_{ic}/D \le 3.0$, Equation 8-4 applies.

$$HW_{ic} = \left[a + bF + cF^2 + dF^3 + eF^4 + fF^5\right]D - 0.5DS_0$$

Equation 8-3.

where:

HWic = 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)

S0 = 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 8-4.

where:

W = width or span of culvert (ft. or m).

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

Regression Coefficients for Inlet Control Equations

Shape and Material	Entrance Type	a	b	c	d	e	f
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
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 $HW_i/D > 3.0$, use an orifice equation, Equation 8-6, 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 8-5.

where:

HWi = inlet control headwater depth (ft. or m)

Q = design discharge (cfs or m^3/s)

k = orifice equation constant ($\sqrt{2gAC}$)

D = rise of culvert (ft. or m).

$$k = 0.6325\frac{Q_{3.0}}{D^{1/2}}$$

Equation 8-6.

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 ($HW_i/D < 0.5$), use an open channel flow minimum energy equation (weir equation) 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.

Refer to the “Charts” in HDS-5 (FHWA, Hydraulic Design of Highway Culverts) for graphical solution of headwater under inlet control. (See References for contact information.)

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 ($d_n < d_c$) and a high tailwater such that subcritical flow or full flow exists in the culvert.

The headwater resulting from flow through 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 tailwater level (possibly).

For practical purposes, when a culvert is under outlet control, you can adjust the headwater by modifying culvert size, shape, and roughness.

Consider inlet control and outlet control 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.

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

You determine the headwater under outlet control by accounting for the total energy losses that occur from the culvert outlet to the culvert inlet. Use Figure 8-4 and Figure 8-5 and associated procedures in Section 4 to analyze or design a culvert.

Outlet control headwater HW_{oc} depth (from the flowline of the entrance) is expressed in terms of balancing energy between the culvert exit and the culvert entrance as indicated by Figure 8-8.

$$HW_{oc} + h_{va} = h_e + h_{vi} + \sum h_f - S_o L + H_o$$

Equation 8-7.

where:

HWoc = 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 8-11

h_f= friction head losses (ft. or m) as calculated using Equation 8-12

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 8-8.

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 when determining outlet control headwater, consider energy balance at outlet, energy losses through barrel, and energy balance at inlet.

When the tailwater controls the outlet flow, use Equation 8-7 to represent the energy balance equation at the conduit outlet. Traditional practice has been to ignore exit losses. If you ignore exit losses, assume that the hydraulic grade line inside the conduit at the outlet, outlet depth, H_o, is the same as the hydraulic grade line outside the conduit at the outlet and do not use Equation 8-10.

$$H_o = TW + h_{TW} + h_o - h_{vo}$$

Equation 8-9.

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. Establish the outlet depth based on the conditions shown below.

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 8-7.
Uniform depth is lower than top of barrel and tailwater exceeds critical depth	Slope is hydraulically steep	Set H_o using Equation 8-7.

Outlet Depth Conditions

If...	And...	Then...
Uniform depth is lower than top of barrel and tailwater 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 8-6. It is necessary to calculate the backwater profile based on the outlet depth, H_o.

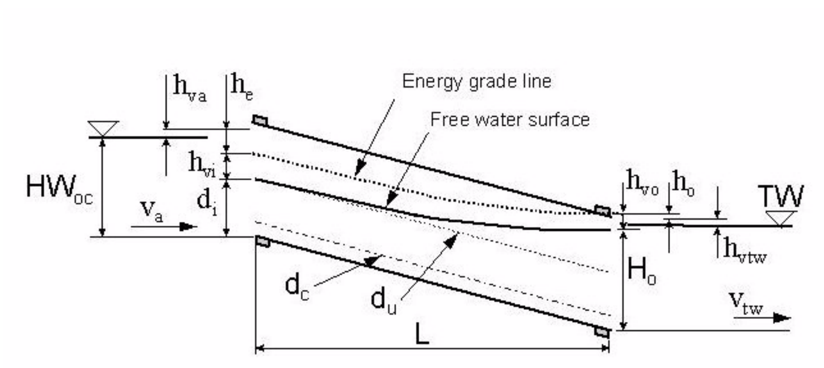

Figure 8-6. Outlet Control Headwater for Culvert with Free Surface

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.

Use the Direct Step Backwater Method to determine the water surface profile (and energy losses) though the conduit. For subcritical flow, begin the calculations 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 you reach the inlet end of the conduit without the calculated depth exceeding the barrel depth (D), you have verified 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 8-7. Calculate the hydraulic grade line based on outlet depth, H_o, at the outlet.

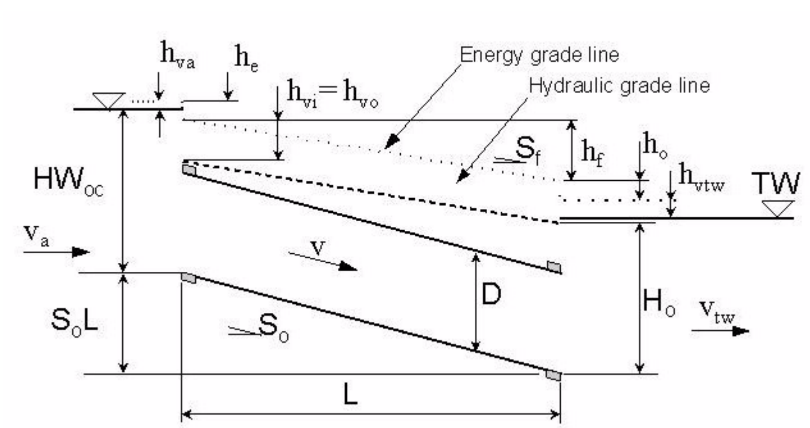

Figure 8-7. Outlet Control, Fully Submerged Flow

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.

Determine the energy loss (friction loss) through the conduit using Equation 8-11.

$$h_f = S_f L$$

Equation 8-10.

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).

Compute the depth of the hydraulic grade line at the inside of the inlet end of the conduit using Equation 8-12. Refer to Energy Balance at Inlet to determine the headwater.

$$H_i = H_o + h_f - S_o L$$

Equation 8-11.

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).

If friction slope (Equation 8-13) 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 8-12.

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 following table on Entrance Loss Coefficients). Take the following steps:

1. Determine the length over which full flow occurs (L_f) is using the geometric relationship shown in Equation 8-14 (refer to the following table on Entrance Loss Coefficients):

$$L_f = \frac{H_o - D}{S_o - S_f}$$

Equation 8-13.

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.

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 8-7. 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 using the direct step method and you reach the inlet end of the conduit, set the calculated depth at the inlet as H_i and refer to Energy Balance at Inlet to determine the headwater.

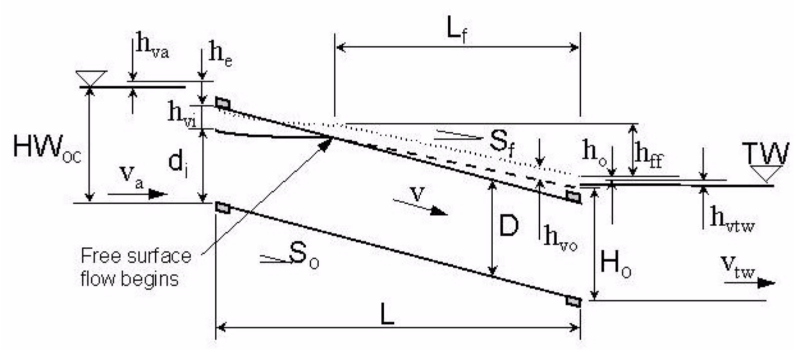

Figure 8-8. Point at Which Free Surface Flow Begins

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 8-9 illustrates this condition. This condition is possible if the theoretical value of uniform depth is higher than the barrel depth. Take the following steps:

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 you reach 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.

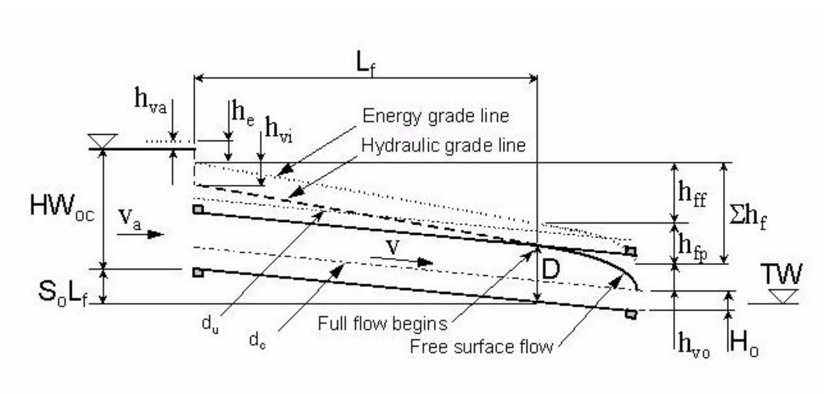

Figure 8-9. Headwater Due to Full Flow at Inlet and Free Surface at Outlet

Energy Balance at Inlet

Compute the outlet control headwater, HW_{oc}, by balancing the energy equation, depicted as Equation 8-15. You will need to know the hydraulic grade at the inside face of the culvert at the entrance. 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 8-14.

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, you may assume that the velocity approaching the entrance is 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. You may need to consider the approach velocity when performing the following tasks:

- estimating 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.

A culvert has an effective flow area similar to the approach channel section so that approach velocities and through-culvert velocities are similar.

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 8-15.

where:

C_e = entrance loss coefficient

V_i = flow velocity inside culvert inlet(fps or m/s).

Select values of C_e from the following table (entrance loss coefficients) based on culvert shape and entrance condition.

Entrance Loss Coefficients (C_e)

Concrete Pipe	C_e
Projecting from fill, socket end (groove end)	0.2
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):	

Entrance Loss Coefficients (C_e)

Concrete Pipe	C_e
◆ 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 varies from inlet control to outlet control and back again in a cyclic pattern 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). Refer to Equation 8-17.

$$v_o = \frac{Q}{A_o}$$

Equation 8-16.

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 8-10.

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 8-11.

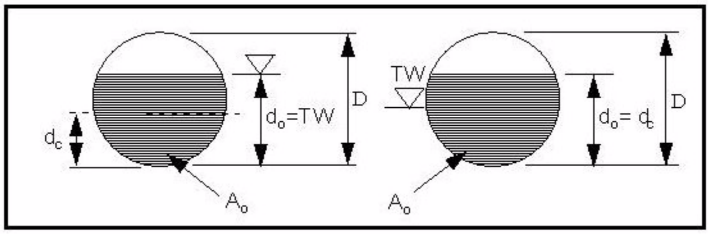

Figure 8-10. Cross Sectional Area based on the Higher of Critical Depth and Tailwater

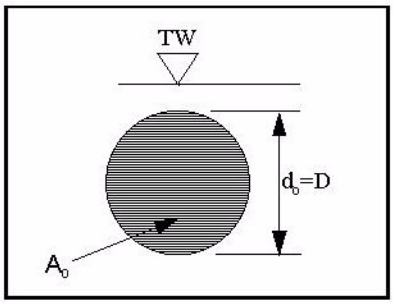

Figure 8-11. Cross Sectional Area Based on Full Flow

Depth Estimation Approaches

For inlet control under steep slope conditions, estimate the depth at the outlet using one of the following approaches:

- Employ 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, you choose an increment (or decrement) of water depth (δd) and compute the distance over which the depth change occurs. 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 8-17.

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 > du, 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 8-18.

$$S_f = \frac{(S_{f2} + S_{f1})}{2}$$

Equation 8-19.

$$\delta L = \frac{\delta E}{S_o - S_f}$$

Equation 8-20.

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 subsection above) 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 above. A hydraulic jump may occur within the culvert (refer to the following subsection for discussion of the hydraulic jump).

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

For a given discharge in any channel when water flows at a depth that is less than critical depth (supercritical flow), a sequent (or conjugate) depth in subcritical flow balances forces due to momentum change and hydrostatic pressure between the respective depths. 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.

The balance of forces is represented using a momentum function, used in Equation 8-22:

$$M = \frac{Q^2}{gA} + A\bar{d}$$

Equation 8-21.

where:

M = momentum function

Q = discharge (cfs or m^3/s)

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.

Figure 8-12 provides a sample plot of depth and momentum function and an associated specific energy plot. By comparing the two curves at a supercritical depth and its sequent depth, you can see 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.

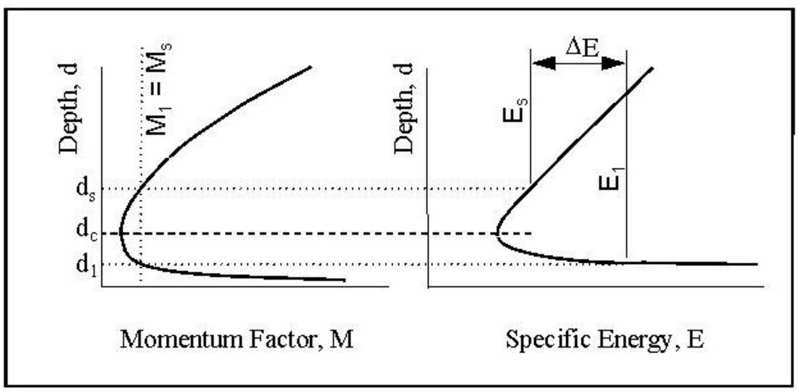

Figure 8-12. Momentum Function and Specific Energy

Determine the potential occurrence of the hydraulic jump within the culvert 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 8-22.

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 8-13, which you calculate by using Equation 8-26.

CAUTION: Some calculators and spreadsheets may give only the principal angle for β in Equation 8-26 (i.e., $-\pi/2$ radians $\leq \beta \leq \pi/2$ radians).

- Use Equation 8-27 to calculate the areas of flow for the supercritical depth of flow and sequent depth.

$$Q^2 = \frac{g\left(A_s\bar{d}_s - A_1\bar{d}_1\right)}{1/A_1 - 1/A_s}$$

Equation 8-23.

where:

Q = discharge, cfs or m^3/s

A_s = area of flow at sequent depth, sq.ft. or m^2

$A_s d_s$ = first moment of area about surface at sequent depth, cu.ft. or m^3

$A_1 d_1$ = first moment of area about surface at supercritical flow depth, cu.ft. or m^3.

$$A\bar{d} = \frac{D^3}{24}\left(3\sin\beta - \sin^3\beta - 3\beta\cos\beta\right)$$

Equation 8-24.

where:

A = first moment of area about water surface, cu.ft. or m^3

D = conduit diameter, ft. or m

β = angle shown in Figure 8-13 and calculated using Equation 8-26.

$$\beta = \cos^{-1}(1 - \frac{2d}{D})$$

Equation 8-25.

$$A = \frac{D^2}{8}\left[2\cos^{-1}\left(1 - \frac{2d}{D}\right) - \sin\left(2\cos^{-1}\left(1 - \frac{2d}{D}\right)\right)\right]$$

Equation 8-26.

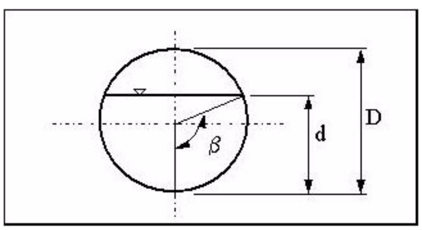

Figure 8-13. Determination of Angle b

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$\bar{d}$, is dependent on the shape of the conduit and depth of flow. Acquire or derive a relationship between flow depth and first moment of area.

Roadway Overtopping

Where water flows both over the roadway and through a culvert (see Figure 8-14), a definition of hydraulic characteristics requires a flow distribution analysis. This is a common problem where a discharge of high design frequency (low probability of occurrence) is applied to a facility designed for a lower design frequency.

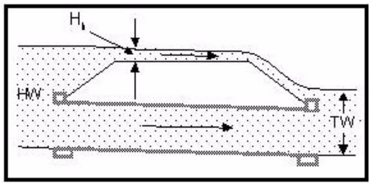

Figure 8-14. Culvert with Overtopping Flow

For example, a complete design involves the application and analysis of a 100-year 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 establishes 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 8-27.

where:

Q = discharge (cfs or m^3/s)

k_t = over-embankment flow adjustment factor (see Figure 8-15)

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 8-16, tailwater will not affect the over-embankment flow if its excess (H_t) over the highway is lower than critical depth of flow over the road, which is 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 8-15 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 8-17). 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.

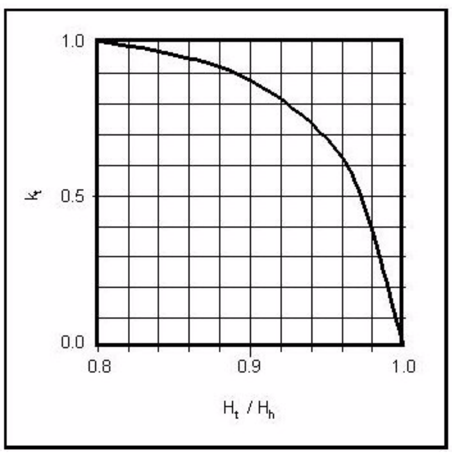

Figure 8-15. Over Embankment Flow Adjustment Factor

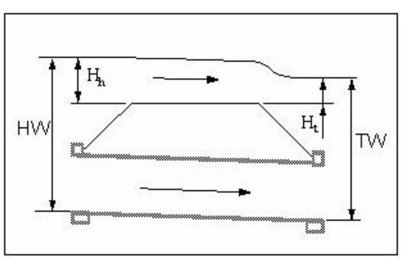

Figure 8-16. Roadway Overtopping with High Tailwater

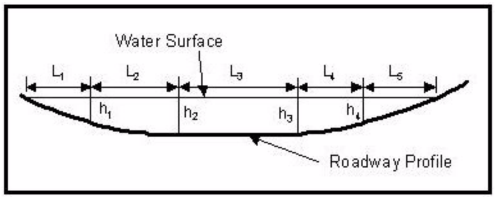

Figure 8-17. Cross Section of Flow over Embankment

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. (3 mm). Consider the current headwater and current assigned culvert flow and calculated roadway overflow 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.

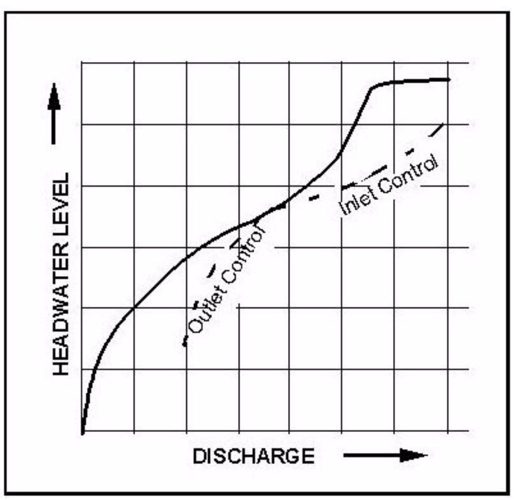

Figure 8-18. Typical Performance Curve

This envelope is termed a 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. Generate the performance curve by performing culvert headwater computations 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, you may need to accommodate an explicit exit loss. 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) to be used in the analysis procedure on balancing Equation 8-27 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 8-28.

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 8-29.

where:

K = loss coefficient which typically varies from 0.5 to 1.

Section 4
Improved Inlets

Inlet Use

An improved inlet may be economical if the culvert is operating under inlet control. 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 of the inlet. If the culvert is operating under outlet control, improved inlets are not effective, and you should not consider them.

Weigh several factors before using improved inlets:

- Improved inlets offer the advantage of increasing the capacity of existing inadequate culverts. This is important where an inlet control culvert serves a watershed that has changed in character from rural to urban or otherwise experienced an increase in the design discharge rate.
- For inlet control, the improved inlet design process typically allows a reduction in the conduit size. Check that this reduction in conduit size does not cause the culvert to revert to outlet control.
- Improved inlets are usually costly. Develop a complete economic analysis that weighs the additional costs associated with the installation and maintenance of the improved inlet against the savings effected by reduced conduit size.
- Available design procedures require a normal entrance for improved inlets. That is, the procedures do not accommodate improved inlets when the face of its inlet is skewed.
- Where heavy debris loads are anticipated, improved inlets can become a serious maintenance problem because of a tendency for some debris to pass the face of the inlet and become lodged in the throat.

The recommended types of improved inlets are top-tapered transitions, side-tapered inlets, and slope-tapered inlets. For specific design procedures for improved inlets, refer to the FHWA publication, *Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5*. (See References for contact information to obtain this publication.)

A simple transition of depth in a rectangular box culvert may improve the hydraulic efficiency. If the box culvert is operating under inlet control, the barrel of the culvert is more hydraulically efficient than the entrance geometry. You may reduce the barrel depth in transition from the original depth to a minimum of 1.0 ft. (0.3 m) greater than the uniform depth of flow. The transition length should be a minimum of 20 ft. (6 m) (see Figure 8-19). This method is arbitrary, and you should use it carefully only when the culvert is definitely operating in inlet control.

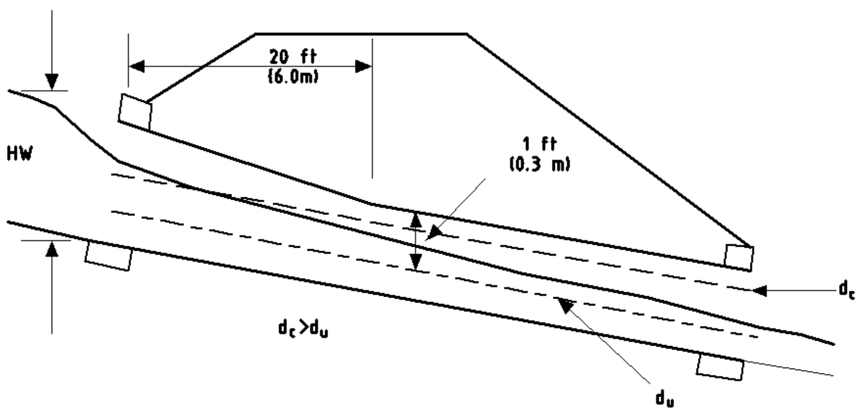

Figure 8-19. Top Tapered Box Culvert

In terms of design and construction, the method is effective, economical, and simple to perform. You may prefer this method for designing a multiple barrel box culvert. Other inlet improvement methods are not feasible for multiple barrel box culverts because of the need to taper or flare the sidewalls of the barrels.

Side-tapered inlets involve a widening of the face area of the culvert by tapering the sidewalls. Such inlets have two possible control sections: the face and the throat (Figure 8-20). Maintain control at the throat for the design discharge in order to realize significant cost savings in the culvert barrel. This type of improvement is similar in operation to the flared inlet for pipes.

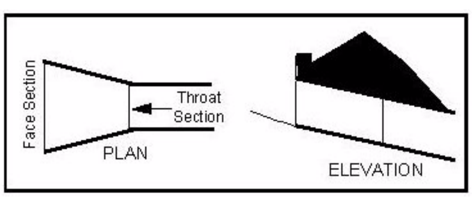

Figure 8-20. Side Tapered Inlet

The slope-tapered inlet incorporates the efficient flow characteristics of side-tapered inlets with a concentration of more of the total available culvert fall at the throat control section. Figure 8-21 shows a slope-tapered inlet. Generally, slope-tapered improvements are not practical for pipe culverts because of their complexity.

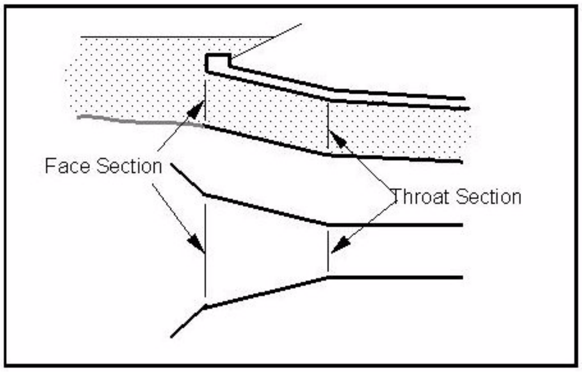

Figure 8-21. Slope Tapered Inlet

Some of the drawbacks of slope-tapered inlets are as follows:

- Slope-tapered inlets have a tendency to allow sediment deposition; this can result in maintenance problems.
- The degree of the slope taper is limited by how flat the remaining portion of pipe can be made without resulting in a mild slope.
- The use of slope-tapered inlets can increase costs of structural excavation because of the lowering of the upstream end of the culvert.

Beveled Inlet Edges

Beveled inlets edges can be useful in the circumstances of outlet control. They effectively reduce the contraction downstream of the culvert face, resulting in a more efficient conveyance of water by the available barrel area (see Figure 8-22). Generally, gaining the hydraulic advantage of beveled edges requires little or no enlargement of the culvert inlet. Thus, structural problems are minor. You may implement beveled edges at little additional expense, and they are effective for culverts operating under either inlet or outlet control. These edges can be a significant improvement in culvert capacity and reductions in the subtended headwater. You can easily adapt them to either pipe or box culverts. The table titled Regression Coefficients for Inlet Control Equations provides polynomial coefficients for some beveled entrance conditions for use in Equation 8-1 (inlet control headwater).

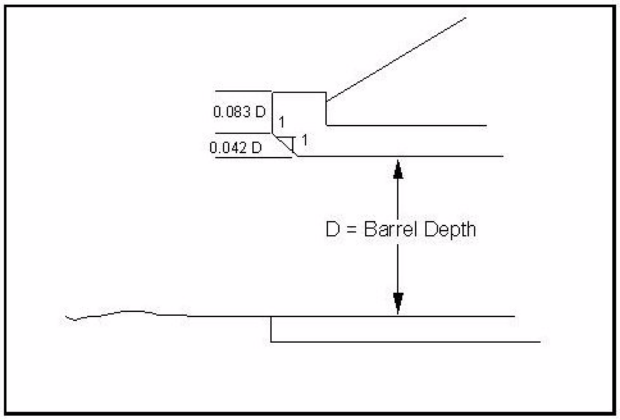

Figure 8-22. Beveled Entrance

Flared Entrance Design for Circular Pipe

In certain instances if a circular pipe culvert of sufficient barrel length is operating under inlet control, a flared entrance as an inlet improvement may serve to increase the hydraulic capacity with a corresponding savings in the initial cost of the culvert barrel. A sufficient barrel length would be such that the reduced cost of the smaller diameter barrel more than offsets the additional cost of the flared inlet.

A flared entrance for a pipe culvert is practical only when steep-slope inlet control conditions exist. Figure 8-23 shows the dimensions of a circular improved inlet.

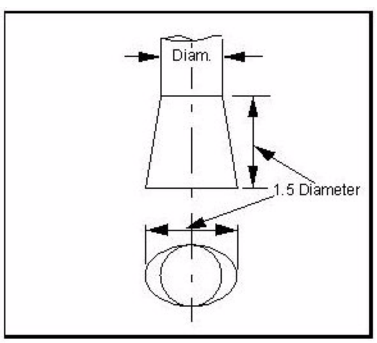

Figure 8-23. Circular Pipe Flared Inlet

For any circular pipe culvert operating under inlet control, you may use a flared inlet to reduce the size of the barrel. Use the design procedure outlined in Design Procedure for Culverts in Section 4. For the inlet control headwater, the table titled Regression Coefficients for Inlet Control Equations provides coefficients for concrete and corrugated metal circular pipe with flared inlets for use in Equation 8-1. Note the following conditions:

- If the culvert is on a mild slope ($d_c < d_u$), a flared inlet is not likely to be any more effective than a bevel.

- If the inlet analysis procedure indicates outlet control, a flared inlet is not an efficient application.
- Trial size is verified when the following conditions are met:

  $HW_{ic} < HW_A$,

  $d_u < d_{c,}$

  $HW_{oc} < Hw_{ic.}$
- If you can verify the trial size, compare costs with a culvert designed without a flared inlet; calculate the culvert outlet velocity in accordance with the procedure outlined for an inlet control culvert.
- Do not cut the flared inlet unit to a skew even if the culvert is skewed with respect to the roadway (see Figure 8-24).
- If you cannot verify the trial size, simply design the culvert without a flared inlet in accordance with the usual procedure.

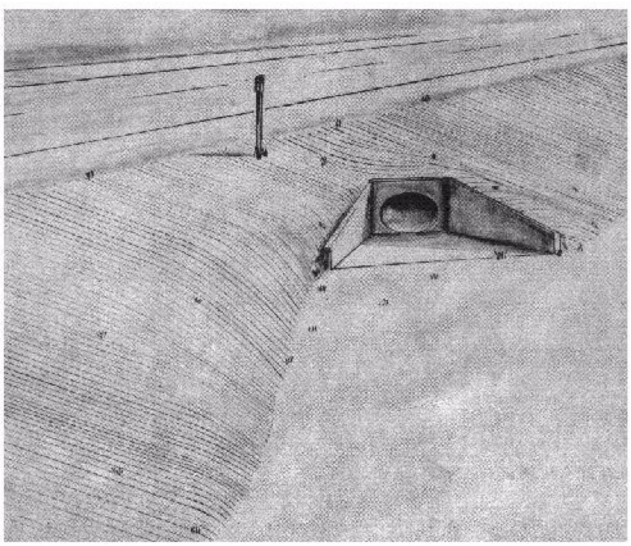

Figure 8-24.

Section 5

Velocity Protection and Control Devices

Excess Velocity

If you consider the outlet velocity to be excessive, several possible solutions are available, for both protection and control to minimize the negative effects of velocity. The excessive velocity may be accommodated.

Accommodation might require taking steps such as deepening the toes of culvert outlet aprons to accommodate channel degradation downstream and purchasing channel easements large enough to accommodate degradation and subsequent local widening of the channel.

Minor configuration changes in the culvert barrel may reduce an excessive velocity to a more acceptable exit velocity. For situations involving excessive outlet velocities in culverts operating under inlet control, it is possible to roughen the conduit or even change geometry of the conduit and yet not affect the headwater characteristics.

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 guidelines -- You should apply channel liners, when used as an outlet velocity protection measure, to the channel area immediately downstream of the culvert outlet for some distance, possibly to the right of way and beyond (with appropriate easement). You may temper an arbitrary limit, such as the right of way, by engineering judgment based on the severity of the velocity and the potential for erosion.
- ◆ Liner types -- 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.
- ◆ Pre-formed outlets -- An effective protection device consists of a pre-formed outlet (scour hole) in the area 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. Protect the recovery reach well 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 8-25 and Figure 8-26. 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 8-25 and Figure 8-26) 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 -- Limited research and some prototype installations show that the use of the sill as depicted in Figure 8-27 is effective in forcing the hydraulic jump in broken-back culverts and in spreading the water back to the natural stream width.
- Roughness baffles -- Figure 8-28 shows an example of roughness baffles that can be effective in inducing turbulence, dissipating energy, and reducing culvert outlet velocity.
- 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*.

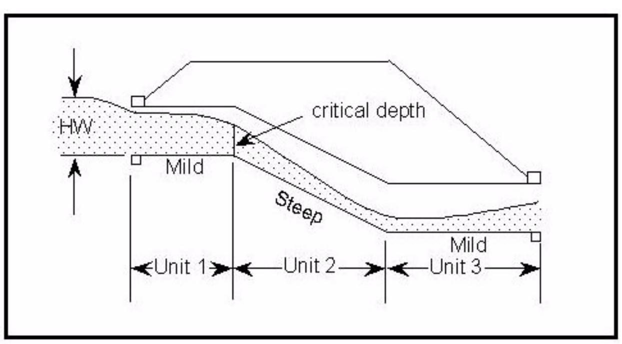

Figure 8-25. Three Unit Broken Back Culvert

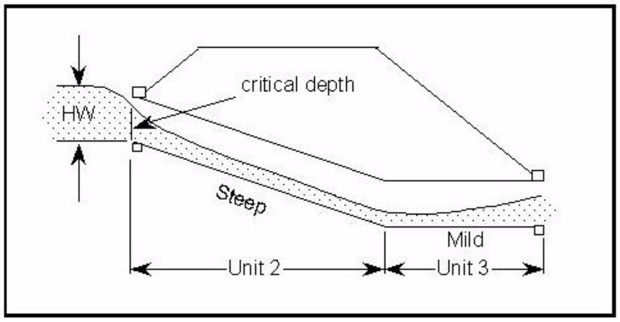

Figure 8-26. Two Unit Broken Back Culvert

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 8-25 and Figure 8-26, 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, ΣL, 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 you may produce it artificially by installing a sill on the downstream apron between wingwalls (refer to the Sills subsection below). Determine the total vertical dimension of this artificial tailwater 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 apply and you should 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, ΣL, to unit 3 length. If ΣL ≥ 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.

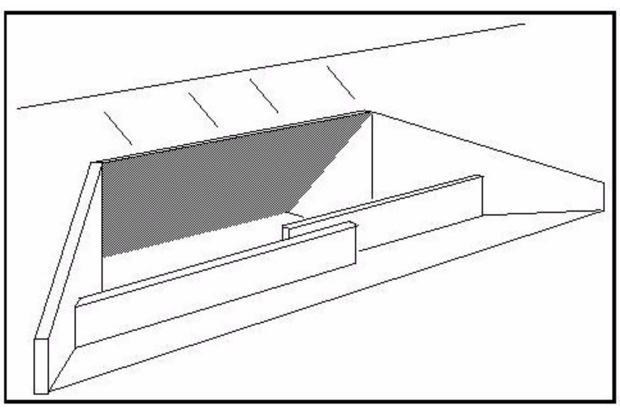

Figure 8-27. Sills.

Sill Guidelines

Do not limit the use of sills to broken-back culverts as they may retard excessive velocities effectively in any type culvert. Some disadvantages of sills are the possible propensity for silting and the waterfall effect that they usually cause. In certain areas, you must maintain the sill frequently to keep it free of sediment deposition.

Historically, the most widely used control has been the use of riprap that covers the channel area immediately downstream from the culvert outlet. Riprap should be installed immediately downstream of the sill for a minimum distance of 10 ft. (3 m) to protect features from the turbulence of the waterfall effect. Locate sills at the midpoint in the downstream culvert wingwall and make them at least half the depth of the culvert barrel.

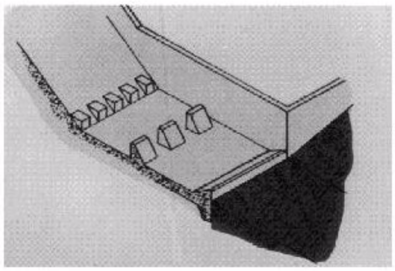

Figure 8-28. Roughness Baffles

Energy Dissipators

Impact basins are effective energy dissipators but are relatively expensive structures (see Figure 8-29).

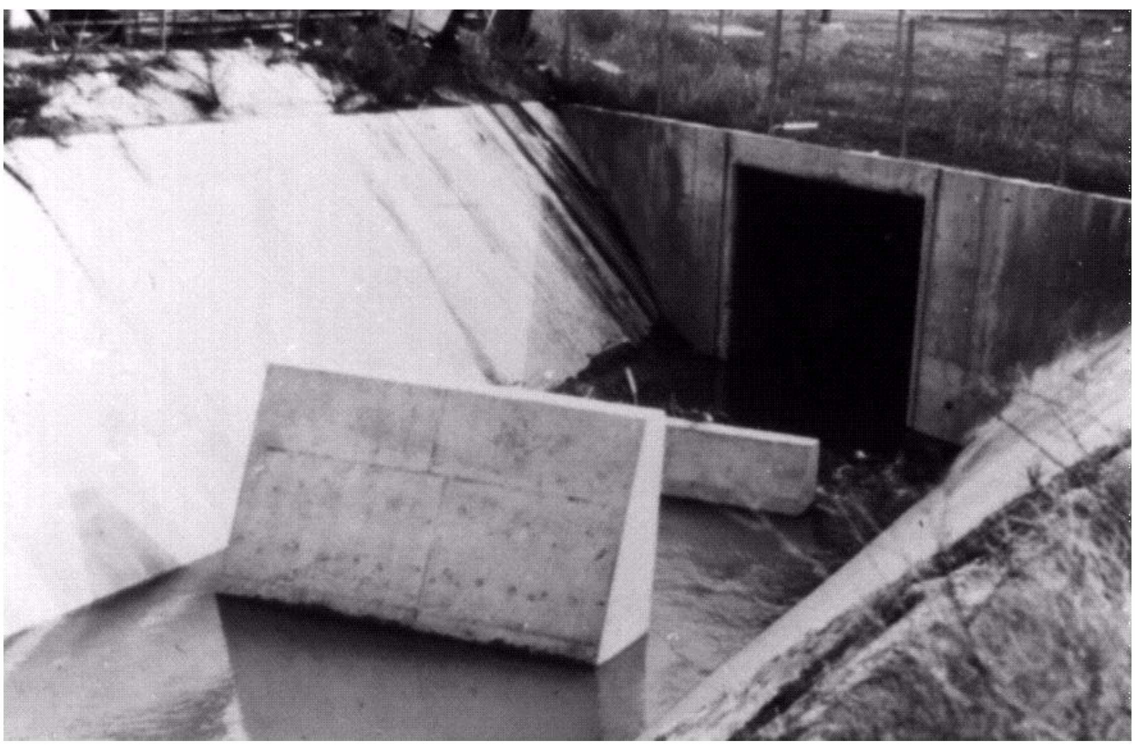

Figure 8-29. Impact Basins

Stilling basins are hydraulically similar to sills (8-30). 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.

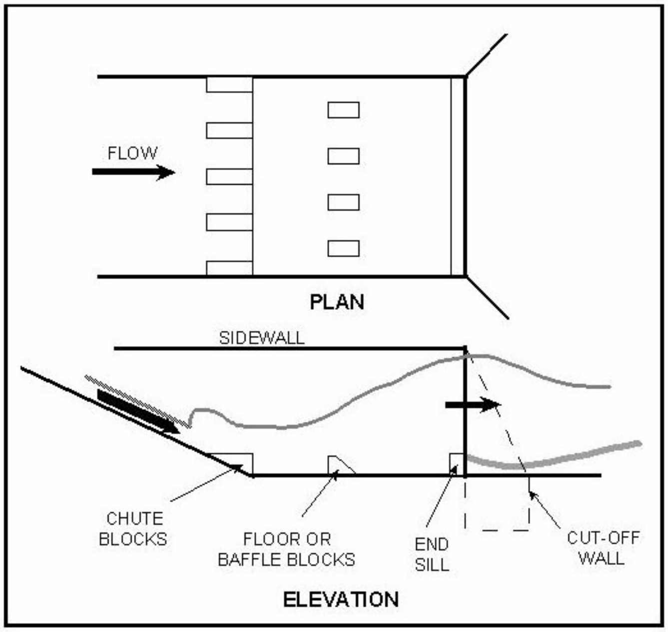

Figure 8-30. Basins

Radial energy dissipators are quite effective but extremely expensive to construct and, therefore, not ordinarily justified (Figure 8-31). 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.

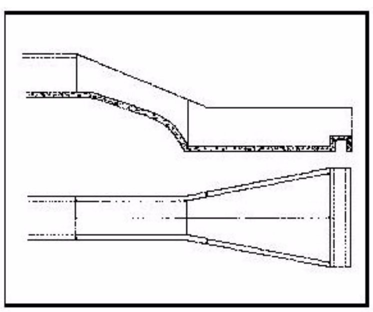

Figure 8-31. Radial Flow Energy Dissipator

Chapter 9
Bridges

Contents:

Section 1
Introduction

Hydraulically Designed Bridges

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, you must recognize and consider the hydraulic requirements of a stream crossing during the development, construction, and maintenance highway phases

This chapter addresses hydraulic engineering aspects of bridge stream crossings. It does not provide detailed information on tidal areas such as bays and estuaries. *Texas Standard Specifications* defines bridges as any structure measuring more than 20 ft. (6 m) along the roadway centerline between the insides of the end walls. Bridges, as distinguished from culverts, are usually supported on piers or abutments.

This chapter addresses structures designed hydraulically as bridges, regardless of length. 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.. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the floodplains.

Section 2
Planning and Location Considerations

Introduction

Generally you select a stream crossing location during the planning and location phase of a highway project. Select the final location only after obtaining detailed survey information and completing preliminary hydraulic studies. Although they are not the sole consideration in bridge location, 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. Identify adverse conditions in the early stages of new location selection so that potential problems receive adequate consideration. If the cost of required structures is prohibitive, consider rerouting the highway.

National Objectives

In site selection, consider national objectives such as the following:

- Observe US Coast Guard navigation clearance requirements for marine traffic
- Reduce the rate of annual increases in flood losses by restricting the use of floodplains. This means that highways in the vicinity of streams must conform to the Federal Emergency Management Agency National Flood Insurance Program requirements. (See the FEMA Designated Floodplains subsection in this section as well as FEMA NFIP in the Project Development Policy Manual for more information.)
- Preserve the wetlands. Wetlands have wildlife habitat, high productivity of food and fiber, and beneficial effects on flooding, pollution, and sediment control.

Select stream-crossing locations that minimize impact to wetlands unless you undertake appropriate mitigation.

Location Selection and Orientation Guidelines

Consider these guidelines in selecting and orienting bridge locations:

- Ordinarily, you should locate and center the bridge 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 it allows for better accommodation of the usual and low flows of the stream.
- Design the bridge waterway opening to provide a flow area sufficient to maintain the through-bridge velocity at no greater than the allowable through-bridge velocity under the circumstances of design discharge.

- Orient headers and interior bents to conform to the streamlines at flood stage. Accomplish this within reason, using standard skew values (15°, 30°, 45°, etc.) where feasible. Locate the toe of slope of the header away from deep channels, cuts, and high velocity areas.
- Locate and orient the bridge headers and piers to minimize the potential for excessive scour.
- If the intrusion of either or both roadway headers into the stream floodplains is more than about 800 ft. (240 m), consider including either relief openings or guide banks.
- Incorporate existing vegetation in the overall bridge plan. Where practicable, leave trees and shrubs 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. On the other hand, you should consider safety and maintenance aspects of retaining vegetation within the right-of-way and near the travel lanes.
- For some configurations, you may need to incorporate roadway approaches that accommodate overflow. Such overflow approaches allow floods larger than the design flow to overtop the roadway, thereby reducing the threat to the bridge structure itself. Of course, this interrupts the function of the roadway, and you need to consider the potential costs associated with such interruption and associated damage to the embankment.

Environmental Considerations

Evaluate potential effects of the crossing site on the environment, including hydraulic aspects, chemical quality, aesthetic aspects, and biological aspects.

Environmental considerations for the hydraulic and physical aspects of water quality at proposed sites are the same concerns historically addressed in evaluating the relative merits of alternate locations. These include the effects of the crossing on velocities and flow distribution, water surface profiles, scour, bank stability, and sediment transport.

Effects of a highway on the chemical quality of surface waters are not ordinarily a consideration in site selection although it is possible that contaminants in the form of minerals or sanitary landfill leachate could be exposed in one location and not at an alternate site. There is, however, some concern for chemical quality at crossing sites near public water supply intakes due to the risk of toxic material spills. Consider the probability of such spills in site selection.

Aesthetic considerations include visual, odor, and taste effects on the surface waters. Consider the aesthetic quality of surface waters in site selection that involves potable water supplies, water contact sports, and fisheries. The visual quality often affected by highways under construction is temporary turbidity.

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.

Coordination with Other Agencies

Coordinate with agencies responsible for proposed or existing water resource development projects in the planning and location phase of highway plan development. This allows for early agreement on cost pro-ration for planned projects and selecting optimal highway locations considering water resources development projects. Coordination with water resource agencies will, at times, provide opportunities to conserve public funds by each agency incorporating provisions in its plans to accommodate the needs of the other. You can undertake mutually beneficial work by either an equitable cost-sharing agreement or construction contract documents that meet the requirements of both agencies.

Surface Water Interests

Numerous local, state, and federal agencies have vested interests in surface waters. These agencies represent interests in the following:

- water rights
- flood control
- drainage
- conservation
- navigation and maintenance of channels
- recreation
- floodplain management
- safety of floodplain occupancy, fish, and wildlife
- preservation of wetlands
- regulation of construction for the protection of environmental values
- gauge stations and apparatus for measuring data

Other local, state, and federal agencies have vested interests in historic and archaeological preservation including historic bridge structures and archaeological resources. Early coordination with these agencies will reveal areas of mutual interest and offer opportunities to conserve public funds and to resolve conflicts between TxDOT plans and those for water resources development and resource protection and preservation. The following agencies are commonly involved with bridge planning and location:

- U.S. Army Corps of Engineers (USACE)
- Federal Emergency Management Agency (FEMA)
- International Boundary and Water Commission (IBWC)
- Texas Natural Resource Conservation Commission (TNRCC)
- Texas Water Commission (TWC)
- U.S. Coast Guard (USCG)
- U.S. Department of the Interior, U.S. Geological Survey (USGS)
- drainage districts
- flood control districts
- levee districts/municipal utility districts
- river authorities
- water authorities.

See References for more information on the specific agencies listed above.

Water Resource Development Projects

Water 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 are never built and may even be permanently stopped by court decisions or regulatory agency actions.

When you choose stream-crossing locations to take advantage of or to accommodate planned water resources development projects (such as reservoirs or stream channel modifications), recognize that the water resources agency plans may never come to fruition. Also recognize that you must design the highway facility for both existing and future site conditions. Carefully study planning and constructing a highway facility at a water resources project. Consider the excess cost of building the facility due to the water resources project in selecting the stream-crossing site.

FEMA Designated Floodplains

The Project Development Policy Manual discusses the National Flood Insurance Program (NFIP) of the Federal Emergency Management Agency (FEMA). The majority of highway crossings involve floodplains that are in FEMA-participating communities. FEMA criteria may often control the design of a bridge over a waterway, and it is important to acknowledge FEMA requirements 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.

Stream Characteristics

All streams change with time, and you can recognize the rate and manner in which they will change. Planning 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. Ensure that highway work within a stream environment does not incur significant change in the stream morphology.

Replacement, Repair, and Rehabilitation

The decision to replace, repair, or rehabilitate a bridge is often made in the planning and location phase of highway project development. You may replace bridges for any of the following reasons:

- structural inadequacies or deterioration
- structural damage from collision
- alignment and geometric inadequacies
- flood-related damage, damage due to scour and debris impact
- inadequate clearances for navigation
- plans for water resources projects

Examine the hydraulic adequacy of an existing crossing before making a decision. The purpose of the examination is:

- to determine if the existing crossing will prove adequate for changed traffic service requirements
- to evaluate flood hazards and risks

The occurrence of rare floods rather than the hydraulic inadequacy of the existing crossing causes some of the problems at a crossing. Serving well over a long period of time does not necessarily assure a stream crossing's hydraulic adequacy. The odds are 2 to 1 that a 20-year old bridge has not experienced a 50-year flood, and over 4 to 1 that a 100-year flood has not occurred during the existence of the same bridge.

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. Check previous methods used by following these steps:

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

When beginning analysis for a cross-drainage facility, first establish the flood frequency curve and the stage-discharge curve according to the principles described in the Concept of Frequency and Peak Discharge Versus Frequency Relations in Chapter 5, and Open Channel Flow in Chapter 6, and make a decision concerning the type of cross-drainage facility. The choice is usually between a bridge or a culvert. Usually choose bridges if the discharge is significant or if the stream to be crossed is large in extent. You might evaluate both types of facilities and make a choice based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves the following:

- determining the backwater associated with each alternative profile and waterway opening(s)
- determining the effects on flow distribution and velocities
- estimating 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.

Flow through Bridges

When flood flow encounters 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 bridge contracts flow, which 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.

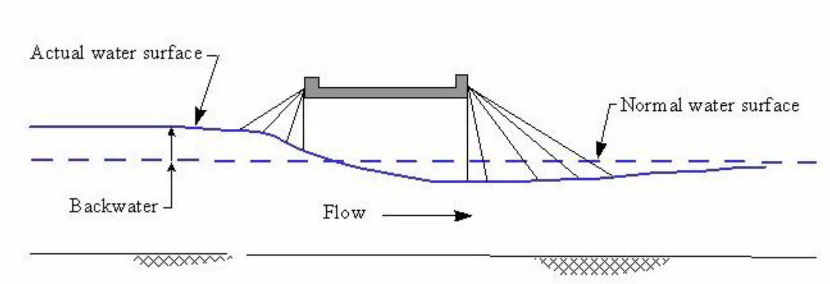

Figure 9-1. Backwater at a Stream Crossing

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.

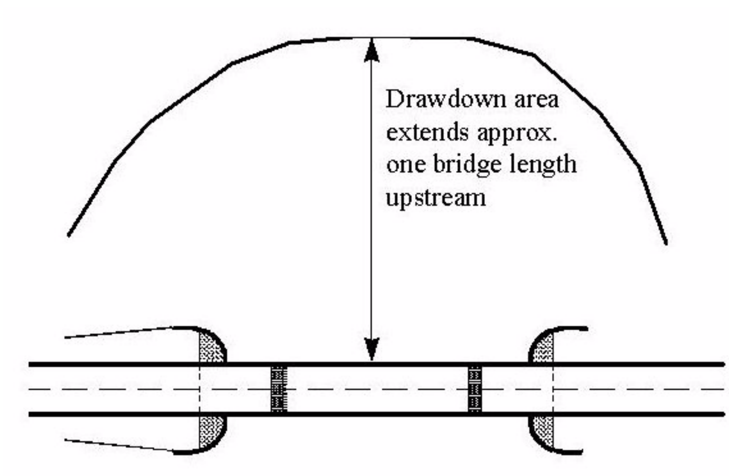

Figure 9-2. Extent of Backwater Drawdown

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 moves toward the bridge opening, the velocity increases. This increase 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.

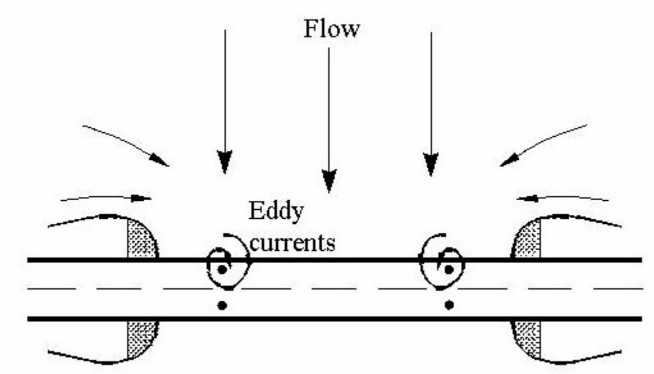

Figure 9-3. Typical Eddy Currents through Bridge Opening

Allowable Backwater Due to Bridges

For design frequency conditions, establish the allowable backwater based on consideration of the risk associated with the potential for incurring flood-related damage to the highway and adjacent property. In general, the department is responsible only for the increase in the upstream water surface (backwater).

The backwater associated with the 100-year event must conform to FEMA NFIP requirements where applicable. (See FEMA Designated Floodplains in Section 3 and the Project Development Policy and Manual for more details.) Where FEMA criteria do not apply, a maximum backwater of 1 ft. (0.3 m) is desirable.

Generally, if you think the potential risk associated with the backwater excessive, consider one or more of the following actions:

- Perform a detailed economic analysis of various design options.
- Minimize or eliminate the backwater to the extent practicable.
- Design channel improvements.
- Determine the incremental area of high-risk inundation and arrange purchase of flood easements or buy out affected property.

Flow Distribution

Any stream crossing that uses a combination of fill and bridge within the floodplain disturbs flow distribution during some floods. However, preserve flow distribution 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, you can minimize the disturbance of flow distribution by establishing 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, as in Figure 9-4, and skewed crossings, as in Figure 9-9, with one-dimensional models only, you need a great deal of intuition, experience, and engineering judgment to supplement the quantitative analysis. Unfortunately, you frequently encounter complex sites 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.

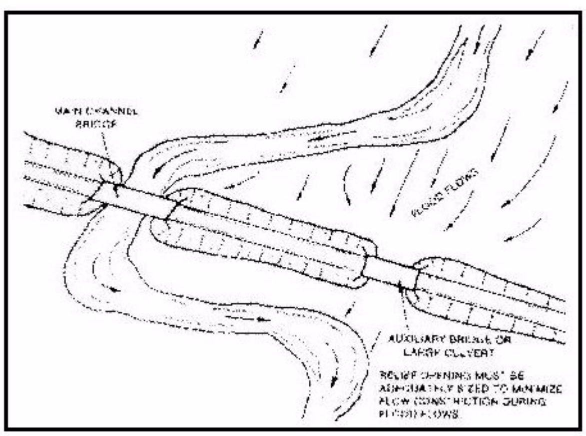

Figure 9-4. Highway Stream Crossing at a Bend

Velocity

A velocity profile exists in the cross section of flow in any bridge opening. Figure 9-5 shows an example of a velocity profile through a bridge opening. The velocity varies significantly within the cross section and has negative velocities (or reverse flows). However, the only 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)

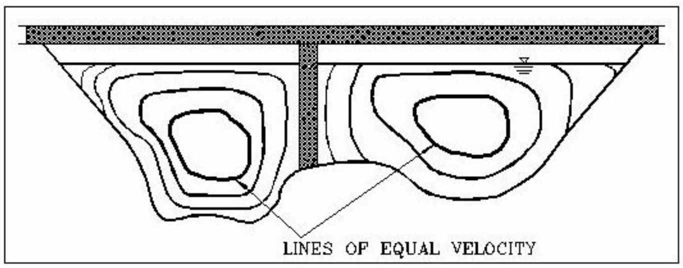

Figure 9-5. Velocity Profile Through Bridge Opening (heavier lines = higher velocity)

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 through-bridge velocity is the basic sizing criterion used for span-type bridges. Generally, the waterway opening defined by the following should cause an average through-bridge velocity of 6 fps (1.8 m/s) or less:

- design high-water
- left and right header slopes
- natural ground profile (or proposed through-bridge channel section)

Accomplish this by moving the header slopes closer together or further apart as necessary. The maximum allowable average through-bridge velocity of 6 fps (1.8 m/s) is arbitrary and may vary across the state. Such variation of this important design criterion usually involves engineering judgment.

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. If the natural stream velocity is already higher than 6 fps (1.8 m/s), you may size the bridge simply by spanning the natural stream without causing restriction. Velocities lower than about 3 fps (1 m/s) ordinarily are not recommended because of the economic disadvantage of longer bridges. However, there may be instances when you need to provide relief structures with design velocities lower than 3 fps (1 m/s).

Bridge Scour and Stream Degradation

A scour analysis is required for new bridges, replacements, and widenings. Generally, design a bridge based on target velocities and to accommodate backwater considerations. If a scour analysis indicates high depths of potential contraction scour, it may be more cost-effective to provide a structure larger than that required by the basic velocity and backwater criteria than to design foundations and armoring to withstand the scour. You can reduce potential for deep local scour 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 include providing longer structures with deep enough foundations to accommodate anticipated degradation or designing deep enough foundations with abutment foundations designed to act as interior bents to allow future lengthening of the bridge. For other possible measures, such as river training techniques, see FHWA-IP-90-014, Stream Stability at Highway Structures (HEC 20) and the Bank Stabilization and River Training Devices subsection in Section 8. (See the Federal Highway Administration for information on obtaining this document.)

Freeboard

Navigational clearance and other reasons notwithstanding, the lowchord 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 minimum freeboard of 2 ft. (0.6 m) to allow for passage of floating debris and to provide a safety factor for design flood flow.

- 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.
- Other constraints may make lower freeboards desirable, but the lowchord must not impinge on the design high water.

Generally, for off-system bridge replacement structures, the lowchord 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 can be any part of the structure that stream flow can strike or impact in its movement downstream. If the stream gets high enough to inundate the structure, then all parts of the roadway and the bridge become part of the complete bridge profile.

For department design, do not inundate the roadway by the design flood, but you may allow inundation by the 100-year flood. In fact, 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 never overtop.

In Figure 9-6, the bridge is at the low point in a sag-vertical curve profile. Extreme examples of this configuration are the use of low bridges in rolling terrain for low-traffic roads that are frequently overtopped and high bridges in rugged terrain that probably will never be threatened by floods. A distinctive feature of this profile is the certainty that the bridge structure will be submerged when any overflow of the roadway occurs.

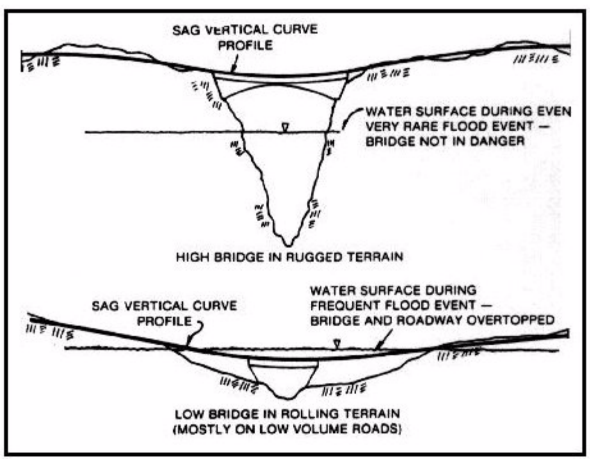

Figure 9-6. Sag-Vertical Curves

If possible, avoid bridges on sag-vertical curves if accumulation of drift in the superstructure is likely. Trapped debris can increase the potential for scour. Also, large drift can produce high impact forces on the structure, possibly causing structural failure, especially if scour has affected the foundations.

If you consider a sag-vertical curve and have even a small probability of overtopping, avoid curbs and use open-type railing 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.

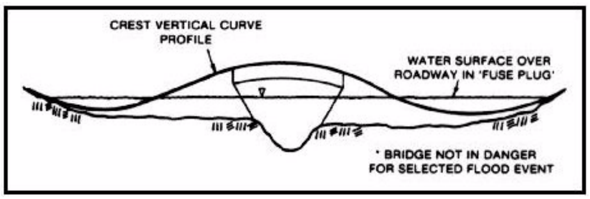

Figure 9-7. Crest Vertical Curve

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 shows a third 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.

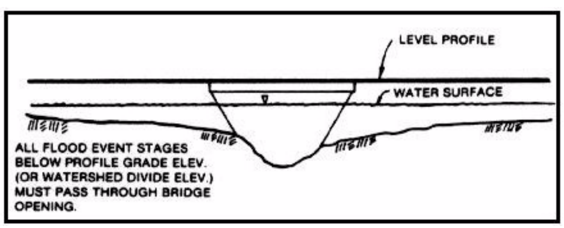

Figure 9-8. Level or Slight Crest Vertical Curve

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. The velocity of flow and depth of the superstructure may impose large hydraulic forces on the bridge superstructure. The accumulation of debris on the upstream side of the structure can increase the effective depth of the superstructure, impose larger hydraulic forces on the bridge superstructure, and increase scour depths.

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

Consider the horizontal alignment of a highway at a stream crossing in selecting the design and location of the waterway opening, as well as the crossing profile. Make every effort 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 9-9, ensure that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow.

Orient bents to as near the skew of the streamlines at flood stage as possible. Skew headers to minimize eddy-causing obstructions. Also, you may want to provide a relief opening at the approximate location of point A to reduce the likelihood of trapped flow and 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 this differential.

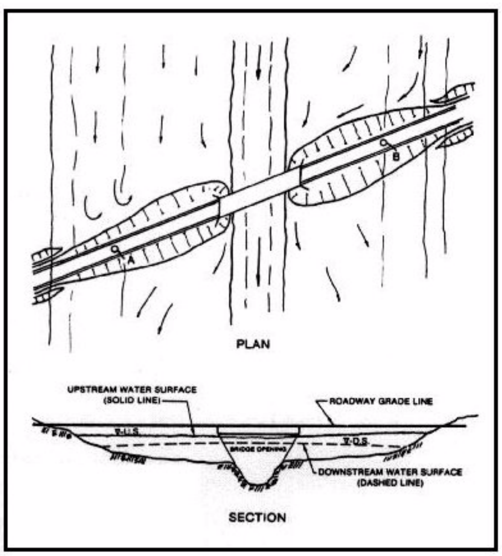

Figure 9-9. Skewed Stream Crossing and Water Surface Differentials

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, after accumulating a head, 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, the department recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft. (240 m). If flow distances along the embankment are greater than recommended, investigate a relief structure that will provide an additional opening. 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.

Also, 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

These discussions of bridge design assume normal cross sections and lengths (perpendicular to flow at flood stage) because you usually assume one-dimensional flow, consider cross sections and lengths at 90° to the direction of stream flow at flood stage.

- If the crossing is skewed to the stream flow at flood stage, normalize all cross sections and lengths before proceeding with bridge length design.
- If the skew is severe and the floodplain is wide, you may need to adjust the analysis to offset the effects of elevation changes within the same cross section.

The following examples illustrate other considerations that can cause a bridge opening to be larger than the bridge that hydraulic design requires.

- Bank protection might be placed in a certain location due to local soil instability or a high bank.
- Bridge costs might be cheaper than embankment costs.
- A highway profile grade line might dictate an excessive freeboard allowance. For sloping abutments, a higher freeboard will result in a longer bridge.
- High potential for meander to migrate, or other channel instabilities may warrant longer opening.

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. Document design and the reasons for any excess opening.

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, you should refer to the documentation of the specific computer program 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.

Flow Zones and Energy Losses

Figure 9-10 shows a plan of typical cross section locations that establish three flow zones that you should consider when estimating the effects of bridge openings.

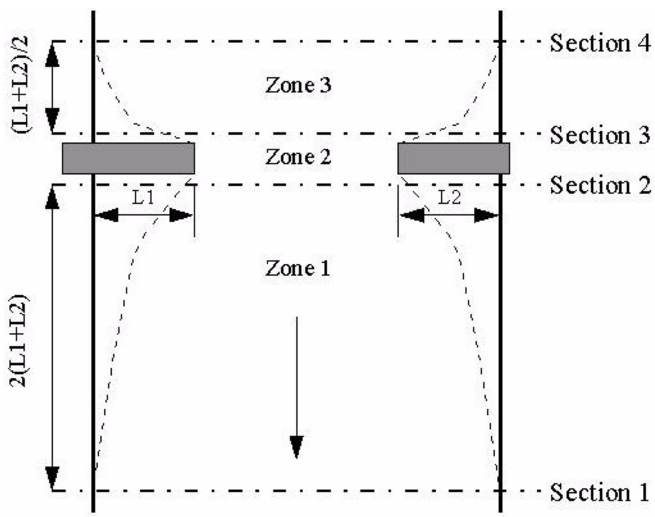

Figure 9-10. Flow Zones at Bridge

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.

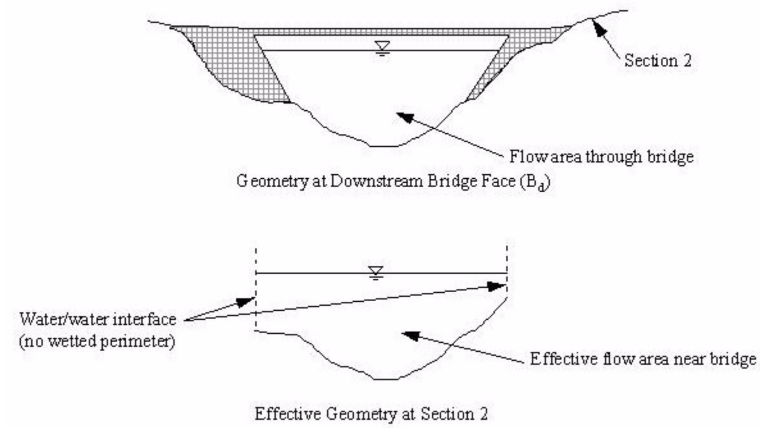

Figure 9-11. Effective Geometry for Bridge (Section 2 shown, Section 3 similar)

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. (This is a requirement for FEMA designated floodplains.)

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 below provides recommended loss coefficients.

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
Typical bridge	0.3	0.5

Recommended Loss Coefficients for Bridges

Transition Type	Contraction (K_c)	Expansion (K_e)
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 the “Low Flow Classes” table below. 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.

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 your own judgment:

- Standard Step Backwater Method (based on balance of energy principle)
- Momentum Balance Method

- WSPRO Contraction Loss Method
- 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.

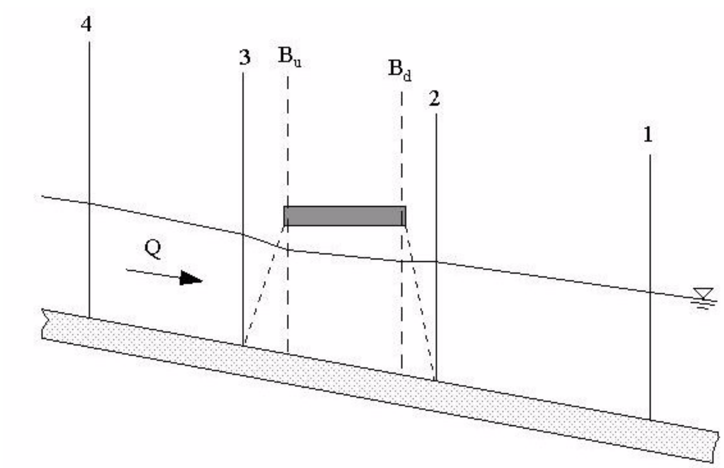

Figure 9-12. Relative Location of Section Geometry

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 you achieve equality within a reasonable tolerance.
3. Determine the momentum correction factor (B), which accommodates natural velocity distributions similar to the energy correction factor, *f* ¿, 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. The “Suggested Drag Coefficients for Bridge Piers” table 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 B_d 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

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 (30o apex)	1.00
Triangular nose (60o apex)	1.39
Triangular nose (90o apex)	1.60
Triangular nose (120o 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.

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 lowchord 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)

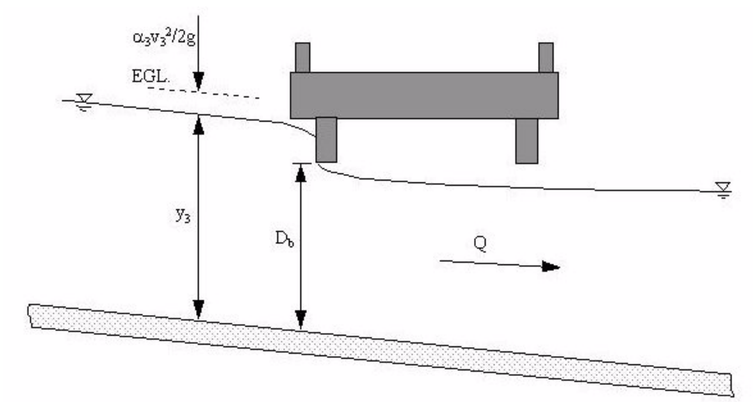

Figure 9-13. Sluice Gate Type Pressure Flow

Figure 9-14 shows a submerged bridge opening with a tailwater lower than the overtopping elevationEquation . 9-8 represents orifice flow. You need to assume successive elevations at cross section 3 (y_3) until the calculated discharge 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:

$\alpha 3$ = kinetic energy correction coefficient

C_d = coefficient of discharge, Equation 9-12

$$C_d = 0.104 \frac{L_c}{b} + 0.7145$$

Equation 9-10.

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)

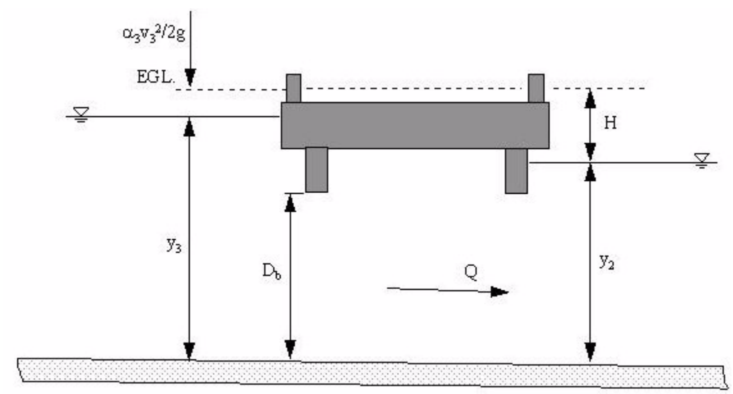

Figure 9-14. Orifice Type Pressure Flow

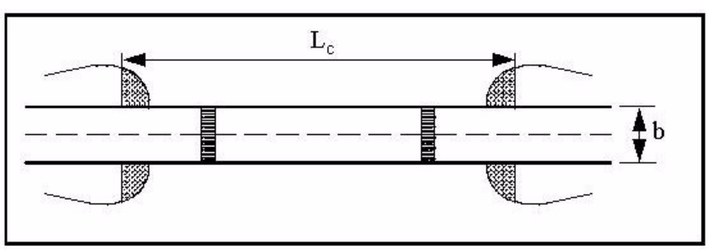

Figure 9-15. Bridge Dimensions for Pressure Flow Analysis

Empirical Energy Loss Method (HDS 1)

Although hand computations are used rarely, the FHWA publication *Hydraulics of Bridge Waterways* (HDS 1, 1978) presents methods for estimating bridge backwater effects. (See Reference for information on obtaining this document.) 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 Bridge 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 sections of Chapter 8, except that you 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, you can consider 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

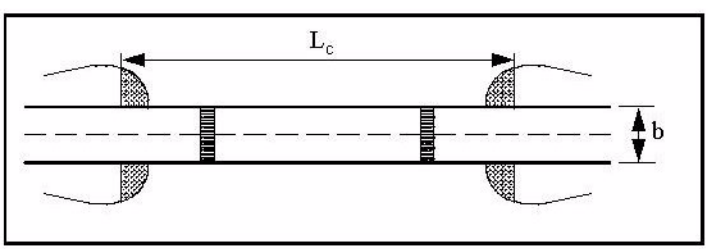

Figure 9-16. Parallel Bridges

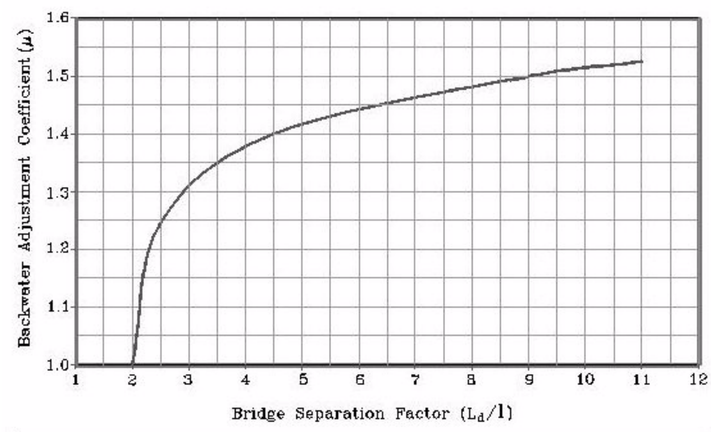

Figure 9-17. Parallel Bridges Backwater Adjustment

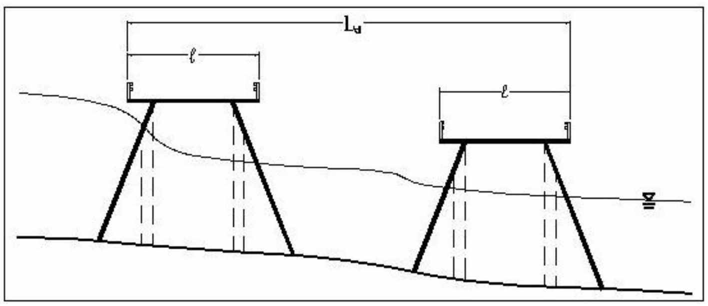

Figure 9-18. Definition of Parameters

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 your selection of methods 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 100-year 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 as discussed in Section 6, Bridge Scour.

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.

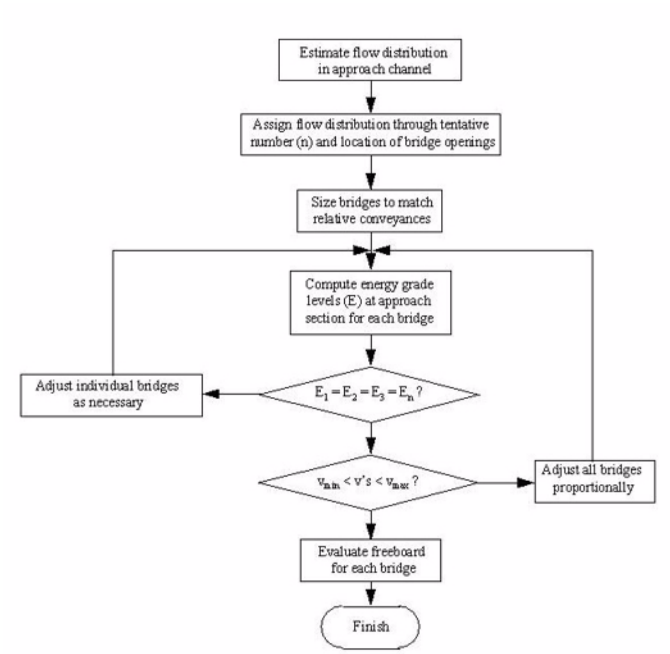

Figure 9-19. Multiple Bridge Design Flowchart

When estimating the design high water at a multiple structure location, you still need 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, your 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, you 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 you must estimate them analytically.

A cumulative conveyance (or discharge) curve is the most straightforward method for estimating these heavy concentrations of flow. Construct a cumulative conveyance curve as follows:

1. Apply the design highwater to a natural stream cross section in the immediate vicinity of the proposed design cross section. The section chosen for this high-water application should be a typical cross section that may control the flow distribution in the reach of the stream in which the structure is located. It will usually be an upstream cross section.
2. Calculate cumulative natural conveyances for each subarea across the section from left bank to right bank.
3. Plot cumulative natural conveyance values across the cross section stationing, as shown in Figure 9-20. The value of the last ordinate is equal to the total conveyance of the stream cross section at a given water surface elevation.
4. Determine the design discharge. Inspect the cumulative conveyance curve, and observe that, in the vicinity of points 1, 2, and 3, the slope of the curve is relatively steep. A rapid increase in natural conveyance with respect to distance across the cross section causes this steepness. These areas of steepness (indicative of flow concentrations) in the cumulative conveyance curve define the approximate best locations for bridges. The points on the curve where the slope is more horizontal define the approximate locations of flow divides. Determine the portion of the design discharge carried between the flow divides by direct calculation or by proportion of relative natural conveyances.

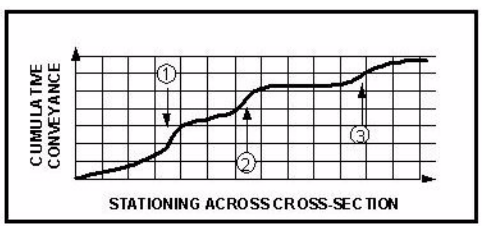

Figure 9-20. Cumulative Conveyance Curve

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, 2 ft. (0.6 m). See Freeboard in Section 3 for more information.

One-dimensional analysis of existing locations involves the same concepts employed for designing new systems: assume that the flood flow will distribute itself to attain a constant energy grade at the approach section. The existing bridges will likely redistribute flow from what approaching channel conditions might otherwise imply. The stagnation points then are a function 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 to adjust stagnation points, determine conveyance apportionment, and analyze each opening.

Section 6

Bridge Scour

Introduction

Scour is the result of the erosive action of flowing water excavating and carrying away material from the bed and banks of streams. Potential scour can be a significant factor in the analysis of a stream crossing system. The design of a crossing system involves an acceptable balance between a waterway opening that will not create undue damage by backwater or suffer undue damage from scour and a crossing profile sufficiently high to provide the required traffic service.

Rates of Scour

The rates of scour in different materials and under different flow conditions depend on erosive power of the flow, erosion resistance of the material, and a balance between sediment transported into and out of a section.

With erosion-resistant materials, final, worst case, or equilibrium scour may not be reached in any one flood but may develop over a long series of events. The methods currently available do not specifically accommodate cohesive bed materials or time-dependency. Therefore, consider the results of any scour calculations only as an indication of the maximum potential scour. Use judgment to decide whether or not calculated depths are likely for the given site conditions and life expectancy of the bridge.

All design projects involving new, rehabilitated, and widened bridges over waterways should include estimates of the potential scour envelope using velocities and flow depths resulting from the 100-year flood and the lower of the 500-year flood and overtopping flood. Basic scour equations are presented here; however, the designer should refer to “Evaluating Scour at Bridges” (HEC 18, 1995) for detailed discussion and analysis procedures.

Evaluate existing bridges for potential failure due to scour. Recommended procedures for performing such evaluations are provided in “Texas Secondary Evaluation and Analysis for Scour” (TSEAS, 1993).

Scour Components

In simple terms, scour consists of long-term aggradation and degradation (natural scour), contraction scour, and local scour.

High local velocities and flow disturbances such as eddies and vortices (refer to Figure 9-21 and Figure 9-22) cause local scour. Generally, experts consider the effects of all three scour components to be additive (see Figure 9-23).

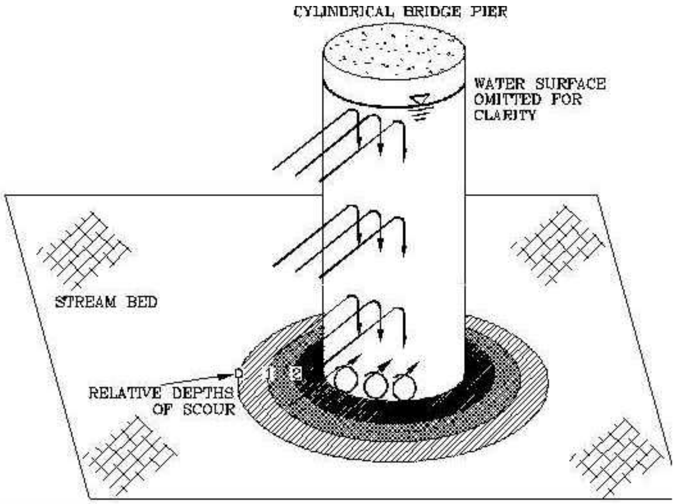

Figure 9-21. Local Scour Due to Eddies

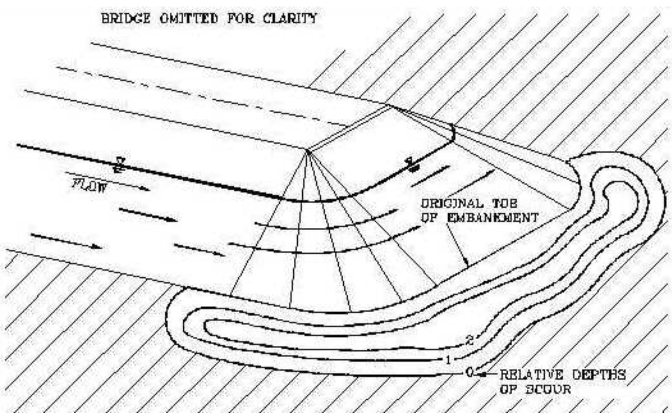

Figure 9-22. Isometric View of Scour at Bridge Abutments

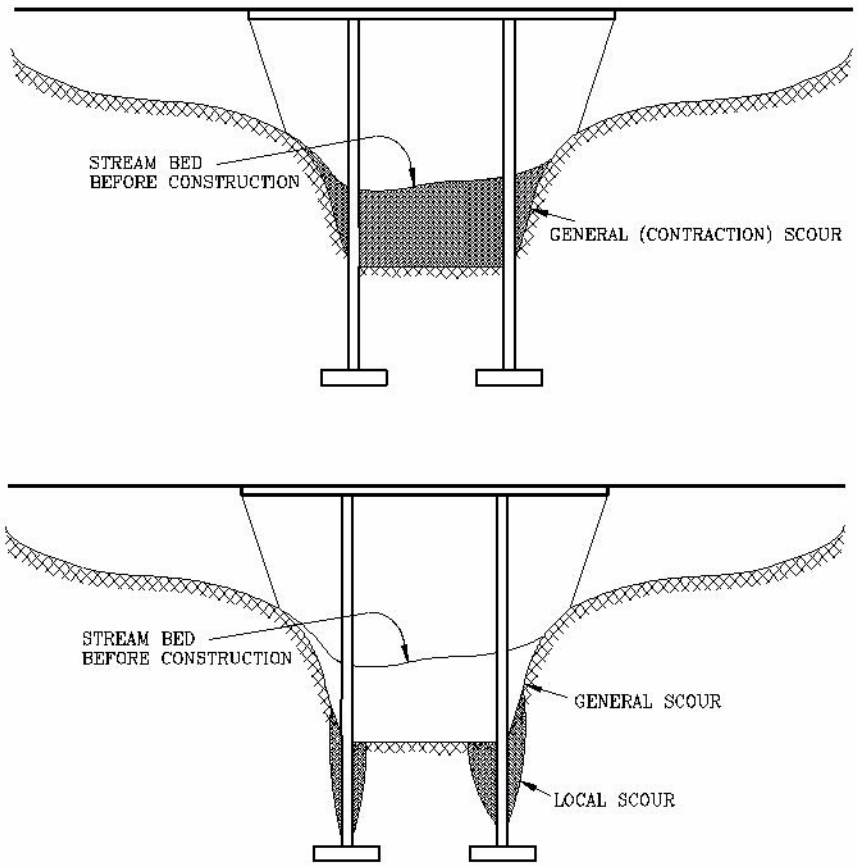

Figure 9-23. Additive Effects of Scour

Natural scour may be the result of lateral stream migration, natural trend of the stream, or some modification to the stream or watershed. Any or all of the following factors may affect the depth and area of natural scour at a waterway opening:

- slope, natural alignment, and shifting of the channel
- type and amount of bed material in transport
- nature and occurrence of flood
- accumulations of debris
- constriction or realignment of flow due to the stream crossing
- layout and geometry of training works
- geometry and orientation of piers
- classification, stratification, and consolidation of bed and sub-bed materials
- placement or loss of riprap and other protective materials
- natural or constructed changes in flow or sediment regimes

- failures such as collapse of a nearby structure

No reasonable, definitive methods are apparent for accurately estimating long-term natural scour. However, consider the potential for long-term natural scour. Generally, projections based an evaluation of the history of the site or ones similar to the site may suffice.

Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage decreases either by a natural contraction or by a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction, thus increasing erosive forces and removing more bed material from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation decreases, the flow area increases, and the velocity and shear stress decrease until relative equilibrium is reached, i.e., until the quantity of bed material that is transported into the reach is equal to that removed from the reach.

Contraction scour is typically cyclic. That is, the bed scours during the rising stage of the runoff event, and fills on the falling stage. The following factors can cause contraction of flow due to a bridge:

- a natural decrease in flow area of the stream channel
- abutments projecting into the channel
- piers blocking a large portion of the flow area
- approaches to a bridge cutting off the floodplain flow

These approaches can cause clear-water scour on a setback portion of a bridge section or relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments.

Depending on the stream flow, contraction scour can be either live-bed or clear-water. Live-bed scour occurs when the bed material upstream of the constriction is in motion. The scour that results at the constriction reflects equilibrium between the sediment transported into the section and that transported away from the section. Under live-bed conditions, scour holes created during the rising stage of a flood often refill during the recession stage.

Clear-water scour occurs when the bed material is not in motion. The sediment transported into the contracted section is essentially zero. Clear-water scour occurs when the shear stress induced by the water flow exceeds the critical shear stress of the bed material. Generally, with clear-water scour, no refilling occurs during the recession of the flood due to the lack of sediment supply. During the initial stages of a flood, clear-water scour could occur followed by live-bed scour at higher flood stages. Typical clear-water scour situations include the following:

- coarse bed material streams
- flat gradient streams during low flow
- local deposits of larger bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation)
- armored stream beds at piers or abutments where tractive forces can be high enough to penetrate the armor layer
- vegetated channel beds at piers or abutments where tractive forces can be high enough to remove the vegetation.

One way to appraise whether clear-water scour or live-bed scour is occurring is to compare the computed average velocity at the upstream approach to the constriction with the velocity at which incipient motion of the bed material (threshold velocity) can be expected. This approach is reasonable as long as the subject portion of channel is not heavily vegetated. You can determine critical velocity using Equation 9-15, which is based on a bed material with a specific gravity of 2.65. If $v_t < v$, the bed material is most likely in motion, and you can consider live-bed scour. If $v_t > v$, the bed material probably is not in motion and you may assume clear-water scour.

$$v_t = 6.36\, y^{1/6} D_{50}^{1/3}$$

Equation 9-15.

where:

v_t = threshold velocity (fps or m/s)

y = depth of flow (ft. or m)

D_{50} = median bed particle size (ft. or m)

Live Bed Contraction Scour Equation

Equation 9-16 compares the flow upstream of the contracted section with flow in the contracted cross section Equation 9-17 then computes the average live bed contraction scour depth (y_{cs}).

$$y_2 = y_1\left(\frac{Q_t}{Q_c}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k1}$$

Equation 9-16.

where:

y_1 = average depth in the upstream main channel (ft. or m)

y_2 = average depth in the contracted cross section (ft. or m)

Q_t = main channel flow upstream of contracted cross section (cfs or m^3/s)

Q_c = main channel flow in contracted cross section (cfs or m^3/s)

W_1 = bottom width of the upstream main channel (ft. or m)

W_2 = bottom width of main channel in the contracted cross section (ft. or m)

k_1 = an exponent determined using the “Exponent (k_1) for Live Bed Contraction Scour Equation” table below

v_s = shear velocity in upstream cross section (ft./s or m/s) = $(gy_1S_1)^{0.5}$

w = fall velocity of bed material (fps or m/s) based on Figure 9-24

D_{50} = mean bed material diameter (in. or mm)

S_1 = slope of energy grade line of main channel (ft./ft. or m/m)

$$y_{cs} = y_2 - y_1$$

Equation 9-17.

Exponent (k_1) for Live Bed Contraction Scour Equation		
v_s/w	**k1**	**Mode of Bed Material Transport**
< 0.5	0.59	Mostly contact bed material discharge
0.5 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

Fall Velocity of Non-Cohesive Particles

	D_{50}		**Geometric Mean Size**		**Fall Velocity, w**					
					32 Degrees		60 Degrees		100 Degrees	
	ft.	mm	ft.	mm	ft./sec.	m/s	ft./sec.	m/s	ft./sec.	m/s
Fine Gravel	0.0131 – 0.0262	4 – 8	0.0185	5.66	1.70	0.0518	1.70	0.0518	1.70	0.0518
Very Fine Gravel	0.00656 – 0.0131	2 – 4	0.00927	2.83	1.10	0.335	1.10	0.335	1.10	0.335
Very Coarse Sand	0.00328 – 0.00656	1 – 2	0.00464	1.41	0.60	0.183	0.70	0.213	0.75	0.229
Coarse Sand	0.00164 – 0.00328	0.5 – 1	0.00232	0.707	0.30	0.091	0.34	0.104	0.40	0.122
Medium Sand	0.000820 – 0.00164	0.25 – 0.5	0.00116	0.354	0.11	0.034	0.15	0.046	0.18	0.055

Fall Velocity of Non-Cohesive Particles

	D_{50}		Geometric Mean Size		Fall Velocity, w					
Fine Sand	0.000410 – 0.000820	0.125 – 0.25	0.00058	0.177	0.048	0.015	0.06	0.018	0.08	0.024
Very Fine Sand	0.000205 – 0.000410	0.0625 – 0.125	0.000290	0.088	0.012	0.004	0.02	0.006	0.03	0.009
Silt	0.000007 – 0.000205	0.0020 – 0.0625	0.0000367	0.011	0.0002	0.00006	0.0004	0.00012	0.00053	0.00016
NOTE: Derived from HEC-18, "Evaluating Scour at Bridges," 1993, and "Highways in the River Environment," 1990.										

Clear Water Contraction Scour Equation

Compute the average depth in the contracted cross section including contraction scour with Equation 9-18.

$$y_2 = 0.2138 \left(\frac{Q_2^2}{D_{50}^{2/3} W_2^2} \right)^{3/7}$$

Equation 9-18.

where:

y_2 = average depth in the contracted section including contraction scour (ft. or m)

W_2 = total width in the sub-section experiencing clear-water scour less the width of any piers in the sub-section (ft. or m)

D_{50} = median particle size diameter (ft. or m) (a suggested minimum for cohesive soils is 0.004 in. or 0.1 mm)

Q_2 = total width in the sub-section experiencing clear-water scour less the width of any piers in the sub-section (ft. or m)

During a flood, bridges over streams with coarse bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges, and then clear-water scour on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour because clear-water scour occurs mainly in coarse bed material streams. In fact, local clear-water scour may not reach a maximum until after several floods. Maximum local clear-water pier scour is about 10 percent greater than the equilibrium local live-bed pier scour.

Local Scour

Local scour involves the removal of material around piers, abutments, spurs, and embankments.

Pier Effect on Scour. The typical effect of a pier is vertical and horizontal vortexes that create a scour hole around the pier. See HEC-18-1995, for a more detailed discussion on pier scour.

Either live-bed or clear-water scour may occur at piers; however, you can use Equation 9-19, which assumes live-bed scour in non-cohesive bed material, to predict either case of pier scour.

$$y_{ps} = 2.0\,K_1 K_2 K_3 K_4 y_1^{0.35} a^{0.65}\,Fr_1^{0.43}$$

Equation 9-19.

where:

y_{ps} = maximum pier scour (ft. or m)

K_1 = correction factor for pier nose shape (see “Correction Factor K_1 for Pier Nose Shape” table below)

K_2 = correction factor for angle of attack (see “Correction Factor K_2 for Angle of Flow Attack” table below)

K_3 = correction factor for bed condition (see “Correction Factor K_3 for Bed Condition” table below)

K_4 = correction factor for armoring of bed material (For most TxDOT design, use 1. The value varies only for a bed material D_{50} in excess of 2.5 in. or 60 mm.)

y_1 = flow depth directly upstream of pier (ft. or m)

a = pier width (ft. or m)

Fr_1 = Froude Number of flow directly upstream of pier

v_1 = mean velocity of flow directly upstream of the pier (fps or m/s)

g = gravitational constant (32.2 ft./s^2 or 9.81 m/s^2)

Correction Factor K_1 for Pier Nose Shape

Shape of Pier Nose	K_1
Square	1.1
Round	1.0
Sharp	0.9

Correction Factor K_1 for Pier Nose Shape

Shape of Pier Nose	K_1
Circular cylinder	1.0
Group of cylinders	1.0

Correction Factor K_2 for Angle of Flow Attack

Angle of Attack	K_2		
	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Correction Factor K_3 for Bed Condition

Bed Condition	H	K_3
Clear-water Scour	N/A	1.1
Plane bed and antidune flow	N/A	1.1
Small dunes	3.0 > H >0.6	1.1
Medium dunes	9.0 > H >3.0	1.1-1.2
Large dunes	> 9.0	1.3

The upstream part of a local scour hole tends to have the shape of a truncated cone with the cone angle approximating the angle of repose of the sediment. Downstream slopes are flatter where the flow mixes with other flow, and a bar is formed downstream of the hole. You can determine the lateral extent of the scour hole from the angle of the material's repose and the depth of scour.

Causes of Scour at Abutments. Scour at abutments is usually caused by turbulence and eddying that result from the redirection of overbank flow into the waterway opening. The maximum scour usually occurs at the upstream face of the header and, depending on the degree of contraction, flow depths, and flow rate in the floodplain, may extend to the first or second interior bent of the bridge.

Several abutment scour equations currently exist and appear in HEC 18-1995. (See Reference for information on obtaining this document.) However, none of the equations presented to date gives acceptable results. Generally, they give inordinately high estimates even for low Froude numbers.

Therefore, the department does not recommend their use. Instead, you should protect abutments to reduce the potential for scour failure.

Total Scour Envelope

In reality, a total scour envelope at any given cross section is the result of a complex interaction of flow, sediment transport, bed material, and time. Currently, the procedures available assume that components of scour (long-term degradation or natural scour, contraction scour, and local scour) act independently and are ultimate depths for non-cohesive bed materials. The total scour envelope, then, is the summation of the individual components at the appropriate locations. Without better methods, assume that the natural degradation and contraction scour depths occur evenly across the portion of the cross section for which they were estimated. Where you consider local scour to occur (at piers and abutments), the total scour is assumed to be the sum of all three of these components: natural degradation, contraction scour, and local scour.

Tidal Scour

Tidal scour is made up of all three scour components. However, it has a dimension that the state-of-the-art equations do not address—the added movement of water that results from tides. Therefore, this scour type requires individual consideration. (The rest of the information in the next subsections regarding tidal scour is taken from the FHWA publication, *Evaluating Scour at Bridges,* HEC 18, 1995. See Reference for information on obtaining this document.)

The analysis of tidal waterways is very complex. The hydraulic analysis must consider the magnitude of the 100- and the 500-year storm surge, the characteristics of the tidal body, and the effect of any constriction of the flow due to natural geometry of the waterway or the presence of a roadway and bridge. In addition, the analysis must consider the longer effects of the normal tidal cycles or long-term aggradation or degradation, natural scour, contraction scour, local scour, and stream instability.

Three-level Approach to Tidal Scour Analysis. A three level approach is recommended to analyze bridge crossings of tidal, similar to that outlined in "Stream Stability at Highway Structures" (HEC 20, 1995).

- Level 1 -- Level 1 includes a qualitative evaluation of the stability and flow in the tidal waterway, and an estimate of the magnitude of tides and storm surges. As noted, Level 1 analysis could follow the procedures outlined in HEC 20. Level 1 tidal analysis evaluates the stability of the inlet, estimates the magnitude of the tides and storm surges and flow in the tidal waterway, and attempts to determine whether the hydraulic analysis depends on tidal or river conditions or both.
- Level 2 -- Level 2 uses engineering analysis to obtain velocity, depths, and discharge for the tidal waterway. Level 2 analysis involves the basic engineering assessment of scour problems at the highway crossing.

- Level 3 -- Level 3 requires a physical model or a two-dimensional mathematical model. At present, no suitable scour equations have been developed specifically for tidal flows. Therefore, use the scour equations developed for inland rivers to estimate and evaluate the tidal scour. However, in contrast to scour at inland river crossings, the evaluation of the hydraulic conditions at the bridge crossing using either WSPRO or HEC-RAS is not usually suitable for tidal flows. The FESWMS (Finite Element Surface Water Model System), a two-dimensional flow computer simulation model, can be used to predict tidal action. Contact the Bridge Division's Hydraulics Branch for consultation on two-dimensional models.

There are three typical types of tidal waterway crossings (see Figure 9-24, Figure 9-25, and Figure 9-26). The crossing must first be defined. Flow into (flood tide) and out of (ebb tide) a bay or estuary is driven by tides and by the discharge into the bay or estuary from upland areas. The problems can be divided into groups. For one group, the flow from the upland areas can be assumed to be negligible, and the ebb and flood in the estuary will be driven solely by tidal fluctuations and storm surges (see Figure 9-27 and Figure 9-28). Alternatively, the effects of tidal fluctuations are negligible when the flow from streams and rivers draining into the bay is large in relationship to the tidal flows. If tidal effects are negligible, then the conventional assessment can be done.

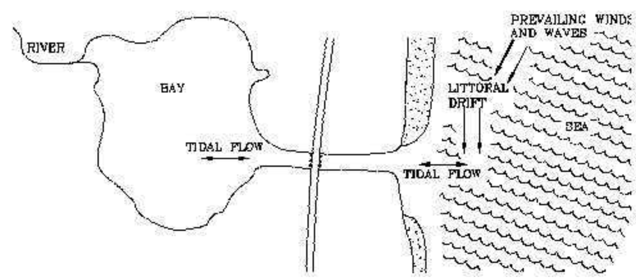

Figure 9-24. Inlet between Open Sea and Enclosed Lagoon or Bay

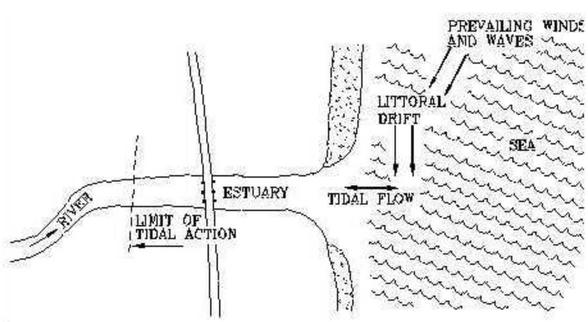

Figure 9-25. River Estuary

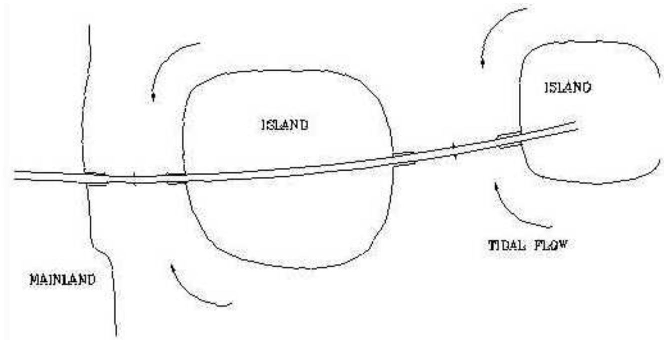

Figure 9-26. Passages Between Islands and Between Mainland and Island

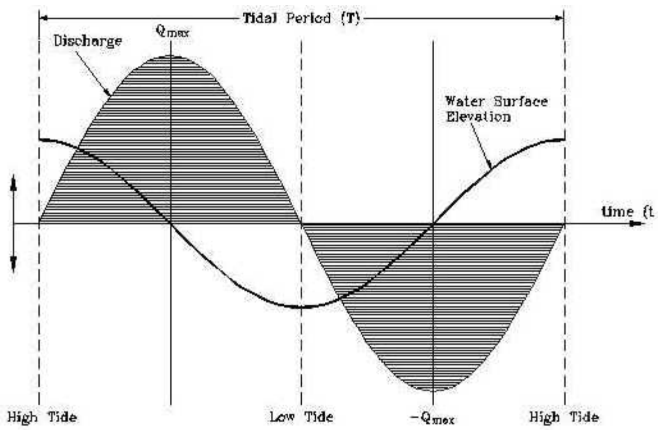

Figure 9-27. Principal Tidal Terms

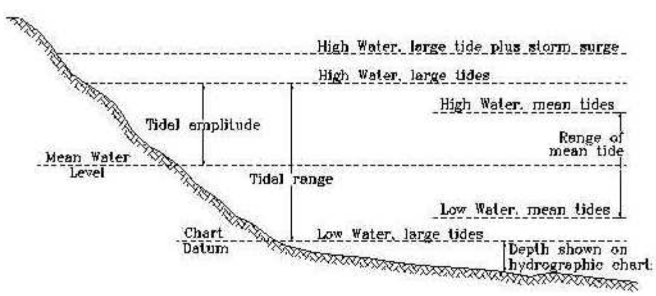

Figure 9-28. Additional Principal Tidal Terms

Because the evaluation of tidal scour is so complex, you must make some assumptions in order to calculate tidal scour.

Unconstricted Waterway Assessment Procedure

This method applies only when the tidal waterway or the bridge opening does not significantly constrict flow:

1. Determine the net waterway area at the crossing as a function of elevation.
2. Determine the tidal prism volume as a function of elevation.

3. Determine the elevation versus time relationship for the 100- and 500-year storm tides. Equation 9-20 represents the ebb tide that starts at the maximum elevation.

$$y = A \cos\theta + Z$$

Equation 9-20.

where:

y = amplitude or elevation of tide above mean water level at time t (ft. or m)

A = maximum amplitude or elevation of storm tide (ft. or m); defined as half the tidal range or half the height of the storm surge

Z = vertical offset to some datum (ft. or m)

θ = angle (degrees) subdividing the tidal cycle (see Equation 9-21).

$$\theta = 360\left(\frac{t}{T}\right)$$

Equation 9-21.

where:

t = time from beginning of cycle (min)

T = total time for complete tidal cycle (min)

4. Determine the discharge, velocities, and depth. Use Equation 9-22 to approximate the maximum discharge in a tidal estuary. Compute the corresponding maximum average velocity in the waterway with Equation 9-23. The velocity determined in this equation represents the average velocity in the cross section that will need to be adjusted to estimate velocities at individual piers to account for non-uniformity of velocity in the cross section. As for inland rivers, local velocities can range from 0.9 to approximately 1.7 times the average velocity depending on whether the location in the cross section was near the banks or near the thalweg of the flow. Studies indicate that the maximum velocity in estuaries is approximately 1.3 times the average velocity.

$$Q_{max} = \frac{3.14\,V}{T}$$

Equation 9-22.

where:

Q_{max} = maximum discharge in the tidal cycle (cfs or m^3/s)

V = volume of water in the tidal prism between high and low tide levels (cu.ft. or m^3)

Compute the corresponding maximum average velocity in the waterway with Equation 9-23:

$$v_{max} = \frac{Q_{max}}{A'}$$

Equation 9-23.

where:

v_{max} = maximum average velocity in the cross section at Q_{max} (cfs or m/s)

A' = cross-sectional area of the waterway at mean tide elevation halfway between high and low tide (sq.ft. or m^2)

5. Evaluate the effect of upland riverine flows on the discharge depth and velocities obtained in Step 4. Depending on the relative magnitudes of the high upland flow and the tidal flow, the effect may range from negligible to significant.
6. Evaluate the discharge, velocities, and depths that were determined in Steps 4 and 5.
7. Evaluate the scour for the bridge using the volumes of discharge, velocity, and depths determined from the above analysis. Use the scour equations recommended for inland bridge crossings.

Procedural Adjustments for Constricted Waterways

The procedures for an unconstricted waterway apply for a constricted waterway, except for Steps 2 and 4. To determine these hydraulic variables when the channel and not the bridge cause the constriction, you can use Equation 9-24 and Equation 9-25 for tidal inlets.

$$v_{max} = C_d \, (2 \, g \, \Delta h)^{1/2}$$

Equation 9-24.

$$Q_{max} = A' \, v_{max}$$

Equation 9-25.

where:

v_{max} = maximum velocity in the inlet (ft./s or m/s)

Q_{max} = maximum discharge in the inlet (cfs or m^3/s)

G = acceleration of gravity (32.2 ft./s^2 or 9.81 m/s^2)

Δh = maximum difference in water surface elevation between the bay and ocean side of the inlet (ft. or m)

A' = net cross-sectional area in the inlet at the crossing (sq. ft. or m^2)

C_d = coefficient of discharge (<1.0)

= $(1 / R)^{1/2}$, where R is the coefficient of resistance and is calculated using Equation 9-26

$$R = K_o + K_b + \frac{2 g n^2 L_c}{h_c^{4/3}}$$

Equation 9-26.

where:

K_o = velocity head loss coefficient on the downstream side of the waterway (taken as 1.0 if the velocity is zero)

K_b = velocity head loss coefficient on the upstream side of the waterway (taken as 1.0 if the velocity is zero)

N = Manning's roughness coefficient

L_c = length of the waterway (ft. or m)

h_c = average depth of flow in the waterway at mean water elevation (ft. or m)

Other Scour Considerations

Highway contractors often use in-stream borrow as a source of quality fill material. Commercial mining of sands and gravel in streams is also common because the material is clean and well graded and the stream replenishes the supply.

Borrow pits, either upstream or downstream of a highway-stream crossing, can cause scour at the bridge. Scour occurs upstream of the borrow because of the increased gradient of the stream bed. The bed load of the stream will be deposited in the borrow area and scour occurs downstream as the stream regains its bed load. The risk that in-stream borrow may pose to a bridge depends on the following conditions:

- amount of material removed from the stream
- effects of the borrow area on flow directions
- location of the borrow area
- size of the stream
- sediment transport capacity of the stream.

Many borrow areas have been filled in by the stream without detriment to nearby bridges during a moderate rise in large streams that carry a large sediment load. If you must, take borrow from areas where sediment deposition occurs, such as point bars and alternate bars, rather than from areas where scour occurs, such as along banks subjected to attack by the stream. If there is any concern about the effects of borrow from a stream, compare the volume of borrow to be excavated with the bed load in the stream.

In-stream mining for aggregates and dredging for navigation and flood control can be extremely damaging in cases where so much material is removed from the streambed that all of the incoming sediment supply is trapped and degradation of long reaches occurs. At some locations, dredging may be necessary, or commercial mining cannot be terminated either by legal action or by purchase. In these cases, measures to stabilize the stream bed elevation and the stream bank may be necessary, or pier and abutment foundations must be set below the expected future elevation of the streambed.

Armoring occurs when a stream or river cannot, during a particular flood, move the more coarse material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially but later be stopped by armoring before reaching the full scour potential again for a given flood magnitude.

When armoring does occur, the coarser bed material tends to remain in place or quickly re-deposit so as to form a layer of riprap-like armor on the streambed or in the scour holes. This armoring effect can decrease scour hole depths that were predicted based on a formula developed for sand or other fine material channels for a particular flood magnitude. When a flood of higher return frequency occurs than that used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring may also cause bank widening, which encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges, as they encourage difficult to assess plan-form changes. Bank widening also spreads the approach flow distribution that in turn results in a more severe bridge opening contraction.

Section 7
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, compare the cost of providing the control measures with 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 the modes of failure.

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 or to protect the streambed around the foundation to prevent scour or reduce the potential for scour.

Primary protection measures at piers include concrete riprap, rock riprap, 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.

You may consider 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, use a superstructure that is as slender as possible with open rails and no curb.

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 rights-of-way are 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 can be significant.

You can minimize the potential for erosion along the toe of approach embankment by avoiding extensive clearing of vegetation and avoiding the use of borrow areas in the adjacent floodplain. You can use embankment protection such as rock, but stable vegetation on the embankment may suffice. Other measures 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.

If you anticipate significant overtopping of the approach embankment during the life of the crossing, you may need to protect the embankment. You can construct the embankment of soil cement or use revetments, usually constructed of rock, wire-enclosed rock, or concrete.

Preventive measures are also needed at some crossings to protect the embankment against wave action, especially at reservoirs. You would usually use riprap of durable, hard rock at these 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 (rock generally preferred to concrete)
- vertical abutment walls

- sheet pile toe walls
- deep foundations of piles or drilled shafts.

Vertical abutment walls founded below anticipated scour depths 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. Use sheet pile only under guidance from the Bridge Division's Geotechnical Branch.

Usually, place revetment 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 does not inhibit scour from the flow contraction and is, therefore, susceptible to failure from undermining. Continue the revetment down below the level of expected scour to protect it and the embankment from failure.

An alternative used on cohesion-less soils is a flexible apron extended to the limits of the expected scour (Figure 9-29). The apron tends to be self-healing because it will settle into any area that scours and inhibit any further scour. Flexible aprons may not work as well on cohesive materials because the apron material does not protect steep faces of failures. Materials commonly used for flexible aprons are rock riprap, articulated concrete, and wire-enclosed rock.

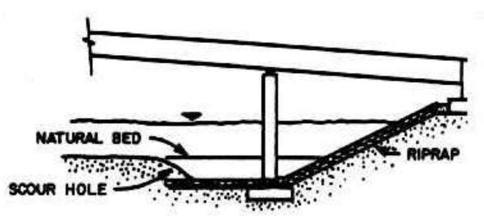

Figure 9-29. Flexible Apron

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 upstream to the upstream end of the guide bank. Where you anticipate that floodwater must flow along the embankment for more than 800 ft. (240 m), consider guide banks a viable appurtenance. Figure 9-30 shows a typical plan form.

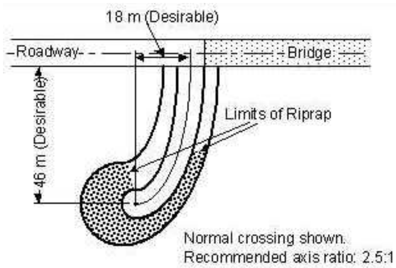

Figure 9-30. Typical Guide Bank

Designers usually construct guide banks from earthen embankment but sometimes from rock. Use revetment for protection of the dike 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.

Keep clearing around the end of the dike to a minimum in wooded floodplains to enhance the effectiveness. A small culvert through the dike, in lieu of a drainage channel around the end to provide local drainage, also helps 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 ft. (30 m) of waterway under the bridge. The suggested shape is based on laboratory experiments, and the length is based on modeling and field data. Optimum shape and length differs for each site and possibly for each flood at a site. Field experience shows, however, that the recommended elliptical shape is usually quite effective in reducing turbulence. If practical reasons require the use of another shape such as a straight dike, expect more scour at the upstream end of the guide banks. You can also use guide banks 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. You may need these measures 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:

- rock riprap
- 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. You can avoid futile attempts at localized control 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, consider stream response to the installation of the measure. For instance, bank stabilization at a crossing can 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 similar streams.

To a large extent, design is an art, and many questions concerning the relative merits of various measures have not been definitively answered. 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).
- Site reconnaissance is imperative. You may perform reconnaissance by on-site inspection, aerial reconnaissance, or aerial photographs taken over a period of years.
- Consider the possibility of using physical model studies at an early stage.
- Inspect the work periodically after construction with the aid of surveys to check results and to modify the design, if necessary.
- The objective of installing bank stabilization and river training measures is to protect the highway. The protective measures themselves are expendable.

Refer to *Stream Stability at Highway Structures* (HEC 20, 1995) for more detailed information regarding bank stabilization and stream training facilities. (See References for information on obtaining this document.)

The effectiveness of protective and training measures in many alluvial streams and the need for the measures may be short-term because the stream will move to attack another location or outflank the installation.

Make a cost comparison of viable options. 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 away from the river hazard
- a larger opening designed to accommodate the hazard
- designing foundations of bridge to accommodate future lengthening (e.g., design abutment foundations sufficiently to allow them to become interior bents at a later date).

When you need measures to protect a highway facility from anticipated actions by a stream, the possibility of a cooperative project with another governmental agency (particularly the U.S. Department of Army, Corps of Engineers). Other agencies have responsibilities and authority to undertake stream stabilization efforts, and mutually beneficial projects may be possible. (See References for information on contacting this agency and others.)

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 helps reduce the potential for scour at the bridge.

Where large volumes of debris are likely to occur, you may need larger spans and high freeboards. Alternatively, you can use debris racks to stop the debris before it reaches the structure.

For even a small probability of total or partial submergence, ensure a minimum potential for the bridge deck to float away. If the dead load of the structure is not sufficient to resist buoyant, drag, and debris impact forces, you may need to anchor the superstructure to the substructure. Provide air holes through each span and between each girder to reduce the uplift pressure.

Fender Systems

Dolphins and fender systems are two slightly different structural systems with the same purpose. For bridges, this purpose is to protect piers, bents, and other bridge structural members from damage due to collision by marine traffic. Dolphin types range from simple pile clusters to massive concrete structures. Fender-system types are less variable, consisting usually of pile-supported stringers, as shown in Figure 9-31.

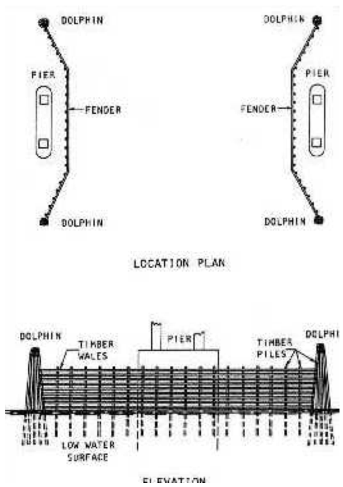

Figure 9-31. Fender Systems

You can often eliminate the need for fender systems by spanning smaller rivers or by placing piers judiciously. Construction costs of long spans may be economically unattractive when compared with shorter spans. However, when all construction and maintenance-related costs are considered, the long span solution may be the most attractive. The bridge designer should receive guidance from the hydraulics designer in the form of estimated depths of flow and depths of scour. This information influences fender lengths, diameters, and spacing, thereby affecting cost comparisons.

Among the maintenance costs are expense of installing navigation lights or a fender system when required by the USCG. These are specialized lamps and fixtures requiring constant attention.

Also consider estimated debris removal costs. When making decisions on span lengths, the bridge designer should consider these factors as well as maintenance costs due to collisions.

In some cases, fender systems may "shield" bridge piers, reducing velocities and scour at the pier. However, this shielding effect can vanish or be modified if the fender system is lost due to collision or unforeseen scour problems. Piers and fender systems introduced into relatively narrow rivers may cause general scour between the fender systems. This scour is usually greatest near the downstream end of the system.

Section 8

Risk Assessment

Introduction

Other engineering considerations, especially geometric standards and navigation requirements, may also override hydraulic design criteria at individual sites. As an example, the traffic service requirements, hydraulics, and economics may indicate a need for a design that would pass a 25-year flood through the bridge opening. However, because of topography or navigation requirements, the roadway profile is high and a 500-year flood would flow through the structure without overtopping the roadway. The hydraulic and economic analyses then become a matter of calculating the risk of backwater, scour, and erosion damage to the highway and other property, and the degree of protection to provide as countermeasures against damage. This section contains discussion of the following:

Purpose of Risk Assessment

By considering alternative designs, you may realize substantial savings in the overall cost of the facility. This analysis of alternatives must consider variations in design frequencies, bridge lengths, and embankment elevations. Variation of design components is encouraged in a hydraulic bridge facility. Such considerations require more time and expense in design and may not be justified for a typical facility that falls within the design frequencies recommended in the Frequency Determination subsection of Chapter 5. However, you may encounter situations that justify a risk assessment. Exercise considerable judgment in arriving at the final design because of the many factors that can become involved in the hydrologic and hydraulic designs for highway crossings.

Risk Assessment Concepts

The following concepts are adapted from *Design of Encroachment on Floodplains Using Risk Analysis* (HEC-17, 1981—see Reference for information on obtaining this document).

Risk Assessment versus Economic (Risk) Analysis. A risk assessment is a general consideration of factors that are likely to indicate the relative risks of one design alternative to another. A detailed economic analysis involves calculation of the probable total costs associated with a range of alternatives.

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 usually based on real-life cost data.

Annual Risk

Annual risk reflects the costs incurred with the repair of the expected damage at each of the flood frequencies. Assume that the probability of a damage loss equals the probability that a flood will be expected in a given year (i.e., the probability of the damage loss is the inverse of the flood frequency).

The economic losses taken into consideration in the annual risk are associated with embankment and pavement damage, interruption to traffic, impact of backwater on adjacent property, damage to superstructure, and damage to bridge due to scour.

Embankment and pavement damage is difficult to foresee, but estimate what it would cost to repair the embankment and pavement if damaged during each flood under consideration.

Calculate the cost associated with traffic delays and detours due to the inundation of a stream crossing by multiplying the traffic risk by the probability of overtopping. Traffic risk includes consideration of traffic restoration time, increased running cost, time losses, and potential accident costs.

In order to simplify risk assessment, make some simplifying assumptions. Figure 9-32 provides an estimate for traffic risk. The graph is based on the following assumptions:

Traffic make-up:	Cars	70%
	Small trucks	20%
	Semi Trailers	10%
Running Costs:	Cars	$ 0.125/km($0.20/Mile)
	Small trucks	$ 0.188/km($0.30/Mile)
	Semi Trailers	$ 0.406/km($0.65/Mile)
Value of Lost Time:	$4/hour/occupant	1.25 occupants per vehicle
Average Detour Time:	2 days	Detour Speed: 80.5 kph (50 mph)

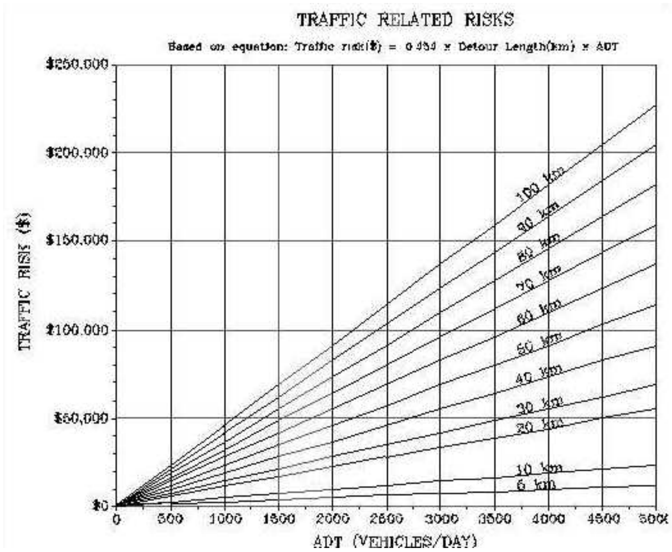

Figure 9-32. Assumed Costs for Traffic Interruption

Superstructure losses are the potential costs to repair damage to a bridge superstructure due to accumulation of trash, impact of debris, and clean-up after the flood. Make a general estimate of the damage as the depth of submergence of the deck in feet or meters times a coefficient. Base this coefficient on a cost per unit length of inundated structure that increases with inundation depth. Base it on the experience of cleanup, repair damage, and placing back in service bridges by floods. If the bridge is above floodwaters, then do not calculate a cost.

Damage due to scour includes the potential cost of repair to damaged piers, abutments, and guide banks.

You can calculate the losses by multiplying a unit cost to repair damage (units are dollars per cubic feet or per cubic meter) and the extent of scour in cubic feet or cubic meters. Base the unit cost to repair damage on past experience of similar crossings.

Annual capital cost is the sum of annualized initial costs incurred in constructing a structure and roadway approach and the long-term operation and maintenance costs. Add construction cost components to obtain the total initial cost that must be amortized over the life of the structure. Make computations in terms of constant dollars using a discount rate instead of the prevailing interest rate in the computations. Using a discount rate lets you estimate all cost by today's prices. Multiply the total construction or capital cost by a capital recovery factor to obtain the annual amortization series. The capital recovery factor (CRF) is defined as "an annuity whose present value is one." Compute the CRF using Equation 9-27. The annual capital cost, then, equals the CRF multiplied by the total initial cost plus the anticipated annual operation and maintenance costs.

$$CRF = \frac{i}{1 - (1 + i)^{-n}}$$

Equation 9-27.

where:

i = discount rate (FHWA recommends using the Federal Water Resources Project discount rate, which was 0.09 in 1996)

n = service life of a structure in years (FHWA recommends 50 years)

Risk Assessment Forms

Do a basic risk assessment using the form titled “Economic and Risk Assessment for Bridge Class Structures.” The assessment includes the calculation of the annual risk cost and annual capital cost. A worksheet for these calculations is provided. Include other factors that seem important to the project, even though the form does not include the item.

Section 9
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 -- Use a solid bridge rail 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 -- A more 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 impervious type.

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. You can improve deck drainage 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.

Locate deck drains 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, provide splash basins to minimize erosion or tie the downspouts into the storm drain conduit. Do not allow drainage to discharge against any part of the structure.

Where practicable, avoid the need to suspend a conduit collection system on the superstructure. When using collection systems, design them 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, provide 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, utilize temporary provisions for protecting the approach fill from erosion until permanent measures are installed and functional.

Chapter 10
Storm Drains

Contents:

Section 1
Introduction

Overview of Urban Drainage Design

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.

The flow of water along a roadway can interfere with or halt highway traffic. These conditions sound and consistent engineering principles and the use of all available data to achieve an acceptable drainage design. The primary aim of urban drainage design is to limit the amount of water flowing along the gutters or ponding at the low areas to rates and quantities that will not interfere with traffic. You can accomplish this goal by placing inlets at appropriate locations to prevent large concentrations of runoff. The most destructive effects of an inadequate drainage system are damage to surrounding or adjacent properties, deterioration of the roadway components, and hazard and delay to traffic caused by excessive ponding in sags or excessive flow along roadway grades.

Overview of Storm Drain Design

Although the design of a storm drain system entails many conventional procedures, certain aspects of a storm drain system design require judgment. You must establish design parameters and criteria, decide layout and component location and orientation, take responsibility for using appropriate design tools, and ensure comprehensive documentation.

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 this process until you develop a storm drain system that satisfies the technical function of collecting and disposing of the runoff and costs the least amount of money.

The proper design of any storm drainage system requires accumulation of certain basic data, familiarity with the project site, and basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design.

Section 2

System Planning and Design Considerations

Design Checklist

- Identify the problem.
- Develop a system plan.
- Establish suitable materials and conduit shapes.
- Establish design criteria.
- Determine outfall channel flow characteristics.
- Identify and accommodate utility conflicts.
- Consider the construction sequence and plan for temporary functioning.
- Recognize other drainage facilities, and accommodate them.
- Determine runoff.
- Design inlets.
- Design conduit.
- Develop a hydraulic grade line analysis.
- Check the final design, and adjust if necessary.
- Document the design.

Problem Identification

As with any kind of project, you 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, optimizing safety and minimizing potential adverse impacts.

Schematic

Preliminary or working schematics featuring the basic components of the intended design are invaluable in the design development. After design completion, the schematic facilitates documentation of the overall plan.

You may include the following items in the working schematic:

- 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 schematic should include the existing physical features of the project area and indicate the location and type of the following:

- streets
- driveways
- 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.

Material and Shape Selection

Consider all possible storm drain materials 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. These characteristics may vary from site to site. It is not cost-effective to declare a rule of thumb that the storm drain system should be of one material exclusive of all others.

Base the choice of material and shape on careful consideration of durability, hydraulic operation, structural requirements, and availability.

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 affect the choice of shape directly. Refer to the Conduit Durability section of Chapter 14 for discussions and design considerations associated with durability. You can usually consult the roadway project's geotechnical report for factors that affect material durability.

The selection of both shape and material for storm drain system components influences the hydraulic capacity. 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.

When choosing both shape and material, consider cover limitations, headroom, and anticipated loading.

Choose materials, shapes, and components that require minimum transportation costs and that are readily available in the geographic region of the project. Items commonly manufactured in standard sizes include prefabricated pipe, inlets, and manholes.

Deviation from standard sized structures is rarely cost-effective. The pipe industry maintains current standard catalogs of nominal fabrication dimensions. Refer to fabricators' catalogs for current lists of generically available sizes and shapes.

Design Criteria

The design frequency is an indication of the level of flooding accommodated by the system without causing an undesirable impact to pavement, structures, traffic, and adjacent facilities and property.

Base the design frequency for a storm drain system design on the following:

- 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) and demand
- a realistic assessment of available funds for the project.

Chapter 5 provides a discussion on design frequency and includes a table of recommended design frequencies.

The allowable ponded width may vary within a single system. For example, an allowable ponded width of one lane of flooding on main lanes and one and one-half lanes for frontage roads may be acceptable. An allowable ponded width is the basis for locating points on the roadway surface at which runoff must be removed. Base the determination of allowable ponded width on such factors as width of roadway, number of lanes, and level of service desired during design frequency.

You may use the following recommended ponded widths with consideration for site specific parameters and limitations:

- 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, and frontage roads.
- Limit ponding to a width and depth that will allow the safe passage of one lane of traffic for minor highways.

The usual TxDOT practice is to design for a non-pressure flow network of collector conduits in most storm drain systems.

Critical elevations are used as comparative values to the key elevations on a developed hydraulic grade line. (See Chapter 6 for more information.) As a rule, a surface water removal system is designed to operate with no impedance or interruption of free fall into the system. Therefore, the system does not perform as predicted by the calculations if the backwater (hydraulic grade line) within the system rises to a level above a curb and gutter grade, a manhole, or any other critical elevation in a storm drain system. Water will either back out on the roadway or runoff will be impeded from entering the system as planned. You need to identify the critical elevations where these problems most likely will exist and compare the resultant hydraulic grade line. Typical critical elevations would be located at the throats of inlets and tops of manholes. For the design frequency, the hydraulic grade line should not exceed the critical elevation.

The usual preference is that flow velocities within the conduit network be no less than 2 fps (0.6 m/s) and no greater than about 12 fps (3.6 m/s). At velocities less than 2 fps (0.6 m/s), sediment deposit becomes a serious maintenance problem. Such slow velocities also indicate an inefficient drainage system. At flow velocities greater than about 12 fps (3.6 m/s), structural damage to the system components becomes a threat. The momentum of flow at higher velocities can cause a damaging impact on the structural components and connections within the system. There may be instances when design velocities outside the range of 2fps and 12 fps (0.6 m/s and 3.6 m/s) are necessary. If so, countermeasures such as greater access for maintenance or strengthened components may be in order.

Outfall Considerations and Features

The outfall of the storm drain system is a key component, and you must coordinate with the demands of the physical and hydraulic characteristics of the system. 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 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, you should assess its ability to convey design flows. If necessary, modify the outfall to ensure minimizing the potential for significant impact.

An outfall for a TxDOT storm drain system must be operated for the life of the system. This implies that TxDOT must have access to all parts of the outfall for purposes of maintenance and to ensure adequate operation of the drainage system. If the outfall is by easement through private property, assure continuing TxDOT access to the outfall within that easement. In many instances, it is necessary to purchase an outfall right-of-way (drainage easement) so that continuing access by the TxDOT is assured.

Special Outfall Appurtenances

When separate storm drain systems intersect, a bubble chamber may be useful to provide a means of connecting the systems. You can design the bubble chamber so that as the water level (hydraulic grade line) in one system rises to a certain level, flow in another system serves as a relief drainage facility.

If the outfall to be used by a storm drain system is permanently or temporarily inadequate to accommodate the flow from the system, you may need to install some type of flow restrictor. The flow restrictor must include a space for runoff detention, allowing a reduced runoff rate to exit into the inadequate outfall.

Flap gates are provided when an outfall might cause the storm drain system to back up. A flap gate allows flow out during lower outfall levels and prevents backflow when the water level is higher. For example, if the storm drain system is to outfall into a tidal basin in which the periodic fluctuation of tides represents a variation of possible outfall water levels, you may need to provide a flap gate at the end of the last downstream run of the system.

Utility Conflicts

Direct consideration and planning toward minimizing conflicts with existing utilities and potential conflicts with future utilities. During design, the order of considerations is as follows:

1. Carefully identify each utility and associated appurtenances that may be in conflict with any part of the storm drain system. Consider in the design 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 you may encounter in an urban situation:
   - Electrical

- telephone or television transmission lines
- water lines
- wastewater lines
- gas lines
- irrigation ditches
- high-pressure fuel facilities
- communication transmission facilities.

2. Where reasonable, relocate components of the storm drain system to avoid utility conflict.
3. When relocation of the storm drain is not feasible, arrange for the relocation or adjustment of the utility. The entity responsible for the utility is usually cooperative in such cases.
4. 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 syphon. The utility company may be on state right-of-way under the agreement that TxDOT may request utility adjustments. However, as a general objective, attempt to minimize the disruption to utilities.

Construction

The construction sequence of the various storm drains can have a major influence on the design. The need to comply with the National Pollutant Discharge Elimination System (NPDES) General Permit for construction activities has increased the importance of proper sequencing.

The system must function, perhaps to a lesser extent, during the time of project construction. It must function adequately (but probably not optimally) both with the rest of the storm drain system and other project aspects. For example, it is usually recommended that storm drain lines be built from downstream to upstream in order to prevent “trapping” storm water during construction. Phase the storm drain system construction to accommodate the following:

- sequences of roadway construction
- traffic control
- cut and fill operations
- utility construction
- structural operations.

Identification of Other Drainage Facilities

You should attempt to identify any existing or proposed facilities that your proposed system is likely to affect or which may affect your proposed system. Examples include the following:

- regional or local storm water detention facilities
- proposed or recent changes to adjacent highway facilities
- municipal master drainage plans
- other major development.

Design Documentation

Design documentation needs for the development of a storm drain design include the following:

- watershed data
- estimates of future development of watersheds
- channel flow characteristics in outfall
- logical inlet locations
- curb and gutter slopes
- transverse slopes
- inlet calculations
- times of concentration to each location (node)
- rainfall intensity calculations
- depth of flow and ponded width of curb/gutter flow
- inlet sizing calculations
- carryover rates
- conduit slopes
- conduit sizing calculations
- conduit run travel times
- critical elevations
- hydraulic grade line elevations.

Documentation Requirements

The design is not complete until the following are documented:

- criteria
- design parameters
- considerations
- calculations.

The documentation serves several important purposes including:

- justification of the design
- reference for review and checking
- reference for potential field changes and future modifications
- potential defense against litigation.

The Storm Drain Documentation Check List (3d) presents required documentation for storm drain systems.

Section 3
Runoff

Hydrologic Considerations for Storm Drain Systems

Show watershed boundaries on the schematic. As inlet locations within the established system are finalized, you can indicate intermediate drainage boundaries. Either show schematically or otherwise describe component parts of contributing watersheds (subareas). See Chapter 4 for discussion of field surveys, and see Chapter 5 for hydrologic considerations.

Flow Diversions

Generally, a storm drain system should accommodate the natural drainage area. Avoid diversion of flow from one watershed to another. Where diversion of flow has already toccurred, you may need to consider the implications of accommodating the diversion. However, it is not the usual practice or aim of TxDOT to divert runoff flows from one major watershed to another. If and when it is unavoidable, you must consider the impacts of flow diversion. You may be required to coordinate with the Texas Natural Resource Conservation Commission (TNRCC) in many instances, and you should investigate this early in the planning and design process. (See Reference for information on contacting the TNRCC.)

Detention

Detention does not change the total volume of runoff. However, the runoff rates change depending on the characteristics of the flood and the detention facility. Such facilities may be in the form of holding reservoirs, large borrow ditches, and underground storage sumps.

TxDOT has not usually incorporated detention into designed systems because the department's chief aim is to remove and dispose of runoff as quickly and effectively as possible. With increased development in Texas, greater runoff rates and quantities have occurred, causing the need for larger and more costly drainage structures. The greater rates and quantities may also damage downstream development.

You may incorporate a detention facility into a design for drainage systems to decrease facility costs and diminish possible damages due to the increased runoff rates and quantities. With this aim, many municipalities, counties, and other entities in Texas have begun to require detention as an integral part of drainage design. Additionally, you may need to design a detention system for multiple use, especially for storm water quantity and quality control.

Determination of Runoff

In a storm drain design, first determine the peak flow runoff. The Rational Method, discussed in Chapter 5, is the method that applies to the vast majority of the types of watersheds that storm drains handle.

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 (called inlet time) and the travel time as the water flows through the storm drain line under consideration (travel time through a conduit). See Procedure to Estimate Time of Concentration in Chapter 5 for more information.

Other Hydrologic Methods

For the urban area under consideration, the TxDOT designer may need to use a special hydrologic method because of some funding arrangements. 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 procedure. (See References for information on contacting this agency.) In other situations, the use of a unit hydrograph procedure may be in order. Refer to NRCS Runoff Curve Number Methods in Chapter 5 for detailed information on the NRCS methods.

Where considerable storage is required in the storm drain system, employ hydrologic routing methods to accommodate peak flow attenuation. Refer to Chapter 5 for information on flood hydrograph routing methods.

Section 4
Pavement Drainage

Design Objectives

A chief objective in the design of a storm drain system is to move any accumulated water off the roadway as quickly and efficiently as possible. Where the flow is concentrated, the design objective should be to minimize the depth and extent of that flow.

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 you must work with the roadway geometric designers to assure efficient drainage in accordance with the geometric and pavement design.

Ponding

Restrict the flow of water in the gutter to a depth and corresponding width that will not cause the water to spread out over the traveled portion of the roadway in a depth that obstructs or poses a hazard to traffic. The depth of flow should not exceed the curb height. The depth of flow depends on the following:

- rate of flow
- longitudinal gutter slope
- transverse roadway slope
- roughness characteristics of the gutter and pavement
- inlet spacing

Place inlets at all low points in the roadway surface and at suitable intervals along extended gutter slopes as necessary to prevent excessive ponding on the roadway. In the interest of economy, use a minimum number of inlets, allowing the ponded width to approach the limit of allowable width specified as a design criterion. In instances such as a narrow shoulder or low grades, you may need to plan a continuous removal of flow from the surface.

Longitudinal gutter slopes should usually not be less than 0.3% for curbed pavements. This minimum may be difficult to maintain in some locations. In such situations, a rolling profile (or sawtooth grade) may be necessary. You may need to warp the transverse slope to achieve a rolling gutter profile. Figure 10-1 shows a schematic of a sawtooth grade profile. Extremely long sag-vertical curves in the curb and gutter profile are discouraged because they incorporate relatively long, flat grades at the sag. Such long, flat slopes tend to distribute runoff across the roadway surface instead of concentrating flow within a manageable area.

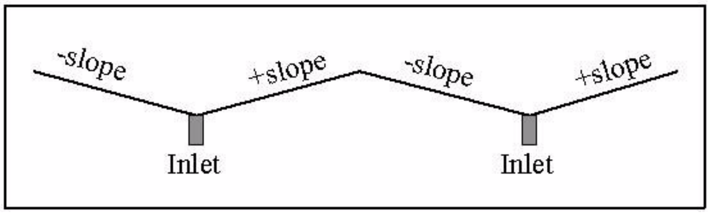

Figure 10-1. Sawtooth Gutter Profile

Transverse Slopes

Except in cases of super-elevation for horizontal roadway curves, the pavement transverse slope is usually a compromise between the need for cross slopes adequate for proper drainage and relatively flat cross slopes that are amenable to driver safety and comfort. Generally, transverse slopes of about 2 % have little effect on driver effort or vehicle operation. If the transverse slope is too flat, more depth of water accumulation is necessary to overcome surface tension. Furthermore, once water accumulates into a concentrated flow in a flat transverse slope configuration, the spread of the flow (ponded width) may be too wide. These characteristics are the chief causes of hydroplaning situations. Therefore, an adequate transverse slope is an important countermeasure against hydroplaning.

For TxDOT projects, a recommended minimum transverse slope for tangent roadway sections is 2%. The recommended maximum transverse slopes for a tangent roadway section is 4%. Refer to the Roadway Design Manual for recommendations concerning super-elevation values for horizontal curves in roadways. Ensure that cross slope transitions, such as those required in reverse curves, are designed to avoid flat cross-slopes in sag vertical curves.

You can effectively reduce the depth of water on pavements by increasing the cross slope for each successive lane in a multi-lane facility. In very wide multi-lane facilities, the inside lanes may be sloped toward the median. However, do not drain median areas across traveled lanes. In transitions into horizontal curve super-elevation, minimize flat cross slopes and avoid them at low points of a sag profile. It is usually in these transition regions where small, shallow ponds of accumulated water, or “birdbaths,” occur.

Use of Rough Pavement Texture

The potential for hydroplaning may be minimized to some extent if the pavement has a rough texture. Cross cutting (grooving) of the pavement is useful for removing small amounts of water such as in a light drizzle. TxDOT discourages longitudinal grooving because it usually causes problems in vehicle handling and tends to impede runoff from moving toward the curb and gutter. 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.

Gutter Flow Design Equations

Figure 10-2 illustrates ponding spread. Ponded width is commonly designated as T.

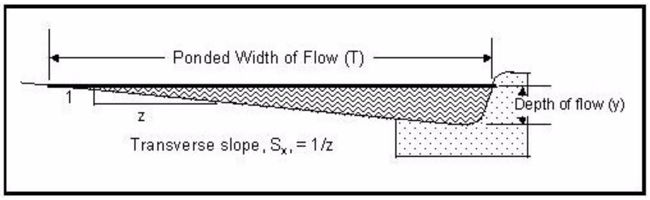

Figure 10-2. Gutter Flow Cross Section Definition of Terms

The ponded width is a geometric function of the depth of the water (y) in the curb and gutter section. For storm drain system design in TxDOT, the depth of flow in a curb and gutter section with a longitudinal slope (S) is taken as the uniform (normal) depth of flow, using Manning's Equation for Depth of Flow as a basis. (See Chapter 6 for more information.) Ordinarily, it would not be possible to solve for uniform depth of flow directly from Manning's Equation. 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.

$$d = z\left(\frac{Q\,nS_x}{S^{1/2}}\right)^{3/8}$$

Equation 10-1.

where:

d = 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 (ft./ft. or m/m)

z = 1.24 for English measurements or 1.443 for metric

Refer to Figure 10-2, and translate the depth of flow to a ponded width on the basis of similar triangles.

$$T = \frac{d}{S_x}$$

Equation 10-2.

where:

T = ponded width (ft. or m)

Determine the ponded width in a sag configuration with Equation 10-2 using depth of standing water or head on the inlet in place of d. Combine Equation 10-1 and Equation 10-2 to compute the gutter capacity using Equation 10-3.

$$Q = \frac{z}{n} S_x^{5/3} S^{1/2} T^{8/3}$$

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

The table below presents suggested Manning's "n" values for various pavement surfaces. The department recommends use of the rough texture values for design.

Manning's n-Values for Street and Pavement Gutters

Type of gutter or pavement	n
Asphalt pavement:	
Smooth texture	0.013
Rough Texture	0.016
Concrete gutter with asphalt pavement:	
Smooth texture	0.013
Rough texture	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016

Equation 10-3 and Equation 10-4 apply to portions of roadway sections having constant cross slope and a vertical curb. Refer to the FHWA publication *"Urban Drainage Design Manual"* (HEC-22, 1996) for parabolic and other shape roadway sections.

Ponding on Continuous Grades

Avoid excessive ponding on continuous grades by placing storm drain inlets at frequent intervals. Determine the gutter ponding at a specific location (such as an inlet) on a continuous grade using the following steps:

1. Determine the total discharge in the gutter based on the drainage area to the desired location. See Runoff for methods to determine discharge.
2. Determine the longitudinal slope and cross-section properties of the gutter. Cross-section properties include transverse slope and Manning's roughness coefficient.
3. 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, 1996)*. For information on obtaining this publication, see References.

Ponding at Approach to Sag Locations

At sag locations, consider sag inlet capacity, flow in the gutter approaching the left side of the sag inlet, and flow in the gutter approaching the right side of the sag inlet, and avoid exceeding allowable ponding:

1. Estimate the apportionment of runoff to the left and right approaches. Considering the limitations of the hydrologic method employed (usually the Rational Method - see information on the Determination of Runoff), it is reasonable to compute the discharge to the sag location based on the entire drainage area and determine the approximate fraction of area contributing to each side of the sag location. Multiply each fraction by the total discharge 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 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.
3. For each side of the sag, calculate the ponded depth and width. Use the appropriate flow apportionment, longitudinal slope, and Equation 10-1. Compute the ponded width using Equation 10-2.

Hydroplaning

As rain falls on the roadway surface, the water accumulates to some depth before overcoming surface tension and running off. A vehicle encountering water on the road may hydroplane, the vehicle's tires planing on top of the accumulated water and sliding across the water surface. Hydroplaning is a function of rainfall intensity and resulting water depth, air pressure in the tires, tread depth and siping pattern of the vehicle tires, condition and character of the pavement, and vehicle speed.

Because the factors that influence hydroplaning are generally beyond the designer's control, it is impossible to prevent the phenomenon. However, minimize the physical characteristics that may influence hydroplaning:

- The greater the transverse slope on the pavement, the less the potential for water depth buildup and potential for hydroplaning. A minimum cross slope of 2% is recommended. The longitudinal slope is somewhat less influential in decreasing the potential for hydroplaning. You must establish coordinate establishment of these slopes with the geometric design to ensure adequate provisions against hydroplaning.
- Studies have indicated that a permeable surface course or a high macrotexture surface course has the highest potential for reducing hydroplaning problems.
- As a guideline, a wheel path depression in excess of about 0.2 in. (5 mm) has potential for causing conditions that may lead to hydroplaning.
- Grooving may be a corrective measure for severe localized hydroplaning problems. However, grooving that is parallel to the roadway traffic direction may be more harmful than useful because of the potential for retarding sheet flow movement.
- Do not use transverse surface drains located on the pavement surface.

Rainfall intensities can be so high in Texas that the designer cannot eliminate the potential for hydroplaning. Because rainfall intensities and vehicle speed are primary factors in hydroplaning, it is incumbent on the driver must be aware of the dangers of hydroplaning. In areas especially prone to hydroplaning where you have employed reasonable measures to minimize the potential for hydroplaning, the department should use wet weather warning signs to warn the driver of the danger.

Vehicle Speed in Relation to Hydroplaning

You can evaluate the potential for hydroplaning using an empirical equation based on studies conducted for the USDOT, (FHWA-RD-79-30 and 31-1979, Bridge Deck Drainage Guidelines, RD-87-014).

Equation 10-5 and Equation 10-6 provide in English and metric units a means of estimating the vehicle speed at which hydroplaning occurs.

(English):

$$v = SD^{0.04}P^{0.3}(TD+1)^{0.06}A$$

Equation 10-5.

Metric:

$$v = 0.9143SD^{0.04}P^{0.3}(TD+0.794)^{0.06}A$$

Equation 10-6.

where:

v = vehicle speed at which hydroplaning occurs (mph or km/h)

SD = [W_d-W_w/W_d]*(100) = spindown percent (10 % spindown is used as an indicator of hydroplaning)

W_d = rotational velocity of a rolling wheel on a dry surface

W_w = rotational velocity of a wheel after spinning down due to contact with a flooded pavement

P = tire pressure (psi or kPa), use 24 psi or 165 kPa for design

TD = tire tread depth (in. or mm), use 2/32-in. or 0.5 mm for design)

WD = water depth, in. or mm (see Equation 10-7)

$$\left[10.409/WD^{0.06}\right]+3.507\ or\ \left\{\left[28.952/WD^{0.06}\right]-7.817\ \right\}*TXD^{0.14}$$

For metric, the greater of

[12.639/WD0.06] + 3.50 or {[22.351/WD0.06] - 4.97} * TXD0.14

NOTE: This equation is limited to vehicle speeds of less than 55 mph (90 km/h).

Water Depth in Relation to Hydroplaning

Equation 10-7 provides for evaluating the depth of storm water on pavement.

$$WD = z\left\{\frac{TXD^{0.11}L^{0.43}I^{0.59}}{S^{0.42}}\right\} - TXD$$

Equation 10-7.

where:

z = 0.00338 for English measurement or 0.01485 for metric

WD = water depth (in. or mm)

TXD = pavement texture depth (in. or mm) (use 0.02 in. or 0.5 mm for design)

L = pavement width (ft. or m)

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

S = pavement cross slope (ft./ft. or m/m)

After calculating water depth, check design speed. If hydroplaning is a concern, several possibilities exist:

- The cross-slope could be increased. Pavement cross-slope is the dominant factor in removing water from the pavement surface. A minimum cross-slope of 2% is recommended.
- Pavement texture could be increased. However, no technical guidance appears to be available on the relationship between texture depth and pavement surface type.
- Reduce the drainage area. If possible, reduce width of drained pavement by providing crowned section or by intercepting some sheet flow with inlets such as slotted drains.
- The speed limit could be reduced for wet conditions.

If physical adjustments to the roadway conditions are not practicable, consider providing appropriate warning of the potential hazard during wet conditions.

Section 5
Storm Drain Inlets

Inlet Types

You can divide inlets used for the drainage of highway surfaces into four major classes:

- Curb opening inlets - See Figure 10-3.
- Grate inlets - See Figure 10-3.
- Slotted drains - Slotted inlets function in essentially the same manner as curb opening inlets, i.e., as weirs with flow entering from the side. See Figure 10-6.
- Combination inlets -- Combination inlets usually consist of some combination of a curb-opening inlet, a grate inlet, and a slotted drain. In a curb and grate combination, the curb opening may extend upstream of the grate. In a grate and slotted drain combination, the grate is usually placed at the downstream end of the grate.

Curb Opening Inlets

Figure 10-3 illustrates a generic example of a typical curb opening inlet. Curb inlets are used in urban sections of highway along the curb line on continuous grades (on-grade) and at sag locations.

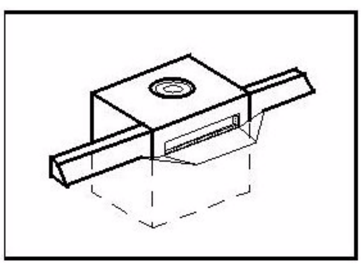

Figure 10-3. Curb Opening Inlet

Most curb opening inlets depend heavily upon an adjacent depression in the gutter for effective flow interception (see Figure 10-4). Greater interception rates result in shorter (and probably, more economical) inlet lengths. However, a large gutter depression can be unsafe for traffic flow moving 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

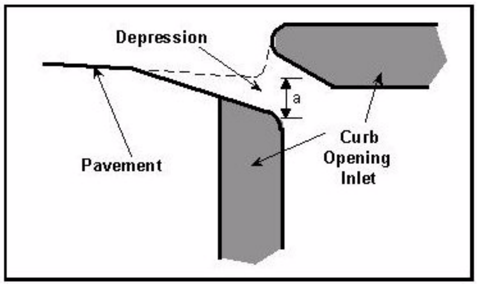

Figure 10-4. Curb Opening Inlet Depression

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 10-5 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.

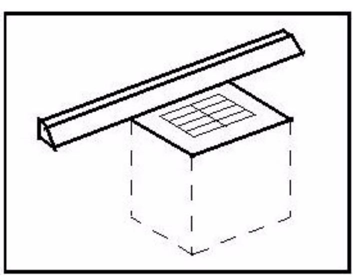

Figure 10-5. Grate Inlet Schematic

For the most part, use grate inlets 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. You may also use them in on-grade situations with curb inlets. Where you expect the grate inlet to intercept gutter flow in an on-grade configuration, the grate openings should be oriented parallel to the gutter flow in order to maximize hydraulic efficiency.

Grate inlets adapt to urban roadway features such as driveways, street intersections, and medians. When grate inlets are specified, assure that the grate configuration and orientation are compatible with bicycle and wheelchair safety. Consult with TxDOT's Statewide Bicycle Coordinator and the Design Division for additional information.

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; their propensity to collect debris make 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.

Slotted Drains

See Figure 10-6 for an illustration of a slotted drain installation. The throat of a slotted drain inlet is ordinarily reinforced for structural integrity. The top of the throat is constructed flush with the surface of the pavement or the gutter.

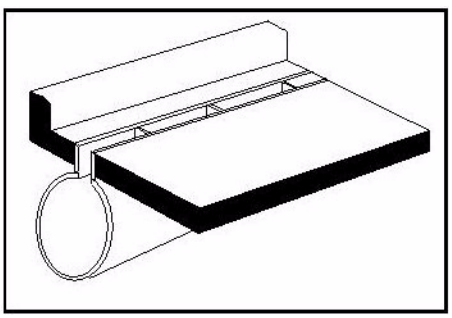

Figure 10-6. Slotted Drain Inlet

Slotted drains may be an alternative to on-grade curb and grate inlets along curb lines. Also, they can be placed across driveways and street intersections.

Design for the removal of sheet flow from the roadway by strategically placing slotted drain pipe installations. Such installations may occur within the traveled way, either transversely or longitudinally. Where drainage is toward the inside of lanes and against median barriers, an installation of slotted drain pipe with appropriate outfall can be effective in removing accumulated runoff.

In asphalt concrete pavement applications, ensure structural integrity either by adequate structural characteristics of the slotted pipe or encasement in concrete such as illustrated in Figure 10-7. Refer to the Bridge Division inlet standards, SD (M), concerning the proper type of slotted drain to use in these situations.

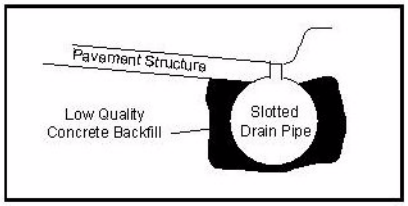

Figure 10-7. Slotted Drain Structural Integrity

Slotted drains have the following advantages:

- They are adaptable to intersections with urban roadway features such as driveways, street intersections, and sidewalks.
- They can accommodate AASHTO HS 20 vehicular traffic as well as bicycles, wheel chairs, and some pedestrian traffic.
- No depression is necessary for hydraulic efficiency.
- Continuous sheet or gutter flow interception is possible at a relatively small cost.
- Construction is very simple and proceeds quickly.
- Pavement overlays or other surface treatment can be accommodated without any effect on the original intended hydraulic characteristics.
- Slotted drain inlets in on-grade configurations are essentially self-cleaning.
- They are aesthetically pleasing.

Disadvantages of slotted drain inlets include the following:

- They have a high propensity to collect debris in sag configurations; therefore, do not use them in sag configurations.
- Effective maintenance access usually requires an adjacent manhole or an adjacent curb opening or grate inlet.
- Slotted drain pipes may have structural connector problems at locations where there are flexible joints in the roadway structure.

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 are generally 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 combined with the grate opening.

Inlets in Sag Configurations

An inlet in a sag configuration is 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, and you must consider some additional items. In a sag configuration, the controlling ponded width can be from one of three origins. The inlet itself may cause a head that translates to a ponded width. Furthermore, as water approaches the sag configuration inlet from each of two directions, the flow in the curb and gutter from each direction subtends its own ponded width. If the sag configuration inlet is in the trough of a vertical curve, the slope in the immediate vicinity of the sag inlet is equal to 0 %. Therefore, no specific slope is available for the computation of gutter flow characteristics. If the low point inlet is located at the intersection of two tangent approach slopes with no vertical curve, use the actual longitudinal slopes for the calculation of flow depths in the gutter.

Because the water or its debris load can go no other place, apply an appropriate safety factor to the inlet size. For grate inlets in sags, the usual safety factor is approximately two. For curb inlets, the ratio can be somewhat less. This is conventional practice for the TxDOT. For example, if a low point grate inlet requires an open area of 4.1 sq.ft. (2.1 m^2) and the standard inlet open area is 4.0 sq.ft. (2.0 m^2), provide two inlets for a total open area of 8.0 sq.ft. (4.0 m^2) (safety factor = 1.9).

In addition, where significant ponding can occur such as in underpasses and in sag-vertical curves, it is good engineering practice to place flanking inlets on each side of the sag location inlet. Analyze flanking inlets as inlets on-grade at some specified distance away from the low point on the sag vertical curve. Often, the specified distance is 50 or 100 ft. (15 or 30 m). The on-grade inlets serve to relieve some or most of the flow burden from the inlet located at the low point. Place the flanking inlets so that they will limit spread on low gradient approaches to the level point and act in relief of the sag inlet if it should become clogged or if the design spread is exceeded.

Median/Ditch Drains

Drains or inlets appearing in ditches and medians are usually grate inlets and are also termed "drop inlets." Often, such an inlet is in a sag (sump) configuration. In sag configurations, drains have a high probability for maintenance problems. As with grate inlets in gutters, grate inlets used in medians or other ditches should usually have the grate bars aligned parallel to the flow. A concrete riprap collar that forms a type of bowl around the inlet will improve the operational characteristics of the facility. If the inlet in the median or ditch is in an on-grade configuration, you may need to provide a downstream dike or “ditch block” as illustrated in Figure 10-8.

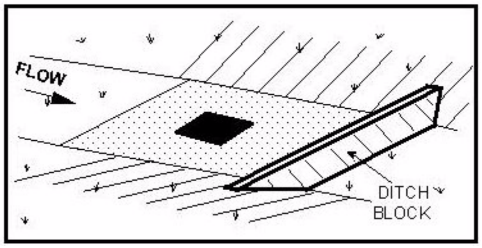

Figure 10-8. Median/Ditch Inlet

Over-side drains, also referred to as drainage chutes, are used when no inlet at the curb and gutter line connects to a storm drain system. An opening in the curb connecting to a scour-resistant channel or chute removes the concentrated flow in the curb and gutter from the roadway. In some instances, you may replace the channel or chute with a small pipe placed in the roadway embankment as illustrated in Figure 10-9.

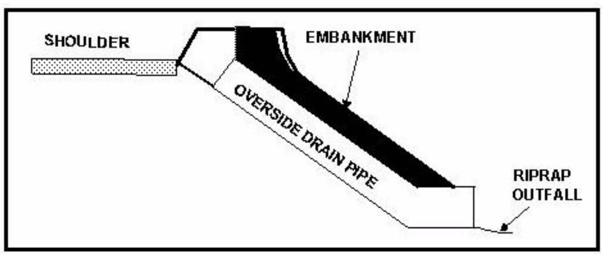

Figure 10-9. Over-Side Drains

Inlet Locations

The inlet location may be dictated either on the basis of physical demands, hydraulic requirements, or both. In all instances, you must coordinate the inlet location with physical characteristics of the roadway geometry, utility conflicts, and feasibility of underground pipe layout.

Establish logical locations early on as permanent and non-adjustable fixtures in the storm drain system. Determine their hydraulic characteristics in the ordinary trial and error process of storm drain design. Logical locations for inlets include sag configurations, near street intersections, at gore islands (see Figure 10-10), and super-elevation transitions.

Inlets with locations not established by physical requirements should be located on the basis of hydraulic demand.

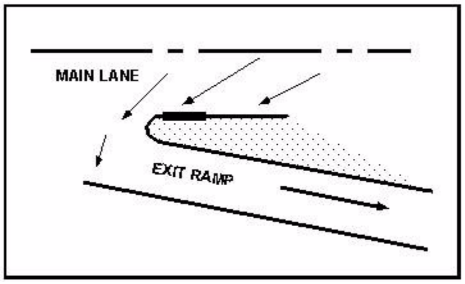

Figure 10-10. Inlet at a Gore Island

Ponded Width Options

An on-grade inlet may be necessary to remove some or all of the flow at that point so that the basic design criterion, allowable ponded width, is not violated. For a given tentative inlet location, determine the ponded width to that point. Figure 10-11 shows interdependence of inlet location, drainage area, discharge, and ponded width. If the calculated ponded width is greater than the allowable ponded width, you have two options:

- Relocate the inlet at a point upstream in the curb and gutter section. This reduces the watershed area and, thus, the peak discharge. The lowered peak discharge causes a smaller ponded width. If this is done, the drainage area to the next downstream location is increased, thus increasing the discharge and ponding.
- Locate an intermediate inlet at some point upstream in the curb and gutter section. This intermediate inlet defines a new watershed from which a reduced discharge flows, reducing the ponded width at the original inlet location.

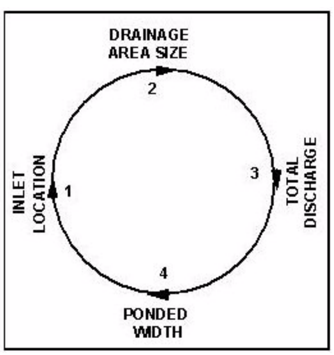

Figure 10-11. Relation of Inlet Location to Design Discharge

If the calculated ponded width is less than or equal to the allowable ponded width, you must decide if it represents an efficient design. Compare the calculated ponded width to the allowable ponded width as a measure of efficiency. If you use all or most of the allowable ponded width, the location is probably efficient. If you use only a small portion of the allowable ponded width, a more efficient location may be possible. In extensive storm drain systems, it should be a design objective to minimize the number of inlets. You may do this effectively by using as much of the allowable ponded width as is possible.

Carryover Design Approach

By using an on-grade inlet to intercept only a portion of the total flow in the gutter, you can make the inlet much more efficient than if all of the flow were to be intercepted. The rate of gutter flow not intercepted is called carryover. This design approach is recommended in those instances where it is not necessary to intercept all of the flow. The approach can be applied only in on-grade inlet configurations.

Figure 10-12 illustrates (in profile) approximately what happens when the inlet is designed to intercept all of the approaching flow. Note the large portion of inlet opening that is not utilized efficiently.

Figure 10-13 illustrates (in profile) approximately what happens when the inlet is designed to intercept less than all of the approaching flow. The remainder of the flow is the carryover. Note that the inlet opening is used much more efficiently for flow interception than the inlet illustrated in Figure 10-12.

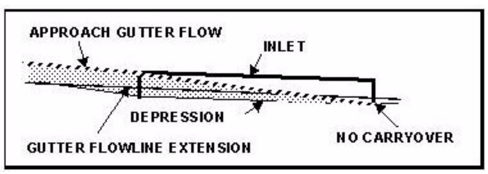

Figure 10-12. Inlet Designed with No Carryover

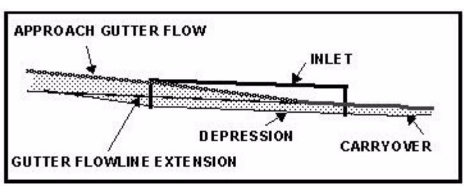

Figure 10-13. Inlet Designed with Carryover

You must accommodate any carryover rates by ultimate interception at some other location (sometimes termed “bypass flow”). Furthermore, the gutter between the two points must accommodate the additional carryover rate. Carryover is not recommended upstream of intersection and driveways, at super-elevation transitions where the cross slope begins to reverse, and below entrance/ exit ramps.

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.

For each on-grade inlet, determine early whether or not carryover is to be a valid design consideration. In some cases due to a logical location of the inlet, no carryover may be allowed. In other cases, while carryover is acceptable, there may not be a convenient location to accommodate the bypass flow.

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 10-8. Figure 10-14 shows the gutter cross section at an inlet.

$$E_0 = \frac{K_W}{K_W + K_0}$$

Equation 10-8.

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)

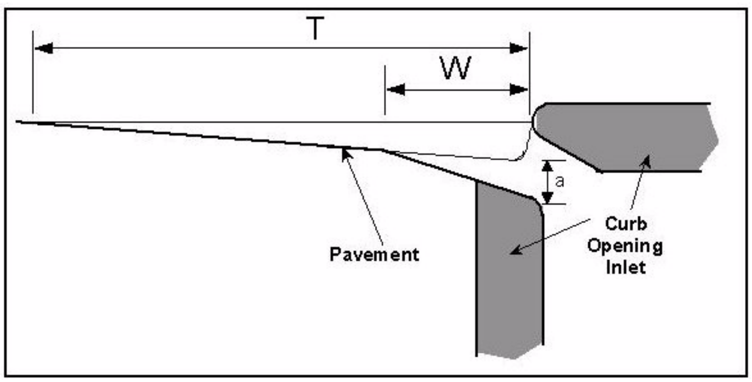

Figure 10-14. Gutter Cross-Section Diagram

Use Equation 10-9 to calculate conveyance, K_W and K_0.

$$K = \frac{zA^{5/3}}{n\,P^{2/3}}$$

Equation 10-9.

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 10-10: 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 10-10.

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 10-11 to calculate the wetted perimeter in the depressed gutter section.

$$P_W = \sqrt{(WS_X = a)^2 = W^2}$$

Equation 10-11.

where:

P_W = wetted perimeter 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)

a = curb opening depression depth (ft. or m)

Use Equation 10-12 to calculate the area of cross section of the gutter section beyond the depression.

$$A_0 = \frac{S_X}{2}(T - W)^2$$

Equation 10-12.

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 (ft. or m)

Use Equation 10-13 to calculate the wetted perimeter of the gutter section beyond the depression.

$$P_0 = T\text{-}W$$

Equation 10-13.

where:

P_0 = wetted perimeter of the depressed gutter section (ft^2 or m^2)

T = calculated ponded width (ft. or m)

W = depression width for an on-grade curb inlet (ft. or m)

3. Use Equation 10-14 to determine the equivalent cross slope (Se) for a depressed curb opening inlet.

$$S_e = S_x + \frac{a}{W} E_o$$

Equation 10-14.

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 10-15.

$$L_r = z\,Q^{0.42}\,S^{0.3}\left(\frac{1}{n\,S_e}\right)^{0.6}$$

Equation 10-15.

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 carryover is allowed, the inlet length is assigned a nominal dimension of at least L_r. Use a nominal length available in standards for curb opening inlets. Do not use the exact value of L_r if doing so requires special details, special drawings and structural design, and costly and unfamiliar construction. If carryover is considered, round the curb opening inlet length down to the next available (nominal) standard curb opening length and compute the carryover flow.

5. Determine carryover flow. In carryover computations, efficiency of flow interception varies with the ratio of actual length of curb opening inlet supplied (L_a) to length L_r and with the depression to depth of flow ratio. Use Equation 10-16 for determining carryover flow.

$$Q_{co} = Q\left(1 - \frac{L_a}{L_r}\right)^{1.8}$$

Equation 10-16.

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)

Carryover rates usually should not exceed about 0.5 cfs (0.03 m^3/s) or about 30% of the original discharge. Greater rates can be troublesome and cause a significant departure from the principles of the Rational Method application. In all cases, you must accommodate any carryover rate at some other specified point in the storm drain system.

6. Calculate the intercepted flow. Calculate the intercepted flow as the original discharge in the approach curb and gutter minus the amount of carryover 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 (d) is less than or equal to 1.4 times the inlet opening height (h), (d1.4H), 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 10-17.

$$Q = C_W L d^{1.5}$$

Equation 10-17.

Rearrange Equation 10-17 to produce the following relation for curb inlet length required.

$$L = \frac{Q}{C_w d^{1.5}}$$

Equation 10-18.

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.

d = head at inlet opening (ft. or m), computed with Equation 10-1.

L = length of curb inlet opening (ft. or m)

2. If the depth of flow in the gutter is greater than the inlet opening height (d > h), determine the length of inlet required considering orifice control. The equation for interception capacity of a curb opening operating as an orifice follows:

$$Q = C_o hL\sqrt{2gh}$$

Equation 10-19.

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_e = effective head at the centroid of the orifice (ft. or m) d_e=d - h/2

Rearranging Equation 10-19 allows a direct solution for required length.

$$L = \frac{Q}{C_o h\sqrt{2gd_e}}$$

Equation 10-20.

3. If both steps 1 and 2 were performed (i.e., h < d ≤ 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

Use the following procedure 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 10-21.

$$L_r = \frac{z\,Q_a^{0.442}\,S^E\,S_x^{-0.849}}{n^{0.384}}$$

Equation 10-21.

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 10-22

S_x = transverse slope (ft./ft. or m/m)

n = Manning's roughness coefficient

$$E = 0.207 - 19.084S^2 + 2.613S - 0.0001S_x^{-2} + 0.007S_x^{-1} - 0.049SS_x^{-1}$$

Equation 10-22.

Equation 10-21 is limited to the following ranges of variables:

total discharge ≤ 5.5 cfs (0.156 m^3/s)

longitudinal gutter slope ≤ 0.09 ft./ft. (0.09 m/m)

roughness coefficient (n) in the curb and gutter: $0.011 \le n \le 0.017$

Because the equations are empirical, extrapolation is not recommended.

2. Select the desired design slotted drain length (La) 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 10-23.

$$Q_{co} = 0.918\, Q\left(1 - \frac{L_a}{L_r}\right)^{1.769}$$

Equation 10-23.

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)

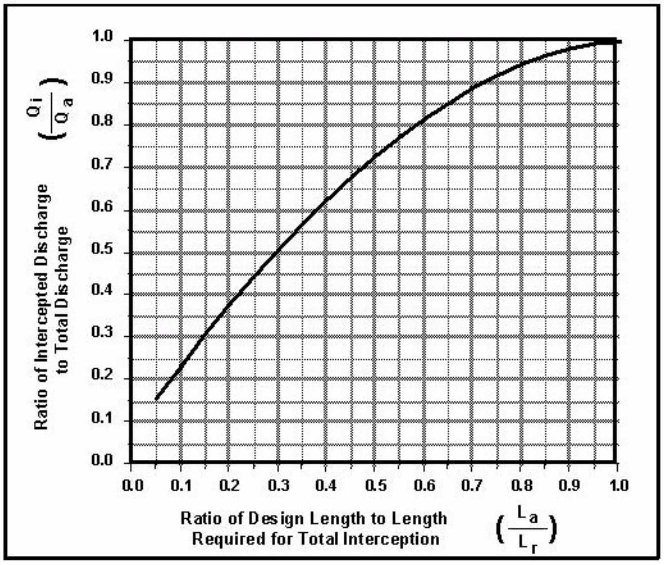

Figure 10-15. Slotted Drain Inlet Interception Rate

As a rule of thumb, you can optimize slotted drain inlets 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.

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, is intercepted by grate inlets, 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 will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 ft. (0.6 m) long, intercepted flow is small. Agencies and manufacturers of grates have investigated inlet interception capacity. For inlet efficiency data for various sizes and shapes of grates, refer to HEC-12.

Bicycle Safety for Grate Inlets On-Grade

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. In certain locations where leaves may create constant maintenance problems, the parallel bar grate may be used more efficiently if bicycle traffic is prohibited.

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 Section 4 (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 10-7. 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 Equation 10-24, Equation 10-25, and Equation 10-26.

$$R_f = 1 - 0.3(v - v_o),\ \text{if } v > v_o$$

Equation 10-24.

$$R_f = 1.0,\ \text{if } v < v_o$$

Equation 10-25.

where:

R_f = ratio of frontal flow intercepted to total frontal flow

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 10-25.

$$v = \frac{2Q}{Ty} = \frac{2Q}{T^2 S_x}$$

Equation 10-26.

Otherwise, compute the section area of flow (A) and calculate the velocity using Equation 10-25:

$$v = \frac{Q}{A}$$

Equation 10-27.

Calculate the minimum velocity (v_o) that will cause splash over the grate using the appropriate equation in tables below.

where:

v_o = splash-over velocity (ft./s or m/s)

L = length of grate (ft. or m)

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	$v_o = 1.762 + 3.117L - 0.451L^2 + 0.033L^3$
Transverse Curved Vane	4.5	$v_o = 1.381 + 2.78L - 0.300L^2 + 0.020L^3$
Transverse 45^o Tilted Vane	4	$v_o = 0.988 + 2.625L - 0.359L^2 + 0.029L^3$
Parallel bars w/ transverse rods	2 parallel/4 trans	$v_o = 0.735 + 2.437L - 0.265L^2 + 0.018L^3$
Transverse 30^o Tilted Vane	4	$v_o = 0.505 + 2.344L - 0.200L^2 + 0.014L^3$
Reticuline	n/a	$v_o = 0.030 + 2.278L - 0.179L^2 + 0.010L^3$

Splash-Over Velocity Calculation Equations (Metric)

Grate Configuration	Typical Bar Spacing (mm)	Splash-over Velocity Equation
Parallel Bars	50	$v_o = 0.676 + 4.031L - 2.13L^2 + 0.598L^3$
Parallel Bars	30	$v_o = 0.537 + 3.117L - 1.478L^2 + 0.358L^3$
Transverse Curved Vane	115	$v_o = 0.421 + 2.78L - 0.984L^2 + 0.215L^3$
Transverse 45^o Tilted Vane	100	$v_o = 0.301 + 2.625L - 1.177L^2 + 0.311L^3$
Parallel bars w/ transverse rods	50 parallel/100 trans	$v_o = 0.224 + 2.437L - 0.869L^2 + 0.192L^3$
Transverse 30^o Tilted Vane	100	$v_o = 0.154 + 2.344L - 0.656L^2 + 0.155L^3$
Reticuline	n/a	$v_o = 0.009 + 2.278L - 0.587L^2 + 0.108L^3$

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 10-28.

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 10-29.

$$E_f = \left[R_f E_o + R_s \left(1 - E_o\right)\right]$$

Equation 10-29.

7. Calculate the interception capacity of the grate, Q_i. Use Equation 10-30. If the interception capacity is greater than the design discharge, skip step 8.

$$Q_i = E_f Q = Q\left[R_f E_o + R_s \left(1 - E_o\right)\right]$$

Equation 10-30.

8. Determine the carryover, CO. Use Equation 10-31.

$$CO = Q - Q_i$$

Equation 10-31.

9. Depending on the carryover, select a larger or smaller inlet as needed. If the carryover 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 ponding depths. A transition to orifice flow begins as the ponded depth increases. Use the following procedure 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. This should be the lower of the curb height and the depth associated with the allowable ponded width. 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 10-32.

$$Q_w = C_w P h^{1.5}$$

Equation 10-32.

where:

Q_w = weir capacity of grate (cfs or m3/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)

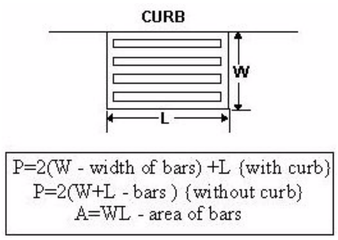

Figure 10-16. Perimeter Length for Grate Inlet in Sag Configuration

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 10-33.

$$Q_o = C_o A \sqrt{2 g h}$$

Equation 10-33.

where:

Q_o = orifice capacity of grate (cfs or m3/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 a 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.

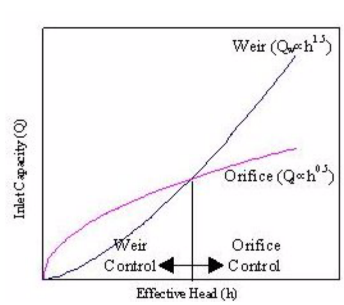

Figure 10-17. Relationship between Head and Capacity for Weir and Orifice Flow

Section 6
Conduit Systems

Conduits

The storm drainage conduit system transports the runoff from the surface collection system (inlets) to the outfall. Although it is an integral component, analyze the conduit system independently of the inlet system.

An inlet location in a storm drain system basically controls the need for a conduit, its slope and horizontal orientation, and its minimum cover requirements.

The configuration of laterals and trunk lines is controlled by the locations of all inlet and roadway layouts and is also affected by utility and foundation locations.

The longitudinal slope of the conduit affects its capacity. The slope of the subject run is tentatively established during the system planning stage of design. Typically, the slope will be approximately parallel to the surface topography. However, you may have to adjust conduit slopes to adapt to critical elevations (such as outfall elevations). You can adjust individual run slopes as necessary to increase capacity, avoid conflicts with utilities, and afford adequate cover for the conduit.

Avoid circular pipe sizes less than 18 in. (450 mm) diameter for main trunk lines or laterals because of difficulties in their construction and maintenance. Some designers prefer to limit the minimum circular diameter to 24 in. (600 mm). Consider the following recommendations on conduit dimensions:

- Standard size pipe use in conduits -- Do not use non-standard sizes of pipe. It is rarely cost effective to specify pipe dimensions requiring special fabrication. Consult with local fabricators, become acquainted with stockpiled dimensions, and use those commonly manufactured sizes in the design.
- Larger versus smaller conduit dimensions -- Avoid discharging the flow of a larger conduit into a smaller one. The capacity of the smaller conduit may technically 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 size of conduit, make an attempt to place the soffits (top inside surfaces) of the two conduits at the same level rather than placing the flow lines at the same level. Where flow lines are placed at the same level, 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.
- Conduit length -- You may approximate the length of the conduit for these calculations. Often, the length is indicated as from centerline-to-centerline of the upstream and downstream nodes

of the subject conduit run. Use the length with the average flow velocity to estimate the travel time within the subject 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 are not pay lengths of conduit; the standard specifications provide that pay lengths include only the actual net length of pipe and not the distance across inlets or manholes where no conduit actually is placed.

Manholes

Place manholes or combination manholes and inlets wherever necessary for clean-out and inspection purposes. It is good engineering practice to place manholes at changes in direction, junctions of pipe runs, and intervals in long pipe runs where the size or direction may not have changed. The table below provides recommended maximum spacing criteria for manholes.

Manholes Spacing Criteria

Pipe Diameter		**Maximum Distance**	
in.	mm	ft.	m
12 – 24	300 - 600	300	100
27 – 36	675 – 900	375	120
39 – 54	1050 – 1350	450	150
=>60	=> 1500	900	300

Round the invert (bottom) of the manhole section to match the inverts of the pipes attached to the manhole to minimize eddying and resultant head losses. For manholes larger than the incoming or outgoing pipes, expansion losses can sometimes be significant.

Detail manholes that are intended as combinations with other functions to include facilities that will serve all the intended functions. In such cases, you may need to consider junction losses.

At junctions of pipelines, right angle intersections are simpler to construct. However, an acute angle junction reduces head losses, and you should consider it where practical. See Figure 10-18 for the contrast. Where junction losses may be of particular concern, consider using acute angle junctions.

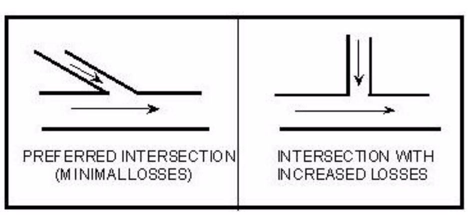

Figure 10-18. Head Losses at Intersections

Inverted Siphons

Inverted siphons carry flow under obstructions such as sanitary sewers, water mains, or any other structure or utility that may be in the path of the storm drain line. Use them only where avoidance or adjustment of the utility is not practical. The storm drain flowline is lowered at an obstacle and is raised again after the crossing. In the design of inverted siphons, we recommend a minimum flow velocity of 3 fps (1 m/s).

In general, the conduit size through the inverted siphon used as 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 an inverted siphon includes slopes of zero and adverse values, account for head losses through the structure using outlines in Chapter 6, Hydraulic Grade Line Analysis. The sources of these losses can be friction, bends, junctions, and transitions.

If the losses are unacceptable, you may need alternative means of avoiding the utility conflict. Provide maintenance access at either or both ends of the inverted siphon as indicated in Figure 10-19.

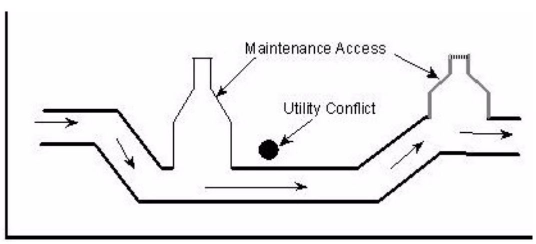

Figure 10-19. Inverted Siphon

Conduit Capacity Equations

Refer to Chapter 6 for calculating channel (conduit) capacity and critical depth.

Conduit Design Procedure

In this procedure, nodes represent point definitions in the network such as junctions and inlets. Runs represent the conduit connections between nodes. 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 re-computed based on cumulative drainage area, runoff coefficient, and longest time of concentration contributing to the particular node.

Use the following steps for the design of conduit systems:

1. Determine the design discharge at each extreme node (inlet). The design discharge for a particular run is based on the watershed area to the upstream node of the run, the associated weighted runoff coefficient, and the rainfall intensity based on the time of concentration (t_c) in the watershed. 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. If the t_c is less than 10 minutes, base the intensity on a t_c of 10 minutes; otherwise, use the actual t_c value. Use this value of t_c in Equation 10-35 for rainfall intensity and compute the discharge using Equation 10-34. Account for the actual time of concentration as this value eventually may become significant even if it is less than 10 minutes.

$$Q = \frac{C\,I\,A}{z}$$

Equation 10-34.

where:

Q = peak discharge (cfs or m^3/s)

C = runoff coefficient

I = rainfall intensity associated with a specific frequency (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 10-35.

where:

I_f = rainfall intensity for frequency (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 in Hydrology. (See Rainfall Intensity-Duration-Frequency Coefficients.)

If the inlet has been designed with carryover, either from or to the inlet, ignore the carryover rate(s) when considering the discharge into the conduit.

Base the intensity 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 upstream end of a conduit run. It also means that you do not simply add discharge rates from approaching watersheds and/or pipe runs; rather, multiply the sum of contributing CA values by an intensity based on the longest time of concentration leading to the point in question.

2. Size the conduit for pressure flow or for non-pressure flow based on Manning's Equation and the design discharge. The 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. For TxDOT, pressure flow design means that the conduit has dimensions smaller than necessary to accommodate the design flow under the terms of Manning's Equation. If it is necessary or useful to design conduits for pressure flow, coordinate such design with the Bridge Division, Hydraulic Branch. To size circular pipe, use Equation 10-36 (depending on material type and associated roughness):

$$D = z\left(\frac{Q\,n}{S^{1/2}}\right)^{3/8}$$

Equation 10-36.

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: select a trial size and compute the capacity. Adjust the size until the computed capacity is slightly higher than the design discharge.

3. Estimate the velocity of flow through the designed conduit. Assume uniform flow as an average depth of flow in the conduit as discussed in Section 2 of Chapter 6. Determine the cross-section area, A_u, at this depth. This is a straightforward procedure for rectangular sections but much more complicated for circular and other shapes. Manufacturers' product information may include tables of depth, area, and wetted perimeter. If not, calculate area and wetted perimeter based on the geometry of the conduit. Then calculate the average velocity of flow (V_a) using the continuity relation shown in Equation 10-37.

$$V_a = \frac{Q}{A_u}$$

Equation 10-37.

4. Calculate the travel time for flow in the conduit from the upstream inlet/node to the downstream node by dividing the length of the conduit by the average velocity of flow. Add this travel time to the time of concentration at the upstream end of the subject run to represent the time of concentration at the downstream end of the run.

NOTE: When accumulating times, base the time of concentration on the actual calculated times, even if it is less than the minimum of 10 minutes.

5. Determine the total drainage area, cumulative runoff coefficient times area, and respective time of concentration. As you complete the design of the most remote runs and the design proceeds downstream through the system, determine the total drainage area, cumulative runoff coeffi-

cient times area, and respective time of concentration for all conduits incoming at a particular node before sizing the conduit run out of that node.

6. Compute the peak discharge for the next run downstream based on the total drainage area upstream contributing to each incoming conduit/run at the node, the cumulative product of the runoff coefficient and contributing area to each incoming conduit/run at the node, the longest time of concentration of all incoming conduits, and, if applicable, inlet time for the node. (This time is used to re-compute intensity in the rational equation for sizing the next downstream conduit run).

NOTE: You can easily determine the area and runoff coefficient if you record the CA values for each watershed as you proceed with design down the system and sum them at each node.

7. Continue this process until you have sized all conduits in the network. In each case, as runs and entering watersheds converge to a node, recalculate the peak discharge for which the exiting conduit is to be designed as the product of an intensity based on the longest time of concentration leading into the node and a summation of all CA values that contribute flow to the node. The discharge, so determined, is not the same as if you have added all approaching discharges because the procedure is fashioned to conform to the general application requirements for the Rational Method. In some instances, calculated discharges can decrease as you carry the analysis downstream (because of a small increase in the accumulated CA as compared to rainfall intensity). 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 Chapter 10.

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 7

Conduit Systems Energy Losses

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.

Junction Loss Equation

For adjoining pipes to be considered a pipe junction, the node and only two inflow pipes (a lateral and a trunk) may enter the junction. The minor loss equation for a pipe junction is in the form of the momentum equation. In Equation 10-38 the subscripts "i", "o", and "1" indicate the inlet, outlet, and lateral, respectively.

$$h_j = \frac{Q_o v_o - Q_i v_i - Q_1 v_1 \cos\theta}{0.5g(A_o + A_i)}$$

Equation 10-38.

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 10-39.

$$h_o = C_o \frac{v^2 - v_d^2}{2g}$$

Equation 10-39.

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

Manhole Loss Equations

Calculate the loss at a manhole where one pipe enters and one leaves using Equation 10-40.

$$h = K \frac{v_o^2}{2g}$$

Equation 10-40.

where the adjusted head loss coefficient (K) is found with Equation 10-41.

$$K = K_o C_D C_d C_Q C_p C_B$$

Equation 10-41.

where:

K_O = initial head loss coefficient based on relative manhole size

C_D = correction factor for pipe diameter

C_d = correction factor for flow depth

C_Q = correction factor for relative flow

C_B = correction factor for benching

C_P = correction factor for plunging flow

The initial head loss coefficient (Ko)) is estimated as a function of the relative manhole size and angle between the inflow and outflow pipes.

$$K_o = 0.1\left[\frac{b}{D_o}\right]\left[1 - \sin\theta\right] + 1.4\left[\frac{b}{D_o}\right]^{0.15}\sin\theta$$

Equation 10-42.

where:

K_O = initial head loss coefficient based on relative manhole size

θ = angle between the inflow and outflow pipes (see Figure 10-20)

b = manhole diameter or width (ft. or m)

D_O = outlet pipe diameter (ft. or m)

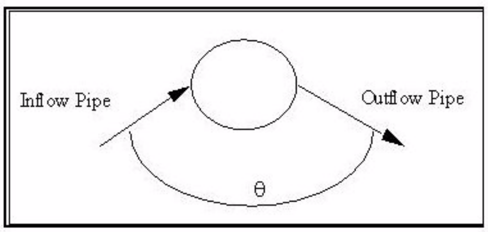

Figure 10-20. Angle Between Inflow and Outflow Pipes

The correction factor for pipe diameter, C_D, can be determined by the following:

$$C_D = \left[\frac{D_o}{D_i}\right]^3$$

Equation 10-43.

where:

C_D = correction factor for variation in pipe diameter

D_I = incoming pipe diameter (ft. or m)

D_O = outgoing pipe diameter (ft. or m)

A change in head loss due to differences in pipe diameter is significant only in pressure flow situations when the depth in the manhole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, only apply it in such cases; otherwise, use $C_D = 1$. Calculate the correction factor for flow depth, C_d, using Equation 10-44.

$$C_d = 0.5\left[\frac{d}{D_o}\right]^{3/5}$$

Equation 10-44.

where:

C_d = correction factor for flow depth

D = water depth in manhole above outlet pipe invert (ft. or m)

D_O = outlet pipe diameter (ft. or m)

This correction factor is significant only in cases of free surface flow or low pressures, when d/D_O ratio is less than 3.2. Water depth in the manhole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. Compute the correction factor for relative flow, C_Q, using Equation 10-45.

$$C_Q = (1 - 2\sin\theta)\left[1 - \frac{Q_i}{Q_o}\right]^{3/4} + 1$$

Equation 10-45.

where:

C_Q = correction factor for relative flow

θ = angle between the inflow and outflow pipes

Q_i = flow in the incoming pipe (cfs or m^3/s)

Q_O = flow in the outlet pipe (cfs or m^3/s)

C_Q = a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes.

To illustrate this effect, consider the following example (see Figure 10-21):

$Q_1 = 0.3\ m^3/s$

$Q_2 = 0.1\ m^3/s$

$Q_3 = 0.4\ m^3/s$

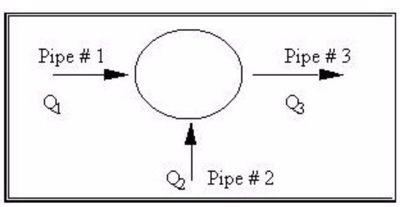

Figure 10-21. Example of Correction Factor for Relative Flow

Solving for the relative flow correction factor in going from the outlet pipe (number 3) to one of the inflow pipes (number 2):

$$C_{Q_{3-2}} = \left[1 - 2\sin(90^\circ)\right]\left[1 - \frac{0.1}{0.4}\right]^{3/4} + 1 = 0.19$$

Equation 10-46.

For a second example, consider the following flow regime:

Q_1=1 cfs

Q_2=3 cfs

Q_3=4 cfs

Calculating C_Q for this case:

$$C_{Q_{3-2}} = \left[1 - 2\sin(90^\circ)\right]\left[1 - \frac{0.3}{0.4}\right]^{3/4} + 1 = 0.65$$

Equation 10-47.

In both of these cases, the flow coming in through pipe number 2 has to make a 90-degree bend before it can go out pipe number 3. In case 1, the larger flow traveling straight through the manhole from pipe number 1 to pipe number 3 assists the flow from pipe number 2 in making this bend. In case 2, a majority of the flow is coming in through pipe number 2. There is less assistance from the straight through flow in directing the flow from pipe number 2 into pipe number 3. As a result, the correction factor for relative flow in case 1 (0.19) is much smaller than the correction factor for case 2 (0.65). The correction factor for plunging flow, C_p , is calculated using Equation Equation 10-48.

$$C_p = 1 + 0.2\left[\frac{h}{D_o}\right]\left[\frac{h-d}{D_o}\right]$$

Equation 10-48.

where:

C_P = correction for plunging flow

h = vertical distance of plunging flow from the center of the outlet pipe (ft. or m)

D_o = outlet pipe diameter (ft. or m)

d = water depth in the manhole (ft. or m)

This correction factor corresponds to the effect of another inflow pipe plunging into the manhole on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 10-21, for example, calculate C_p for pipe number 2 when pipe number 1 discharges plunging flow. Consider the plunging flow that results from flow entering through the inlet into the manhole in the same manner. Only apply the correction factor when h is greater than d.

The table below presents correction factors for benching, C_B. Benching refers to how the conduit is placed with respect to the manhole as follows:

- Depressed floor -- The manhole bottom is lower than the storm drain conduit.

- Flat floor -- The manhole bottom is flush with the storm drain conduit.
- Half bench -- The bottom of the manhole is grouted or shaped to match up with the bottom half of the conduit.
- Full bench -- The bottom of the manhole is grouted or shaped to the top of the storm drain conduit.

Correction Factor for Benching

	Correction Factor, C_B	
Bench Type	Pressure Flow ($d/D_O > 3.2$)*	Free Surface Flow ($d/D_O < 1.0$)*
Flat or Depressed Floor	1.0	1.0
Half Bench	0.95	0.15
Full Bench	0.75	0.07
* If $1.0 < d/D_O < 3.2$, use linear interpolation between pressure flow and free surface flow coefficients.		

Chapter 11
Pump Stations

Contents:

Section 1
Introduction

Purpose of Pump Stations

A pump station mechanically lifts storm water runoff. In general, gravity outfalls are the primary and preferred means of releasing flow from storm drain systems. However, a pump station becomes necessary in the following instances:

- if it is physically impossible to drain by gravity alone
- if it is uneconomical to use a gravity system due to the required length and depth to reach receiving water
- if the receiving water level would inundate the roadway and adjacent property by backing up through the storm drain system

The need for pump stations is much more a function of the highway geometric design than of climatic factors. Planners can design pump stations to be unobtrusive, efficient, and reliable.

In the planning stages, you can obtain valuable advice and assistance from the following sources:

- representatives of pump manufacturers
- contractors who have had experience in pump station construction
- representatives of utility firms that will supply power to the station, such as electricity, natural gas, and diesel fuel

Security and Access Considerations

Protect and secure the pump station facility with fences, gates, and locks. When planning the fencing, provide adequate access for service and maintenance vehicles.

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 Bridge Division’s Hydraulics Branch, about the quality of the runoff water. Refer to the TxDOT Environmental and Policy Manuals for more information on environmental concerns, policies, and agencies.

Section 2

Pump Station Components

Overview

A pump station includes the following:

- Pumps—Pump selection depends on station layout, required pump rate, wet well depth, and 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 provide multiple pumps rather than a single pump of appropriate size. Smaller pumps are usually cheaper, and with multiple pumps, the loss of one will not shut down the entire pump station.
- 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.
- Power sources—The power source is usually 480-volt, three-phase electrical service provided by the local utility. Depending on availability, a redundant, secondary electrical service feed from a different electrical substation can provide regular power if the primary service power is interrupted. Every pump station should have an on-site standby electrical generator regardless of the presence of redundant utility power because the type of storm that makes a pump station necessary is also the type of storm that interrupts utility power. Standby generators are usually powered by diesel or natural gas and rarely by gasoline. Fuel type depends on the size of the standby unit, the fuel source and availability, the site layout, and local economics.
- Controls—Control circuitry includes the flood level at which the pump station will be activated, sequence of operation, activation of the standby generator when necessary, deactivation when the flood event has passed, and operation of any night security lighting. Controls may also include automatic communication with a central office on the station's status regarding water levels, pump readiness, utility electrical power, standby generator fuel level, security, or other central office concerns.
- Structures—The structure should meet requirements for public safety, local extreme weather conditions, site security, and maintenance operation. Consider also aesthetics and the possible need for future expansion.
- Wet well sumps— The wet well sump receives the inflow of storm water prior to pumping. The sump serves as a storage space so that, as the storm progresses, the storm hydrograph peak may be attenuated. You can accomplish this analytically by following the Hydraulic Design Procedure in this chapter. Design the wet well sump with provisions for screening trash and other debris associated with the storm water and convenient access for the removal of accumulated debris and silt.

- Discharge conduits—While waters are usually discharged to a storm drain system, sometimes the discharge point is a wetland, mud flat, or creek. 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.

Full discussion of the design and specification of a pump station is beyond the scope of this manual. Consult appropriate design specialists for the electrical, mechanical, and structural components of a pump station.

Section 3

Pump Station Hydrology

Methods for Design

The hydrology developed in the associated storm drain system should serve as a firm basis for discharge determination in pump station design. The two most typical design methods are as follows:

- Rational Method for pumps -- Because small watershed sizes (less than 200 acres or 80 ha) are usually associated with pump station facilities, the Rational Method for determining peak runoff is often used. To determine peak runoff, use the Rational Equation (Equation 5-3).
- Synthetic hydrograph -- In most cases, the synthetic hydrograph is adequate for the pump station design basis. Some situations may require other methods of discharge estimation or the development of a natural runoff hydrograph. The department bases its drainage facilities designs on the assumption that the peak discharge continues for an indefinite period of time. However, since it is usually practical to detain a portion of a flood drained by a pump station in a wet well sump, the procedure for design/analysis of pump stations incorporates detention storage of the flood. This effectively attenuates the runoff hydrograph since a portion of the incoming flood is temporarily stored in the sump. A major ingredient of a facility design incorporating detention storage is the incoming runoff hydrograph. For expedience, the department assumes a runoff hydrograph as illustrated in Figure 11-1.

The storm drain system associated with the pump station may have a design basis of less than 50 years. However, engineers recommend developing a design capable of accommodating at least a 50-year flood because the pump station is generally used when drainage by gravity from a low point is inadequate or impractical.

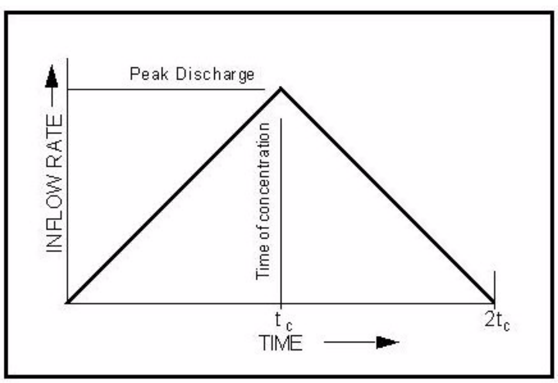

Figure 11-1. Runoff Hydrograph

Section 4
Pump Station Design Procedure

Design Guidelines

The allowable water level is the maximum elevation that you allow storm water to pond in the low point of the roadway section. Properly sized pumps (for the design storm) should maintain the ponded elevation of the storm water equal to or below the allowable water level at peak discharge.

The available flood storage is the volume of water that can pond in the system below the allowable water level and above the pump cut-off elevation. This includes water that would be ponded in roadway ditches, gutters, pipes, boxes, inlets, manholes, and wet well sumps.

Pump Characteristics

The following procedure and example provide some guidelines for the hydraulic design of a pump station, and the following table lists the elements required for the hydraulic design of a pump station.

Required Elements for Pump Station Hydraulic Design

Element	Description
Input flow rate	The total storm flood collected into the pump station, calculated from hydrology
Plan and low point cross section of roadway	See Figure 11-2 and Figure 11-3, for example.
Water surface elevation of discharge	Higher elevation to which the water must be raised
Pump cut-off elevation	Elevation at which the last pump shuts off
Number of pumps	A minimum of two pumps should be provided.
Wet well sump dimensions	The process of establishing a suitable size is often one of trial and error to optimize costs. As the sump size is increased, the required pump capacity decreases.

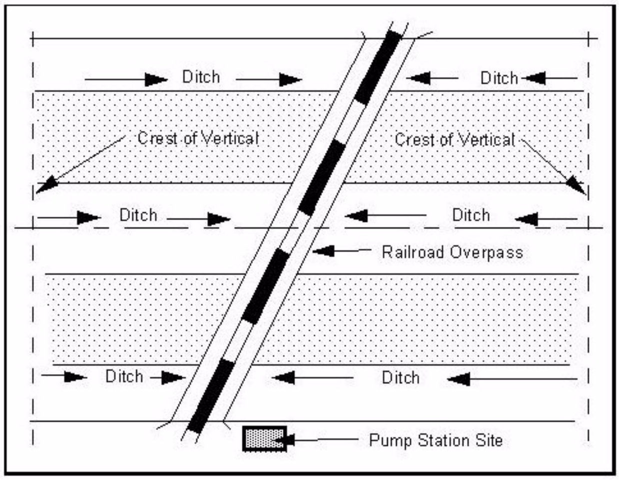

Figure 11-2. Pump Station Schematic

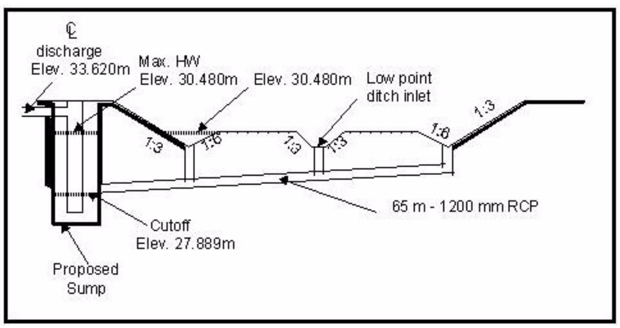

Figure 11-3. Typical Cross Section

Hydraulic Design Procedure

Use the following procedure to design a hydraulic pump station. The steps include sample calculations using both English and metric measurement units.

1. To determine the estimated peak rate of runoff from the watershed, use the Rational Equation, (Equation 5-3). For example, using e= 0.581, b = 48.5 in. (1231.9 mm), d = 10.1, and time of concentration = 10 minutes for a 50-yr frequency and a rational equation “CA” of 7.18 (2.87):

English	Metric
$I_{50} = \frac{48.5}{(10+10.1)^{0.581}} = 8.48 \text{ in/hr}$	$I_{50} = \frac{1231.9}{(10+10.1)^{0.581}} = 215.5 \text{ mm/hr}$
For a total CA of 7.18 ac: $Q_{50} = 7.18 \times 8.48 = 60.9 \text{ cfs}$	For a total CA of 2.87 ha: $Q_{50} = 2.87 \times 215.5 \div 360 = 1.718 \text{ m}^3/\text{s}$

2. The available storage represents available space below the allowable high-water elevation and above the pump cut-off elevation. The spaces in which storm water can be stored before flooding occurs include sump storage, pipe storage, ditches, and total effective ponded volume below elevation. Using the cross section shown in Figure 11-3 with a circular sump of 15-ft. (4.6-m) diameter:

	English	Metric
Sump storage	$(100-91.5)\times\pi\times\frac{15^2}{4} = 1502.1 \text{ cu.ft.}$	$(30.480-27.889)\times\pi\times\frac{4.6^2}{4} = 43.06 \text{ m}^3$
Pipe storage	$213\times\pi\times\frac{4^2}{4} = 2676.6 \text{ cu.ft.}$	$65\times\pi\times\frac{1.2^2}{4} = 73.51 \text{ m}^3$
Estimated ditch storage	Below 100 ft. = 8641.5 cu.ft.	Below 30.480 m = 244.70 m^3
Total effective storage	Below 100 ft. = 12,820.2 cu.ft.	Below 30.480 m = 361.27 m^3

3. Even though the 50-year design discharge is based on a storm duration equal to the time of concentration, the area can experience 50-year storms of various durations. Each duration is associated with a different intensity. With storage added as another factor, you must evaluate the system to determine the adjusted peak, which represents the average pump capacity required.
   - For a synthetic runoff hydrograph, assume that the rain occurs at a constant intensity for a certain duration. Refer to Figure 11-4 for the following discussion. The inflow into the storage area is assumed to vary in a straight line from zero rate at the beginning of the rain to the point of maximum runoff rate at a time equal to the time of concentration.
   - At a time equal to the time of concentration, the rain ceases (if duration equals time of concentration), and runoff rate varies from the maximum rate to zero in a period of time also equal to the time of concentration.

- If the storm duration is longer than the time of concentration, the maximum rate of runoff is not momentary but continues at a constant rate until the storm ceases.
- The area under the rate versus time curve yields the volume of water to be accommodated.

Because the maximum rate of runoff for a specific intensity is obtained through the Rational Formula, the total volume that will flood above the maximum allowable is Equation 11-1:

$$Q = \frac{CIAD}{z}$$

Equation 11-1.

where:

CIA = Total runoff (see Chapter 5) in cfs (or m^3/sec)

D = Storm duration in seconds

z = 1 for English measurement units and 360 for metric

The average pump capacity (APC) in cfs (or m3/sec.) required to remove the flood volume is:

$$APC = \frac{\text{EXCESS VOLUME}}{\text{DURATION}}$$

Equation 11-2.

Inspect the following tables to determine the maximum required average pump capacity. This solution is only preliminary. A final design combination depends upon a thorough analysis of pump size combinations and pump initiation time schedules during the runoff event.

Average Pump Capacity Requirements (English)

1	2	3	4	5	6	7
Duration (Minutes)	Duration (Seconds)	Intensity (in/hr I=b/(t+d)e	Discharge (cfs) Q=CIA	Flood Volume (col 4 * col 3)	Excess Volume (col 5 – total storage)	APC (cfs)
5	300	10.0	71.9	21,578	8,757	29.2
6	360	9.7	69.3	24,946	12,126	33.7
7	420	9.3	66.9	28,102	15,282	36.4
8	480	9.0	64.7	31,074	18,254	38.0
9	540	8.7	62.7	33,883	21,063	39.0
10	600	8.5	60.9	36,548	23,728	39.5
11	660	8.2	59.2	39,084	26,264	39.8
12	720	8.0	57.6	41,506	28,685	39.8
13	780	7.8	56.2	43,823	31,003	39.7

Average Pump Capacity Requirements (English)

1	2	3	4	5	6	7
14	840	7.6	54.8	46,046	33,226	39.6
15	900	7.5	53.5	48,183	35,363	39.3
16	960	7.3	52.3	50,242	37,422	39.0
17	1020	7.1	51.2	52,229	39,409	38.6
18	1080	7.0	50.1	54,149	41,329	38.3
19	1140	6.8	49.1	56,008	43,188	37.9
20	1200	6.7	48.2	57,810	44,989	37.5
21	1260	6.6	47.3	59,558	46,738	37.1
22	1320	6.5	46.4	61,258	48,437	36.7
23	1380	6.3	45.6	62,911	50,091	36.3
24	1440	6.2	44.8	64,521	51,700	35.9
25	1500	6.1	44.1	66,090	53,270	35.5

Average Pump Capacity Requirements (Metric)

1	2	3	4	5	6	7
Duration (Minutes)	Duration (Seconds)	Intensity (mm/hr $I=b/(t+d)^e$	Discharge (m^3/s) Q=CIA/360	Excess Volume (col 5 – total storage)	Flood Volume (col 4 * col 3)	APC (m^3/S
5	300	254.4	2.028	608.5	247.3	0.824
6	360	245.1	1.954	703.5	342.3	0.951
7	420	236.7	1.887	792.6	431.3	1.027
8	480	229.0	1.826	876.4	515.1	1.073
9	540	222.0	1.770	955.6	594.3	1.101
10	600	215.5	1.718	1,030.7	669.5	1.116
11	660	209.5	1.670	1,102.3	741.0	1.123
12	**720**	**203.9**	**1.626**	**1,170.6**	**809.3**	**1.124**
13	780	198.8	1.585	1,235.9	874.7	1.121
14	840	193.9	1.546	1,298.6	937.4	1.116
15	900	189.4	1.510	1,358.9	997.6	1.108
16	960	185.1	1.476	1,417.0	1,055.7	1.100

Average Pump Capacity Requirements (Metric)

1	2	3	4	5	6	7
17	1020	181.1	1.444	1,473.0	1,111.7	1.090
18	1080	177.4	1.414	1,527.1	1,165.9	1.080
19	1140	173.8	1.386	1,579.6	1,218.3	1.069
20	1200	170.4	1.359	1,630.4	1,269.1	1.058
21	1260	167.2	1.333	1,679.7	1,318.4	1.046
22	1320	164.2	1.309	1,727.6	1,366.4	1.035
23	1380	161.3	1.286	1,774.2	1,413.0	1.024
24	1440	158.5	1.264	1,819.6	1,458.4	1.013
25	1500	155.9	1.243	1,863.9	1,502.6	1.002

The average pump capacity (APC) requirement reaches a maximum for a 12-minute duration storm. These examples with large storage were chosen to illustrate the storage effect. In sites where storage is small and will not offer any significant adjustment to the peak of the runoff hydrograph, use APC equal to the maximum rate of discharge from Step 1. Select two or more pump sizes that will furnish the total desired average pump capacity determined from the "Average Pump Capacity Requirements" tables above. Determine what nominal pump sizes are available in the area of the project. For these examples, the following pump sizes are available:

English Example	Metric Example
5,000 gpm = 11.1 cfs	16 m^3/min = 0.27 m^3/sec.
6,000 gpm = 13.4 cfs	20 m^3/min = 0.33 m^3/sec.
7,000 gpm = 15.6 cfs	24 m^3/min = 0.40 m^3/sec.
8,000 gpm = 17.8 cfs	28 m^3/min = 0.47 m^3/sec.
9,000 gpm = 20.0 cfs	32 m^3/min = 0.53 m^3/sec.
10,000 gpm = 22.3 cfs	36 m^3/min = 0.60 m^3/sec.

By inspection and as a preliminary solution, three 7,000-gpm (24-m^3/min) pumps provide 46.8 cfs (1.20 m^3/sec) discharge to satisfy the required APC. Other combinations would work, but this selection allows for uniformity of parts. A complete hydraulic and economic analysis is necessary for any considered configuration of pumping capacity. Often, a small utility pump (e.g., a capacity of 1200 gpm or 4 m^3/min) is used in the combination of pumps. Such a pump then would serve to cycle on and off during small runoff events while the larger capacity pumps would be reserved for the larger runoff events. This often is referred to as a sump pump.

Analysis of Initiation Times for 3-7,000 gpm – 46.8 cfs Pumps

A	B	C	D	E	F	G	H	I	J
Elapsed Time	Inflow Rate	Increm. Inflow Volume	Accum. Inflow Volume	Outflow per Pump			Remaining in Sump		
(min)	(cfs)	(cu.ft.)	(cu.ft.)	Pump #1 (cu.ft.)	Pump #2 (cu.ft.)	Pump #3 (cu.ft.)	Pump #1 (cu.ft.)	Pump #2 (cu.ft.)	Pump #3 (cu.ft.)
0	0	0	0	0					
1	5.8	173	173	0	0	0	173	173	173
2	11.5	519	692	0	0	0	692	692	692
3	17.3	865	1,556	0	0	0	1,556	1,556	1,556
4	23.1	1211	2,767	936	0	0	1,831	1,831	1,831
5	28.8	1556	4,324	1,872	0	0	2,452	2,452	2,452
6	34.6	1902	6,226	2,808	0	0	3,418	3,418	3,418
7	40.4	2248	8,474	3,744	936	0	4,730	3,794	3,794
8	46.1	2594	11,068	4,680	1,872	0	6,388	4,516	4,516
9	51.9	2940	14,008	5,616	2,808	0	8,392	5,584	5,584
10	57.6	3286	17,294	6,552	3,744	936	10,742	6,998	6,062
11	57.6	3459	20,753	7,488	4,680	1,872	13,265	8,585	6,713
12	57.6	3459	24,212	8,424	5,616	2,808	15,788	10,172	7,364
13	51.9	3286	27,497	9,360	6,552	3,744	18,137	11,585	7,841
14	46.1	2940	30,437	10,296	7,488	4,680	20,141	12,653	7,973
15	40.4	2594	33,032	11,232	8,424	5,616	21,800	13,376	7,760
16	34.6	2248	35,280	12,168	9,360	6,552	23,112	13,752	7,200
17	28.8	1902	37,182	13,104	10,296	7,488	24,078	13,782	6,294
18	23.1	1556	38,739	14,040	11,232	8,424	24,699	13,467	5,043
19	17.3	1211	3,9949	14,976	12,168	9,360	24,973	12,805	3,445
20	11.5	865	40,814	15,912	13,104	10,296	24,902	11,798	1,502
21	5.8	519	41,333	16,848	14,040	11,232	24,485	10,445	0
22	0	173	41,506	17,784	14,976	12,168	23,722	8,746	0

Analysis of Initiation Times for 3-7,000 gpm – 46.8 cfs Pumps

A	B	C	D	E	F	G	H	I	J
23	0	0	41,506	18,720	15,912	13,104	22,786	6,874	0
24	0	0	41,505.61	19,656	16,848	14,040	21,850	5,002	0

Analysis of Initiation Times for 3-24 m^3/min Pumps

A	B	C	D	E	F	G	H	I	J
Elapsed Time	Inflow Rate	Increm. Inflow Volume	Accum. Inflow Volume	Outflow per Pump			Remaining in Sump		
(min)	(m^3/s)	(m^3)	(m^3)	Pump #1 (m^3)	Pump #2 (m^3)	Pump #3 (m^3)	Pump #1 (m^3)	Pump #2 (m^3)	Pump #3 (m^3)
0									
1	0.163	4.877	4.877	0	0	0	4.877	4.877	4.877
2	0.325	14.632	19.509	0	0	0	19.509	19.509	19.509
3	0.488	24.387	43.896	0	0	0	43.896	43.896	43.896
4	0.650	34.141	78.038	24	0	0	54.038	54.038	54.038
5	0.813	43.896	121.934	48	0	0	73.934	73.934	73.934
6	0.975	53.651	175.585	72	0	0	103.585	103.585	103.585
7	1.138	63.406	238.990	96	24	0	142.990	118.990	118.990
8	1.301	73.160	312.151	120	48	0	192.151	144.151	144.151
9	1.463	82.915	395.066	144	72	0	251.066	179.066	179.066
10	1.626	92.670	487.735	168	96	24	319.735	223.735	199.735
11	1.626	97.547	585.282	192	120	48	393.282	273.282	225.282
12	1.626	97.547	682.829	216	144	72	466.829	322.829	250.829
13	1.463	92.670	775.499	240	168	96	535.499	367.499	271.499
14	1.301	82.915	858.414	264	192	120	594.414	402.414	282.414
15	1.138	73.160	931.574	288	216	144	643.574	427.574	283.574
16	0.975	63.406	994.980	312	240	168	682.980	442.980	274.980
17	0.813	53.651	1,048.631	336	264	192	713.829	449.829	257.829
18	0.651	43.946	1,093.776	360	288	216	733.776	445.776	229.776
19	0.488	34.180	1.127.956	384	312	240	743.956	431.956	191.956
20	0.326	24.415	1,152.371	408	336	264	741.371	408.371	144.371

Analysis of Initiation Times for 3-24 m^3/min Pumps

A	B	C	D	E	F	G	H	I	J
21	0.163	14.649	1,167.020	432	360	288	735.020	375.020	87.020
22	0.001	4.883	1,171.902	456	384	312	715.902	331.902	19.902
23	0.000	0.000	1,171.902	480	408	336	691.902	283.902	0.000
24	0.000	0.000	1,171.902	504	432	360	667.902	235.902	0.000

4. Analyze the proposed pump operation using the selected pump sizes and the storm duration requiring the largest average pump capacity. The tables above and the figures below illustrate a systematic method for developing the necessary information. In this example, Figure 11-5 represents a plot of the pump operation determined in the calculations of the preceding tables. The inflow volume is the area under the rate versus time curve for the increment of time being considered. The starting time of the pumps can be varied, if necessary, to keep the volume in the storage area (below maximum allowable elevation) below the storage volume determined in Step 2 above, to minimize the cycling operation of the pumps, and to provide for the most economical operation of the pumps. You can find an optimum combination by varying pump sizes and pump initiation times. It is important to offset pump initiation times by at least one minute to avoid a power overload. A computer spread sheet solution is useful.
5. Develop a stage vs. storage curve for setting cut-on/cut-off elevations. You may perform this step at any time up to this point in designing a hydraulic pump.
6. To compute the total dynamic head requirement for the pump, including losses and any safety factor; use Equation 11-3. The total dynamic head (H_{TD}) requirement is defined according to the expression in Equation 11-3. The friction loss for any geometry of discharge conduit is Equation 11-4. The velocity head for any geometry of discharge line is Equation 11-5. Losses in pump valves, fittings, bends, and transitions in the discharge conduit system are affected by specific characteristics of the system. Refer to Chapter 10 for equations for loss estimates in bends and transitions. Refer to pump system appurtenance manufacturer's literature for recommendations concerning losses in pump valves, fittings, and other appurtenances. Also, refer to *Hydraulic Design of Pumping Stations* (CDS 5, 1982) for additional guidance on estimating minor losses.

$$H_{TD} = h_s + h_f + h_v + \sum h_p + h_{sf}$$

Equation 11-3.

where:

h_{td} = total dynamic head (ft. or m)

h_s = static head (height through which the water must be raised) (ft. or m)

h_f = friction loss in the discharge line (ft. or m)

h_v = velocity head (ft. or m)

Σh_p = summation of losses due to friction in water passing through the pump valves, fittings, and other items (ft. or m)

hsf = safety factor (ft. or m) Usually 1 ft. (0.3 m) is adequate for accounting for possible silting or other unpredictable losses

$$h_f = L\left|\frac{Qn}{zAR^{2/3}}\right|^2$$

Equation 11-4.

where:

L = length of discharge line (ft. or m)

Q = discharge rate (cfs or m^3/s)

n = Manning's roughness of conduit

z = 0.4644 (0.3116 metric)

A = cross-sectional area of conduit (sq.ft. or m^2)

R = hydraulic radius of discharge conduit when running full (ft. or m)

$$h_v = \frac{v^2}{2g}$$

Equation 11-5.

where:

v = velocity of design discharge in discharge conduit operating at full flow (fps or m/s)

g = acceleration due to gravity (32.2 ft./s2 or 9.81 m/s^2)

7. The following standard power equation, describing work with respect to time, is used to determine the minimum horsepower for the pump driver and assumes an efficiency, E. Use the manufacturer's capabilities to base assumptions of efficiency. See Equation 11-6

$$P = \frac{\gamma\, Q\, H_{TD}}{550\, E}$$

Equation 11-6.

where:

P = required power (HP or kW)

γ = unit weight of water = 62.4 lbs./cu.ft. or 9.810 kN/m^3)

Q = rate of discharge to be pumped, (cfs or m^3/s)

HTD = total dynamic head as described in Step 6 above (ft. or m)

E = anticipated efficiency of motor. 550=conversion from ft.-lbs./s to horsepower, for English only. (For this example, assume 80% efficiency.)

In the English example, the discharge conduit is large enough so that the total dynamic head comprises significant values of only 18.93-ft. static head, a 1-ft. safety factor, and 2-ft. additional losses. Total TDH = 20 ft. Therefore, the power requirement for 1-13.2 cfs pump is computed as:

$$P = \frac{62.4 \times 13.2 \times 20}{550 \times 0.80} = 37.4\text{Hp}$$

Equation 11-7.

In order to assure sufficient power and conform to nominal available power ratings, use 40 Hp minimum.

In the metric example, the discharge conduit is large enough so that the total dynamic head comprises significant values of only 5.73-m static head, a 0.3-m safety factor, and 0.07-m additional losses. Total TDH = 6.1 m. Therefore, the power requirement for 1–24 m^3/min pump is computed as

$$P = \frac{9.81 \times 0.4 \times 6.1}{0.80} = 29.9\text{kW}$$

Equation 11-8.

In order to assure sufficient power and conform to nominal available power ratings, use 30 kW minimum.

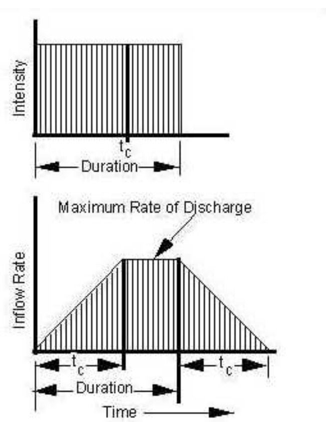

Figure 11-4. Design Rainfall and Runoff for Pump Station

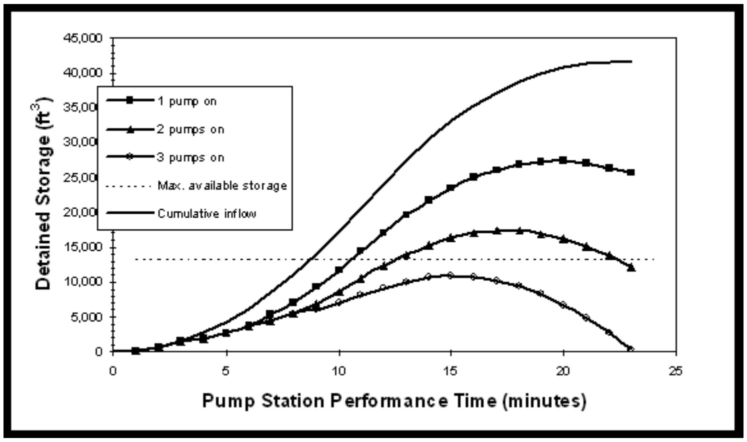

Figure 11-5. Typical Pump Station Performance (Three 7,000 gal/min Pumps)

Average Pump Capacity Requirements

A final design combination requires a thorough analysis of pump size combinations and pump initiation time schedules during the runoff event. Sites where storage is small will not offer any significant adjustment to the peak of the runoff hydrograph. When this occurs, use APC = maximum rate of discharge from Step 1. Select two or more pump sizes that will provide the total desired average pump capacity determined from the preceding tables. (A two-pump minimum is recommended practice. You could have a situation requiring only one pump.)

Pump Sizes

Determine what nominal pump sizes are available in the area of the project. The tables show that three pumps will satisfy the average pump capacity requirements. A complete hydraulic and economic analysis is necessary for any possible configuration of pumping capacity.

Chapter 12
Reservoirs

Contents:

Section 1
Introduction

Function of Reservoirs

In this chapter flood control concepts apply to retention, detention, and sedimentation basins. Storm water runoff is stored in reservoirs by either the detention of a desired portion of the runoff or by the retention of the runoff until the basin becomes dry. This chapter deals primarily with large reservoirs and their impact on highway facilities and vice versa.

Impact of Reservoirs on Highways

Reservoirs can impact highways by affecting the following:

- the natural storm runoff
- the highway alignment and/or location
- the embankment stability
- the risk of highway overtopping

Natural storm runoff affects highways that are downstream from the reservoir. The remaining three impacts affect highways that border the reservoir and cross the reservoir proper or cross the impounded stream(s) just upstream of the reservoir.

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 Natural Resource Conservation Commission (TNRCC)

See References for information on contacting these 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

Reservoirs often affect highways, and when they do, the department coordinates with the involved agency, which usually initiates contact with the department. See the Project Development Policy Manual for TxDOT policy on Highway Adjustments for Reservoir Construction.

When an agency makes contact, the department analyzes the proposal and evaluates all apparent impacts to the highway facility. Adverse impacts to the highway include relocation, revision of the highway profile, embankment protection, and adjustment of structures.

Mitigation of impacts resulting from construction of a reservoir is the responsibility of the reservoir agency. The two regulatory agencies most involved with streams are the TNRCC and the Federal Emergency Management Agency (FEMA). See References for information on contacting these agencies.

Although large reservoir sponsors usually present comprehensive design packages, 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.

Private ventures have sponsored reservoirs in Texas, but the department generally does not deal directly with private projects because it has no consistent machinery 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).

Section 3

Reservoir Design Factors

Hydrology Methods

The primary hydraulic factors involved in the design of a reservoir include hydrology methods, flood storage potential, and reservoir discharge facilities. Several different methods are available for predicting runoff rates. Some of the more productive methods are described in Chapter 5; however, you may use more sophisticated hydrologic methods. For department consideration, the peak runoff rate for the drainage area served by a reservoir should be associated with a flood event having a minimum recurrence interval of 50 years (Q_{50}). For department consideration, determine the magnitude of the 50-year event by procedures provided in Chapter 5, specifically the following procedures:

- NRCS Runoff Curve Number Methods
- Design Rainfall Hyetograph Methods
- Flood Hydrograph Routing Methods

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 the requirement for flood water storage. Provision of sediment storage space 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, consider only the storage provided for flood water.

Check the adequacy of the proposed storage by routing the hydrograph with the peak flow through the proposed reservoir. Consider the following:

- ordinate/time association of the flood hydrograph
- available reservoir storage
- capacity of the reservoir outlet works

Through a routing process, consider the factors of the hydrograph, storage, and outlet relations simultaneously. Several flood routing techniques are useful for department analysis. Chapter 5 discusses the Storage Indication Routing Method, the most prominent and productive of these techniques.

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. The operational releases can exist for a long period of time and can even threaten the highway with sustained inundation. For this reason, carefully evaluate the design notes and operating schedules.

Section 4

Reservoirs Upstream of Highway

Peak Discharge

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 documented as completely with design notes. Therefore, the department's 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.

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.

Water flowing out of the reservoir can be deprived of sediment. Whether or not a reservoir has been designed with storage capacity for sediment, significant sediment deposition usually occurs within the reservoir wherever sediment supply rates are appreciable. The sediment concentration in the water released from the reservoir is likely much lower than that entering the reservoir. A possible effect of sediment deprivation downstream is an increase in the potential for stream erosion due to a deficit between the sediment carrying capacity and the actual sediment concentration of the released flow. Increased stream degradation can jeopardize the integrity of the foundations of downstream highway structures.

Design Adequacy

The department 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, make no allowances for the reservoir when designing the highway facility. Ignore the existence of the reservoir and do not expect consistent flood attenuation.

Future Liability

The period of future liability encompasses the years following initial construction of the reservoir. The potential for incurring liability arises from the adverse effects on property owners either upstream or downstream of the reservoir. Occurrence of an event that is greater than the design flood or dam failure can create potential for liability if the reservoir is not properly maintained. However, the department does not sponsor reservoirs upstream from the highway and, therefore, does not accept any liability. The department functions as another property owner adversely affected by failure of the reservoir system.

Section 5

Criteria for 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 you must cross a reservoir, set the highway profile high enough to reduce the risk of overtopping, and stabilize the embankment 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.

Adjustments to Existing Highways

When a proposed reservoir is expected to impound floodwater on an existing highway location, adjust the highway to meet the same conditions of structure size and embankment elevation and protection that apply to new locations. Also, upgrade the roadway to meet current geometric design standards. All adjustments to the highway are usually the responsibility of the reservoir sponsor. (See Policy on Highway Adjustments for Reservoir Construction in the Project Development Policy Manual for more information.) Reservoirs that fall into this category are often major facilities, and the reservoir designs are usually well documented and available for the department's use in its analysis.

Minimum Top Establishment

Measure the roadway embankment elevation at the point of low shoulder (crown line), as shown in Figure 12-1.

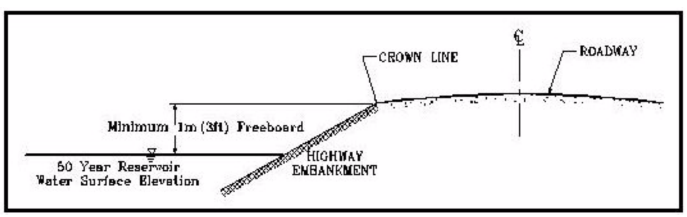

Figure 12-1. Reservoir Freeboard Requirement

As a general criterion for establishing minimum top of embankment elevation, set the top of embankment no lower than the elevation created by the higher of the following conditions:

- the 50-year (frequency) reservoir surface elevation for the entire reservoir, plus a minimum freeboard of 3 ft. (1 m).
- the elevation of the 50-year flood backwater curve as depicted in Figure 12-2, plus a minimum 3 ft. (1 m) of freeboard to the low chord elevation of any structure

- the elevation of the 500-year flood backwater for interstate highways and evacuation routes

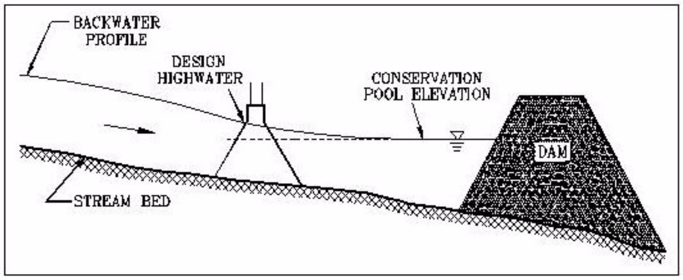

Figure 12-2. 50-Year Flood Backwater Profile

Basis for Minimum Embankment Elevation

Base the 50-year reservoir surface elevation on the entire watershed contributing runoff at the dam site. Request the 50-year reservoir surface elevation from the reservoir engineers. If the information is unavailable, calculate the level based upon an inflow hydrograph having a peak rate of inflow equal to the 50-year discharge and a storage routing in accordance with Chapter 5. For analysis, assume that the reservoir level is at conservation pool elevation when the 50-year flood begins.

Structure Location

Locate the structure or structures in accordance with the stream crossing design process and guidelines outlined in Chapter 9. These procedures generally base the location of the structures on flow and velocity distributions across the channel section. In some cases, you may need additional openings in the highway embankment near the borders of the reservoir to ensure reservoir circulation.

For the completed reservoir conditions, the minimum structure length should accommodate a 50-year design discharge. Base the associated 50-year design flood surface elevation at the crossing on a calculated backwater curve that begins at the reservoir conservation pool elevation (see Figure 12-2). Base the velocity through the openings on the waterway area below the 50-year flood surface elevation (without wind effects). Determine the 50-year design discharge, based on the drainage area above the highway site, in accordance with department procedures described in Chapter 7, specifically the (Standard) Step Backwater Method, and determine the 50-year flood water level in accordance with the department procedures described in Channel Analysis Methods, Chapter 7.

Measure the structure height, or freeboard, from the 50-year water surface to the lowest point of the superstructure. The low chord of the structure should be a minimum of 3 ft. (1 m) above the highest of one of the same conditions used to establish the basis for minimum embankment elevation (see the indicated subsection above for more information). Adjust the roadway embankment approach grades to the structure so that there is at least 3 ft. (1 m) of freeboard for the structure. Check the possible need for freeboard in excess of 3 ft. (1 m) to accommodate anticipated recreational use or other uses.

Embankment Protection

Embankment protection is required from the toe of the highway embankment up to an elevation equal to the sum of conservation pool elevation, 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 ft. (1 m) 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 vegetal cover with a strong root system is adequate and very economical.

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 sponsors can help decide which to use for a given situation.

Rock Riprap

Consider the following elements of rock riprap:

- 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 is 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. This 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.
- rock riprap design – 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:

where:

Wa = weight of the median sized stone (lbs. or kN)

W_{max} = weight of the maximum sized stone (lbs. or kN)

W_{min} = weight of the minimum sized stone (lbs. or kN)

γ = unit weight of stone = 62.4G lbs./cu.ft. ($9.81G\ kN/m^3$)

T = thickness of the riprap layer (in. or mm)

H_s = significant wave height (ft. or m)

α = slope angle from the horizontal in degrees

G = specific gravity of the stone material

g = gravitational acceleration (32.2 ft/s2 or 9.81 m/s^2)

NOTE: Determine masses in kg by multiplying weights in kN by 1000 and dividing by g.

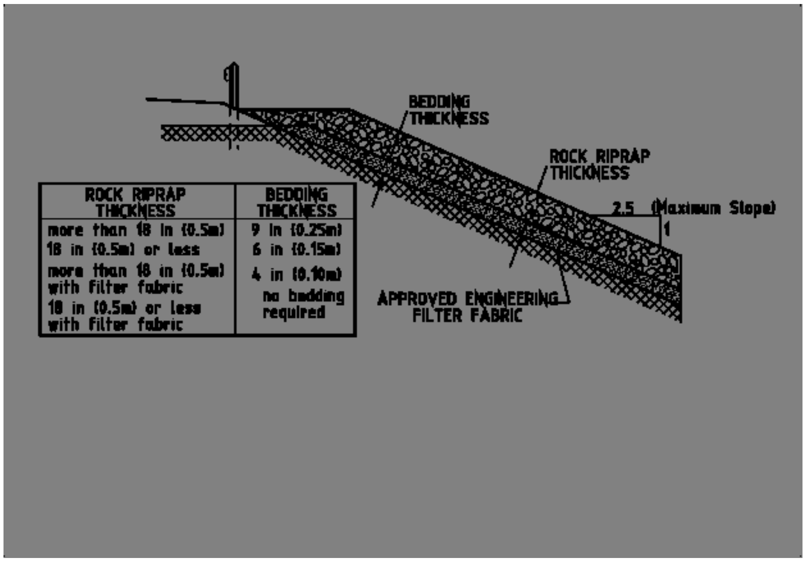

Figure 12-3. Rock Riprap Specifications

Soil-Cement Riprap

Soil-cement riprap consists of layers of soil cement on the slope placed in prescribed lifts. Figure 12-4 shows completed soil-cement slope protection. 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.

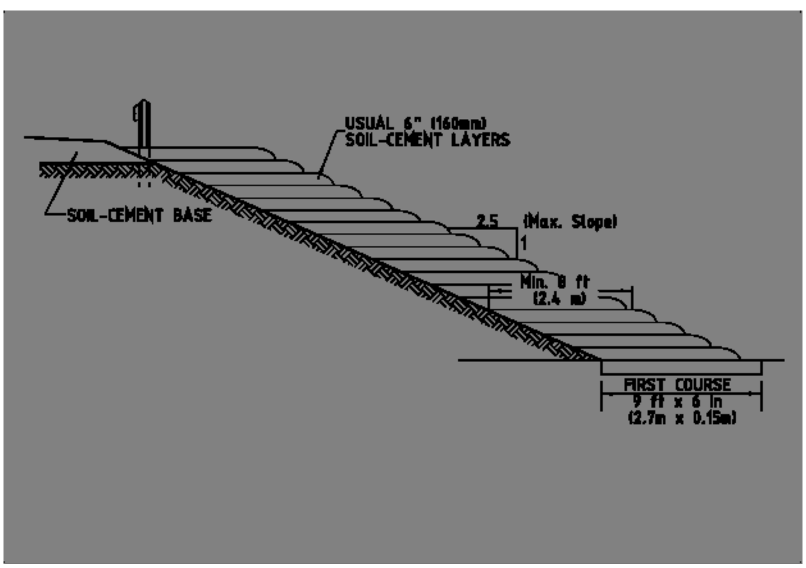

Figure 12-4. Soil Cement Riprap Specifications

Articulated Riprap

This type of riprap is usually fabricated so that the individual elements are keyed together, and 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—can move as a unit with the slope and still remain intact. There are several commercial sources of articulated riprap. Consider each for price, performance, and experience.

Concrete Riprap

Concrete usually consists of slope paving of 4 to 6 in. (100 to 150 mm) 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 bulges and falls 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. This type of 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

Urbanization, which includes transportation activities, generally reduces surface perviousness and loosens soils. The results can increase flooding, soil erosion, sedimentation, stream bank erosion and channel enlargement, and pollution of surface and subsurface waters.

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 runoff quantity and quality problems are termed “best management practices” (BMPs). Flooding and water quality development, urban development, and pollution from highways benefit from BMPs.

Recognition is growing that providing for rapid disposal of rainfall runoff from developing areas has increased the frequency of flooding in downstream areas. Furthermore, water quality problems in surface waters often stem from nonpoint as well as point sources of pollution. Water quality goals for surface waters cannot be achieved by separation of combined sewers or tertiary treatment of sewage but require abatement of pollution from nonpoint sources as well.

Where existing developed areas are downstream of more recent development, as is the predominant sequence of development in the United States, massive investments in flood control works or storm sewer outfalls from developing areas are sometimes required to reduce flood damage. Where flood control is not feasible, flooding reduces property values and may lead to abandonment of property. The alternative to downstream flood control works or the abandonment of flood hazard areas 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, you may need additional or alternative storm water management measures to provide source control of storm water pollution.

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 you can make sound judgments as to the overall usefulness of the facility. (For more information on EIS, refer to the *Environmental Procedures in Project Development Manual*.)

Requirements for Construction Activities

The TxDOT publication *Storm Water Management Guidelines for Construction Activities* (TxDOT, 1993) details the department's procedures and recommended BMPs to be included in a Storm water Pollution Prevention Plan (SW3P) for proposed projects. Though the U.S. Environmental Protection Agency (EPA) National Pollution Discharge Elimination System (NPDES) permit requirements currently require SW3Ps for projects disturbing in excess of 5 acres (2 ha), you should utilize erosion control specifications. Also, we recommend appropriate BMPs for all construction projects.

Storm Drain Systems Requirements

The National Pollution Discharge Elimination System 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. Refer to the Division of Environmental Affairs 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 all of its sediment load.

Schematically, Figure 13-1 illustrates the typical forces involved in soil erosion.

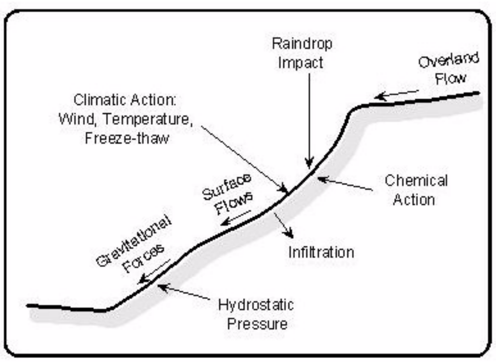

Figure 13-1. Typical Forces in Soil Erosion

Soil erosion is either natural or accelerated:

- Natural erosion is a geological process over which humans have little or no control. Natural erosion may range from extremely slow to rapid, depending on various factors. For example, where humans have disturbed land by construction, there may be a sudden, rapid increase in the rate of erosion, thus producing accelerated erosion.
- Accelerated erosion is the type of erosion that should be controlled during highway construction and after the highway is completed.

It is usually not practical for the department to reduce erosion generated upstream of the highway. If possible, avoid locations with high erosion potential. In areas of considerable natural erosion and accelerated erosion, document the quantity of sediment that reaches a stream before highway construction begins 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. 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. Erosion and sedimentation are usually undesirable from an environmental standpoint. They can also be detrimental to the roadway by causing significant maintenance problems.

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. Use this technology. 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 Bridge Division's Hydraulics Branch for detailed information.

To deal adequately with the erosion and sediment problem, you must understand erosion and sedimentation processes, develop erosion and sediment control plans, schedule construction operations for erosion and sediment control, construct specific erosion and sediment control measures (when, where, and how), and monitor and maintain water quality.

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

Examine the natural drainage pattern, including subsurface flow, for the alternate routes considered. You must also study the drainage pattern beyond the vicinity of the proposed highway location 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, you must examine established patterns of drainage (as contrasted to natural patterns).

Stream Crossings

Whenever practical, make stream crossings at stable reaches of a stream. Avoid meanders in the stream that are subject to shifting. 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. See Chapter 9 for more details on planning and location.

Make crossings as nearly as practical at a right angle to the direction of flow. Give emphasis to the direction of the flood flow where it is different from that of the low water. Try to minimize the number of stream crossings and the disturbance of streambeds. Avoid crossing and then re-crossing the same stream.

Always consider the direction, rate, and volume of flood flow at various stages in the location of bridge openings. Try to avoid undue scour and erosion that might result in a complete change in the river channel. Meandering streams have inherent problems of no good places to cross because the sinusoidal pattern of the stream naturally tends to progress in a downstream direction.

Encroachments on Streams

If a possible highway alignment will encroach upon a stream, consider moving the highway away from the stream to avoid erosion and sedimentation problems. Make channel changes to avoid encroachments or for any other reasons cautiously and with the Bridge Division, Hydraulics Branch.

For an existing roadway that already encroaches on or near a stream, plan improvements or rehabilitation work to minimize further encroachment. If the stream impinges and encroaches on the highway, you may need to protect the highway itself.

Public and Industrial Water Supplies and Watershed Areas

If possible, avoid the crossing of a catchment area of a water supply. Such crossings could entail building costly temporary facilities for the water supply. Problems with industrial water supplies may be as great as those with a public water supply. Some industries require higher quality water than is required for drinking water. When you cannot avoid crossing a water supply catchment area, determine any corrective measures and their costs before making the choice of the route.

Geology and Soils

Ground conditions encountered in the field directly result from geologic processes operating on and within the earth. Knowledge of the area's geology allows the highway designer to detect potential problem areas and anticipate subsidence, landslides, and erosion problems.

You can sometimes avoid areas and problems in route selection for a new roadway. For an existing roadway, however, recognize problems and take precautions in the design.

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. (See References for information on contacting these agencies.) You can often apply research on a particular soil type to soils of the same type in other locations. Local NRCS offices can give much assistance in both soil identification and erosion control measures applicable to the local area.

Coordination with Other Agencies

Contact local offices of the USACE, NRCS, and other agencies, such as the TNRCC. Their plans or projects might affect or be affected by the location of a proposed highway, or by improvements or changes to an existing roadway. Contact these agencies so you can also learn of their projects for controlling bank erosion, their plans for protective works, and their 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. Unfortunately, this is in direct contradiction to the usual aim of the geometric designer. The traveling public favors smooth, non-rolling profiles. You must sometimes make compromises to satisfy both demands. Depressed roadways and underpasses require careful consideration of drainage design to avoid deposition of sediment and debris on the highway and in drainage facilities. Blend or fit alignment and grade, consistent with highway safety criteria, to the natural landscape to minimize cut and fill sections and reduce erosion and costly maintenance. 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

Make slopes of the roadway cross section as flat as possible and consistent with soil stability, climatic exposure, geology, proposed landscape treatment, and maintenance procedures.

Vary the cross section, 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. Avoid the concentration of storm water at the top of cuts. Follow these guidelines 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. You must protect outlets for such high velocity chutes from scour. Occasionally, you cannot avoid streams in cut sections; they require special attention.
- serrated slopes – In some areas of Texas, serrated cut slopes help establish vegetative cover on decomposed rock or shale slopes. You can serrate any material that is rippable or that will hold a vertical face for a few weeks until vegetation becomes established.
- shoulder drains -- Where you cannot establish vegetation 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 you cannot avoid concentration of flows. 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. You can protect such channels 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. Dissipate the energy along the chute or at the outlet is usually necessary. In highly erosive soil, you may need to provide watertight joints to prevent failure of the facility.

Make variations in channel alignment gradual, particularly if the channel carries flow at high velocity. Whenever practical, make changes in alignment on the flatter gradients to prevent erosion caused by the overtopping of the channel walls. Although usually more expensive, rectangular channel sections are preferred on curves of paved channels to give a more positive control of the flow.

Line channels if the bank and bed material will erode at the prevailing velocities. Protective linings for channels and streams can be very expensive. Make a special effort to develop the most cost-effective erosion protection, including maintenance, for the particular location. See Chapter 7 for more information.

Several applications are effective for both channel and bank protection during the design phase of a project, 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. Refer to "Design of Riprap Revetment" (HEC 11, 1989) and "Design of Roadside Channels with Flexible Linings" (HEC 15, 1988) for detailed guidance.

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.

You must exercise special care to avoid creating safety hazards and to prevent expensive maintenance.

Dissipate the energy of the high velocity flow at the outlet of culverts and chutes where necessary, or protect the area subject to scour by riprap or other types of protection. Some velocity control devices are illustrated in Chapter 8. The HEC-14 (1983) illustrates other fixtures and energy dissipaters along with techniques for rigid design.

Section 3

Inspection and Maintenance of Erosion Control Measures

Inspections

Preventive maintenance built into the highway in the location 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. Conduct periodic inspections of drainage and erosion control measures shortly after completion of construction so that you can locate and correct deficiencies before they develop into major problems. Discuss deficiencies in design or in construction procedures with the engineering staff to avoid similar deficiencies on future projects. We encourage you to coordinate responsibilities for erosion control measures among design construction and maintenance sections by establishing and maintaining a continuing and clear communication system between these entities.

Embankments and Cut Slopes

Embankments and cut slopes are especially vulnerable to erosion. Make maintenance equipment operators aware that damage to ground cover at such locations can create serious erosion problems that are difficult to correct. Emphasize surveillance of these areas by maintenance personnel because such areas are not easily seen from the roadway.

Channels

Channels, whether active streams or open roadside ditches, are vulnerable to erosion, especially for a period of time after construction. Periodically and after significant storms, maintenance personnel should inspect these facilities for any erosion that will require remedial work.

Keep intercepting channels clean and free of brush, trees, tall weeds, and other material that lowers the capacity of the channel. When channel deterioration reduces channel capacity, overflow may occur frequently. 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. Take care to inspect for holes and eroded areas under paved channels to prevent collapse of rigid sections. Remove or repair projections and joint offsets that cause splash and possible erosion. 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. Carefully analyze failures during construction before performing remedial work because changes in the original construction may be indicated.

Repair to Storm Damage

Repair storm damage as quickly as possible in order to avoid additional damage. Such damage may indicate that additional protection is needed. A damaged area only restored 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, advise those responsible so that they can take proper actions to protect the structure. Try to establish and maintain a system of record keeping and documentation regarding erosion/scour problems and flood events respective to highway facilities.

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.

Chapter 10 details storm drain system planning and design considerations. Chapter 11 gives pumping stations design and operation considerations. The hydrologic methods for analysis of detention and retention systems are detailed in Flood Hydrograph Routing Methods, Chapter 5. Outfall channel design and operation considerations and procedures are detailed in Channel Analysis Methods, Chapter 7.

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.

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. The department rarely is involved in non-structural measures in association with transportation projects. The table below lists some of the measures for reducing and delaying urban storm runoff recommended by the Natural Resources Conservation Service.

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

Figure 13-2 illustrates how storage facilities, such as detention basins, can be used for flood control. The inflow hydrograph is developed according to one of the procedures described in Flood Hydrograph Routing Methods, Chapter 5. The outflow hydrograph is also specified so that the peak discharge is below the maximum flow permitted. The shaded area represents the storage volume required to produce the specified outflow from the given inflow hydrograph.

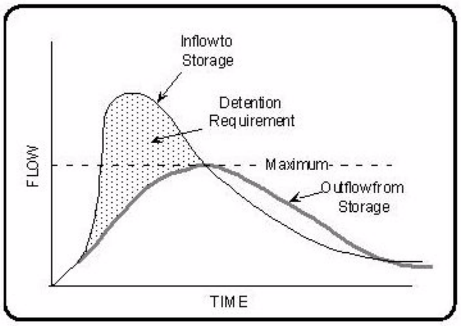

Figure 13-2. Attenuation of Peak Flow

Of the measures listed in "Measures for Reducing and Delaying Urban Storm Runoff" table, 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 5 for details of hydrograph routing by the Storage Indication Routing Procedure. 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
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
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.

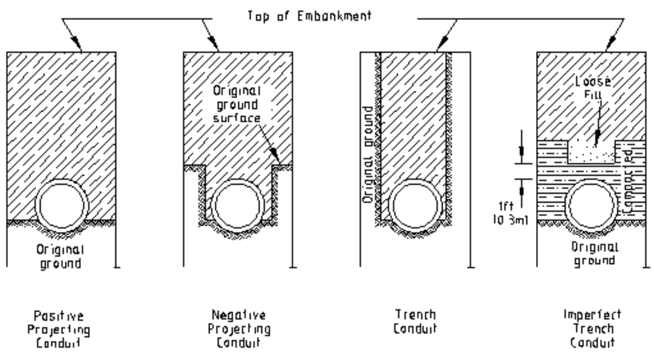

Figure 14-1. Pipe Installation Conditions

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.

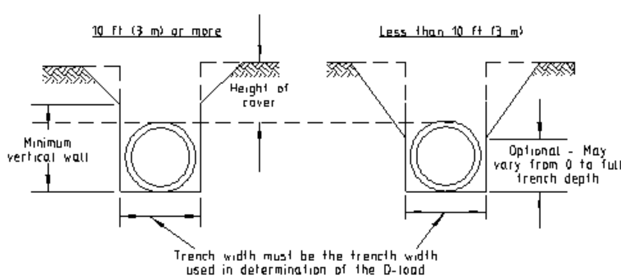

Figure 14-2. Permissible Trench Shapes

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.

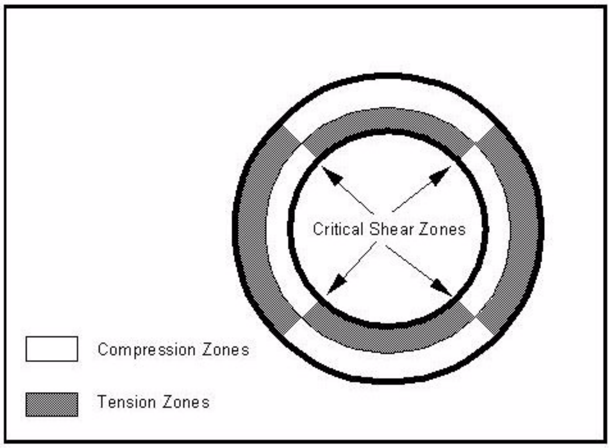

Figure 14-3. Critical Shear Stress Zones for Rigid Pipe

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.

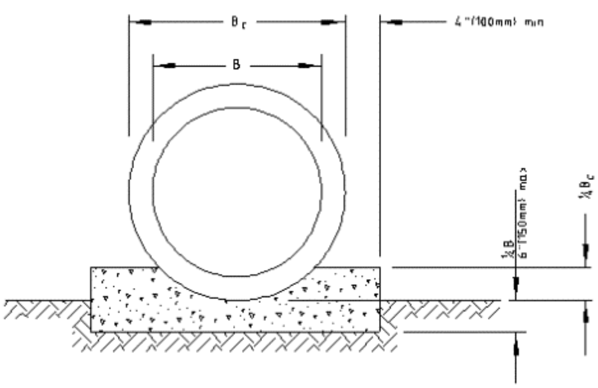

Figure 14-4. Class A Bedding

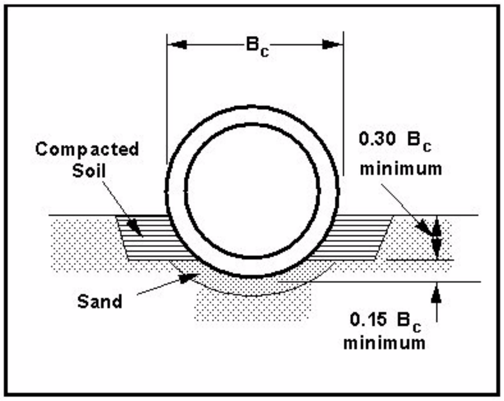

Figure 14-5. Class B Bedding

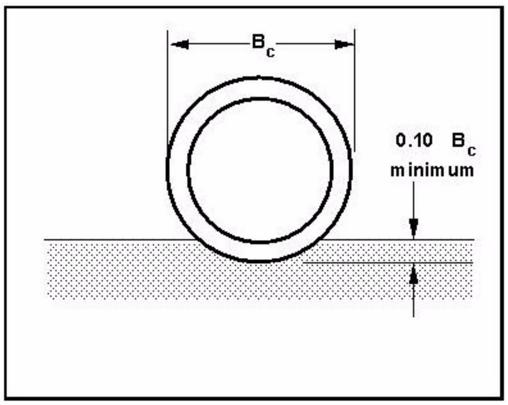

Figure 14-6. Class C Bedding

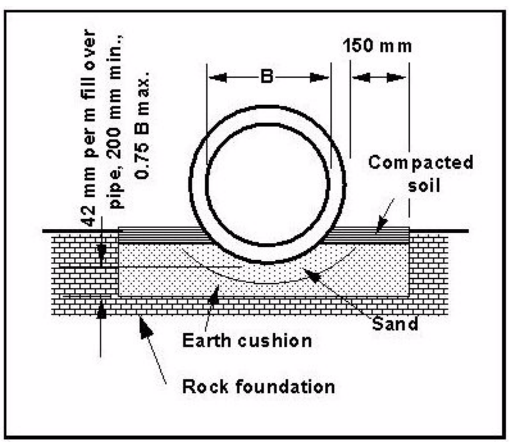

Figure 14-7. Class C Bedding on Rock Foundation

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.