Chapter 7 – HYDROLOGY AND HYDRAULICS

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CHAPTER 7

HYDROLOGY AND HYDRAULICS

7.1 GENERAL

This chapter identifies the hydrologic and hydraulic related policies, standards, standard practices, criteria, guidance, and references approved for use in developing highway and bridge designs in the Federal Lands Highway Programs. Refer to Chapter 1 for definitions of policy, standards, standard practices, criteria, and guidance. Where appropriate, relevant procedures, instructional aids, and publications such as engineering manuals, AASHTO guidelines, federal regulations, and computer programs are referenced. Detailed descriptions and examples of technical methods or procedures are not included. Users of this chapter are expected to be knowledgeable in the use of all referenced methods and procedures, and otherwise stay informed of current, related technologies.

The chapter is organized by topics within broad categories of related work. Policies, standard practices, standards, criteria, and guidance are condensed and addressed separately for the user under each topic. In addition, a quick reference guide that summarizes standards and criteria by topic is provided in Exhibit 7.1–A. Compliance with all policies and standards in this manual is essential to ensure consistency in project development throughout Federal Lands Highways projects. Although policy cannot be compromised, flexibility of standards is sometimes necessary to meet project-specific objectives. (See Section 7.1.9 for exceptions and variances to standards.)

As changes in policies, standards, or criteria occur, updates to this chapter will be made as described in Section 1.1.5

The information presented in this section will be applied as Standard Practices to any and all hydraulic work executed to develop and deliver projects of the Federal Lands Highway Programs.

Refer to [EFLHD – CFLHD – WFLHD] Division Supplements for more information.

7.1.1 QUICK REFERENCE GUIDE

Exhibit 7.1–A provides a quick reference guide for the standards, criteria, and recommended methods provided in this chapter. Wherever possible, numerical standards and criteria are listed. Links are provided to applicable sections in this chapter and to recommended methods outside the PDDM. See Section 7.1.6.1 for the definition of high- and low-standard roadways.

Exhibit 7.1–A QUICK REFERENCE GUIDE

Topic	Standard	Criteria	Method Reference
HYDROLOGY			
Peak Flow Methods			HDS 2, HEC 22, NEH Part 630, TR-55, WRI 024168, Bulletin 17B
Hydrograph Methods			HDS 2, WinTR-55
ROADWAY HYDRAULICS			
Culverts	Capacity Design and Stability Design: High-Standard road: 50-year flood Low-Standard road: 25-year flood Roadside ditch: 10-year flood Capacity Check Flood: Evaluate potential for adverse impacts for the overtopping flood Exception: See Floodplain Encroachments	Headwater New: WSEL ≤ bottom of aggregate base layer Existing: WSEL ≤ shoulder hinge point. HW/D ratio: 1200 mm [48”] or smaller = 1.5 Larger than 1200 mm [48”] = 1.2 Other: WSEL limited by unacceptable hazards to human life or property. Minimum Size: Cross-road culvert = 600 mm [24”] Parallel culvert = 450 mm [18”] Slope: Stream Crossings: Match streambed Ditch Relief: Min. = 2%, Max. = 10% Cover Pipe Anchors: Concrete > 10% slope, Metal > 25% slope	HDS 5, HEC 14

Exhibit 7.1–A QUICK REFERENCE GUIDE *(Continued)*

Topic	Standard	Criteria	Method Reference
Ditches	Capacity Design: 10-year flood Stability Design: Permanent Linings 10-year flood Temporary Linings: 2-year flood	Depth: New: WSEL ≤ bottom of aggregate base layer Existing: WSEL ≤ shoulder hinge point Slope: Min. = 0.5% Stability: Permissible shear stress	HDS 3, HEC 15
Pavement Drainage	Capacity Design: 10-year flood, 50-year in sumps	Spread: High-Standard road: 900 mm [3’] into one travel lane, Low-Standard road: Half of one travel lane Depth: On-grade and Sags: Allowable spread, not to exceed curb height, Sumps and Parking Areas: 150 mm [6”]. Inlet Clogging Factor: Grate Inlets in sag or sump, 50%	HEC 21, HEC 22
Storm Drains	Capacity Design: 10-year flood, 50-year in sumps	Minimum Size: 375 mm [15”]. Minimum Slope: Pipe-full velocity ≥ 0.9 m/s [3 ft/sec]	HEC 22
Energy Dissipators	Design Standard: Range of discharges	Design Guidance: Natural or stable channel velocity	HEC 14
Alternative Pipe Materials	Service Life: 50-years Minimum Pipe Classification: RCP: Class II Metal: 1.63 mm [0.064”]		FHWA-RD-97-140, Caltrans Chapter 850

Exhibit 7.1–A QUICK REFERENCE GUIDE *(Continued)*

Topic	Standard	Criteria	Method Reference
RIVER HYDRAULICS			
Floodplain Encroachment	Design Flood: 100-year Check Flood: Overtopping flood, not to exceed 500-year	FEMA Regulated Base Floodplain with Detailed Study: With floodway defined, no floodway encroachment With no defined floodway or no detailed study, rise ≤ 0.3 m [1.0’] Unregulated Base Floodplain: Rise ≤ 0.3 m [1.0’]	HEC-RAS
Scour and Stream Stability			HDS 6, HEC 18, HEC 20,HEC 23
Bridged Waterways	Capacity Design: Design Flood: 50-year Check Flood: Greater of overtopping flood or 100-year, not to exceed 500-year Stability Design: Design Flood: 100-year Check Flood: 500-year	Freeboard: 0.6 m [2.0’], greater where potential for debris or ice Stability Design: Design Flood: Normal geotechnical and structural safety factors Check Flood: Safety Factor ≥ 1.0	HEC-RAS, HEC 18, HEC 20, HEC 23
Longitudinal Embankments	Capacity Design: High-Standard road: 50-year flood Low-Standard road: 25-year flood Check Flood: Greater of overtopping or 100-year Stability Design: High-Standard road: 50-year flood Low-Standard road: 25-year flood	Capacity Design: Freeboard: 0.6 m [2.0’]	HEC 14, HEC 23

Exhibit 7.1–A QUICK REFERENCE GUIDE *(Continued)*

Topic	Standard	Criteria	Method Reference
Retaining Walls	Longitudinal Flow Scour: Wall height > 2 m [6.5']: 100-year Wall height ≤ 2 m [6.5'] on High-Standard road: 50-year Wall height ≤ 2 m [6.5'] on Low-Standard road: 25-year Pipe Penetrations: High-Standard road: 50-year Low-Standard road: 25-year	Stability Design: Normal geotechnical and structural safety factors	HEC 14, HEC 23
Low-Water Crossings	Allowable Uses: ADT ≤ 200 or existing feature Capacity Design: Vented: 10-year Stability Design: 25-year flood	Capacity Design: Vented: No overtopping Stability Design	Low Volume Roads Engineering, HDS 5, HEC 20, HEC 23
Channel Changes	Capacity Design: Duplicate existing stream characteristics Stability Design: High-Standard road: 50-year Low-Standard road: 25-year	Capacity Design Stability Design	HDS 6, HEC 20, HEC 23
Scour and Stream Instability Counter-measures			HDS 6, HEC 11, HEC 14, HEC 23

Exhibit 7.1–A QUICK REFERENCE GUIDE *(Continued)*

Topic	Standard	Criteria	Method Reference
COASTAL HYDRAULICS			
General			HEC 25
Hydrology			HEC 25, EM 1110-2-1100
Scour and Stream Stability			HDS 6, HEC 18, HEC 20, HEC 23, HEC 25
Bridged Waterways	Capacity Design: 50-year storm tide plus wave height Stability Design: Design Flood: 100-year Check Flood: 500-year	Design Criteria Same as riverine except freeboard measurement reference datum	HDS 6, HEC 11, HEC 23, HEC 25
Roadway Embankments	Capacity Design: High-Standard road: 50-year storm tide plus wave height Low-Standard road: Highest astronomic tide plus 25-year wave height Stability Design: High-Standard road: 50-year storm tide plus wave height Low-Standard road: 25-year wave	Capacity Design: High Standard road Freeboard: 0.6 m [2.0’] Stability Design	HEC 14, HEC 23
Scour and Stream Instability Counter-measures			HDS 6, HEC 11, HEC 14, HEC 23

7.1.2 PROJECT MANAGEMENT AND COORDINATION

The identification and definition of project development activities needed to deliver Federal Lands Highway projects is typically achieved through an interdisciplinary team approach, led by a project manager. Consequently, to ensure consistency and effectiveness, it is essential that hydraulic related work be planned and executed in close coordination with the project manager and the other technical disciplines involved in the project (e.g., environment, roadway design, bridge design, etc.). Coordination may include the establishment of design standards and criteria different from those contained in this chapter. Such coordination may require direct contact with the partner agencies or other stakeholders.

7.1.3 RECONNAISSANCE AND SCOPING

Project reconnaissance and scoping is a combination of conducting field inspections and gathering existing engineering data needed to identify and quantify a highway's deficiencies and needs. The information is then assessed to identify a course of action for investigating improvement alternatives and conducting necessary engineering analyses that will ultimately result in a preferred alternative. Within Federal Lands Highways, these activities are collectively referred to as a Project Scoping Study as described in Section 4.5.1.

The project scoping study initially identifies the major needs, issues, constraints, scope, and feasibility of proposed improvements from which the more comprehensive, interdisciplinary preliminary engineering activities, surveys, investigations, environmental studies, and analysis can be effectively planned and budgeted. This includes the major elements of hydrologic and hydraulic work necessary to develop the project. The results of the study are summarized and documented in a Project Scoping Report as described in Section 4.5.2 and Section 4.5.2.12.9.

The following list includes broad categories of information that would be expected to be sought, collected, and used, as a standard practice for the reconnaissance and scoping, whenever available and applicable.

- Previous Hydrology/Hydraulic Studies and Reports
- Hydrological Data (rainfall, gage data, flood history, etc.)
- Aerial/Site Photography
- Survey and Mapping
- Land use, Ground cover, Soils information
- Fluvial Geomorphic data (plan forms, bed and bank sediment characteristics, etc.)
- As-Built Plans
- Bridge Inspection Reports
- Maintenance Reports

7.1.3.1 New vs. Rehabilitated Structures

The type of work proposed for drainage structures will affect the level of hydrologic and hydraulic analysis and the applicability of the standards and criteria presented in this chapter.

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

- A culvert that is to be extended to accommodate roadway widening
- A culvert needing repair due to heavy corrosion
- A bridge deck to be reconstructed or widened
- A cross drainage structure beneath a road that is to be reconstructed
- A structure being retrofitted for fish passage
- Pavement drainage improvements

Include an appropriate assessment of the existing physical condition and the hydraulic performance of all cross-drainage structures in the scoping and reconnaissance efforts. The findings of the assessment will lead to recommendations as to whether existing structures are to be replaced, rehabilitated, modified, abandoned, or left undisturbed.

7.1.3.2 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. AASHTO HDG Vol. I — AASHTO Highway Drainage Guidelines, *Volume I – Hydraulic Considerations in Highway Planning and Location*

2. AASHTO HDG Vol. VIII — AASHTO Highway Drainage Guidelines, *Volume VIII – Hydraulic Aspects in Restoration and Upgrading of Highways*

7.1.4 RISK CONSIDERATIONS

This chapter presents policy, standards, criteria, and guidance for general application on projects undertaken by the Federal Lands Highway Divisions. These standards and criteria represent the minimum for most projects. Consequently, conformance with these standards and criteria may not ensure that all risks have been fully addressed. A project can be fully compliant with the policy, standards, and criteria described within this chapter, yet still incur an inappropriate level of risk. Consequently, all sources of potential risk will be considered as part of the hydrology/hydraulic investigation for all hydraulic structures on all projects in order to determine whether modified site-specific standards or criteria are appropriate. The consideration of risk will typically begin with the evaluation of an applicable check flood, as defined in Section 7.1.7.

For the purposes of this chapter, risk is defined as the consequences associated with the probability of flooding attributable to the project, including the potential for property loss and hazard to life during the service life of the highway. If the consideration of risks appears to warrant design standards or criteria other than those outlined in this chapter, a risk assessment will be conducted. As described below, the assessment of risk can either be qualitative or quantitative in nature. If the results of the assessment confirm that lower standards are warranted, the assessment will be documented through the design exception process (see Section 7.1.9) and coordinated with project management.

7.1.4.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. 23 CFR 650A — Code of Federal Regulations, Title 23, Part 650 Subpart A, *Location and Hydraulic Design of Encroachments on Flood Plains*
2. NS 23 — Federal-Aid Policy Guide, *Non-regulatory Supplements to Title 23*
3. HEC 17 — FHWA HEC 17, *Design of Encroachments on Flood Plains Using Risk Analysis*, 1981.

7.1.4.2 Qualitative Risk Assessment

When necessary, most projects will require only a qualitative risk assessment. A qualitative risk assessment may determine that the standards and criteria of this chapter are appropriate or inappropriate based on such considerations as the presence or absence of upstream structures that could be impacted by the project, the perceived economic impact of temporary road closures, the environmental impact, or the cost of the roadway facility itself.

7.1.4.3 Quantitative Risk Analysis

Highly complex or expensive projects or those with particularly high levels of risk may justify detailed and quantitative risk analyses. A quantitative risk analysis provides a detailed economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least total expected cost to the public. This type of analysis supports the appropriate design discharge and criteria based on the economic comparison of alternatives rather than a set of predetermined design frequencies and criteria such as those presented in this chapter. Federal Lands Highway projects will rarely require quantitative risk analyses.

7.1.5 BASELINE VS. PROPOSED CONDITIONS

The hydrologic and hydraulic analysis will include a comparison of proposed conditions (post-project) to baseline conditions when the project includes one or more of the following:

- An encroachment onto a floodplain designated by FEMA
- A structure that is defined as a bridge (total span greater than or equal to 6.1 m [20.0']

Projects that do not include either item listed above may require a comparison of proposed conditions to baseline conditions based on site-specific risk.

Baseline conditions may represent either existing, pre-project conditions, or some pre-existing state, depending on project and partner agency requirements. Comparing the post-project conditions to baseline conditions allows an accurate assessment and documentation of the impacts of the project and the associated risks to neighboring properties and facilities. Bases for comparison may include, but will not necessarily be limited to:

- The water-surface profile for floods of various frequencies
- The average and maximum channel velocities
- The waterway's capacity to entrain and transport sediment
- The long-term and flood-event stability of the channel in the project vicinity

The comparison between baseline and proposed conditions may refer to more than one alternative proposed condition, depending on the needs of the project.

7.1.6 DESIGN STANDARDS AND CRITERIA

7.1.6.1 Roadway Classifications

For the design of roadway hydraulic structures, the design standards and criteria will vary based on the roadway classification. There are two roadway classifications used in this chapter, defined below:

- *High-Standard Road* – A roadway will be classified as a high-standard road if any of the following conditions apply to any section of the project:

    ◊ Design speed > 70 km/hr [45 mph]
    ◊ Design Average Daily Traffic (ADT) > 1500
    ◊ Designated as a critical access road

    Examples of critical access roads are emergency evacuation routes, sole access to a community, or sole access to critical facilities, such as hospitals, power plants, water supply and wastewater treatment facilities.

- *Low-Standard Road* – All others.

7.1.6.2 New Structures

The standards and criteria presented in this chapter represent the minimum acceptable for projects involving new drainage structures or replacements of existing structures. Exceptions to standards may be justified by a qualitative risk assessment or a detailed risk analysis.

7.1.6.3 Existing and Rehabilitated Structures

The design standards and criteria of this chapter need not be considered minimum for existing structures to be retained or rehabilitated. However, where condition or performance problems are evident, existing structures will be evaluated against the standards and criteria contained in this chapter. Where problems are not evident, consider the estimated service life and future performance of the existing structure in relation to the design standards and criteria, the overall roadway facility and scope of other roadway improvements when deciding to retain, rehabilitate or replace existing structures.

The goal of a rehabilitation design should be to increase the hydraulic performance toward those standards if appropriate and cost effective. A rehabilitation design should not decrease the safety characteristics of the existing facility. As with all projects, the needs, desires, and regulations of partner agencies and local authorities must be considered when establishing project-specific standards and criteria.

7.1.7 CAPACITY VS. STABILITY DESIGN

The capacity standards relate to the ability of the structure to convey the discharge rate anticipated for the design event. Stability standards relate to the ability of the structure or facility to withstand the discharge, velocity, shear stress, and scour induced by the design event without collapsing or sustaining substantial damage. Where appropriate, the later sections of this chapter define design and check flood standards separately for the capacity of the structure and the stability of the structure.

7.1.8 DESIGN AND CHECK FLOODS

The design of a drainage system begins with the selection of an appropriate design flood frequency. The later sections of this chapter define the standards for determining the design flood for various drainage structures or features on Federal Lands Highway projects. Where appropriate, the chapter also defines check flood standards. The purpose of evaluating a check flood is to assess the potential consequences or risks associated with floods exceeding the design flood. A flood that exceeds the capacity design may cause road overtopping, for example, and extensive damage to structures in the floodplain. A flood that exceeds the stability design flood for a bridge may undermine a foundation and lead to failure of the structure.

If evaluation of the check flood indicates undue risk, then an increase of the design flood above the normal standard should be considered for that structure, or the design should incorporate other measures to reduce the level of risk. Small structures, such as small-diameter culverts, will seldom require a formal check flood evaluation. Risk potential will be quickly assessed by evaluating impacts associated with roadway or structure overtopping elevation.

7.1.9 DESIGN EXCEPTIONS/VARIANCES

Deviation from standards cited within this chapter will require formal justification and approval by project management and the facility owner. (See Section 9.1.3 for a description of the Design Exception process). Significant deviations from the criteria cited within this chapter will be justified, approved by the local Federal Lands Hydraulics Office, and documented in the project file.

7.1.10 QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and assurance procedures (QC/QA) will be incorporated and executed in all hydrology and hydraulics investigations, evaluations, and designs. Those responsible for hydrology and hydraulics activities will define the QC/QA procedures early in the project and provide signed documentation as evidence of conforming to the procedures throughout the duration of the hydrologic and hydraulics activities.

7.1.11 DOCUMENTATION AND DELIVERABLES

The type and nature of documentation and deliverables required will vary depending upon the project. The later sections of this chapter define the documentation required for each type of hydraulic element of the project. Typical hydraulic design projects will include the following submittal requirements:

- Hydraulics Reconnaissance Documentation. Summarize the following hydrologic/ hydraulic tasks:

    - ◊ Data collection
    - ◊ Needed hydrologic and hydraulic analyses
    - ◊ Definition of baseline hydraulic conditions, as required

  Also incorporate this documentation into the Project Scoping Report described in Section 4.5.2.

- Preliminary Hydraulics Documentation. Summarize the following commensurate with the potential risks and adverse impacts:

  ◊ Applicable design standards and criteria
  ◊ Alternatives considered and evaluated and the results of the evaluations
  ◊ Required risk assessment or analysis
  ◊ Preliminary design recommendations

  This documentation represents the Location Hydraulic Study required by 23 CFR 650A. Also incorporate this documentation into the Preliminary Engineering Study Report described in Section 4.10.1. Information developed during this phase of development may be incorporated into the project environmental document, as appropriate. Therefore, close coordination with the local Federal Lands Environmental Office may be required.

- Final Hydraulics Documentation. Support the final design of the selected alternative. Fully document, to a level commensurate with project complexity and risk, the following:

  ◊ Project description
  ◊ Base data and sources
  ◊ Analytical approaches, methods, and results
  ◊ Design approaches and methods
  ◊ Final design recommendations
  ◊ Supporting information

Documentation will typically include the following support information when applicable and appropriate:

- Annotated maps and aerial photographs
- Drainage area data
- Field survey data
- Field photographs
- Floodplain mapping with cross-section locations/orientation
- Manual and electronic calculations
- Flood history data
- Applicable correspondence
- Required QC/QA documentation

7.1.12 APPLICABLE LAWS

This section presents the federal laws and regulations relating to hydrology and hydraulics.

7.1.12.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. AASHTO HDG Vol. V — AASHTO Highway Drainage Guidelines, *Volume V – Legal Aspects of Highway Drainage*

2. AASHTO MDM Chap. 2 — AASHTO Model Drainage Manual, *Chapter 2 – Legal Aspects*

7.1.12.2 FHWA Policy

Certain federal regulations comprise FHWA policy. All Federal Lands projects will conform to FHWA policy. The policy statements of particular interest in hydrology and hydraulics include:

1. 23 CFR 650A — **FHWA Policy 23 CFR 650 Subpart A** – *Location and Hydraulic Design of Encroachments on Floodplains*. This federal law establishes policy affecting any project that includes an encroachment on a base floodplain. See Section 7.4.1 for a detailed discussion of this policy

2. 23 CFR 650C — **FHWA Policy 23 CFR 650 Subpart C** – *National Bridge Inspection Standards.* This federal law defines the national standards for the proper safety inspection and evaluation of all highway bridges including the evaluation of bridges for scour susceptibility in accordance with the guidance outlined in Technical Advisory T 5140.23.

3. 23 CFR 650H — **FHWA Policy 23 CFR 650 Subpart H** – *Navigational Clearances for Bridges.* This federal law requires coordination with the United States Coast Guard (USCG) and United States Army Corps of Engineers (USACE) in providing adequate vertical and horizontal clearance for navigation on navigable waterways.

4. 23 CFR 635D — **FHWA Policy 23 CFR 635 Subpart D** – *General Material Requirements*,

7.1.12.3 Other Federal Laws

Other federal laws may affect hydraulic tasks, analyses, design, or construction of Federal Lands Highway projects. These laws are formulated under the following legislative acts:

- The National Environmental Policy Act (1969)
- The Flood Disaster Protection Act (1973)
- The Rivers and Harbors Act (1899)
- The Federal Water Pollution Control Act (1972)
- The Fish and Wildlife Coordination Act (1956)
- The Tennessee Valley Authority Act (1933)
- The Coastal Zone Management Act (1972)
- Wild and Scenic Rivers Act (1968)

7.1.12.4 State and Local Laws

At the state and local levels, the most common water-related legal concerns involve diversion, collection, concentration, quality, obstruction, erosion, and sedimentation. The reconnaissance and scoping effort should identify the state and local laws affecting the design of the project and the appropriate agencies to be contacted for coordination relating to those laws. Since laws related to these problems vary from state to state, the following is a brief generalization of each topic as it relates to this chapter:

7.1.12.4.1 Diversion

Diversion relates to the detention, or changing the course, of a stream or drainage way from its natural or existing condition. Depending on the type of resource system (human or natural) that the diversion affects, the state laws will vary in their scope of jurisdiction. Water diversions should be evaluated for their impact on property owners upstream, downstream, and adjacent to the project. Changes in the flow characteristics due to the diversion may require mitigation with the affected property owners. Diversions should be evaluated for their impact upon fish and wildlife habitat. The state fish and wildlife agencies should be contacted for questions of jurisdiction and possible mitigations. Design diversions of streams or drainage ways to preserve flow conditions that are as similar as possible to those that existed before the diversion while still accomplishing the highway design objectives. A comparison of baseline versus proposed conditions will allow for identification, quantification, and mitigation of impacts related to diversions.

7.1.12.4.2 Storm Water Management

A highway drainage system can collect or concentrate floodwaters, causing discharge rates at the point of discharge to exceed those discharge rates that would naturally occur without the project. A comparison of baseline versus proposed conditions will allow for identification, quantification, and mitigation of impacts related to collection and concentration, including potential water quality concerns.

7.1.12.4.3 Obstruction

Drainage structures form partial obstructions that can cause backwater upstream, increase velocities in the structure area, and cause other hydraulic impacts. A comparison of baseline versus proposed conditions will allow identification, quantification, and mitigation of impacts related to the obstructions caused by drainage structures.

7.1.12.4.4 Stream Erosion and Sedimentation

Highways and their structures can have pronounced impacts on erosion and sedimentation characteristics of a water resource system. If the flow characteristics of rivers and streams are significantly changed, then the erosion and sedimentation characteristics will also be changed.

7.1.12.4.5 Floodplain Management and Administration

Local and state agencies are responsible for managing development within base floodplains. Compliance with FHWA Policy 23 CFR 650A will normally ensure that the local and state floodplain ordinances and statutes are satisfied.

7.2 HYDROLOGY

The hydrologic analysis is a necessary component to the design and evaluation of highway hydraulic structures. The calculation of the design flood is contingent on several factors, the primary two being selection of a design flood standard and an appropriate hydrologic method.

For any given site, there may be several methods available for estimating flows and their return periods. No single method is applicable to all watersheds. Engineering judgment and a good understanding of hydrology are essential in selecting the method to be used in a particular design or for a given watershed. The method chosen should be a function of drainage area (i.e., size and type), availability of data, the validity of the method for the site, land use, and the degree of accuracy desired. When applicable, several methods should be used and the results compared before selecting the most appropriate method.

7.2.1 REFERENCES

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 2	FHWA HDS 2, *Highway Hydrology*, Publication Number NHI-02-001, 2002
2.	HEC 22	FHWA HEC 22, *Urban Drainage Design Manual*, Publication Number NHI-01-021, 2001
3.	AASHTO MDM Chap. 7	AASHTO Model Drainage Manual, *Chapter 7 – Hydrology*
4.	AASHTO HDG Vol. II	AASHTO Highway Drainage Guidelines, *Volume II – Hydrology*
5.	NEH Part 630	NRCS National Engineering Handbook, *Part 630 – Hydrology*
6.	TR-55	NRCS TR-55, *Urban Hydrology for Small Watersheds*, 1986.
7.	WinTR-55	NRCS WinTR-55, Windows version if TR-55 program
8.	WRI 024168	USGS WRI 02-4168, The National Flood Frequency Program, Version 3: *A Computer Program for Estimating Magnitude and Frequency of Floods for Ungaged Sites*
9.	NFF	USGS National Flood Frequency (NFF) computer program
10.	Bulletin 17B	Bulletin 17B of the Hydrology Subcommittee, *Guidelines for Determining Flood Flow Frequency*, 1982.

11. PFDS — Precipitation Frequency Data Server, National Weather Service

12. Terraserver-USA — Terraserver-USA, free public access to maps and aerial photographs of the United State

13. National Map — USGS National Map

7.2.2 DESIGN STANDARDS

The selection of a design flood standard is the first step in the design of highway hydraulic structures. The minimum design flood standards for each type of hydraulic analysis or design are provided in this chapter, and summarized in the quick reference guide in Exhibit 7.1–A.

7.2.3 DESIGN GUIDANCE

7.2.3.1 Peak Flow vs. Hydrograph

Depending on the type of hydraulic investigation, either a peak discharge will be computed or a hydrograph will be developed. The majority of highway drainage structures are analyzed and designed using only the peak discharge for a given design flood. A hydrograph (time distribution of discharge) may be required where either the volume of runoff or the storm duration is needed.

Hydrographs will be used for the design or evaluation of highway hydraulic structures where roadway overtopping duration, storage routing, sediment routing, or unsteady flow modeling are required.

7.2.3.2 Statistical vs. Deterministic

All analytical methods can be grouped into two broad categories of deterministic and statistical models. Deterministic methods model the physical aspects of the rainfall-runoff process, where each element of the runoff process is accounted for, generally based on empirical equations. Statistical methods utilize measured gage data and procedures of statistical analysis to determine flood-frequency relationships.

Simple statistical or deterministic methods are often sufficient for applications within this chapter. More sophisticated models, such as the U.S. Army Corps of Engineers' HEC-HMS and the NRCS TR-20 programs, which use deterministic unit hydrograph methods, may be required and are acceptable for both peak flow and hydrograph needs.

7.2.3.3 Urban vs. Rural

Land use changes affect watershed hydrology and also impact the applicability of hydrologic methods used for design. Urbanization, channelization, and other land use changes (e.g., logging) result in a decrease in infiltration and depression storage, a decrease in travel time, and an increase in runoff volume, resulting in an increase to the peak discharge. The engineer should be aware of past and proposed changes in the watershed land use when selecting a hydrologic method and performing the hydrologic calculations.

Urbanization can also have an adverse impact on stream morphology. There can be a temporary increase in sediment supply due to construction-site erosion, and a long-term reduction in sediment production. Urbanization also typically increases the normal base flow in stream channels. These changes can result in channel stability problems, both lateral and vertical, that may impact highway structures.

Regional regression equations are primarily for natural, undeveloped watersheds. Development should be accounted for using urban regression equations, where available. For regions where urban regression equations have not been specifically developed, both the NFF program and HDS 2 provide methods and procedures for calculating a peak discharge for urban areas, based on the drainage area, the peak discharge for the same watershed in a natural condition, and a basin development factor, which measures the degree of urbanization in the watershed.

7.2.3.4 Potential Future Development

In general, the hydrologic investigations will only account for existing land use conditions, which includes planned development that is funded and has received approval from the local land use permitting authority. Future development may be accounted for in circumstances where the partner agency has a cooperative agreement with the land developer or local community.

7.2.3.5 Local Procedures

There are many local hydrologic procedures or regional modifications to general hydrologic procedures. The engineer may use local procedures within their limits of applicability, with advanced approval of Federal Lands Highway Hydraulics and concurrence of the partner agency. Local procedures are encouraged for use as a check method when available and applicable.

7.2.3.6 Previous Studies

Results of previously documented hydrologic studies may be used with advanced approval of the local Federal Lands Hydraulics Office, if the engineer is confident in the applicability of the hydrologic method and correctness of the calculations.

7.2.3.7 Historical Observations

Field data can sometimes be obtained that can be used to estimate the discharge of historical floods through stage-discharge relationships or open-channel flow calculations. Useful information might include high water marks, bridge inspection reports, and eyewitness reports of overtopping depths of highways and bridges.

Flows determined by historical observations should be used when available as a check on other methods. Flood-frequency magnitudes should not be developed solely from this method because of the small number of observations and inherent inaccuracies.

7.2.3.8 Special Considerations

The standard hydrologic procedures are appropriate for the majority of highway design projects. Conditions that may require special hydrologic investigation and represent hydrological design challenges not anticipated by standard hydrologic procedures include:

- Wetland mitigation analysis and design
- Snowmelt flood hydrology
- Arid lands runoff

Chapter 9 of HDS 2 addresses hydrologic methods and procedures that are associated with such conditions.

7.2.3.9 Data Sources

Data needs frequently include information on the watershed (maps, topography, soils, and land use), stream flow records, and precipitation records. Data must be reliable, accurate, and as current as possible. The sources for the required data may be the partner agency, federal agencies, or state, and local agencies. The geoSpatial Data Acquisition (GSDA) website provides a clearinghouse for much of the publicly available digital data. Acceptable sources of commonly needed data are described below.

In addition to the data sources described in the following sections, hydrologic modeling data may be compiled by state departments of transportation or local flood control agencies (typically in a drainage manual or criteria and procedures manual). Reference Chapter 3 of HDS 2 for information on required data and acceptable sources.

7.2.3.9.1 Stream Flow

The major source of stream flow information is the U.S. Geological Survey (USGS). The USGS stream flow database, including daily, monthly, and annual stream flow statistics is available on the Internet. Also, the U.S. Army Corps of Engineers (USACE), Bureau of Reclamation, and U.S. Forest Service collect stream flow data. Other potential sources of data are state and local governments, utility companies, water-intensive industries, and academic institutions.

7.2.3.9.2 Rainfall

The major source of precipitation data is the National Weather Service (NWS), an agency of the National Oceanic and Atmospheric Administration (NOAA). Historically, NWS publications have been the primary source for precipitation depth-duration-frequency data across the United States. The following NWS publications can be accessed from the Internet:

- Technical Paper 40 – *Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 hours and Return Periods from 1 to 100 Years* (1961)
- Technical Paper 42 – *Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands*
- Technical Paper 43 – *Rainfall-Frequency Atlas of the Hawaiian Islands for Areas to 200 Square Miles, Durations to 24 Hours, and Return Periods from 1 to 100 Years*
- HYDRO 35 – *Five to 60-minutes Precipitation Frequency for Eastern and Central United States* (1977)
- NOAA Atlas 2 – *Precipitation Frequency Atlas of the Western United States* (1973)
- *Short Duration Rainfall Relations for the Western United States* (1986)
- NOAA Atlas 14 – *Precipitation Frequency Atlas of the United States* (Vol. 1 and 2, 2004)

The PFDS should be referenced to ensure that the most up-to-date publication is used for the hydrologic design calculations. For raw rainfall data, NOAA's National Climatic Data Center (NCDC) can be referenced.

Other sources of rainfall data may include state and local agencies. Specifically, many state departments of transportation and local flood control agencies have developed Intensity-Duration-Frequency (IDF) curves and rainfall hyetographs that may be of use to the engineer.

7.2.3.9.3 Land Use

Land use data are available in different forms, including aerial photographs and zoning maps. Data can be obtained from a broad variety of sources, such as state and local planning organizations. The USGS has a nationwide network of maps (1:100,000 and 1:24,000 scale) and aerial photographs. The USGS maps can be obtained in print. The USGS maps and aerial photographs can be accessed from Terraserver-USA or the USGS National Map.

7.2.3.9.4 Soil Type

Information on soil type is needed for some hydrologic methods, primarily NRCS methods, including TR-55. The major source of information on soil types is the NRCS, which has prepared soil maps for most of the counties in the country. The NRCS Soil Survey publications can be obtained from the NRCS or county extension service. The NRCS also has a website that allows online viewing of soil survey maps and reports. The TR-55 publication and Chapter

7 of NEH Part 630 of the NRCS *National Engineering Handbook* give a correlation between NRCS soil type and hydrologic soil group. For soil types not identified in those publications, a correlation can be found in the NRCS Soil Survey for the county.

7.2.3.9.5 Topographic Maps

Topographic mapping can be obtained from a broad variety of sources, such as state and local planning organizations. The USGS has a nationwide network of maps (1:100,000 and 1:24,000 scale) that can be obtained in print and digital formats. The USGS maps can be accessed from Terraserver-USA or the USGS National Map.

7.2.4 RECOMMENDED METHODS FOR ESTIMATING PEAK FLOW

Peak flow estimates obtained by one method should be compared to estimates obtained by other applicable methods. Significant differences may indicate the need to review data from other comparable watersheds or the need to obtain historical data.

7.2.4.1 Ungaged Watersheds

There are many methods available for estimating peak flows at sites without gages. These methods include the Rational Method, Natural Resource Conservation Service (NRCS, formerly SCS) methods, US Geological Survey (USGS) regression equations, and other local methods. Following are brief descriptions of the most frequently used methods:

7.2.4.1.1 Rational Method

The Rational Method is the most commonly used procedure for estimating peak flows from urban, rural, or combined areas for watersheds smaller than 80 hectares [200 acres]. Perform hydrologic calculations using the Rational Method in accordance with the methods presented in HDS 2 *Highway Hydrology.* Additional guidance for the usage of the Rational Method in the design or evaluation of urban storm drain systems is given in HEC 22 *Urban Drainage Design Manual*.

The rainfall intensity is determined using the time of concentration and an Intensity-Duration-Frequency (IDF) curve. IDF curves may be available from state departments of transportation or local flood control agencies. For states that are included in the NOAA Atlas 14, an IDF curve can be obtained directly from the NWS PFDS. For states not yet covered by NOAA Atlas 14, follow the procedures given in Appendix A of HEC 12.

7.2.4.1.2 NRCS Methods

The NRCS Technical Release 55, *Urban Hydrology for Small Watersheds*, commonly referred to as TR-55, provides a graphical peak discharge method that is applicable for small drainage areas (time of concentration between 0.1 and 10 hours).

The NRCS has also released the WinTR-55 computer software package, which will calculate peak flows for watersheds with areas smaller than 6,500 hectares [25 square miles].

Further background information on TR-55 and NRCS hydrologic methods in general, can be found in NEH Part 630 of the NRCS *National Engineering Handbook.* The NRCS method was developed for rolling agricultural and rolling undeveloped land, but is applicable to urbanized areas. Specific application of the NRCS methods to the design of highway drainage structures can be found in Chapter 5 of HDS 2 and Chapter 3 of HEC 22.

7.2.4.1.3 Regional Regression Equations

Regression equations are one of the most commonly accepted methods for estimating peak flows for watersheds without gages or sites with insufficient gage data. Regional regression equations are an extrapolation of data from nearby watersheds with similar hydrologic, physiographic, and climatological characteristics. The USGS, in cooperation with the States, has developed a comprehensive series of regional regression equations for most of the United States into the National Flood Frequency (NFF) computer program. The USGS has also published documentation for the NFF program and for each of the States. These regression equations permit peak flows to be estimated for recurrence intervals ranging from 2 to 500 years for natural streams. Regression equations are developed using independent variables (i.e., basin characteristics) within given ranges for each state and hydrologic region. To ensure the stated accuracy of the estimated discharges, the equations should only be applied within the range of independent variables utilized in their development.

The regional regression equations used in the NFF program are primarily for natural, undeveloped watersheds, although some urban regression equations have been developed. For regions where urban regression equations have not been specifically developed, both the NFF program and HDS 2 provide methods and procedures for calculating a peak discharge for urban areas, based on the drainage area, the peak discharge for the same watershed in a natural condition, and a basin development factor, which measures the degree of urbanization in the watershed.

7.2.4.2 Gaged Watersheds

When a sufficient period of record is available, a desirable method for determining the peak flow is a flood-frequency analysis of flows that have occurred at or near the site. Analyzing flood-frequency relationships from actual streamflow data uses records of past events and statistical relationships to predict future flow occurrences. The best circumstance for estimating peak flows is to have a stream gage near the site for a large number of years. The more years of record, the more accurate the estimate will be. It is recommended that the period of record should be at least 10 years. Where the site being studied is on the same stream and near a gaging station, peak discharges can be adjusted to the site by drainage area ratios using drainage area to some power. For this method to be valid, the gage data used must be homogeneous, i.e., no significant changes in the characteristics of the drainage basin or climatological patterns have occurred over the period of record.

Several of the more popular analysis techniques include Log-Pearson Type III, Normal and Log-Normal, and Gumbel Extreme Value Distributions. Log-Pearson Type III will be used unless it can be shown that the data does not fit this distribution function. Refer to Chapter 4 of HDS 2 and Bulletin 17B for analysis methods of gaged data. The USGS PeakFQ computer program is a method for performing Log-Pearson Type III analyses on raw gaging data. Regional equations may improve peak flow estimate at gaged sites by weighting the statistical analysis estimate with the regression estimate.

7.2.4.3 Guidance on Peak Flow Method Selection

Select methods for calculating the peak flow that are appropriate for the size and hydrologic characteristics of the tributary watershed. Discretion in the selection of the most appropriate method is given to the engineer. General guidance on the applicability of peak flow methods is given as follows:

- For streams with gaging data, with a sufficient period of record (a minimum of 10 years, refer to Chapter 4 of HDS 2), it is recommended that the engineer perform an appropriate statistical analysis of the flood frequency.
- In ungaged watersheds less than 80 ha [200 acres], Rational Method is applicable
- In ungaged watersheds greater than 80 ha [200 acres], regional regression equations or the NRCS TR-55 method are typically applicable.

7.2.5 RECOMMENDED METHODS FOR COMPUTING HYDROGRAPHS

7.2.5.1 Unit Hydrographs

Unit hydrograph techniques are used to approximate the rainfall-runoff response from a watershed. A unit hydrograph is defined as the direct runoff resulting from an excess rainfall event that falls uniformly over the watershed at a constant intensity and has a volume equal to one unit of depth over the watershed. Unit hydrographs are either determined from gaged data or are derived using empirically based synthetic unit hydrograph procedures.

Unit hydrographs are most accurate when based on continuous readings from stream and rainfall gages. When gage data is not available for stream crossings, the NRCS, Snyder, or Clark synthetic unit hydrographs methods may be used. Documentation for unit hydrograph methods can be found in Chapter 6 of HDS 2.

The most common unit hydrograph method for computing a discharge hydrograph for highway drainage structures is the NRCS procedure documented in NEH Part 630 of the NRCS *National Engineering Handbook.* The WinTR-55 computer program is generally applicable for areas less than 6,500 ha [25 square miles], with additional limitations set by the time of concentration for the watershed. Specific application of the NRCS methods to the design of highway drainage structures can be found in Chapter 6 of HDS 2.

7.2.5.2 Regional Regression Equations

The National Flood Frequency program contains a procedure for computing a dimensionless hydrograph, representing the average runoff for a given peak discharge. The hydrograph is not representative of any rainfall distribution. Runoff calculations performed using regional regression equations should be done in accordance with the methods and procedures documented for the NFF computer program. Specific application of the USGS regression equations to the design of highway drainage structures can be found in Chapter 6 of HDS 2.

7.2.5.3 Storage Routing

Where detention ponds are required for Federal Lands Highway projects, such as for storm water management applications, storage routing can be performed using the Storage-Indication method as documented in Chapters 7 and 8 of HDS 2 and Chapter 8 of HEC 22.

Storage routing may also be used to evaluate existing or rehabilitated culverts that do not have the capacity to convey the peak discharge prescribed by the applicable standard.

7.2.6 REPORTING

All hydrologic analyses will be supported by appropriate documentation, which at a minimum will include:

- Data and data sources
- Reference for methods used
- Assumptions
- Conclusions
- Recommendations

7.3 ROADWAY HYDRAULICS

7.3.1 CULVERTS

Culverts are physically simple structures used to convey surface runoff through, around, and away from roadways and associated facilities. They typically consist of a pipe barrel with an inlet and outlet structure. Although simple structurally, the hydraulic design of culverts requires the investigation of numerous physical, operational, and regulatory elements during the data collection phase, which must then be applied, as appropriate, during project development. Examples of physical elements include geometrics (e.g. size, shape, length, alignment, material roughness, slope, and entrance treatments); and hydraulic characteristics (outlet tailwater depth, outlet velocity, headwater depth, scour/erosion potential, sediment transport, debris production). Operational elements include frequency of maintenance and vehicular safety.

Regulatory elements may include federal and state hydraulic criteria such as the requirements of the National Flood Insurance Program (NFIP) administered by the Federal Emergency Management Agency (FEMA). Other federal laws/regulations that may impact culvert design include: NEPA, Fish & Wildlife Act, TVA, Coastal Zone Management Act, and Wild & Scenic Rivers Act.

Refer to [EFLHD – CFLHD – WFLHD] Division Supplements for more information.

7.3.1.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 5	FHWA HDS 5, *Hydraulic Design of Highway Culverts*, Publication Number NHI-01-020, 2005
2.	HEC 14	FHWA HEC 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, 1983
3.	AASHTO MDM Chap. 9	AASHTO Model Drainage Manual, *Chapter 9 – Culverts*
4.	AASHTO HDG Vol. IV	AASHTO Highway Drainage Guidelines, *Volume IV – Hydraulic Design of Culverts*
5.	FLH Standard Drawings	Federal Lands Highway Standard Drawings, current edition.

7.3.1.2 Standard Practices

7.3.1.2.1 Floodplain Encroachment

If a waterway crossing constitutes a new or expanded encroachment on a base (100-year) floodplain that is regulated by FEMA or potentially creates an adverse impact to private property or insurable buildings, reference Section 7.4.1 for details on appropriate design policy, standards, and criteria, as well as guidance on FEMA coordination, if required.

7.3.1.2.2 New vs. Rehabilitation

All existing culverts identified to be retained as part of a roadway rehabilitation project will receive an appropriate evaluation of condition, hydraulic performance and long term risk to determine whether replacement or rehabilitation is necessary. Inform partner agency of all condition and performance problems if correction is not included within the project scope.

7.3.1.3 Design Standards

7.3.1.3.1 Capacity Design

<u>Design Flood</u>

The design flood standards for culverts are based on two roadway classifications – High Standard and Low Standard (reference Section 7.1.6).

- *High Standard:* Design cross culverts using the following standards:
    - ◊ Culverts will convey runoff from the 50-year flood
    - ◊ Culverts for temporary detours will convey runoff from the 10-year flood, unless seasonal construction justifies a lower standard
- *Low Standard:* Design cross culverts using the following standards:
    - ◊ Culverts will convey runoff from the 25-year flood
    - ◊ Culverts for temporary detours will convey runoff from the 2-year flood, unless seasonal construction justifies a lower standard
- *Roadside Ditches:* Culverts required for roadside ditches should be designed to convey the runoff from the 10-year flood for both High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

Check Flood

Evaluate the potential for adverse impacts to private property or insurable buildings upstream of the roadway at the roadway overtopping elevation. If such adverse impacts can occur, refer to Section 7.4.1 for direction on applicable design policy, standards, criteria, and guidance.

7.3.1.3.2 Stability Design

Design Flood

The stability design flood standards for culverts are based on two roadway classifications – High-Standard and Low-Standard (reference Section 7.1.6).

- *High-Standard:* roadway culverts and embankments at culvert locations will be stable for the 50-year flood
- *Low-Standard:* roadway culverts and embankments at culvert locations will be stable for the 25-year flood

7.3.1.3.3 Surveying and Mapping

When survey is needed to quantify hydraulic impacts, refer to Chapter 5 for standards on survey and mapping for culverts.

7.3.1.4 Design Criteria

7.3.1.4.1 Headwater Elevation

The headwater elevation is defined as the water-surface elevation (WSEL) at the culvert entrance. There are three sets of criteria used to determine the allowable headwater elevation: 1) new vs. existing culvert, 2) ratio of headwater depth to culvert diameter or rise (HW/D), where depth is measured from the water surface to the inlet invert, and 3) site-specific reference elevations. The criterion that results in the lowest headwater elevation will govern the design.

New vs. Existing

- *New Culverts*: Headwater elevation will not be greater than the bottom of the aggregate base layer for the roadway pavement structure at the local roadway low point.
- *Existing Culverts:* Headwater elevation will not be greater than the shoulder hinge point at the local roadway low point (i.e. ponding will not be allowed to spread onto the shoulder of the roadway).
- *Temporary Culverts:* Headwater elevation will not be greater than the shoulder hinge point at the local roadway low point (i.e. ponding will not be allowed to spread onto the shoulder of the roadway).

<u>HW/D Ratio</u>

- *1200 mm [48”] equivalent and smaller culverts:* Limit ratio to 1.5.
- *Larger than 1200 mm [48”] equivalent culverts:* Limit ratio to 1.2.
- *Debris or Sediment:* A ratio range of 0.8 to 1.0, depending on severity, is suggested where the potential for heavy debris or sediment bed loads are a concern.

<u>Reference Elevations</u>

Elevations that represent unacceptable hazards to human life or private property, or exceed local sub-basin divides such as ditch invert elevations that would allow runoff to flow away from the desired crossing point.

7.3.1.4.2 Minimum Size

To limit maintenance problems due to debris or sedimentation and to facilitate inside access to culverts, minimum pipe size criteria are:

- 600 mm [24”] or equivalent for cross-road culverts
- 450 mm [18”] or equivalent for parallel culverts in roadside ditches and channels

7.3.1.4.3 Slope

Site conditions determine the slope for a particular cross culvert. For determining appropriate slope, cross culverts can be divided into two categories:

<u>Ditch Relief</u>

For culverts used as cross-drains to carry away intermittent roadside ditch water, the pipe slope should not be flatter than 2% whenever possible, with 0.5% being the minimum. Where practical, the pipe slope should equal or exceed the roadside ditch grade. The maximum slope should not exceed 10% for concrete pipe, or 25% for metal pipes, without using pipe anchors.

<u>Stream Crossings</u>

These culverts are individually designed to carry the design discharge from a basin without exceeding the allowable headwater criteria. The pipe slope will generally conform to the average streambed flow line and should match the channel elevations on both the upstream and downstream sides.

7.3.1.4.4 Cover

Refer to FLH Standard Drawings for the minimum and maximum cover on pipes.

7.3.1.4.5 Pipe Anchors

Pipe anchors are required for any exposed pipe (i.e., laid on embankment fill or natural ground). Additionally, because culverts placed on very steep slopes can experience joint separation, incorporate pipe anchors for concrete pipe on a slope of 10% or greater and for corrugated metal pipes on a slope of 25% or greater.

7.3.1.4.6 Materials

Refer to Section 7.3.6 for standards and guidance regarding the selection of alternative materials. All proposed culvert installations will meet the selected design criteria regardless of which alternative material is selected.

7.3.1.5 Design Guidance

7.3.1.5.1 Alignment

The recommended maximum culvert skew, relative to the roadway centerline, is 45 degrees.

7.3.1.5.2 Entrance Treatments

The culvert end treatments affect hydraulic efficiency, embankment stability, aesthetics, and safety for run-off-the-road vehicles. There are several types of entrance treatments for culverts:

- Thin edge projecting
- Mitered to conform to slope
- Flared end section
- Square edge in a headwall (with or without wingwalls)
- Beveled edge in a headwall (with or without wingwalls)
- Grooved end projecting
- Side-tapered and slope-tapered inlets

For the design of new structures, flared end sections are recommended for 1200 mm [48”] equivalent and smaller pipes. For larger pipes, a headwall end treatment is recommended to offset buoyant forces. Headwalls are also recommended for multiple pipe installations. Beveled edges should be used on all headwalls. For long culverts operating under inlet control conditions, tapered inlets, also known as “improved inlets,” may be used to increase hydraulic efficiency and allow the designer to reduce the pipe size.

For existing, lengthened, or rehabilitated structures with insufficient capacity to convey the design discharge, the designer should consider adding a more efficient entrance treatment.

7.3.1.5.3 Outlet Treatments

Scour at culvert outlets is a common occurrence that can undermine and cause failure of the culvert system. For most culverts, standard riprap outlet protection is sufficient. Refer to Section 7.3.5 for discussion on applications where the outlet velocity, relative to soil erodibility, dictates the use of an energy dissipator.

7.3.1.5.4 Fish Passage

At some culvert locations, the ability of the structure to accommodate migrating fish is an important design consideration. For these sites, consult state fish and wildlife agencies early in the roadway planning process. For existing culverts that obstruct fish passage, modifications can often meet the fish and wildlife agencies' design criteria. Design standards, criteria, and guidance for fish passage are provided in Section 7.5.1 of this document.

7.3.1.5.5 Camber

Under high fill conditions, the engineer should incorporate sufficient camber to allow for settlement. Refer to FLH Standard Drawings for the recommended camber.

7.3.1.5.6 Open-Bottom Culverts

Open-bottom culverts, either concrete or metal, are sometimes designed for fish passage, environmental, aesthetic, or economic reasons. These structures have a natural bottom and must be supported on both sides by a foundation. Because of the likelihood of local scour, evaluate and design the foundations using bridge criteria, unless they can be founded on bedrock. Refer to Section 7.4.3 for information on foundation design.

7.3.1.5.7 Box Culverts

Use standard drawings from the applicable State, unless a custom design is required. If a custom design is required, consult the Bridge Design Group.

7.3.1.6 Recommended Methods

Design and evaluate culverts for hydraulic performance according to the methods and procedures presented in HDS 5 *Hydraulic Design of Highway Culverts.*

For standard riprap outlet protection, refer to FLH Standard Drawings or the methods in HEC 14. For outlets requiring energy dissipators, refer to Section 7.3.5.

7.3.1.7 Reporting

Documentation on the design of culverts should contain, at a minimum, the following data, as applicable:

- Project identification
- Location of proposed installations
- Drainage area map and site topography
- Stream profile and cross sections
- Information on existing structures
- Historical highwater data
- Site investigation data (e.g., stream stability information)
- Hydrologic design computations
- Hydraulic design calculations and culvert performance curves
- Economic analysis

7.3.1.8 Plans

In the plans for culvert installations, include the following for each culvert location:

- Size
- Alignment
- Length
- Acceptable materials, including class, gauge, and any special coatings
- Joint gasket treatments, if any
- End treatment
- Cover depth
- Camber, if any

For the location and design of simple riprap outlet protection, include the following for each culvert location:

- Dimensions and extent of riprap
- Gradation
- Bedding and Filter Material
- Grading or slope details, if needed

In addition, culvert pipe 1200 mm [48"] or equivalent and larger will include individual cross sections showing slope, inlet/outlet invert elevations, design headwater or headwater/diameter ratio, design discharge, drainage area, and any special foundation work or end treatment. Headwalls, energy dissipators, or riprap must be shown. Also include any necessary FLH Standard Drawings or special detail drawings.

Include a Drainage Summary Sheet in the plans for all culverts. Show maximum pipe cover, structure excavation, type of pipe (e.g., wall thickness, size, length), and acceptable alternative pipe materials. See Division Supplements for an example Drainage Summary Sheet.

7.3.2 DITCHES

Ditches are engineered channels, such as roadside ditches in cut sections, toe-of-slope ditches, and interceptor ditches placed at the top of cut slopes. Capacities will be less than 1.5 cms [50 cfs]. This section addresses the design of ditches, including selecting the appropriate design frequency, and evaluating the physical geometry (shape, slope, side slopes, roughness, depth, and freeboard) and channel stability (velocity, shear stress, and channel lining).

For the design or evaluation of channels with capacities of 1.5 cms [50 cfs] or greater, refer to the River Hydraulics Section 7.4.

7.3.2.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 3	FHWA HDS 3, *Design Charts for Open Channel Flow*, 1961
2.	HDS 4	FHWA HDS 4, *Introduction to Highway Hydraulics*, FHWA Publication Number NHI-01-019, 2001
3.	HEC 15	FHWA HEC 15, *Design of Roadside Channels with Flexible Linings*, FHWA Publication Number IF-05-114, 2005
4.	HEC 22	FHWA HEC 22, *Urban Drainage Design Manual*
5.	AASHTO MDM Chap. 8	AASHTO Model Drainage Manual, *Chapter 8 – Channels*
6.	AASHTO HDG Vol. VI	AASHTO Highway Drainage Guidelines, *Volume VI – Hydraulic Analysis and Design of Open Channels*

7.3.2.2 Design Standards

7.3.2.2.1 Capacity Design

Design Flood

Design roadside ditches for the 10-year flood for both High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

7.3.2.2.2 Stability Design

Design Flood

Design roadside ditches for stability for the 10-year flood for both High- and Low-Standard roadways. (Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.)

Temporary Linings

Temporary linings may be needed to protect ditches from erosion over the transitional period before permanent protective vegetation can become established. Design temporary channel linings to be stable for the 2-year flood.

7.3.2.3 Design Criteria

7.3.2.3.1 Depth

Depth is defined as the allowable depth of flow relative to the ditch invert.

New Ditches

Limit the design depth to the elevation of the bottom of the aggregate base layer for the roadway pavement structure.

Existing Ditches

When evaluating capacity of existing ditches, limit the depth to the elevation of the shoulder hinge point on the roadway (i.e. flow should not spread onto the shoulder of the roadway).

7.3.2.3.2 Slope

Minimum ditch slope is 0.5% where possible. Where practical, provide a desired 1.0% minimum ditch slope.

7.3.2.3.3 Stability

Design all engineered channels to be stable for the prescribed stability discharge based on permissible shear. The shear stress approach focuses on stresses developed at the interface between the channel boundary and flowing water. The permissible shear stress is the maximum that will not cause serious soil erosion from the channel bed or banks. Acceptable channel linings are outlined in HEC 15 and identified in the FLH Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects (FP).

7.3.2.3.4 Ditch Relief

Design permanent ditch relief (culverts, spillways, or inlets), as necessary, to meet conveyance or stability criteria.

7.3.2.3.5 Outlet Protection

Ditch outlets will be stable for the stability design discharge. Scour at ditch outlets is a common occurrence that can undermine and cause embankment failure. For most outlets, properly designed riprap outlet protection is sufficient. Reference Section 7.3.5 for discussion on applications where the outlet velocity, relative to soil erodibility, dictates the use of an energy dissipator to prevent excessive outlet scour.

7.3.2.4 Design Guidance

7.3.2.4.1 Cross Section Shape

Ditch cross sections are typically designed based on minimum standard dimensions that permit easy construction and maintenance with highway equipment. Minor drainage channels may have vee, trapezoidal, rectangular, parabolic, or triangular shapes.

7.3.2.4.2 Slope

The ditch slope need not follow that of the roadbed. Although preferred, the roadside ditch geometry need not be standardized for any length of highway. Wider, deeper, or flat-bottom ditches may be used as required to meet different amounts of runoff, channel slopes, lining types, and distances between points of discharge. Ditch relief structures should be provided, where necessary, to maintain the standard ditch section to the extent possible.

7.3.2.4.3 Erosion Protection

In areas where vegetation will not provide adequate protection, the channel may be lined with rock or stone of suitable size, or with asphalt or concrete. Smooth linings generate higher velocities than rough linings such as stone and vegetation and may require energy dissipation devices at the outlets.

7.3.2.5 Recommended Methods

Design roadside channels using methods given in HEC 15, *Design of Roadside Channels with Flexible Linings.* Evaluate the channel stability for the immediate post-construction condition and for the final condition using the permissible shear stress, as documented in HEC 15. The values for permissible shear stress are given in HEC 15.

The permissible shear stress values for many temporary and permanent erosion control blankets have been determined in laboratory studies by manufacturers. The engineer may use a manufacturer-specified permissible shear stress, if developed according to ASTM D6460, *Standard Test Method for Determination of Erosion Control Blanket (ECB) Performance in Protecting Earthen Channels from Stormwater-Induced Erosion.*

7.3.2.6 Reporting

Documentation on the design of roadside channels should contain the following minimum data:

- Project identification
- Location of proposed work
- Design discharge and frequency
- Hydrologic calculations
- Channel cross section and gradient
- Type of lining
- Design calculations

7.3.2.7 Plans

The plans will show all details necessary to construct the channel according to the hydraulic design. The following information should be included, at a minimum:

- Location
- Alignment
- Slope and elevations
- Cross section (bottom width, side slope, depth)
- Channel linings (both temporary and permanent)
- Special structure details, if any

7.3.3 PAVEMENT DRAINAGE

Pavement drainage refers to the above-ground hydraulic considerations associated with the design of systems to collect and drain runoff from roadways with curb and gutter. Design considerations include selecting the storm event, defining surface drainage patterns, limiting the allowable spread (extent of water on the road surface), locating and spacing inlets, and special considerations associated with sag locations. This section provides design discussion and guidance on all areas of roadway surface drainage, including bridge deck drainage.

7.3.3.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HEC 21	FHWA HEC 21, *Bridge Deck Drainage*, FHWA Publication Number SA-92-010, 1993
2.	HEC 22	FHWA HEC 22, *Urban Drainage Design Manual*
3.	AASHTO MDM Chap. 13	AASHTO Model Drainage Manual Chapter 13 – *Storm Drainage Systems*
4.	AASHTO HDG Vol. IX	AASHTO Highway Drainage Guidelines Volume IX – *Storm Drain Systems*

7.3.3.2 Design Standards

7.3.3.2.1 Capacity Design

Design Flood

These standards apply to both High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *On-grade, Sags, and Parking Areas:* Design the roadway conveyance and collection systems (i.e. gutter flow and inlet design) for the 10-year flood.
- *Sumps:* Roadway sumps are defined as deep roadway sags that must have storm drain systems to outlet runoff and limit gutter depths. In roadway sump locations where a storm drain system is the only outlet, design the drainage inlet system to accommodate the 50-year flood.

7.3.3.3 Design Criteria

7.3.3.3.1 Spread

Spread refers to the allowable width of flow encroachment onto the pavement section during storm events. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *High-Standard Roadways:* Limit the spread to 900 mm [3'] of one travel lane for gutter flow, both on-grade and in roadway sags.
- *Low-Standard Roadways:* Limit the spread to half of one travel lane for gutter flow, both on-grade and in roadway sags.

7.3.3.3.2 Depth

Applies to High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

On-grade and Sags

Flow depth at the curb should not exceed the curb height or the allowable spread for the design discharge.

Sumps

Limit the depth of flow at the gutter flowline to 150 mm [6"].

Parking Areas

For inlets adjacent to curbs, flow depth should not exceed the curb height. For sags limit the depth of flow at the gutter flowline to 150 mm [6"].

7.3.3.3.3 Inlet Clogging Factor

Applies to High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

On-grade

Assume that on-grade inlets are not subject to debris clogging, unless clogging is a known problem.

Sumps and Sags

- *Grate Inlets:* Design grate inlets in roadway sags and parking areas using an inlet clogging factor of 50 percent. In other words, reduce the grate perimeter or open area parameters by 50 percent of the actual values.
- *Curb-opening Inlets:* Assume that curb-opening inlets are not subject to debris clogging, unless clogging is a known problem
- *Rehabilitation Projects:* Assume all inlets are not subject to clogging, unless clogging is a known problem.

7.3.3.4 Roadway Design Guidance

The roadway pavement and geometry should be designed for the efficient removal of rainfall from the traveled lanes of the roadway. The roadway pavement materials and finishes, cross-slope, and longitudinal slope should be designed to promote the removal of water from the traveled lanes.

In rural areas, avoid the use of curbed sections whenever possible to avoid runoff concentration and potential erosion.

7.3.3.4.1 Gutter Flow

A gutter is defined as the section of roadway next to the curb that conveys water during a storm runoff event. Gutter cross sections have a triangular shape with the curb forming the near-vertical leg of the triangle. The gutter may have a uniform cross slope or a composite cross slope. Composite gutter sections are encouraged, where possible, because of the associated increase in gutter capacity and inlet efficiency.

7.3.3.4.2 Inlet Location

There are numerous locations where inlets are required based on the geometry of the roadway. The following list includes locations where inlets are recommended based solely on roadway geometry:

- At all low points in the gutter grade
- Immediately upstream of median breaks, entrance/exit ramp gores, cross walks, and street intersections, i.e., at any location where a concentrated flow path could flow onto the travel lanes
- Immediately upgrade of bridges (to prevent water from flowing onto bridge decks)
- Immediately downstream of bridges (to intercept bridge deck drainage)
- Immediately up grade of cross-slope reversals
- Immediately up grade from pedestrian cross walks
- On side streets immediately upgrade from intersections
- At the end of channels in cut sections
- Behind curbs, shoulders, or sidewalks to drain low areas

Additional on-grade inlets will be spaced to meet the allowable spread criteria. The minimum recommended capture efficiency for on-grade inlets is 70%.

Where curbs are used, runoff from cut slopes and areas off the right-of-way should, wherever possible, be intercepted by ditches at the top of slopes or in a swale behind the curb. This reduces the amount of water that has to be picked up by the inlets and the amount of mud and debris carried onto the pavement.

7.3.3.4.3 Inlet Type

Select the type of inlet to best meet the design criteria, considering cost, hydraulic efficiency, interference with traffic, pedestrian and bicycle safety, and debris clogging. Grate inlets, curb-opening inlets, slotted drain pipe inlets, or a combination of curb opening and grate inlets may be used for intercepting runoff. Some of the major operational characteristics for each inlet type are provided below. Refer to the following FLH Standard Drawings list for general application of Federal Lands inlets:

- Type 1 Catch Basin – Grate Inlet with a tilt-bar grate (Type A or B), intended for use on-grade in a curb and gutter section or in a ditch flowline
- Type 2 Catch Basin with Down Drain – Grate Inlet with a tilt-bar grate (Type A or B), intended for use on-grade in a curb and gutter section, roadway in fill
- Type 5A Inlet – Grate Inlet with a P 64 x 108 [P 2.5 x 4.25] grate, for use on-grade or in sags
- Type 6B Inlet – Grate inlet with a cast iron grate, for use in valley gutters or parabolic ditches
- Type 7A/B Inlet – Grate inlet with wide bar-spacing, for use in a ditch flowline

<u>Grate Inlets</u>

Grate inlets consist of a collection box below the gutter, covered with a grate.

- *Continuous Grade*
    - ◊ Grate inlets on a continuous grade will intercept all or nearly all of the gutter flow passing over the grate, or the frontal flow. A portion of the flow along the side of the grate will be intercepted, depending on the cross slope of the pavement, the length of the grate and flow velocity.
    - ◊ On-grade grate inlets maintain interception capacity on steeper slopes.
    - ◊ Interception capacity of grate inlets is reduced by debris clogging.
    - ◊ The length of grate inlets is relatively inflexible. Increased length typically does not significantly affect interception capacity
- *Sag Locations*
    - ◊ A grate inlet in a sag location operates as a weir at shallow depths and as an orifice at greater depths.
    - ◊ In a sag the length of the grate inlet can be varied to increase interception capacity.
    - ◊ Interception capacity of grate inlets is reduced by debris clogging.

<u>Curb-opening Inlets</u>

Curb-opening inlets are vertical openings in the curb, covered by a top slab.

- Curb-opening inlets are relatively free of clogging tendencies and offer little interference to traffic operation.

- Curb-opening inlets may be preferred over grate inlets in locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.
- Curb-opening inlets are preferred on longitudinal grades 3 percent or less because of decreasing capture capacity and efficiency at steeper grades.

Slotted Inlets

Slotted inlets consist of a pipe cut along its longitudinal axis with perpendicular bars used to maintain a continuous opening.

- Slotted inlets function in essentially the same manner as curb opening inlets on a continuous grade.
- Slotted drains are susceptible to clogging and can be difficult to maintain.
- Due to the high potential for debris clogging, the use of slotted drain inlets located in sags is discouraged.

Combination Inlets

Combinations of grate and curb-opening inlets can be used. Combination inlets can either be equal-length or sweeper inlets, where the curb opening extends upstream of the grate.

- *Equal-length Combination Inlets*
    - ◊ Equal-length combination inlets have both a grate and a curb opening, with the same length.
    - ◊ Equal-length combination inlets on a continuous grade are not recommended because the capacity is not appreciably greater than with the grate inlet alone.
    - ◊ Equal-length combination inlets are recommended in sag locations because of increased capacity and the fact that the curb opening provides relief should the grate inlet become clogged.
- *Sweeper Inlets*
    - ◊ Sweeper inlets have both a grate and a curb opening, with the curb opening being longer than the grate in the upstream direction.
    - ◊ Sweeper inlets on a continuous grade are relatively free of debris clogging tendencies and can be used where increased interception efficiency is required.

Median and Roadside Ditch Inlets

- Grate inlets similar to those used for pavement drainage may be used to drain medians and roadside ditches. Additionally, since bicycle safety is typically not a factor at these locations, these inlets/grates should provide maximum open area to minimize clogging potential.

- Grate inlets should be flush with the ditch bottom and cross drainage structures should be continuous across the median unless the median width makes this impractical.
- Ditches tend to erode at grate inlets. Paving around the inlets may help prevent erosion and may increase the interception capacity of the inlet marginally by reducing bypass flow.
- Small dikes placed immediately downstream of median or ditch inlets can ensure complete interception of the flow.

7.3.3.5 Bridge Deck Design Guidance

The hydraulic principles of bridge deck drainage are similar to roadway drainage principles. The surface drainage, gutter flow and inlet design standards, criteria, and guidance provided in the previous sections all apply to bridge deck drainage, but are complicated by the structural and architectural requirements of bridges. The bridge deck inlets tend to be small to conform to structural requirements and, as such, tend to clog easily. Down-drain pipes can detract from the bridge aesthetics, and encased piping has serious maintenance considerations.

Wherever possible, do not design bridge deck profiles with sags or low points because small inlet sizes and potential for debris clogging make them difficult to drain.

Wherever possible, design bridges to meet roadway drainage criteria without the use of bridge deck inlets. Typically, bridges are built with uniform gutter geometry, as opposed to the more effective composite gutter section. Where required by criteria, on-grade inlet spacing may be determined both by allowable spread criteria and bridge pier spacing.

Roadway inlets should be placed up-gradient of bridges to reduce or eliminate runoff onto the bridge deck.

Roadway inlets should also be placed down-gradient of bridges to capture runoff from the bridge deck. This is especially critical where a curbed gutter section does not extend beyond the bridge abutment. Concentrated runoff from the bridge deck in these situations could precipitate erosion, which could cause damage to the abutment fill.

7.3.3.6 Recommended Methods

Design and evaluate the pavement drainage system performance according to the methods and procedures presented in HEC 22 *Urban Drainage Design Manual.* For bridge deck drainage design, HEC 21 *Design of Bridge Deck Drainage* is the recommended reference for information on detailed design methods and procedures.

7.3.3.7 Reporting

The design of a roadway drainage facility should be supported by documentation containing, at a minimum, the following information:

- Project identification
- Location of proposed installation
- Roadway gradient and applicable cross section
- Design discharge and frequency
- Gutter discharge and spread calculations
- Type and size of inlets
- Inlet efficiency calculations
- Data on intercepted and bypass flows

7.3.3.8 Plans

Design roadway drainage improvements to reflect the roadway gradient and cross sections given on the plans. For the location and design of inlets, prepare plans showing all details necessary to construct the improvements according to the hydraulic design, including the following:

- Location
- Type and size of inlets
- Special structure details, if any
- Drainage Summary Sheet

7.3.4 STORM DRAINS

A storm drain is the portion of the roadway drainage system that receives runoff from multiple inlets and conveys it through a series of pipes to an outfall. The design of storm drain systems includes selecting the proper hydrologic method and recurrence interval, sizing the pipe, locating access structures, determining energy losses, and computing the hydraulic grade line to determine free surface flow versus pressure flow.

7.3.4.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HEC 22	FHWA HEC 22, *Urban Drainage Design Manual*
2.	AASHTO MDM Chap. 13	AASHTO Model Drainage Manual, *Chapter 13 – Storm Drainage Systems*
3.	AASHTO HDG Vol. IX	AASHTO Highway Drainage Guidelines, *Volume IX – Storm Drain Systems*
4.	AISI Sewer Design	American Iron and Steel Institute, *Modern Sewer Design*

7.3.4.2 Design Standards

7.3.4.2.1 Capacity Design

Design Flood

The following design flood standards apply to both High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- On-Grade: 10-year flood
- Sumps: 50-year flood

Roadway sumps are defined as deep roadway sags that must have storm drain systems to outlet collected runoff and limit gutter depths.

7.3.4.3 Design Criteria

Design storm drains to flow full (i.e., no pressure) for the design event whenever possible.

7.3.4.3.1 Minimum Size

The minimum size for storm drain pipe is 375 mm [15"] or equivalent.

7.3.4.3.2 Minimum Slope

Design storm drains with slope sufficient to develop a self-cleaning velocity of 0.9 m/s [3 ft/s] when flowing full (reference Table 7-7 in HEC 22). Slope less than 0.5% should be avoided for constructability reasons.

7.3.4.3.3 Hydraulic Gradeline

Compute the hydraulic gradeline (HGL) over the full length of storm drains with four or more inlets connected in series.

In storm drain sections where the hydraulic gradeline for the design flood must exceed the pipe soffit (i.e., the pipe flows under pressure), the hydraulic gradeline for the design flood will remain below the ground elevation at all inlets and access structures, and watertight gaskets should be specified for the pipe joints.

7.3.4.3.4 Access Structures

Locate access structures to provide access for inspection and maintenance. Inlet structures are considered access structures and should be designed accordingly. Access structures are typically located based on maintenance requirements and at changes to the storm drain alignment or profile, including locations where:

- Two or more storm drains converge
- Pipe size changes
- Abrupt change in alignment occurs
- Abrupt change in slope occurs
- At intermediate points according to spacing given in Exhibit 7.3–A

Exhibit 7.3–A ACCESS STRUCTURE MAXIMUM SPACING

Pipe Size, mm [in]	Maximum Spacing, m [ft]
375 – 600 [15 – 24]	90 [300]
675 – 900 [27 – 36]	120 [400]
1050 – 1350 [42 – 54]	180 [600]
1500 and up [60 and up]	300 [1000]

7.3.4.3.5 Materials

Refer to Section 7.3.6 for standards and guidance regarding the selection of alternative materials. All proposed storm drain installations will meet the selected design criteria regardless of which alternative material is selected.

7.3.4.4 Design Guidance

7.3.4.4.1 Storm Drain Profile

Where practical, match the pipe soffit elevations (high point inside pipe) at all junctions, rather than the pipe invert elevation. Invert elevations for same size pipes should be offset to account for losses in access structures. This technique will help prevent backwater profiles from rising and upstream velocities from decreasing.

Where possible, the pipe size should not decrease in the downstream direction, even though the capacity of the smaller pipe may be greater due to a steep slope. Exceptions are to be considered when tying into an existing system.

The storm drain profile should be designed as close to the surface as possible, taking minimum cover depths and utility conflicts into consideration.

7.3.4.4.2 Hydraulic Gradeline

If the computed hydraulic gradeline is higher than allowed by criteria, energy losses can be reduced by increasing the pipe size or designing more hydraulically efficient access structures.

7.3.4.4.3 Outlet Treatment

Outlet channels will be stable for the design discharge. Use standard wing wall outlet protection where applicable. Reference Section 7.3.5 for design of energy dissipators when required.

7.3.4.5 Recommended Methods

Design and evaluate the storm drain system performance according to the methods presented in HEC 22 *Urban Drainage Design Manual*, or approved equivalent.

7.3.4.6 Reporting

The design of the storm drain and evaluation of the hydraulic gradeline should be supported by documentation containing, at a minimum, the following information:

- Project identification
- Location of proposed installation
- Hydrologic design computations
- Hydraulic design calculations

7.3.4.7 Plans

For the location and design of storm drains, prepare plans showing all details necessary to construct the improvements according to the hydraulic design, including the following:

- Size
- Alignment
- Length
- Slope and inlet/outlet invert elevations
- Inlet, access structure locations
- Acceptable materials, including class, gauge, and any special coatings
- Joint gasket treatments, if any
- Outlet treatment

Information placed on the plans will include individual profile sheets showing design discharge, drainage area, hydraulic gradeline, and any special access structure details. Show maximum pipe cover, structure excavation, type of pipe (e.g., wall thickness, size, length), and acceptable alternative pipe materials on Drainage Summary Sheet. End Treatments, energy dissipators, or riprap must be shown. Include any necessary FLH Standard Drawings or special detail drawings.

7.3.5 ENERGY DISSIPATORS

Local scour at channel, culvert, and storm drain outlets is a common occurrence. The natural runoff is usually confined to a lesser width and greater depth as it passes through a conveyance system. An increased velocity results with potentially erosive capabilities at the conveyance outlet. Turbulence and erosive eddies form also as the flow expands to conform to the natural channel. In addition to the hydraulic characteristics of the flow at the outlet, the erosive characteristics of the outlet channel bed and bank material, and the amount of sediment and other debris in the flow are contributing factors to scour potential.

Where the local scour potential exceeds the protective capabilities of standard outlet treatments, an energy dissipator design is typically required. The focus of this section is on the special design requirements for energy dissipators.

7.3.5.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. HEC 14 — FHWA HEC 14, *Hydraulic Design of Energy Dissipators for Culverts & Channels*
2. AASHTO HDG Vol. IV — AASHTO Highway Drainage Guidelines, *Volume IV – Hydraulic Design of Culverts*

7.3.5.2 Design Standard

Evaluate the performance of energy dissipators (i.e. velocity reduction) over a range of discharges. The range of discharges will include the lowest discharge for which scour is a concern to the design of the applicable conveyance system. Select the dissipator that provides acceptable performance over this range of discharges.

7.3.5.3 Design Criteria

Discharge outflow to the downstream channel at velocities that are compatible with the erosion characteristics of the outlet channel bed and bank material. If the outlet channel is stable, the natural channel velocities would be an appropriate dissipation target. If the outlet channel is unstable, the concern becomes a stream stability problem that may or may not include local energy dissipation as a solution. Refer to Section 7.4.2 for guidance on evaluating stream stability.

7.3.5.4 Design Guidance

In order to release stormwater discharge at a stable channel velocity, there are several alternatives for outlet protection:

- no protection required (no scour potential or expected scour can be tolerated)
- simple riprap outlet protection (standard outlet treatment)
- minimal outlet protection with performance monitoring
- formal energy dissipator

Riprap protection at pipe outlets is appropriate where moderate outlet velocities exist. At some locations, the use of a rougher culvert material may alleviate the need for a special outlet protection device. Preformed scour holes (approximating the configuration of naturally formed holes) dissipate energy while providing a protective lining to the streambed. There are many situations where riprap structures are impractical even at low to moderate flow conditions. Energy dissipators can be designed easily and are suitable for a wide variety of site conditions. In some cases, they are more economical than large riprap basins, particularly where long-term costs are considered. Various types of energy dissipation structures are identified in HEC 14.

7.3.5.5 Recommended Methods

Design and evaluate the performance of energy dissipators according to the methods presented in HEC 14. HEC 14 also contains procedures for estimating scour hole dimensions at pipe outlets.

7.3.5.6 Reporting

The design of the energy dissipators for culvert, pipe, or channel outlets should be supported by documentation containing, at a minimum, the following information:

- Project identification
- Location of proposed installation
- Hydraulic design calculations

7.3.5.7 Plans

For the location and design of simple riprap outlet protection, prepare plans showing all details necessary to construct the improvements according to the hydraulic design, including the following:

- Dimensions and extent of riprap
- Gradation
- Bedding and Filter Material or Geotextile
- Grading or slope details

For the location and design of energy dissipators, prepare plans showing all details necessary to construct the improvements according to the hydraulic design, including the following:

- Location
- Structural Details
- Additional riprap channel protection

7.3.6 ALTERNATIVE PIPE MATERIALS

It is Federal Lands Highway policy to specify alternative drainage pipe materials on all projects where feasible and to comply with the provisions of 23 CFR 635.411. All suitable pipe materials, including reinforced concrete, steel, aluminum, and plastic will be considered as alternatives for all new cross culverts and storm drain pipes on Federal Lands Highway projects. Not all pipe materials are appropriate or applicable for all storm drain applications. The design of alternative drainage pipe materials should consider functionally equivalent performance in three areas: structural capacity, durability and service life, and hydraulic capacity. The service life and hydraulic capacity issues are addressed in this section.

7.3.6.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 635.411	Title 23 CFR 635.411 in 23 CFR 635D, *General Material Requirements*
2.	FHWA-FLP-91-006	FHWA, *Durability of Special Coatings for Corrugated Steel Pipe*, FHWA Publication FLP-91-006
3.	FHWA-RD-97-140	FHWA, *Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe*, FHWA Publication RD-97-140, 2000.
4.	AASHTO HDG Vol. IV	AASHTO Highway Drainage Guidelines, *Volume IV – Hydraulic Design of Culverts*
5.	AASHTO HDG Vol. XIV	AASHTO Highway Drainage Guidelines, *Volume XIV – Culvert Inspection and Rehabilitation*
6.	Caltrans Chapter 850	California Department of Transportation Highway Design Manual, Chapter 850 – *Physical Standards*

7.3.6.2 Design Standards

7.3.6.2.1 Service Life

Design cross culvert and storm drain pipes with a minimum maintenance-free service life of 50-years, regardless of pipe material selection. A shorter service life may be used for temporary installations, and a longer service life may be considered in unusual situations.

7.3.6.2.2 Minimum Pipe Classification

Use Class II as the minimum for all reinforced concrete pipes. Determine appropriate pipe class from FLH fill height FLH Standard Drawings.

Use a minimum wall thickness of 1.63 mm [0.0625"] for all steel and aluminum pipes. The appropriate minimum structural metal thickness will be determined from approved FLH fill height tables.

7.3.6.3 Design Guidance

7.3.6.3.1 Service Life

The durability and service life of a storm drain pipe is directly related to the environmental conditions encountered at the site and the type of materials and coatings from which the pipe was fabricated. The two primary causes of early failure in drainage pipe materials are corrosion and abrasion.

Corrosion gradually wears away at the pipe walls by chemical action, and can occur from both the soil and water sides of the pipe. Abrasion wears away at the interior pipe wall by friction from suspended or bed-load sediment.

7.3.6.3.2 Data Collection

Corrosion

Representative pH and resistivity determinations are required in order to specify pipe materials capable of providing a maintenance-free service life. Samples are taken in accordance with the procedures described in AASHTO T 288 and T 289. Samples should be taken from both the soil and water side environments to ensure that the most severe environmental conditions are selected for determining the service life of the drainage pipe. Soil samples should be representative of backfill material anticipated at the drainage site. Avoid taking water samples during flood flows or for two days following flood flows to ensure more typical readings. In locations where streams are dry much of the year, water samples may not be possible or necessary. In areas of known uniform pH and resistivity readings, a random sampling plan may be developed to obtain the needed information.

In corrosive soil conditions where water side corrosion is not a factor, consider specifying less corrosive backfill material to modify the soil side environment. The mitigating effect of the specified backfill should be taken into account in making alternative pipe materials selections in situations where the soil side conditions control the design.

Abrasion

An estimate of the potential for abrasion is required in order to determine the need for invert protection. Four levels of abrasion are referred to in this guidance and the following guidelines are established for each level:

- *Level 1*. Nonabrasive conditions exist in areas of no bed load and very low velocities. This is the condition assumed for the soil side of drainage pipes.

- *Level 2*. Low abrasive conditions exist in areas of minor bed loads of sand and velocities of 1.5 m/s [5 ft/s] or less.

- *Level 3*. Moderate abrasive conditions exist in areas of moderate bed loads of sand and gravel and velocities between 1.5 m/s and 4.5 m/s [5 ft/s and 15 ft/s].

- *Level 4*. Severe abrasive conditions exist in areas of heavy bed loads of sand, gravel, and rock and velocities exceeding 4.5 m/s [15 ft/s].

Abrasion levels are intended as guidance to help the engineer consider the impacts of bed-load wear on the invert of pipe materials. Sampling of the streambed materials is not required, but visual examination and documentation of the size of the materials in the stream bed and the average slope of the channel will give the designer guidance on the expected level of abrasion. Where existing culverts are in place in the same drainage, the conditions of inverts should also be used as guidance. The expected stream velocity should be based upon a typical flow (i.e., 2-year flow and less) and not a 10-or 50-year design flood.

7.3.6.3.3 Reinforced Concrete Pipe

Reinforced concrete pipe (AASHTO M 170M) is typically specified as an alternative whenever environmental conditions permit. The appropriate pipe class is determined from approved FLH fill height FLH Standard Drawings. If the following guidance on corrosion and abrasion limitations is used, reinforced concrete pipe can be assumed to have a minimum service life of 50 years.

Corrosion

Reinforced concrete pipe should not be specified for extremely corrosive conditions where the pH is less than 3.0 and the resistivity is less than 300 Ω-cm [118 Ω-in]. Where the pH is less than 4.0, or the pipe is exposed to wetting and drying in a salt or brackish water environment, protective coatings (e.g., epoxy resin mortars, poly vinyl chloride sheets) should be used. When the sulfate concentration is greater than 0.2% in the soil or 2,000 parts per million in the water, Type V cement should be specified. When the sulfate concentration is greater than 1.5% in the soil or 15,000 parts per million in the water, Type V cement should be used with a sulfate resistant pozzolan. A higher cement ratio may also be used (e.g., AASHTO Class V pipe design).

Abrasion

On installations in severe abrasive environments, consider using seven or eight sack concrete or increasing the cover over the reinforcing steel.

7.3.6.3.4 Steel Pipe with Metallic Coatings

Steel pipe will typically be specified as an alternative when the environmental conditions permit. The appropriate minimum structural metal thickness is determined from approved FLH fill height tables. Federal Lands Highway design policy assumes that steel pipe will provide a useful, maintenance-free service life for a period of time beyond the point of first perforation. This assumes an acceptable risk for most Federal Lands Highway projects, but at locations with erodible soils, large traffic volumes, or high fills where replacement or repair would be unusually difficult or expensive, consider increasing the steel plate by one standard thickness. In unusual situations where very high fills and severe abrasion are combined, or where other environmental concerns would make replacement of a pipe culvert very costly or impractical, consider using a pipe one size larger in diameter to permit re-lining in the future by insertion of another pipe.

The following types of steel pipe with metallic coatings are considered as alternatives on Federal Lands Highway projects:

- Galvanized steel (AASHTO M 218)
- Aluminum coated steel (Type 2) (AASHTO M 274)

Corrosion

Under non-abrasive and low-abrasive conditions, the service life of steel pipe with metallic coatings may be determined based upon corrosion (i.e., pH and resistivity) factors determined from Exhibit 7.3–B, which shows the relationship between service life and corrosion for plain galvanized steel pipe. It has been adapted from the California Department of Transportation "Method for Estimating the Service Life of Steel Culverts," California Test 643. The curves have been modified to show the expected average service life of pipe with a steel thickness of 1.63 mm [0.0625"] assuming a useful, maintenance-free service life 25 percent longer than the number of years to first perforation. Under moderate and severe abrasive conditions, abrasion protection must also be considered.

Under nonabrasive and low abrasive conditions, the metal thickness of galvanized and aluminum coated steel (Type 2) alternatives should be determined from Exhibit 7.3–B based on the resistivity and pH of the site. The minimum metal thickness of steel pipe, as determined from FLH standard fill height tables, may have to be increased, or the additional life of a protective coating may have to be added, in order to provide a 50-year service life. The results included in FHWA-FLP-91-006 indicate that within the environmental range of 5.0 through 9.0 pH and resistivity equal to or greater than 1500 Ω-cm [590 Ω-in], aluminum coated steel (Type 2) can be expected to give a service life of twice that of plain galvanized pipe.

Exhibit 7.3–B can be used to determine various combinations of increased thicknesses, aluminum coated steel (Type 2), and protective coatings to achieve a 50-year service life, but in no case may the metal thickness specified by the structural requirements be reduced.

Abrasion

Under nonabrasive and low abrasive conditions, the metal thickness of the galvanized, galvalume, and aluminum coated steel alternatives, as determined from Exhibit 7.3–B, should be used.

On installations in moderate abrasive environments where protective coatings are not required for corrosion protection, the thickness of the metal should be increased by one standard metal pipe thickness determined from the diagram for average service life of plain galvanized culverts (see Exhibit 7.3–B) or invert protection should be provided. Invert protection may consist of bituminous coating with invert paving with bituminous concrete, Portland cement concrete lining, installation of metal plates or rails, or velocity reduction structures.

On installations in severe abrasive environments where protective coatings are not required for corrosion protection, the thickness of the metal should be increased by one standard metal pipe thickness determined from the diagram for average service life of plain galvanized culverts (see Exhibit 7.3–B) and invert protection should be provided. Invert protection may consist of installation of metal plates or rails, or velocity reduction structures.

Protective coatings are not suitable for corrosion protection in moderate-abrasive and severe abrasive locations. Metal pipes should not be specified in moderate and severe abrasive environments where coatings are required to protect against water-side corrosion.

7.3.6.3.5 Non-Metallic Protective Coatings for Steel Pipe

Protective coatings may be used to provide additional protection from corrosion or abrasion resulting in an extended service life. Coatings to protect against corrosion may only be used in non-abrasive and low abrasive environments.

The additional service life noted below in **bold** for each type of protective coating, for corrosion protection, are from Part V of FHWA-FLP-91-006. The added service is applicable only to non-abrasive and moderate abrasive conditions. All of the following types of steel pipe with non-metallic coatings are considered as alternatives on Federal Lands Highway projects:

Bituminous coating

Bituminous coatings (AASHTO M 190) can be expected to add **10 years** of service to the water side and **25 years** life to the soil side service life of pipe as determined from Exhibit 7.3–B. Bituminous-coated pipe should not be used in low abrasive environments.

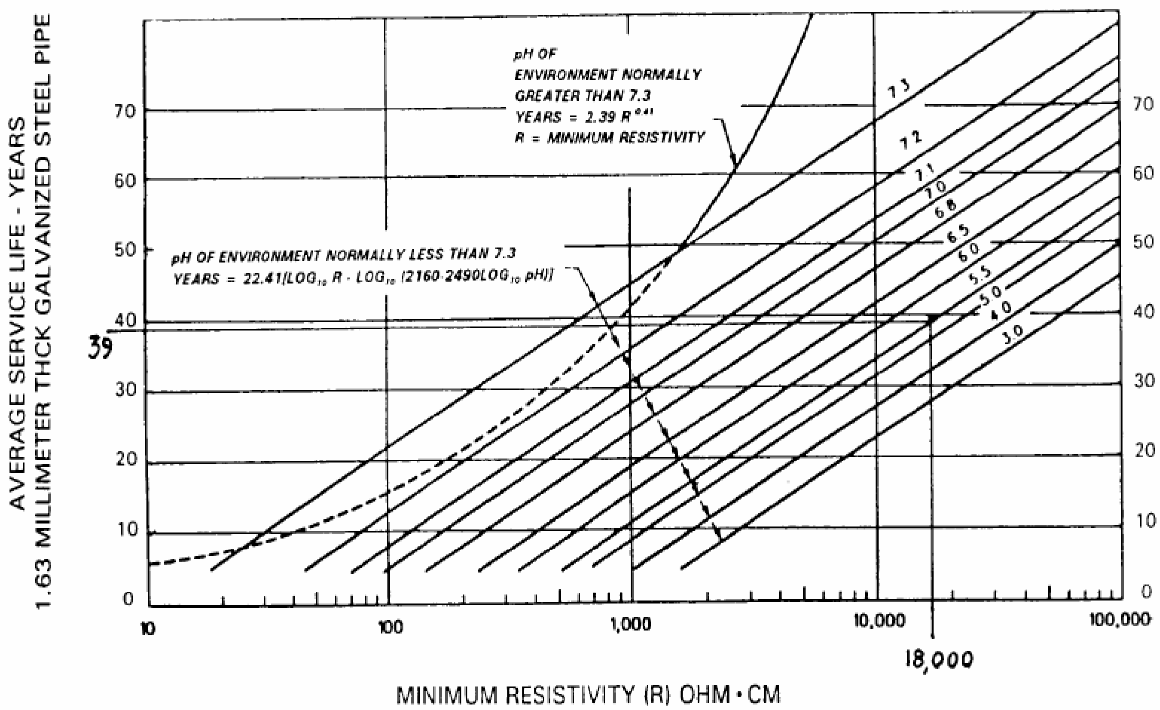
Bituminous paving and coating

Bituminous paved invert with bituminous coatings (AASHTO M 190) can be expected to add **25 years** life to water side locations. Under moderate abrasive conditions, bituminous paved pipe may be used for invert protection where corrosion protection is not required.

Exhibit 7.3–B ESTIMATING STEEL PIPE SERVICES LIFE

(Metric)

FEDERAL LANDS HIGHWAY
MODIFIED CALIFORNIA METHOD CHART
FOR ESTIMATING SERVICE LIFE OF
PLAIN GALVANIZED STEEL CULVERTS



Service Life Estimation Chart
For Average Service Life of Plain Galvanized Culverts

THICKNESS FACTORS

THICKNESS, mm	1.32	1.62	2.00	2.77	3.50	4.27
FACTOR *	0.8	1.0	1.2	1.7	2.2	2.6

* MULTIPLY THE AVERAGE SERVICE LIFE BY THE THICKNESS FACTOR

Notes:

1. The curves in this Chart are based on the data in FHWA-FLP-91-006 which uses the factors in California Test 643, "Method for Estimating the Service Life of Steel Culverts". These factors increased the estimated service life by 25% after first perforation.
2. The Chart has also been modified to reflect a minimum metal thickness of 1.63 mm.
3. Under conditions with pH between 5 and 9, and above $R \geq 1500$, the average service life determined for plain galvanized culverts should be multiplied by 2.0 for Aluminum coated steel, (Type 2).

Concrete lining

Concrete lining (ASTM A 849) can be expected to add **25 years** of service life. Due to the natural cracking of concrete, the concrete lining should be applied over an asphalt coating if corrosion protection is needed. Under moderate abrasive conditions, concrete lined pipe may be used for invert protection where corrosion protection is not required.

Polymer coating

Ethylene Acrylic Acid Film coatings (AASHTO M245 and M246) should provide an additional **30 years** service life with a 0.25 mm [0.009"] thickness.

Aramid fiber bonded coating.

Only limited data is available for the service life of aramid fiber bonded coated (ASTM A 885) and epoxy coated pipes. No additional service life is currently credited with this policy.

7.3.6.3.6 Aluminum Alloy Pipe

Aluminum alloy pipe (AASHTO M 196M) will typically be specified as an alternative when environmental conditions permit. The appropriate minimum structural metal thickness is determined from approved FLH fill height tables. Within the following limits of corrosion and abrasion, aluminum alloy pipe can be assumed to have a service life of 50 years. Additional service life may be achieved where required by abrasion with the addition of protective coatings or additional metal thickness as discussed below:

Corrosion

An aluminum alloy should be allowed if the pH is between four and nine and the resistivity is greater than 500 Ω-cm [200 Ω-in]. An aluminum alloy alternative can also be considered for use in salt and brackish environments when embedded in granular, free draining material.

Abrasion

On installations in non-abrasive and low-abrasive environments, abrasion protection is not required.

On installations in moderately abrasive environments, the thickness should be increased by one standard metal thickness or invert protection should be used. Invert protection may consist of bituminous coating and invert paving with bituminous concrete or Portland cement concrete, installation of metal plates or rails, or velocity reduction structures.

On installations in severe abrasive environments, the thickness of the metal should be increased by one standard metal pipe thickness from that determined for low-abrasive conditions and invert protection should be provided. Invert protection may consist of installation of metal plates or rails or velocity reduction structures.

7.3.6.3.7 Plastic Pipe

Polyethylene and polyvinyl chloride plastic pipe may be specified as alternatives for pipe diameters and minimum resin cell classifications shown in the AASHTO's *Standard Specifications for Highway Bridges*, Division I Design, Section 18, Soil Thermoplastic Pipe Interaction Systems. The thickness of the plastic alternatives must meet the structural requirements of AASHTO's *Standard Specifications*. The assumed service life of plastic pipe designed in accordance with AASHTO Section 18 is 50 years. The maximum allowable fill heights for pipe materials listed below is determined from approved FLH standard fill-height tables which include the following plastic pipe materials:

- Smooth wall polyethylene (ASTM F 714)
- Corrugated polyethylene (AASHTO M 294)
- Ribbed polyethylene (ASTM F 894)
- Smooth wall polyvinyl chloride (AASHTO M 278 and ASTM F 679)
- Ribbed polyvinyl chloride (AASHTO M304 and ASTM F 794)

<u>Corrosion</u>

Plastic alternatives may be specified without regard to the resistivity and pH of the site.

<u>Abrasion</u>

Under nonabrasive and low-abrasive conditions, polyethylene and polyvinyl chloride alternatives should be allowed. Plastic alternatives should not be used under moderate and severe abrasive conditions without invert protection.

<u>Maximum Size</u>

Limit the size of plastic pipe to 1200 mm [48”] under mainline roads.

The locations selected for use of plastic pipes should address partner agency concerns of possible damage due to fire, ultraviolet sunlight, and rodents.

7.3.6.4 Recommended Methods

Design and evaluate the design service life for galvanized steel culvert and storm drain pipes by the Modified California method presented in Exhibit 7.3B. Refer to FHWA-RD-97-140, *Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe,* for design guidance on aluminized material.

7.3.6.5 Reporting

Documentation of the design service life of culvert and storm drain pipes should be included in the design reporting.

7.3.6.6 Plans

For culvert and storm drain pipes, include information on pipe material, size, class, gauge, and any special coatings in the Plan Drainage Summary.

7.4 RIVER HYDRAULICS

7.4.1 FLOODPLAIN ENCROACHMENTS

When a Federal Lands Highway project involves an encroachment on a base (100-year) floodplain, the location and design of the project must comply with FHWA Policy 23 CFR 650A, *Location and Design of Encroachments on Flood Plains*. This section identifies the standards and criteria arising from this policy and their applicability (see Section 7.4.1.2). It also provides guidelines for ensuring compliance.

Typically, one is referring to the standards and criteria of this section because of direction received from another section within this chapter that involves floodplain encroachments (e.g., bridges, culverts, etc.). Such direction is given when a proposed project includes a new or expanded encroachment on a base floodplain regulated by the Federal Emergency Management Agency (FEMA), or contains the potential for adversely impacting private property or insurable buildings on or near a base floodplain, as defined below.

7.4.1.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 650A	FHWA Policy 23 CFR 650 Subpart A, *Location and Design of Encroachments on Flood Plains*
2.	NS 23 CFR 650A	Federal-Aid Policy Guide, Non-regulatory Supplement to Title 23 CFR 650 Subpart A, Attachment 2, *Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency (FEMA)*
3.	44 CFR Subchapter B	Code of Federal Regulations, 44 CFR Sections 59 to 77, *National Flood Insurance Program (NFIP)*
4.	HEC 17	FHWA HEC 17, *The Design of Encroachments on Flood Plains using Risk Analysis*
5.	AASHTO MDM Chap. 2	AASHTO Model Drainage Manual, *Chapter 2 - Legal Aspects, Section 2.5 – National Flood Insurance Program*
6.	AASHTO HDG Vol. I	AASHTO Highway Drainage Guidelines, *Volume I – Hydraulic Considerations in Highway Planning and Location*

7.4.1.2 Standard Practices

If a proposed project includes a new or expanded encroachment on a FEMA regulated base floodplain, or contains the potential for adversely impacting private property or insurable buildings on or near the base floodplain, the design will comply with the FEMA standards and criteria used to administer the National Flood Insurance Program in accordance with 23 CFR 650A, Section 650.115(a)(5), in addition to the other applicable standards and criteria contained within this chapter. These standards and criteria apply as minimums, regardless of the hydraulic structure proposed or the encroachment type (i.e., transverse or longitudinal).

For the purposes of this chapter, adverse impacts to private property or insurable buildings will be defined, respectively, as follows:

- Damage to existing real or fixed private property, caused directly by the project during a 100-year flood, over the service life of the project

- Increased 100-year water-surface elevations that impact existing, insurable buildings

If a FEMA map revision request is anticipated, project management will be notified immediately to determine how the coordination process will be handled, and how a revision will be developed (e.g., development and evaluation of alternatives). The revision request will receive concurrence from Federal Lands Highway, the project partner, and the local floodplain administrator.

7.4.1.3 Design Standards

7.4.1.3.1 Capacity Design

<u>Design Flood</u>

Design the encroachment using the 100-year (base) flood.

<u>Check Flood</u>

Use the overtopping flood for evaluating encroachment impacts. If the overtopping flood is less than the base flood, or so large as to not be practicable, then use the greatest flood that may be reasonably estimated to pass through the structure, such as the 500-year flood, as the check flood.

7.4.1.3.2 Survey and Mapping

When survey is needed to quantify hydraulic impacts, refer to Chapter 5 for standards on survey for floodplain mapping.

7.4.1.4 Design Criteria

7.4.1.4.1 FEMA Regulated Base Floodplain

With Detailed Study (i.e., FIRM or FBFM map, report, and modeling information available)

- Floodway defined – Do not encroach upon floodway (bridge piers excepted)
- No Floodway defined – Do not exceed 0.3 m [1'] rise (or local standard if more strict)

No Detailed Study (i.e., FHBM map available)

- Do not exceed 0.3 m [1'] rise based on own pre- and post-project water-surface profile models

7.4.1.4.2 Unregulated Base Floodplain

Do not exceed 0.3 m [1'] rise based on own pre- and post-project water-surface profile models.

7.4.1.5 Design Guidance

7.4.1.5.1 Floodplains Identified on NFIP Maps

Where National Flood Insurance Program (NFIP) maps are available, their use is mandatory in determining whether the project will involve an encroachment upon a base floodplain. If a particular encroachment cannot be designed to meet FEMA standards and criteria, then coordination with FEMA is necessary, as described in the Non-Regulatory Supplement, Attachment 2 (NS 23 CFR 650A).

An encroachment upon a base floodplain identified on NFIP maps, for which a regulatory floodway has been established, will be considered consistent with NFIP standards and criteria if the highway and structure components are kept outside the regulatory floodway. An encroachment having components other than bridge piers within the regulatory floodway should be avoided wherever practicable.

If an encroachment upon a regulatory floodway cannot be avoided, it will be designed to cause no rise in the floodway profile. The floodplain administrator of each affected local community must be contacted and must concur that the project, as designed, will cause no rise in the base flood profile. An example of this is a project to replace an existing low-water crossing in a regulatory floodway with higher road profile and a bridge. Unless the new bridge is built with both abutments outside the floodway, then the higher-profile embankment leading to the bridge constitutes an encroachment upon the floodway.

An encroachment upon a base floodplain identified on NFIP maps, for which no regulatory floodway has been established, will be designed to cause no more than 0.3 m [1.0'] rise in the base flood profile, unless more strict local criteria are applicable and appropriate. Many states, counties, and municipalities have ordinances mandating more restrictive criteria than those listed above. It is imperative to determine the extent and nature of state, county, and municipal floodplain regulations early in the reconnaissance and scoping phase of the project.

7.4.1.5.2 Coordination with FEMA

Coordination with FEMA is required when the project includes an encroachment upon a base floodplain identified by NFIP maps and the applicable standards and criteria cannot be satisfied. Typically, the coordination includes a map revision request in order to incorporate changes to the effective water-surface profile model; increases to the base flood profile, floodway profile, or base flood inundation limits; or to revise the regulatory floodway encroachment limits.

Whenever a project requires a physical map revision, a Conditional Letter of Map Revision (CLOMR) will be submitted to FEMA and their approval received prior to construction. Once the construction is completed, a survey may be required to verify that the project was constructed as represented in the CLOMR request, and a Letter of Map Revision (LOMR) will typically be requested.

When a project includes an encroachment upon a regulatory floodway and the no-rise criteria cannot be met, NFIP regulations mandate that a CLOMR request pursuant to 44 CFR Subchapter B Section 65.12 (*Revision of flood insurance rate maps to reflect base flood elevations caused by proposed encroachments*) be submitted to and approved by FEMA prior to construction of the project. When an encroachment meets FEMA/local standards and criteria on a base floodplain with a detailed regulatory study, FEMA, or the local floodplain administrator may request to obtain a copy of the updated water-surface profile model and study report.

Failure to comply with these regulations can lead to NFIP program sanctions against the affected local community.

7.4.1.5.3 Role of Community Floodplain Administrators

The responsibility for enforcing floodplain regulations lies with the local community (state, county, or municipality) having land use jurisdiction. This is true for floodplains identified by NFIP maps and those not included in the NFIP. The regulations of relevant local communities must be examined early in the reconnaissance and scoping phase of the project. Coordination with FEMA on a given project usually implies and includes coordination with the floodplain administrator of the local community. If a project requires revisions to the NFIP maps, for example, the revision request must be approved by the community floodplain administrator. It is important, therefore, to identify the names and contact information of the floodplain administrators of the communities affected by a project early, and to remain in frequent contact with the floodplain administrators as the project progresses.

7.4.1.6 Reporting

The reporting requirements for this section will be consistent with those applicable to the encroachment or structure type, as described in other sections of this chapter.

7.4.1.7 Plans

Show the following information in the project plans for encroachment structures:

- The magnitude, approximate probability of exceedance and, at appropriate locations, the water-surface elevations associated with the overtopping flood or the largest flood that may be reasonably estimated, such as the 500-year flood

- The magnitude and water-surface elevation of the base flood, if larger than the overtopping flood

7.4.2 SCOUR AND STREAM STABILITY

Any crossing of, or encroachment onto a natural river, stream or floodplain by a highway facility calls for an evaluation of the scour potential and the stability of the stream. This section identifies key technical references for assessment of scour and stream stability and provides some specific guidance for application to Federal Lands Highway projects.

7.4.2.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	T 5140.23	Technical Advisory T 5140.23, *Evaluating Scour at Bridges*, 1991
2.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*, FHWA Publication NHI-01-004, 2001.
3.	HEC 18	FHWA HEC 18, *Evaluating Scour at Bridges*, FHWA Publication NHI-01-001, 2001
4.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*, FHWA Publication NHI-01-002, 2001
5.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*, FHWA Publication NHI-01-002, 2001
6.	FHWA RD-86-126	FHWA Report Number RD-86-126, *Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping*, 1987
7.	NCHRP 533	NCHRP Report 533, *Handbook for Predicting Stream Meander Migration*, 2004

7.4.2.2 Standard Practices

The potential for scour and stream instability will be considered when designing highway facilities that interface with natural rivers, streams, or floodplains. Accordingly, an assessment or evaluation of the potential for scour and stream instability will be conducted at a level commensurate with the risk of damage to the facility. The design will protect the highway facility from stream instability and scour at an appropriate level, in accordance with the applicable sections of Chapter 7 for each type of drainage structure or facility.

7.4.2.3 Design Standards and Criteria

Select the design standards and criteria for stability against scour and stream instability in accordance with the applicable sections of Chapter 7, as referenced below:

- Bridge foundations, see Section 7.4.3
- Embankment stability in overtopping, see Section 7.4.3
- Longitudinal embankments, see Section 7.4.4
- Retaining walls, see Section 7.4.5
- Low water crossings, see Section 7.4.6
- Channel changes, see Section 7.4.7
- Scour and stream instability countermeasures, see Section 7.4.8

7.4.2.4 Design Guidance

7.4.2.4.1 General Approach to Stream Stability and Scour Analysis

HEC 20 describes the systematic analysis approach as having three levels, progressing from simple concepts and qualitative assessment to detailed numerical and physical modeling:

Level 1: Application of simple geomorphic concepts and other qualitative assessment methods.

Level 2: (if necessary after Level 1 assessment) Application of basic hydrologic, hydraulic, and sediment transport engineering concepts.

Level 3: (if necessary after Level 2 analysis) Application of detailed numerical or physical modeling studies.

In the majority of cases, the Level 2 analysis will provide a reliable, somewhat conservative evaluation of the potential threat from scour and stream instability. The design of the facility can then account for and protect against the threat. In such cases a Level 3 analysis is not required. Certain circumstances may justify a Level 3 study. Some examples are listed below:

- The hydraulics of the site are too complex for one-dimensional analysis and a two-dimensional model is required (see Section 7.4.3)

- The scour estimates are too conservative to be practicably accommodated in design and refined approaches are needed, such as:

◊ Accounting for potential duration-limited scour
◊ Conducting physical modeling to represent site-specific conditions

- The scour will potentially be arrested or reduced by a scour-resistant horizon (e.g. bedrock, sandstone, shale, or stiff clay) and an assessment of the erodibility of the material is justified

- The degradation potential may be too complex for simple analysis and a sediment transport modeling study is justified

7.4.2.4.2 Scour Components

The analysis of scour potential at a bridge or other highway facility should consider several scour components, generalized as follows:

- Long-term bed elevation change
- General scour
- Contraction scour
- Local scour

Long-Term Bed Elevation Change

Aggradation and degradation are the vertical raising and lowering, respectively, of the streambed over relatively long distances and time frames. Such changes can be the result of both natural and man-induced changes in the watershed. Long-term bed change can occur in perennial streams that flow year round and in ephemeral desert arroyos. Its progression can take many forms, such as headcuts (vertical channel bed discontinuities) migrating upstream, progressive incision of a low-flow channel, or gradual lowering or raising across the streambed over time. Evaluation of the potential for long-term aggradation or degradation must consider the effects of a range of flow conditions over a long period of time, rather than focusing solely on the effect of a single event. HEC 20 provides extensive guidance on evaluating the potential for long-term bed change.

General Scour

General scour is a lowering of the channel bed elevation due to the natural downstream sediment transport capacity of a stream. Physical changes to the stream environment are not required to produce general scour. Common examples of general scour that occurs naturally are scour at the outside of a channel bend, scour at a confluence of two streams, and scour that occurs due to a change in stream gradient. For design purposes, general scour is usually evaluated on an event-specific basis, considering one or more flood conditions. Guidance on evaluating the potential for general scour is available in HDS 6, HEC 18, HEC 20, and HEC 23.

Contraction Scour

Contraction scour is a specific type of general scour that results when the flow area is constricted, for example when a bridged waterway has less flow area under the bridge than upstream. Its effects are usually localized in the vicinity of the constriction. Contraction scour is event-specific and is usually analyzed for one or more flood conditions (e.g. the stability design flood and check flood). HEC 18 provides detailed guidance on evaluating contraction scour.

Local Scour

The scour caused by, and in the immediate vicinity of an obstruction such as a bridge pier or abutment is referred to as local scour. Local scour can also be caused by other localized conditions, such as high-velocity flow impinging on a wall, sudden drops, or scour at the tip of a spur. Local scour is usually evaluated on an event-specific basis considering one or more flood conditions (e.g. the stability design flood and check flood). Local scour at bridges can be evaluated using the guidance of HEC 18. The evaluation of local scour in other contexts is aided by the guidance in HDS 6, HEC 20, and HEC 23.

7.4.2.4.3 Lateral Migration

Lateral migration of the stream channel is another potential long-term threat to highway facilities. Lateral migration can undermine bridge abutments, piers, embankments, retaining walls, and other facilities that were originally located at the top of the channel bank or set back from the channel. If lateral migration is a potential threat to a highway facility, the design should accommodate the channel migration by providing adequate foundation depth or should prevent the migration by the use of appropriate countermeasures. HEC 20 and NCHRP 533 provide guidance on evaluating and predicting meander migration. HEC 23 provides guidance on designing stream instability countermeasures.

7.4.2.4.4 Bridge Scour

Reference Elevations

Use the lowest channel bed elevation as the pier scour reference elevation for all main channel bridge piers, unless non-erodible material allows otherwise. Use the main channel hydraulic input variables and reference elevation for piers located outside but near the main channel, when the potential for channel migration exists. Use the lowest channel bed elevation as the abutment scour reference elevation for abutments located in or adjacent to the main channel, unless non-erodible material allows otherwise.

Debris at Piers

Consider the potential for debris to accumulate on the piers during a flood. If the potential is moderate to high, account for the debris by artificially increasing the pier width in the scour calculations or by some other rational approach.

Abutment Scour

If accommodating the computed local abutment scour depth in the foundation design is not practicable, consider using an abutment scour countermeasure to prevent the formation of the local scour. If a countermeasure is used, then design the abutment foundation to accommodate the sum of the estimated contraction scour and long-term degradation.

7.4.2.4.5 Incipient Motion

Chapter 5 of HEC 18 provides a critical velocity equation to determine whether the scour conditions are live-bed or clear-water. This equation is generally reliable for sand-bed channels. It is not always reliable for coarse bed material such as gravel or cobbles. To determine whether clear-water or live-bed conditions apply at a site with coarse bed material, consider developing a modified critical velocity equation using the detailed derivation data provided in Appendix C of HEC 18.

7.4.2.4.6 Sediment Transport Modeling

Sediment transport modeling (sediment routing analyses) is a Level 3 approach that is warranted only rarely. It is an appropriate approach when the Level 2 methods are producing results that are obviously too conservative. When sediment transport modeling is being considered, the context is usually a perceived threat of the long-term degradation component of total scour.

Commonly used sediment transport modeling programs include:

- BRI-STARS, available from the FHWA
- HEC-6, available from the U.S. Army Corps of Engineers

When undertaking sediment transport modeling, the engineer must take care to calibrate the model and should apply extensive engineering judgment to the interpretation and use of the results.

7.4.2.5 Recommended Methods

The methodologies described in HEC 18 and HEC 20 provide a systematic approach to evaluating scour potential and assessing stream instability and should be followed wherever practicable.

7.4.2.6 Reporting

In addition to the reporting requirements described in Section 7.1.11, the following items are required when scour evaluation, stream stability analysis, or sediment transport analysis have been performed.

- Sediment gradation curves
- Scour components investigated
- Scour equations/approach used
- Hydraulic input variable values (e.g. velocity, depth, and angle of attack) for scour calculations

- Sediment transport modeling assumptions, if applicable
- Calibration results
- Sediment sampling locations and frequencies, if sediment transport calculations were performed
- Sediment transport function used, if sediment transport calculations were performed
- Findings, conclusions, and recommendations

7.4.3 BRIDGED WATERWAYS

This section applies to the hydraulic design of waterway crossings involving bridges. For the purposes of this section, bridges are defined as structures that consist of a superstructure or deck supported by abutments, with or without piers, usually with an open bottom. This section typically does not apply to closed-bottom culverts even if their total span is greater than or equal to 6.1 m [20'].

7.4.3.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 650A	Code of Federal Regulations, Title 23, Part 650 Subpart A, *Location and Hydraulic Design of Encroachments on Flood Plains*
2.	NS 23 CFR 650A	Federal-Aid Policy Guide, Non-regulatory Supplement to Title 23 CFR 650 Subpart A, Attachment 2, *Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency (FEMA)*
3.	T 5140.23	Technical Advisory T 5140.23, *Evaluating Scour at Bridges*, 1991
4.	HDS 1	FHWA HDS 1, *Hydraulics of Bridge Waterways*, 1978
5.	HEC 18	FHWA HEC 18, *Evaluating Scour at Bridges*
6.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*
7.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
8.	HEC-RAS	USACE HEC-RAS, *Hydraulic Reference Manual*

9.	AASHTO HDG Vol. VII	AASHTO Highway Drainage Guidelines, *Volume VII – Hydraulic Analysis for the Location and Design of Bridges*
10.	AASHTO HDG Vol. VIII	AASHTO Highway Drainage Guidelines, *Volume VIII – Hydraulic Aspects in Restoration and Upgrading of Highways*
11.	Guide to Bridge Hydraulics	Transportation Association of Canada, *Guide to Bridge Hydraulics*

7.4.3.2 Standard Practices

7.4.3.2.1 Floodplain Encroachments

If a waterway crossing constitutes a new or expanded encroachment on a base (100-year) floodplain that is regulated by FEMA or potentially creates an adverse impact to private property or insurable buildings, reference Section 7.4.1 for details on appropriate design policy, standards, and criteria, as well as guidance on FEMA coordination, if required.

7.4.3.2.2 Existing Bridges

Known Scour Problems

An appropriate scour analysis will be conducted on any existing bridge within the project limits that has known scour problems or concerns and has not had such an analysis conducted previously. This will be done regardless of the project type. The bridge owner will be informed of the results of the scour analysis and asked to update National Bridge Inventory, Item 113, accordingly.

Substantial Rehabilitation

To identify hydraulic consequences of proposed work, conduct full capacity and stability analyses on any bridge that is to be substantially rehabilitated. Substantial rehabilitation is defined as the addition or modification of a foundation element, any work that reduces the hydraulic opening of the bridge, or any work that changes the flow distribution at the crossing. Incorporate results of the analyses into the project, as necessary, to meet current standards. Scour countermeasures will be designed and installed as necessary to achieve the foundation stability design standard either before or as a part of the rehabilitation project. Existing bridge piers will be considered stable against scour if protected with a suitably designed countermeasure.

7.4.3.2.3 Scour Countermeasures

Scour countermeasures will not be used to protect or reduce scour at new bridge piers.

7.4.3.3 Design Standards

The hydraulic design of bridged waterways requires the definition of standards for capacity and foundation stability. The following standards apply to bridges on both High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

7.4.3.3.1 Capacity Design

Design Flood

Design bridges to convey the 50-year flood with appropriate freeboard. Freeboard is defined as the vertical clearance between the design-flood water surface and the low chord of the bridge. The required height of freeboard is defined in Section 7.4.3.4.

Check Flood

Use the greater of the 100-year flood or the overtopping flood as the standard check flood for water surface increase caused by the crossing.

The overtopping flood is defined as the discharge rate at which water would begin to flow over the top of the bridge deck or the approach roadways. If overtopping is not practicable then use the greatest flood that may be reasonably estimated to pass through the bridge, such as the 500-year flood.

Temporary Bridges

The capacity design standard for temporary bridges depends on the roadway classification. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *High-Standard Roadways:* Design temporary bridges to remain open to traffic during a 10-year flood.
- *Low-Standard Roadways:* Design temporary bridges to remain open to traffic during a 2-year flood.

7.4.3.3.2 Stability Design

Foundations

The stability design of a bridge foundation refers to its ability to withstand scour. Refer to Section 7.4.2 for guidance on evaluating scour at bridges.

- *Design Flood:* Design bridge foundations to withstand the estimated worst-case scour up through the 100-year flood.
- *Check Flood:* Use a larger flood, on the order of a 500-year event, as the check flood. Provide supporting documentation when using a flood frequency other than 500-year for the check flood.

<u>Approach Embankments</u>

Some bridged waterway crossings will be designed to allow overtopping of the approach embankments. In such cases design the embankment, with armoring if necessary, to remain stable in overtopping floods up to the 50-year event. Refer to Section 7.4.8 for guidance on the design of embankment protection measures.

<u>Temporary Bridges</u>

The stability design standard for temporary bridges depends on the roadway classification. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *High-Standard Roadways:* Design temporary bridges to remain stable in a 10-year flood.
- *Low-Standard Roadways:* Design temporary bridges to remain stable in a 2-year flood.

7.4.3.3.3 Survey and Mapping

When survey is needed to quantify hydraulic impacts, refer to Chapter 5 for standards on survey and mapping for bridged waterways.

7.4.3.4 Design Criteria

7.4.3.4.1 Capacity Design

<u>Freeboard</u>

- Design all bridges with a minimum freeboard of 0.6 m [2.0’]
- Design the bridge with 1.0 m to 1.5 m [3.5’ to 5.0’] of freeboard when the potential for woody debris is significant
- Design the bridge with 1.5 m to 3.0 m [5.0’ to 10.0’] of freeboard when the potential for ice flows during flood season is significant

Freeboard is defined as the vertical clearance between the design-flood water surface and the low chord of the bridge superstructure. Freeboard design provides a measure of protection to the bridge by reducing the chance of superstructure inundation and impact from floating debris.

The reference datum for measuring the freeboard is the computed water-surface elevation at the upstream face of the bridge. A bridge with a straight-grade profile will meet or exceed the freeboard criterion along its entire length. A bridge with an arched or vertical-curve profile will meet or exceed the freeboard criterion along at least half of its length.

The above freeboard criteria do not apply to temporary bridges.

7.4.3.4.2 Stability Design

Design Flood

Design bridge foundations to withstand the estimated total scour with normal geotechnical and structural safety factors. Assume that all streambed material above the total scour elevation has been removed and is not available for load bearing or lateral support.

Check Flood

Design bridge foundations to withstand the estimated total scour with geotechnical and structural safety factors of at least 1.0. Assume that all streambed material above the total scour elevation has been removed and is not available for load bearing or lateral support.

Countermeasures may be designed at abutments to prevent the formation of local scour. If a suitably designed countermeasure is used, design the abutment foundations to be stable with appropriate geotechnical and structural safety factors assuming the estimated contraction scour and any predicted degradation has occurred. Countermeasures will not be used at new bridge piers.

Refer to Section 7.4.2 for guidance on evaluating scour at bridges. Refer to Section 7.4.8 for standards, criteria, and guidance on the design of countermeasures.

7.4.3.5 Design Guidance

The hydraulic design of bridged waterways requires the investigation of numerous physical, operational, and regulatory elements during the data collection phase, which must then be applied, as appropriate, during project development. Examples of physical elements include geometrics (e.g. length, width, alignment, abutment type, pier type, deck profile, approach roadway profile); and hydraulic characteristics (e.g. freeboard, velocity, flow distribution, potential overtopping of approach roadways, scour potential, sediment transport, debris potential). Operational elements include inspection and maintenance requirements.

7.4.3.5.1 Bridge vs. Culvert

The typical channel or floodplain crossing will present an obvious need for either a culvert or a bridge based on the width of the channel or floodplain and the discharge to be conveyed. For some crossings it will be difficult to determine if a bridge or culvert is most suitable. Accordingly, the following general advantages of bridges and culverts are offered as guidance:

Bridges have the following advantages over culverts:

- Less susceptibility to clogging with sediment and debris
- The waterway increases with rising water surface until water begins to submerge the superstructure

- With properly designed foundations, the structure can accommodate streambed degradation

- Scour can potentially increase the bridge waterway capacity

- Bridge deck widening does not usually affect hydraulic capacity

- Substantially less fill volume may be required, especially for high-profile roadways

Culverts have the following advantages over bridges:

- Require less structural maintenance than bridges
- The capacity can sometimes be increased by installing improved inlets
- Usually easier and quicker to build than bridges
- Scour associated with the structure is localized and easier to control
- Upstream storage can be used to reduce peak discharge
- Profile-grade raises and widening projects sometimes can be accommodated by extending culvert ends

7.4.3.5.2 Bridge Rehabilitation

Most bridge rehabilitation projects cannot be cost-effectively designed to significantly improve the capacity of the bridge with regard to either freeboard or water-surface profile. Avoid to the extent practicable decreasing the freeboard or increasing the water-surface profile.

7.4.3.5.3 Approach Roadway Overtopping

It may be beneficial to design bridged waterway crossings to allow the flood to overtop the approach roadways. Allowing overtopping to occur at an elevation below the bridge low-chord often provides a high-capacity alternate flow path across the alignment, which leads to the following potential benefits:

- Reduces the probability of damaging pressure-flow and buoyancy conditions
- Reduces the peak velocity inside the bridge waterway
- Reduces the potential for scour to threaten the bridge foundations
- May preserve historic flow distributions
- May prevent excessive increase to the water-surface profile in large floods

The decision to allow overtopping of the approach roadways at a particular flood frequency should be supported by a risk assessment of the possible adverse consequences, which include:

- Loss of traffic serviceability during the overtopping period
- Loss of emergency vehicle access across the waterway during the overtopping period
- Possible loss of the road surface and embankment
- Potential damage to the bridge abutment by erosion of the adjacent approach embankment

If an overtopping condition is allowed, the road profile should be designed to keep the overtopping flow away from the bridge abutments.

7.4.3.5.4 Location of Bridge Abutments and Relief Openings

The bridge opening waterway should be designed so the velocity of water through the structure will not damage the highway facility or adjacent property. The acceptable velocities should be based on the characteristics of the individual site. These characteristics include the following:

- Natural stream velocities
- Bed materials
- Scour considerations (see Section 7.4.2)

Avoid placing abutments within the main channel of a natural stream or in other areas of relatively high natural flow concentration and velocity. Locate abutments and relief openings to preserve the natural flow distribution to the extent practicable Extensive guidance material can be found in the AASHTO HDG, Volume VII *Hydraulic Analysis for the Location and Design of Bridges*.

7.4.3.5.5 Pier Spacing, Shape and Orientation

Piers should be designed to minimize flow disruption and scour potential. The number of piers located in any channel should be limited to a practical minimum and piers should not be located in the main channel of small streams. Piers that are properly oriented with the flow do not significantly increase the water-surface profile. A solid pier will not collect as much debris as a pile bent or a multiple column bent. Rounding or streamlining the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier. Circular-shaped, single-column piers provide a benefit by eliminating the adverse affect of high attack angles.

7.4.3.5.6 Hydraulic Analysis and Modeling

The application of the standards, criteria, and guidance presented in this section requires a hydraulic analysis to determine the water-surface profile and flow distribution. It is necessary, at a minimum, to analyze a baseline (pre-project) condition and one or more proposed (post-project) conditions.

The most common and usually most appropriate approach to bridge hydraulic analysis is to compute a water-surface profile through 1-dimensional computer modeling. For guidance on applying 1-dimensional hydraulic models to bridged waterways, see the user documentation for HEC-RAS. Particularly useful is the HEC-RAS *Hydraulic Reference Manual*, Chapter 5 and Appendices B and D.

Some specific bridged waterway sites may not be suitable for 1-dimensional analysis. A key limitation in applying 1-dimensional models to bridge projects is the fact that flow contraction and expansion are often significant factors. The lateral components of velocity, which are ignored in 1-dimensional modeling, can be significant in the vicinity of the bridge. The water-surface elevation is assumed constant along a cross section in 1-dimensional modeling, when in reality the water surface can vary significantly along a cross section near the bridge, especially at skewed crossings.

Two-dimensional hydraulic models are formulated without the aforementioned limitations of 1-dimensional models. They are typically more difficult to develop and run, but can provide a far superior understanding of the hydraulics when the bridged waterway is complex.

7.4.3.6 Recommended Methods

7.4.3.6.1 One-Dimensional Computer Model

HEC-RAS is available from the U.S. Army Corps of Engineers (USACE).

7.4.3.6.2 Two-Dimensional Computer Model

FESWMS-2DH is available from the FHWA.

For efficient model development and post-processing, FESWMS-2DH should be used in conjunction with the graphical user interface SMS.

7.4.3.7 Reporting

Section 7.1.11 provides a general list of submittal requirements for hydraulic design projects. Specific deliverables for the analysis and design of bridged waterways will include at a minimum:

- Exhibit showing cross section locations and orientations
- A plot of the baseline water-surface profile compared to the proposed-condition water-surface profile resulting from the recommended design
- For the capacity design discharge: the water-surface elevation upstream of the bridge; the vertical clearance between the water surface and the lowest point on the low chord; and the percentage of the low chord length that meets the freeboard criterion
- The maximum velocity through the bridge opening for the capacity design discharge
- The predicted total scour depths and post-scour elevations at each substructure element (shown both graphically and in tabular form)
- Calculations for individual scour components
- Design calculations for any proposed scour countermeasures (i.e. riprap sizing calculations, etc.)

- Design sketches of any proposed scour countermeasures (i.e. abutment riprap, embankment protection, etc.)

7.4.3.8 Plans

Include the following information, as a minimum, in the bridge drawings:

- Location, geometry, and axis alignment of abutments
- Location, geometry, and axis alignment of piers
- Elevations of spread footing bases or pile tips for each abutment and pier
- Existing topography and grading contours in the plan drawing
- Water-surface elevation upstream of the bridge from the capacity design flood in the elevation drawing
- Waterway cross-section geometry in the elevation drawing
- Locations, dimensions, and details for any proposed scour countermeasures
- Magnitude, frequency, and water-surface elevation of overtopping flood or the check flood if overtopping is not possible
- Magnitude, frequency, and water-surface elevation for the 100-year flood if greater than the overtopping flood

7.4.4 LONGITUDINAL EMBANKMENTS

Hydraulic consideration is required when a longitudinal roadway encroachment on a 100-year floodplain is unavoidable. This section provides standards, criteria, and guidance for the hydraulic design of longitudinal embankments that encroach on base floodplains.

7.4.4.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*
2.	HEC 11	FHWA HEC 11, *Design of Riprap Revetments*
3.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*

4. HEC 23 — FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*

7.4.4.2 Standard Practices

7.4.4.2.1 Floodplain Encroachments

Longitudinal floodplain encroachments on 100-year floodplains should be avoided wherever practicable. If a project requires an encroachment on a 100-year floodplain that is regulated by FEMA or potentially creates an adverse impact to private property or insurable buildings, reference Section 7.4.1 for details on appropriate design policy, standards, and criteria, as well as guidance on FEMA coordination, if required.

7.4.4.2.2 Use of Scour and Stream Instability Countermeasures

The stability standards presented in this section will usually be met by using a suitably designed countermeasure to prevent damage to the embankment. Refer to Section 7.4.8 for standards, criteria, and guidance on the design of countermeasures.

7.4.4.3 Design Standards

The standards presented here apply to longitudinal embankments, with or without retaining walls that support roadways for which the profile grade is controlled by riverine water-surface elevations.

7.4.4.3.1 Capacity Design

<u>Design Flood</u>

Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *High-Standard Roadways:* Design longitudinal embankments to provide adequate freeboard in the 50-year flood.
- *Low-Standard Roadways:* Design longitudinal embankments to provide adequate freeboard in the 25-year flood.

Freeboard is defined in Section 7.4.4.4.

<u>Check Flood</u>

Use the greater of the 100-year flood or the overtopping flood as the standard check flood for evaluating impacts to private property or insurable buildings. The overtopping flood is defined as the discharge rate at which water would begin to flow over the road surface.

7.4.4.3.2 Stability Design

Design Flood

Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *High-Standard Roadways:* Design longitudinally encroaching embankments with protection as needed to remain stable in the 50-year flood.
- *Low-Standard Roadways:* Design longitudinally encroaching embankments with protection as needed to remain stable in the 25-year flood.
- *Retaining Walls:* Refer to Section 7.4.5.

7.4.4.4 Design Criteria

The criteria presented here apply to longitudinal embankments, with or without retaining walls, that support roadways for which the profile grade is controlled by riverine water-surface elevations.

7.4.4.4.1 Capacity Design

Design longitudinal floodplain encroachments with a minimum freeboard of 0.6 m [2.0']. Freeboard is defined as the vertical distance between the design water surface and the bottom of the aggregate base layer of the pavement structure.

7.4.4.4.2 Stability Design

Demonstrate that the embankment is reasonably expected to remain stable, with or without protection by countermeasures, up through the stability design flood throughout the intended service life of the embankment.

7.4.4.5 Design Guidance

7.4.4.5.1 Scour Mechanisms

Consider the following scour mechanisms in evaluating the potential scour threat to a longitudinal embankment within a 100-year floodplain:

- Long-term lateral instability in the form of channel migration
- Low-flow channel impingement, if the embankment will be located within a broad, sandy waterway that has a highly active low-flow channel meandering within it

- Bank erosion of an adjacent stream channel
- Contraction scour, if the longitudinal embankment will form a significant constriction to the waterway
- Local scour by impinging flow, if flood flows will impact the embankment at a significant angle
- Bendway scour, if the embankment will be located at the outside of a bend
- Outlet scour, if a cross drain or storm drain exits through the embankment
- Potential flanking or undermining of scour countermeasures intended to protect the embankment

7.4.4.6 Recommended Methods

Hydraulic analysis is necessary to determine the water-surface profile of the design flood for the purpose of establishing the profile grade that will provide adequate freeboard. Hydraulic analysis is also necessary to determine impacts to private property or insurable buildings. HEC-RAS modeling is an appropriate approach for most designs.

Refer to HEC 23 for approaches to estimating impinging-flow scour, bendway scour, and low-flow channel impingement scour. HEC 14 provides a method of estimating scour at cross drain and storm drain outlets.

7.4.4.7 Reporting

Section 7.1.11 provides a general list of submittal requirements for hydraulic design projects. Specific deliverables for the hydraulic analysis and design of longitudinal embankment encroachments will include at a minimum:

- A map or aerial photograph of the affected waterway reach showing the embankment location and hydraulic model cross section locations.
- A cross-section plot of the waterway showing the embankment, at the approximate point of maximum encroachment by the embankment
- A plot of the baseline water-surface profile compared to the proposed-condition water-surface profile resulting from the design alternatives
- The predicted total scour depths and post-scour elevations at intervals along the toe of the embankment
- Design calculations for any proposed embankment protection (i.e. riprap sizing calculations)

- Design sketches of any proposed embankment protection, showing longitudinal extent, required thickness of protection, and termination requirements (i.e. toe downs and end terminations)

7.4.4.8 Plans

If protection has been designed for the embankment, then the following must be included on the final design plans:

- Details and dimensions of any required protection

- Discharge rate and water-surface elevations at appropriate locations along the embankment profile for the capacity design flood and the 100-year flood

7.4.5 RETAINING WALLS

Some roadways include retaining walls to minimize fill quantities, longitudinal encroachments on adjacent floodplains or channels, and other environmental impacts. Hydraulic consideration is warranted when a proposed highway retaining wall is to be located within a 100-year floodplain, or if a cross drain or storm drain is designed to exit through a retaining wall. Scour at the retaining wall foundation must be prevented or the foundation must be designed for stability against the predicted scour. This section provides standards, criteria, and guidance for the hydraulic design and protection of retaining wall foundations.

7.4.5.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. HDS 6 — FHWA HDS 6, *River Engineering for Highway Encroachments*
2. HEC 14 — FHWA HEC 14, *Design of Energy Dissipators for Culverts and Channels*
3. HEC 20 — FHWA HEC 20, *Stream Stability at Highway Structures*
4. HEC 23 — FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*

7.4.5.2 Standard Practices

7.4.5.2.1 Floodplain Encroachment

Longitudinal floodplain encroachments on 100-year floodplains, with or without retaining walls, should be avoided wherever practicable. If a project requires an encroachment on a 100-year floodplain that is regulated by FEMA or potentially creates an adverse impact to private property or insurable buildings, reference Section 7.4.1 for details on appropriate design policy, standards, and criteria, as well as guidance on FEMA coordination, if required.

7.4.5.2.2 Use of Scour and Stream Instability Countermeasures

The stability standards presented in this section will be met by designing the retaining wall foundation to withstand the estimated scour associated with the stability design flood, or by using a suitably designed countermeasure to prevent the formation of all or a portion of the estimated scour. Refer to Section 7.4.8 for standards, criteria, and guidance on the design of countermeasures.

7.4.5.3 Design Standards

The design standards for the hydraulic design and protection of retaining wall foundations depend on the wall height and the roadway classification. Refer to the definition of High- and Low-Standard roadways in Section 7.1.6.

7.4.5.3.1 Stability Design

The hydraulic stability of a retaining wall foundation refers to its ability to withstand scour. Two different types of scour can potentially threaten a retaining wall foundation. First, flow along the wall from the channel or floodplain on which the wall is located (longitudinal flow) can cause scour potentially throughout the entire length of the wall foundation. Second, flow from cross drain or storm drain outlets penetrating the wall can cause local outlet scour. Each case has a set of stability standards presented below.

Longitudinal Flow

- *Wall Height > 2 m [6.5']:* Design retaining wall foundations to withstand the estimated worst-case longitudinal scour up through the 100-year flood.

- *Wall Height 2 m [6.5'] or Less on High-Standard Roadways:* Design retaining wall foundations to withstand the estimated worst-case longitudinal scour up through the 50-year flood.

- *Wall Height 2 m [6.5'] or Less on Low-Standard Roadways:* Design retaining wall foundations to withstand the estimated worst-case longitudinal scour up through the 25-year flood.

Pipe Penetrations

Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

- *High-Standard Roadways:* Design retaining wall foundations to withstand the estimated worst-case outlet scour up through the 50-year flood.
- *Low-Standard Roadways:* Design retaining wall foundations to withstand the estimated worst-case outlet scour up through the 25-year flood.

7.4.5.4 Design Criteria

7.4.5.4.1 Stability Design

Longitudinal Flow

Design retaining wall foundations to withstand the estimated total scour (as defined in Section 7.4.2) from the stability design flood with normal structural and geotechnical safety factors. Assume that all streambed material above the total scour elevation has been removed and is not available for bearing or lateral support.

Pipe Penetrations

Design retaining wall foundations to withstand the estimated local outlet scour from the foundation-stability design flood with normal structural and geotechnical safety factors. Assume that all streambed material above the local scour elevation has been removed and is not available for bearing or lateral support.

7.4.5.5 Design Guidance

7.4.5.5.1 Scour Mechanisms

Consider the following scour mechanisms in evaluating the potential scour threat to a retaining wall segment. (See Section 7.4.2 for guidance.)

- Long-term degradation, if the wall is located within or immediately adjacent to a stream channel
- Long-term lateral instability in the form of channel migration
- Low-flow channel impingement, if the wall will be located within a broad, sandy waterway that has a highly active low-flow channel meandering within it
- Bank erosion
- Contraction scour, if the wall forms a significant constriction to the waterway

- Local scour by flow impinging on the wall, if flood flows will impact the wall at a significant angle
- Bendway scour, if the wall will be located at the outside of a bend
- Bed forms, if the wall is located within a sand-bed channel
- Outlet scour, if a cross drain or storm drain exits through the wall
- Potential flanking or undermining of scour countermeasures intended to protect the wall

7.4.5.5.2 Pipe Exit Configuration

Pipe exits from retaining walls that include drops (the invert of the exit pipe being above the toe of the wall) should be avoided whenever practicable. Such drops will be allowed, if required, as long as any additional scour potential caused by the drop is accommodated. The preferred horizontal alignment for pipes exiting a retaining wall is perpendicular to the wall.

7.4.5.6 Recommended Methods

Refer to HEC 23 for approaches to estimating impinging-flow scour, bendway scour, and low-flow channel impingement scour. HEC 14 provides a method of estimating scour at cross drain and storm drain outlets.

7.4.5.7 Reporting

Section 7.1.11 provides a general list of submittal requirements for hydraulic design projects. Specific deliverables for the hydraulic analysis and design of retaining wall encroachments will include at a minimum:

- The predicted total scour depths and post-scour elevations at intervals along the wall segment
- Design calculations for any proposed scour countermeasures (i.e. riprap sizing calculations)
- Design sketches of any proposed scour countermeasures

7.4.5.8 Plans

If scour calculations have been performed or countermeasures have been designed for the retaining wall, then the following must be included on the final design plans.

- Details and dimensions of any required scour countermeasures

- Stability design discharge with water-surface elevations at appropriate locations along the wall

7.4.6 LOW-WATER CROSSINGS

Low-water stream crossings can provide safe, cost-efficient alternatives to bridge and culvert crossings for certain low-volume roads, provided the streamflow and road-use conditions are suitable. This section provides standards, criteria, and guidance on the design of low-water crossings.

7.4.6.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	FHWA CFL-03-003	FHWA CFL-03-003, Low Water Crossing Study
2.	Low Volume Roads Engineering	U.S. Forest Service and U.S. Agency for International Development, *Low Volume Roads Engineering-Best Management Practices Field Guide*, 2003
3.	HDS 1	FHWA HDS 1, Hydraulics of Bridge Waterways
4.	HDS 5	FHWA HDS 5, Hydraulic Design of Highway Culverts
5.	HDS 6	FHWA HDS 6, River Engineering for Highway Encroachments
6.	HEC 20	FHWA HEC 20, Stream Stability at Highway Structures
7.	HEC 23	FHWA HEC 23, Bridge Scour and Stream Instability Countermeasures

7.4.6.2 Standard Practices

7.4.6.2.1 Allowable Uses

Low-water crossings will not be used on roadways with an ADT greater than 200, unless such crossing is a desirable, existing feature.

7.4.6.2.2 Classes and Applications

Two classes of low-water crossings are possible on Federal Lands Highway projects: vented crossings and unvented crossings. A vented crossing has a hydraulic opening beneath the road surface for low flows, while an unvented crossing has no opening beneath the road surface. The selection of the class to use for a particular project is dependent on the following:

- Vented Crossing:
  - ◊ Daily access required, AND
  - ◊ Crosses a perennial stream characterized by daily flows
- Unvented Crossing
  - ◊ Daily access not required, OR
  - ◊ Crosses an ephemeral stream with only intermittent, short-duration flows

7.4.6.2.3 Floodplain Encroachment

If a low-water crossing is in an NFIP floodplain or the potential exists for adversely impacting private property or insurable buildings, refer to Section 7.4.1 for relevant policy, standards, and criteria, as well as for guidance on FEMA coordination.

7.4.6.3 Design Standards

7.4.6.3.1 Capacity Design

Vented

Design vented low-water crossings to convey the 10-year flood beneath the road.

Unvented

Not applicable since all flow must pass over the roadway.

7.4.6.3.2 Stability Design

Design all low-water crossings to remain stable under worst-case scour conditions up through the 25-year flood.

7.4.6.4 Design Criteria

7.4.6.4.1 Capacity

Vented

No overtopping by the design flood.

Unvented

Not applicable since all flow must pass over the roadway.

7.4.6.4.2 Stability

Vented

Evaluate the foundation of any open-bottom structure for scour susceptibility according to guidance in Section 7.4.3.

Vented and Unvented

Design the crossing to remain stable under worst-case scour conditions up through the stability-design flood. Demonstrate that the embankment is reasonably expected to remain stable.

Design to withstand applicable scour components, e.g., drop scour, culvert outlet scour, and long-term degradation (refer to Section 7.4.2). The use of scour countermeasures is acceptable (refer to Section 7.4.8).

7.4.6.5 Design Guidance

Every low-water crossing should be posted with signs on both approaches instructing motorists to stay out of the crossing when it is flooded.

7.4.6.5.1 Vented

Hydraulic Operation

A vented low-water crossing will typically operate as a culvert for flows up to the capacity design flow and as a broad crested weir combined with a culvert for flows exceeding the capacity design flow.

Scour

A vented low-water crossing will experience the potential for culvert-type outlet scour combined with drop-scour. The scour potential will be exacerbated if the downstream reach experiences degradation. The stability design must accommodate or prevent the formation of scour on the downstream side of the crossing.

Fish

Fish passage concerns may be a factor in the design of vented low-water crossings. Refer to Section 7.5.1 for guidance on designing crossings to prevent creating a barrier to fish passage.

7.4.6.5.2 Unvented

Maintenance

An unvented low-water crossing will typically be used in an arid or semi-arid setting and will be overtopped whenever the watershed produces runoff. The flow will often leave behind a deposit of sediment that may require clearing before reopening the road to traffic. Consider surfacing the low-water crossing with a hard surface to facilitate the quick removal of sediment deposits by heavy equipment without damage to the road.

Hydraulic Operation

An unvented low-water crossing will typically operate hydraulically as a broad-crested weir.

Scour

If a significant hydraulic drop occurs from the upstream side to the downstream side of an unvented crossing, the potential exists for scour (termed drop-scour) on the downstream side. The scour potential will be exacerbated if the downstream reach experiences degradation. The stability design must accommodate or prevent the formation of scour on the downstream side of the crossing.

In addition to preventing or accommodating scour on the downstream side, the road surface should be protected along the entire wetted perimeter up to the anticipated water-surface elevation from the stability design flood.

7.4.6.6 Recommended Methods

The U.S. Forest Service and U.S. Agency for International Development, *Low Volume Roads Engineering-Best Management Practices Field Guide* provides practical advice in developing the design of low-water crossings.

Chapter VIII of HDS 1 provides detailed guidance on the hydraulic analysis of roadway overtopping conditions. HDS 5 is an important reference in the analysis of the culvert-type flow through the openings of vented low-water crossings.

HEC 20 provides useful detailed guidance on evaluating the stability of the stream reach of interest. HEC 23 contains extensive guidance on the prediction of drop-scour and the design of countermeasures to prevent failure of the crossing from scour and stream instability.

If water-surface elevation impacts are a concern, it may be necessary to compute a water-surface profile through the affected stream reach.

7.4.6.7 Reporting

Section 7.1.11 provides a general list of submittal requirements for hydraulic design projects. Additional specific deliverables for hydraulic design of low-water crossings will include:

- The water-surface elevation upstream of the crossing for the capacity design discharge (vented crossings)

- The maximum velocity through the low-flow opening for the capacity design discharge (vented crossings)

- Calculations of applicable scour components and total scour

- Design calculations for the proposed scour countermeasures

- Design sketches of the crossing and the proposed scour countermeasures

7.4.6.8 Plans

Include the following information, as a minimum, in the drawings for the crossing:

- Locations of low-flow openings, if any
- Elevations of footings or piles if used as foundations
- Existing topography and grading contours in the plan drawing
- The water-surface elevation upstream of the crossing from the capacity design flood
- Magnitude and frequency of the capacity design flood
- Waterway cross-section geometry in the elevation drawing
- Extent of road surface protection
- Locations, dimensions, and details for the proposed scour countermeasures

7.4.7 CHANNEL CHANGES

Some projects require realignments of stream channels to avoid or to mitigate potential hydraulic problems at a highway crossing location. Properly designed channel changes can reduce the hazard of flood damage to a highway crossing by reducing skew and curvature, and sometimes by providing a larger main channel.

This section provides standards, criteria, and guidance related to the design of channel changes for those situations where they cannot be avoided. It addresses only the hydraulic aspects of channel relocation. For guidance on the environmental aspects of relocation (e.g., restoration of biological or ecological components), see Section 7.5.2.

7.4.7.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. HDS 6 FHWA HDS 6, *River Engineering for Highway Encroachments*

2.	HEC 11	FHWA HEC 11, *Design of Riprap Revetment*
3.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*
4.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
5.	AASHTO HDG Vol. I	AASHTO Highway Drainage Guidelines, *Volume I – Hydraulic Considerations in Highway Planning and Location*
6.	AASHTO HDG Vol. VII	AASHTO Highway Drainage Guidelines, *Volume VII – Hydraulic Analysis for the Location and Design of Bridges*
7.	AASHTO HDG Vol. X	AASHTO Highway Drainage Guidelines, *Volume X – Evaluating Highway Effects on Surface Water Environments*

7.4.7.2 Standard Practices

Alterations or relocations of existing stream channels will be avoided wherever practicable. Where channel changes cannot be avoided, close coordination with Environmental Office staff, resource agencies, and the partner agency will be initiated during the reconnaissance and scoping phase and continue throughout the design of the project.

The design of channel changes will consider the impacts to stream stability and to the riparian environment and will mitigate those impacts to the extent practicable.

If a channel change is proposed in a floodplain that is regulated by FEMA, or if it potentially creates an adverse impact to private property or insurable buildings, refer to Section 7.4.1 for details on appropriate design policy, standards, and criteria as well as guidance on FEMA coordination, if required.

7.4.7.3 Design Standards

7.4.7.3.1 Capacity Design

To the extent practicable, the channel change will duplicate the existing stream characteristics including:

- Stream capacity
- Width
- Depth
- Slope
- Sinuosity
- Bank cover
- Side slopes
- Flow and velocity distribution over the full range of discharges up to and including the 100-year flood

7.4.7.3.2 Stability Design

Where instability of a relocated stream channel may threaten the highway infrastructure, channel migration countermeasures will be provided. The design standard for the countermeasures depends on the classification of the roadway. Refer to the definition of High-Standard and Low-Standard roadways in Section 7.1.6.

- *High Standard:* Design channel migration countermeasures to withstand the worst conditions up through the 50-year flood.
- *Low Standard:* Design channel migration countermeasures to withstand the worst conditions up through the 25-year flood.

7.4.7.4 Design Criteria

7.4.7.4.1 Capacity Design

To minimize the potential biological and ecological impacts, avoid using prismatic channel geometries with neat grading lines wherever practicable.

7.4.7.4.2 Stability Design

Demonstrate stability in the horizontal and vertical dimensions, when required.

Bioengineering treatments for both vertical and lateral stability within the relocated reach of the channel may be used if the stability of the channel is not compromised relative to the standards and criteria presented in this section.

7.4.7.5 Design Guidance

Addressing potential impacts to the stability of the stream and to the riparian environment is a multi-disciplinary challenge involving the application of geomorphic analysis, hydraulic engineering, and stream habitat evaluation.

Geomorphic analysis is required to evaluate the range of potential responses of the stream to the proposed channel change, and to guide the design of the channel change to minimize the adverse responses. The basic types of potential response needing investigation include degradation, aggradation, or lateral instability. These responses can affect the channel upstream and downstream of the proposed channel change, as well as the relocated reach itself. An appropriate geomorphic analysis considers the initial state of the stream system and its degree of sensitivity to the channel change being considered. It makes use of established stream-response relationships as well as an understanding of geomorphic threshold conditions.

Refer to Section 7.4.2 for Standards, Criteria, and Guidance related to scour and stream instability. Refer to Section 7.4.8 for Standards, Criteria, and Guidance in the design of stream instability and countermeasures.

7.4.7.6 Recommended Methods

Chapters 4 and 6 of HEC 20, Chapter 5 of HDS 6 and Section 4 of the AASHTO Highway Drainage Guidelines, *Volume X* are good starting references for the geomorphic analysis.

The application of hydraulic engineering to channel change designs entails supplementing the geomorphic analysis with quantitative evaluations of the potential for stream instability and designing countermeasures against stream instability.

HEC 20, HEC 23, and HDS 6 contain recommended methods for hydraulic engineering applications to channel changes.

7.4.7.7 Reporting

Document through appropriate analysis, calculations, and judgment that the relocated channel, together with any associated channel stability protection measures, is reasonably expected to remain stable under worst-case conditions up to the design flood. Items to be documented include:

- Comparison of water-surface impact expected for each channel change alternative being considered
- Qualitative comparison of adverse impacts for each channel change alternative being considered
- Cross-section plots of the proposed relocated channel reach at key locations
- Design calculations for any proposed stream instability countermeasures (i.e. riprap sizing calculations, etc.)
- Design sketches of any proposed stream instability countermeasures

7.4.7.8 Plans

The project plans should include the following for any proposed channel change:

- Plan/layout drawing of the proposed channel relocation, including contour grading and showing the connection to the existing channel at the upstream and downstream ends of the channel change
- Cross section drawings at sufficient locations to allow adequate construction staking
- Details and dimensions of any proposed stream instability countermeasures

7.4.8 SCOUR AND STREAM INSTABILITY COUNTERMEASURES

This section provides standards, criteria, and guidance for designing countermeasures to protect Federal Lands Highway facilities from scour and stream instability.

7.4.8.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*
2.	HEC 11	FHWA HEC 11, *Design of Riprap Revetment*
3.	HEC 14	FHWA HEC 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*
4.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
5.	AASHTO HDG Vol. VII	AASHTO Highway Drainage Guidelines, *Volume VII – Hydraulic Analysis for the Location and Design of Bridges*
6.	AASHTO MDM Chap. 17	AASHTO Model Drainage Manual, *Chapter 17 – Bank Protection*

7.4.8.2 Standard Practices

The potential for scour and stream instability will be considered when designing highway facilities that interface with natural rivers, streams, or floodplains (see Section 7.4.2). Where it is impracticable or inappropriate to accommodate the estimated scour and stream instability in the design of the facility, countermeasures will be used to mitigate the potential for damage.

7.4.8.2.1 Bridge Piers

New piers will be designed so that they withstand the estimated total scour depth from the design flood without the need for countermeasures (see Section 7.4.3). The piers of bridges to be rehabilitated may be protected from scour by countermeasures as appropriate.

7.4.8.2.2 Floodplain Encroachments

Countermeasure installations themselves may encroach upon base floodplains and be subject to FHWA Policy 23 CFR 650A. If such an encroachment is in an NFIP mapped floodplain or if the encroachment produces potential adverse impacts to private property or insurable buildings, refer to Section 7.4.1 for related policy, standards, criteria, and guidance.

7.4.8.3 Design Standards

7.4.8.3.1 Stability Design

Scour and stream instability countermeasures will be designed to meet the appropriate stability standards for the structures they are intended to protect. Specific references to appropriate standards are provided below:

Culvert Outlets

Refer to Sections 7.3.1 and 7.3.5.

Foundations of Bridge Abutments and Existing Piers

Refer to Section 7.4.3.

Bridge Approach Embankments

Refer to Section 7.4.3.

Longitudinal Embankments

Refer to Section 7.4.4.

Protection of Retaining Wall Foundations

Refer to Section 7.4.5.

Low-Water Crossings

Refer to Section 7.4.6.

Channel Changes

Refer to Section 7.4.7.

Adjacent Streambanks

If the stream stability assessment indicates that streambank erosion or migration of a nearby channel may threaten the highway facility, install countermeasures to stabilize the channel banks. The design standards for protection of streambank countermeasures depend on the roadway classification. Refer to the definition of High- and Low-Standard roadways in Section 7.1.6.

- *High Standard:* Design the protection to withstand the worst scour conditions up through the 50-year flood.

- *Low Standard:* Design the protection to withstand the worst scour conditions up through the 25-year flood.

7.4.8.4 Design Criteria

The general design criteria for scour and stream instability countermeasures are as follows:

Demonstrate that the countermeasure is reasonably expected to remain stable and to protect the facility under worst-case conditions up through the stability design flood throughout its intended service life.

Provide appropriate termination details to prevent undermining or flanking of the countermeasure by scour and erosion processes not arrested by the countermeasure itself. A countermeasure intended to prevent local scour, for instance, must be protected from undermining by the sum of the estimated contraction scour and long-term degradation.

7.4.8.5 Design Guidance

7.4.8.5.1 Minimizing the Need for Countermeasures

Where practicable, it is usually preferable to design the facility so that countermeasures are not necessary. This can be accomplished by avoiding route locations through areas of high scour potential, or by designing the foundations of bridges and retaining walls to accommodate the estimated potential scour. Designing to avoid the need for countermeasures provides the following benefits:

- Avoids the additional cost associated with building countermeasures
- Avoids the considerable maintenance commitment usually associated with countermeasures
- Preserves the natural dynamics of the stream system
- Minimizes impacts to wetlands and riparian habitat
- Minimizes Section 404 wetlands permit requirements

7.4.8.5.2 Selection

Many different types of countermeasures, and variations of each type, have been used to protect highway facilities. At a minimum, the selection should consider:

- A verified need for the countermeasure (make sure the countermeasure is needed and that the design can’t practicably be modified to avoid the need)
- The function of the countermeasure to address the need

- The compatibility of the countermeasure with the geomorphology of the stream channel
- The acceptability of any environmental impacts associated with the countermeasure, or the potential to mitigate the impacts
- The capital cost of the countermeasure
- The maintenance and inspection requirements of the countermeasure

7.4.8.5.3 Inspection and Maintenance

Most countermeasure installations for protection of highway facilities are designed with the expectation of some maintenance requirements. A typical riprap revetment, for example, needs regular inspection to verify its continuing functionality. A long-term maintenance commitment is needed to ensure the continued performance of a countermeasure through the expected service life of the highway facility.

7.4.8.6 Recommended Methods

The design of protection for structures, streambanks, and longitudinal embankments can be aided by the procedures found in several references, including:

1.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*
2.	HEC 11	FHWA HEC 11, *Design of Riprap Revetment*
3.	HEC 14	FHWA HEC 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*
4.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
5.	AASHTO MDM Chap. 17	AASHTO Model Drainage Manual, *Chapter 17 – Bank Protection*
6.	Caltrans Chap. 870	California Department of Transportation Highway Design Manual, Chapter 870 – *Channel and Shore Protection-Erosion Control*
7.	EM 1110-2-1601	U.S. Army Corps of Engineers EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*
8.	Denver USDCM	Denver Urban Drainage and Flood Control District, *Urban Storm Drainage Criteria Manual*, Volume 2 (particularly useful for the design of grade control structures)

7.4.8.7 Reporting

The reporting requirements listed below should be integrated with those for the specific types of facilities that the countermeasures are designed to protect. The reporting items listed below are the minimum expected for countermeasure design.

- Description of need and countermeasure alternatives considered
- Description of selection criteria
- Documentation demonstrating the suitability and stability of the proposed countermeasure design, including calculations
- Design sketches of the proposed scour countermeasures

7.4.8.8 Plans

Include the following information, as a minimum, in the project drawings:

- Locations, dimensions, and details of any proposed scour and stream instability countermeasures

7.5 ENVIRONMENTAL HYDRAULICS

The topics included in this section on environmental hydraulics all require interdisciplinary design or analysis. Early and frequent coordination with the local Federal Lands Environmental Office, resource agencies, regulatory agencies, and the partner agency is often required. The role of the hydraulics engineer will typically be support, such as that required for permit application and acquisition, and review of deliverables from specialty contractors, as requested.

7.5.1 FISH PASSAGE

The necessity to protect fish life and provide for fish passage can affect many decisions regarding bridge, culvert, channel change, riprap design, and construction requirements. Because of their relatively small size, the ability of these structures to accommodate migrating fish is an important design consideration. Consult state and local fish and wildlife resource agencies early in the roadway planning process when fish passage issues are anticipated. For existing culverts that obstruct fish passage, modifications can be used to meet fish passage criteria. Fish passage will be accommodated when need is verified by project scoping studies.

7.5.1.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 650A	Code of Federal Regulations, Title 23, Part 650 Subpart A, *Location and Hydraulic Design of Encroachments on Flood Plains*
2.	HDS 5	FHWA HDS 5, Hydraulic Design of Highway Culverts
3.	AASHTO MDM Chap. 9	AASHTO Model Drainage Manual, *Chapter 9 – Culverts*
4.	AASHTO HDG Vol. IV	AASHTO Highway Drainage Guidelines, *Volume IV – Hydraulic Design of Culverts*

7.5.1.2 Design Standards and Criteria

Roadway crossing structures to accommodate fish passage will comply with the applicable standards and criteria of this chapter. The selected design will be reasonable in terms of satisfying social, environmental, and economic constraints.

7.5.1.3 Design Guidance

7.5.1.3.1 Culverts

Because fish passage needs are particularly acute and frequent at culvert locations, many fish and wildlife agencies have established design standards and criteria for fish passage through culverts. Design considerations include discharge, maximum allowable velocity, minimum water depth, substrate characteristics, maximum culvert length and gradient, type of structure, and construction scheduling. Final designs should consider these standards and criteria as well as those of this chapter.

<u>New vs. Retrofit</u>

The design of new or replacement culverts that must provide fish passage should seek to replicate the natural stream hydraulics and processes, such as sediment transport characteristics, over a range of discharges up to and including the roadway design flood. The design should concentrate low-flows to provide adequate passage depth and provide overbank relief for high flows to minimize scour and degradation of the main channel.

For highway rehabilitation or restoration projects, where an existing culvert has been identified as a fish passage barrier, the engineer should consider alternatives for retrofitting the existing structure to meet fish passage requirements. It is possible that the addition of baffles inside the culvert, weirs downstream of the culvert, or other treatments can meet the hydraulic criteria for local fish passage and design storm conveyance. An economic evaluation should be performed to determine the most cost effective alternative.

<u>Open-Bottom Culverts</u>

Open-bottom culverts, either concrete or metal, are sometimes designed for fish passage, environmental, aesthetics or economic reasons. These structures typically have a natural bottom and must be supported on both sides by a foundation. Because of the likelihood of local scour, evaluate and design the foundations using bridge criteria, unless they can be founded on bedrock. Refer to Sections 7.4.2 and 7.4.3 for information on foundation design in areas where scour is a potential.

<u>Oversized or Depressed Culverts</u>

As an alternative to an open-bottom culvert, where fish passage or environmental conditions require an alluvial bottom, the designer may use an oversized culvert, with the invert buried below the channel invert elevations and a portion of the culvert bottom filled with alluvial material. Sometimes baffles are necessary to hold the channel material in place.

<u>Culverts with Baffles</u>

Many baffle configurations have been shown to be satisfactory in decreasing the velocity or increasing water depth through the culvert. Baffles are especially useful for making existing culverts fish passable in retrofit situations. The addition of baffles may cause culverts to flow in outlet control at relatively low discharge rates. Neglecting the culvert area occupied by the baffles may not adequately account for the energy losses from turbulence generated by the baffles.

Downstream Weirs

Weirs are also useful in retrofit applications. They are typically constructed downstream of the culvert to increase tailwater and increase flow depths through the culvert. Weirs must be designed for stability during high flows and also provide for fish passage. This may require means for fish to bypass the weir.

Special Treatment

In wide, shallow streams, one barrel of a multiple barrel culvert installation can be depressed slightly to concentrate low-flows, thus providing for fish passage.

7.5.1.3.2 Bridges (reserved)

7.5.1.4 Recommended Methods

Use local or regional guidance, methods, or procedures, as available and applicable. Examples include:

- Federal resources agency procedures (e.g., USFS, USFWS, NMFS)
- State DOT Memorandums of Agreement with resource agencies (e.g., Alaska, Maine)
- State procedures (e.g., California, Oregon)

7.5.2 STREAM RESTORATION AND REHABILITATION

7.5.2.1 Standard Practice

Stream restoration/rehabilitation is a highly interdisciplinary task requiring close coordination with the Environmental Office, resource agencies, and the partner agency. This task may be undertaken as part of needed channel relocation work or as an independent environmental mitigation or habitat enhancement effort. The role of hydraulics is to provide:

- Appropriate protection for the roadway
- Compatibility with geomorphic and biological factors at the site
- Cost-effective design

For detailed guidance on the stability aspects of stream restoration or rehabilitation work, reference Sections 7.4.2, 7.4.7, and 7.4.8.

7.5.2.2 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 650A	Code of Federal Regulations, Title 23, Part 650 Subpart A, *Location and Hydraulic Design of Encroachments on Flood Plains*
2.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*
3.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*
4.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
5.	AASHTO MDM Chap. 15	AASHTO Model Drainage Manual, *Chapter 15 – Surface Water Environment*
6.	AASHTO HDG Vol. X	AASHTO Highway Drainage Guidelines, *Volume X – Evaluating Highway Effects on Surface Water Environments*
7.	Stream Corridor Restoration	Federal Interagency Stream Restoration Working Group, *Stream Corridor Restoration: Principles, Processes, and Practices*

7.5.2.3 Design Standards and Criteria

The design standards and criteria applied to stream restoration and rehabilitation design will comply with the applicable standards and criteria of this chapter. The selected design will be reasonable in terms of satisfying social, environmental, and economic constraints.

7.5.2.3.1 Plan Form and Geometry

Replicate the historical plan form and channel geometries, if known. Where historical geometries are unknown, use the appropriate dominant discharge (2- to 10-year discharge) and regime theory to establish appropriate plan form and channel geometries.

7.5.2.3.2 Stability Checks

Conduct stability checks of plan form and channel geometry over a range of discharges up to and including the 50-year flood.

7.5.2.4 Design Guidance

In the process of restoration and rehabilitation of streams and aquatic habitat, the goal is not a static, immovable channel. Rather, the goal is to restore the stream to a reasonably stable, naturalistic system that exhibits a state of dynamic equilibrium.

7.5.2.5 Recommended Methods

Design and evaluate the hydraulic engineering aspects of stream restoration and rehabilitation according to the methods and procedures presented in Chapter 7 of HEC 20, HEC 23, and the Federal Interagency Stream Restoration Working Group, *Stream Corridor Restoration Principles, Processes, and Practices.*

7.5.3 WETLANDS

7.5.3.1 Standard Practice

Road construction and roadway operation can have numerous impacts on wetland chemistry, biology, surface hydrology, and groundwater hydrology. Wetland design and analysis is a highly interdisciplinary task requiring close coordination with the Environmental Office, resource agencies, and the partner agency. The primary role of the hydraulics engineer is for support and review of deliverables from specialty contractors, as requested.

7.5.3.2 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 2	FHWA HDS 2, *Highway Hydrology*
2.	23 CFR 771	23 CFR 771, *Environmental Impact and Related Procedures*
3.	23 CFR 777	23 CFR 777, *Mitigation of Environmental Impacts to Privately Owned Wetlands*
4.	AASHTO MDM Chap. 15	AASHTO Model Drainage Manual, *Chapter 15 – Surface Water Environment*
5.	AASHTO HDG Vol. X	AASHTO Highway Drainage Guidelines, *Volume X – Evaluating Highway Effects on Surface Water Environments*

7.5.3.3 Design Standards and Criteria

The design standards and criteria applied to wetland design will comply with the applicable standards and criteria of this chapter. The selected design will be reasonable in terms of satisfying social, environmental, and economic constraints.

7.5.3.4 Design Guidance

The design of wetlands should be performed by specialists. The primary role of the hydraulics engineer is for support and review as requested.

7.5.4 STORMWATER MANAGEMENT

7.5.4.1 Standard Practice

Where required by federal, state, or local stormwater management policies, standards, and criteria, stormwater discharge controls will be incorporated into Federal Lands Highway projects. Controls on both water quantity and quality are typical.

7.5.4.2 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. HEC 22 — FHWA HEC 22, *Urban Drainage Design Manual*
2. AASHTO MDM Chap. 12 — AASHTO Model Drainage Manual, *Chapter 12 – Storage Facilities*
3. AASHTO HDG Vol. IX — AASHTO Highway Drainage Guidelines, *Volume IX – Guidelines for Storm Drain Systems*
4. NPDES Regulations
5. State and Local Stormwater Management Manuals
6. FHWA PD-96-032 — FHWA PD-96-032, *Evaluation and Management of Highway Runoff Water Quality*
7. FHWA EP-00-002 — FHWA EP-00-002, Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring

7.5.4.3 Design Standards and Criteria

Conduct project-specific hydrologic/hydraulic analyses, and design necessary facilities to support compliance with federal, state, and local stormwater management requirements, standards, and criteria.

The design standards and criteria applied to stormwater management design will comply with the applicable standards and criteria of this chapter. The selected design will be reasonable in terms of satisfying social, environmental, and economic constraints.

7.5.4.4 Recommended Methods

Federal, state, and local requirements often govern the design of stormwater management facilities. Where applicable, those design methods should be used and supplemented with the analysis and design methods recommended in this chapter.

Methods specific to storage routing analysis and outlet structure design for retention/detention basins are provided in Chapter 8 of HEC 22.

7.6 COASTAL HYDRAULICS

7.6.1 GENERAL

Tidal waterways and coastal shorelines present special challenges to the design of highway facilities. This section provides references, standards, criteria, and guidance specific to the design of highway facilities in coastal areas.

Certain elements of analysis and design in coastal areas require technical knowledge specific to the field of coastal engineering. The design of critical facilities in coastal areas, therefore, will usually require attention from a qualified coastal engineer.

7.6.1.1 Tide Levels and Wave Heights

The hydrology and hydraulics of coastal shorelines and tidal waterways are dominated by factors that are typically nonexistent or of little consequence in inland streams. The most significant factors distinguishing tidal waters from inland streams are the effects of the tides and wind-generated waves. Tidal water elevations and wave heights are, therefore, the two key elements of coastal hydraulic analysis that define design water surface elevations. Hydraulic design of roadway facilities along coastlines or crossing tidal waterways will consider the affects of tidal water elevations and their cyclical fluctuations, along with storm surges and wave heights, as appropriate. Refer to FHWA HEC 25, *Tidal Hydrology, Hydraulics and Scour at Bridges*, for definitions of terminology specific to tidal waterways and coastal areas.

7.6.1.2 Vertical Datum Reconciliation

Published tide heights are usually referenced to the Mean Lower Low Water (MLLW). The relationship between MLLW and any fixed vertical datum such as the North American Vertical Datum of 1988 (NAVD 88) and the National Geodetic Vertical Datum of 1929 (NGVD 29, often simply termed Mean Sea Level) varies widely depending on location along the coast. Resources are available that quantify the relationship between MLLW and NAVD 88, which can then be converted for NGVD 29. Chapter 6 of HEC 25 explains how to find and use these resources.

7.6.2 HYDROLOGY

Hydrologic analysis for projects along shorelines or crossing tidal waterways primarily involves the prediction of tidal water elevations and wave heights (i.e., design water surfaces). These predictions may be required for normal conditions unaffected by storms, for conditions resulting from severe storms or both. The appropriate recurrence interval or level of severity to be analyzed will depend upon the design standards presented later in this section.

7.6.2.1 References

The following references provided source information for the development of the guidance of this subsection (most recent editions apply):

1.	HEC 25	FHWA HEC 25, *Tidal Hydrology, Hydraulics and Scour at Bridges*
2.	EM 1110-2-1100	U.S. Army Corps of Engineers EM 1110-2-1100, *Coastal Engineering Manual*

7.6.2.2 Standard Practices

7.6.2.2.1 Roadway Facilities Along Shorelines

The hydrologic analysis of a project along a shoreline of an ocean or bay will predict the following elements for astronomic and storm events of appropriate severity:

- Peak tidal elevations
- Wind-generated wave heights

7.6.2.2.2 Roadway Facilities Crossing Inlets, Tidal Channels and Bays

The hydrologic analysis of a project crossing an inlet, other tidally dominated channel, or bay, will predict the following elements for astronomic and storm events of appropriate severity:

- Water-level hydrograph at the project location resulting from combined astronomic tides and storm-surge conditions
- Discharge hydrograph at the project location resulting from combined astronomic tides and storm-surge conditions
- Wind-generated wave heights

7.6.2.2.3 Roadway Facilities Crossing Estuaries

The hydrologic analysis of a project crossing an estuary (a tidally affected reach at the mouth of a river or stream) will predict the following elements for astronomic and storm events of appropriate severity:

- Peak discharge rates for riverine floods along with approximate riverine flood duration and time-to-peak estimate

- Water-level hydrograph at the project location resulting from combined astronomic tides and storm-surge conditions
- Discharge hydrograph at the project location resulting from combined astronomic tides and storm-surge conditions
- Wind-generated wave heights

The hydrologic analysis of estuaries requires an investigation of the probability of a severe flood coinciding with an extreme astronomic tide or storm-surge condition.

7.6.2.3 Design Guidance

The hydrologic determinations called for above require analysis methods that are usually not relevant to inland rivers and streams. Essentially the design processes involve:

- Estimating the magnitude and timing of the ocean's rise and fall for the event or condition of interest
- Estimating the discharge hydrograph or the peak discharge rate at the location of interest in response to the tidal rise and fall
- If appropriate, combining the tidal discharge information with the riverine flow
- Estimating the water-surface elevation associated with the peak discharge
- Developing an appropriate design wave height prediction, usually a function of the wind speed, the fetch, and the depth of the waterway

7.6.2.4 Recommended Methods

The processes described above can be achieved by simple or complex analysis methods, depending on project needs. Chapters 2 through 4 of HEC 25 describe various available methods and their appropriate application.

A common approach for developing wave height predictions is to assume a hurricane-force wind and use the U.S. Army Corps of Engineers EM 1110-2-1100, *Coastal Engineering Manual* to determine the "significant wave height."

7.6.2.5 Reporting

Section 7.6.2.2 describes the hydrologic elements that are to be predicted depending on the project situation. Those elements must be reported and must be supported by appropriate documentation, which will include, at a minimum:

- Data and data sources
- Reference for methods used

- Assumptions
- Conclusions
- Recommendations

7.6.3 FLOODPLAIN ENCROACHMENTS

The National Flood Insurance Program has designated special flood hazard areas for coastal shorelines and tidal waterways. As with inland floodplains, coastal flood hazard areas are delineated for base flood (100-year) conditions. Consequently, the requirements of Section 7.4.1 also apply to projects encroaching on FEMA regulated coastal floodplains. The impacts of roadway projects encroaching on coastal floodplains are typically less critical than on riverine floodplains. Unlike riverine floodplains, the base flood elevations of coastal floodplains other than estuaries are not typically affected by roadway encroachments. The flood elevations of non-estuary coastal floodplains are set by the effects of astronomic tides, storm surges, and waves, which are not sensitive to the presence of roadway encroachments.

7.6.3.1 References

The following references provided source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 650A	Code of Federal Regulations, Title 23, Part 650 Subpart A, *Location and Hydraulic Design of Encroachments on Flood Plains*
2.	NS 23 CFR 650A	Federal-Aid Policy Guide, Non-regulatory Supplement to Title 23 CFR 650 Subpart A, Attachment 2, *Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency (FEMA)*
3.	FEMA Appendix D	FEMA Guidelines and Specifications for Floodplain Mapping Partners – Appendix D, *Guidance for Coastal Flooding Analyses and Mapping*

7.6.3.2 Standard Practices

Refer to Section 7.4.1.2.

7.6.3.3 Design Standards

Refer to Section 7.4.1.3.

7.6.3.4 Design Criteria

Refer to Section 7.4.1.4. Note that encroachments on coastal floodplains other than estuaries rarely cause any rise to the base flood elevations.

7.6.3.5 Design Guidance

Because coastal flood levels are driven by tides, storm surges, and waves, they are typically not affected by highway encroachments. Water-surface-elevation impact studies are usually not required, therefore, for projects encroaching on the floodplains of shorelines, bays, or inlets. Projects encroaching on estuary floodplains may cause an adverse impact, depending on the importance of riverine flooding compared to coastal flooding at the location of interest. Refer to Section 7.4.1.5 for more comprehensive guidance on the design of floodplain encroachments and coordination with floodplain administration officials (FEMA, state, and local).

7.6.3.6 Reporting

Refer to Section 7.4.1.6.

7.6.3.7 Plans

Refer to Section 7.4.1.7. The magnitude (discharge rate) of the flood will not be applicable except in the case of estuaries.

7.6.4 SCOUR AND STREAM STABILITY

Scour and stream instability present potential threats to highway facilities in coastal areas, just as in the riverine context. This section provides standards, criteria, and guidance related to scour and stream instability specifically in coastal areas.

7.6.4.1 References

The following references provided source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	T 5140.23	Technical Advisory T 5140.23, *Evaluating Scour at Bridges*, 1991
2.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*
3.	HEC 18	FHWA HEC 18, *Evaluating Scour at Bridges*
4.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*

5.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
6.	HEC 25	FHWA HEC 25, *Tidal Hydrology, Hydraulics and Scour at Bridges*
7.	FHWA RD-86-126	FHWA Report Number RD-86-126, *Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping*
8.	AASHTO HDG Vol. XI	AASHTO Highway Drainage Guidelines, *Volume XI – Highways Along Coastal Zones and Lakeshores*
9.	EM 1110-2-1100	U.S. Army Corps of Engineers EM 1110-2-1100, *Coastal Engineering Manual*

7.6.4.2 Standard Practices

The potential for scour and stream instability will be considered when designing highway facilities that interface with shorelines and tidal waterways. Accordingly, an assessment or evaluation of the potential for scour and stream instability will be conducted at a level commensurate with the risk of damage to the facility. The design will protect the highway facility from stream instability and scour at an appropriate level, in accordance with the applicable sections of Chapter 7 for each type of drainage structure or facility.

7.6.4.3 Design Standards and Criteria

Select the design standards and criteria for stability against scour and stream instability in accordance with the applicable sections referenced below:

- Bridge foundations, see Section 7.6.5
- Roadway embankments, see Section 7.6.6
- Scour and stream instability countermeasures, see Section 7.6.7

7.6.4.4 Design Guidance

Refer to Section 7.4.2.4 and consider additional guidance related specifically to scour and stream instability in coastal areas.

Even though the standards and criteria associated with scour and stream instability are often the same for inland-area and coastal-area projects, the processes causing scour can be quite different.

7.6.4.4.1 Wave Attack Considerations

Wave attack can cause scour at facilities located along shorelines. Embankment side slopes, for instance, can be destroyed by waves through impact, run-up, or rebound, unless protected.

7.6.4.4.2 Causes of Degradation

Degradation in an inlet is usually caused by a sediment imbalance in the tidal flows through the inlet. The degradational trend can be initiated by construction of coastal protection works that stop or impede the littoral drift of sediment from reaching the inlet, or by another nearby inlet to the same bay becoming closed.

7.6.4.4.3 Flow Reversal

Since the flow reverses directions in a tidal waterway the contraction scour and local scour potential often must be determined for flow in both directions, and the worst case used for design.

7.6.4.4.4 Short Duration of High Discharge and Velocity

If the scour potential is being estimated for a short-duration event, such as a hurricane storm surge condition, consider the possibility that the scour-causing flows will not last long enough to develop the full equilibrium scour potential. Contraction-scour calculations can be modified to account for the time-rate of scour (see HEC 25, Chapter 5).

7.6.4.4.5 Riverine vs. Tidal Scour Conditions

For bridges that could be subject to scour from both extreme riverine floods and extreme tidal storm events, it may be necessary to analyze the scour for both conditions and design for the worst case.

7.6.4.5 Recommended Methods

HEC 25 provides a description of Level 1, Level 2, and Level 3 analysis approaches for tidal waterways. It also gives detailed guidance analyzing tide levels, hydraulics, and scour potential in tidal waterways.

For estimates of wave scour, refer to the U.S. Army Corps of Engineers EM 1110-2-1100. This manual provides guidance on wave height prediction as a function of wind speed, wind duration, the fetch of the water body, and the water depth within the fetch.

7.6.4.6 Reporting

Refer to Section 7.4.2.6.

7.6.5 BRIDGED WATERWAYS

This section provides standards, criteria, and guidance specific to bridges over tidal waterways.

7.6.5.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	23 CFR 650A	Code of Federal Regulations, Title 23, Part 650 Subpart A, *Location and Hydraulic Design of Encroachments on Flood Plains*
2.	T 5140.23	Technical Advisory T 5140.23, *Evaluating Scour at Bridges*, 1991
3.	HEC 18	FHWA HEC 18, *Evaluating Scour at Bridges*
4.	HEC 20	FHWA HEC 20, *Stream Stability at Highway Structures*
5.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
6.	HEC 25	FHWA HEC 25, *Tidal Hydrology, Hydraulics and Scour at Bridges*
7.	HEC-RAS	USACE HEC-RAS, *Hydraulic Reference Manual*
8.	AASHTO HDG Vol. VII	AASHTO Highway Drainage Guidelines, *Volume VII – Hydraulic Analysis for the Location and Design of Bridges*
9.	AASHTO HDG Vol. VIII	AASHTO Highway Drainage Guidelines, *Volume VIII – Hydraulic Aspects in Restoration and Upgrading of Highways*
10.	AASHTO HDG Vol. XI	AASHTO Highway Drainage Guidelines, *Volume XI – Highways Along Coastal Zones and Lakeshores*

7.6.5.2 Standard Practices

7.6.5.2.1 Floodplain Encroachments

New or expanded encroachments on 100-year coastal floodplains should be avoided wherever practicable. If a project requires an encroachment on a 100-year coastal floodplain that is regulated by FEMA or potentially creates an adverse impact to private property or insurable buildings along estuaries, reference Sections 7.6.3 and 7.4.1 for appropriate design policy, standards, and criteria, as well as guidance on FEMA coordination, if required.

7.6.5.3 Design Standards

The following standards apply to bridges on both High- and Low-Standard roadways. Refer to the definitions of High- and Low-Standard roadways in Section 7.1.6.

7.6.5.3.1 Capacity Design

Design bridges to provide the appropriate freeboard above the 50-year storm-tide elevation plus the 50-year wave height.

Temporary Bridges

- *High-Standard Roadways:* Design temporary bridges to remain open to traffic while experiencing the highest astronomic tide plus the 10-year wave height.
- *Low-Standard Roadways:* Design temporary bridges to remain open to traffic while experiencing the highest astronomic tide plus the 2-year wave height.

7.6.5.3.2 Stability Design

The stability design of a bridge foundation refers to its ability to withstand scour.

Design Flood

Use the worst-case scour-producing event up through the 100-year event as the design flood. See HEC 25 for an expanded description of the stability design event.

Check Flood

Use a more severe storm, on the order of a 500-year event, as the check flood. Provide supporting documentation when using an event frequency other than 500-year for the check flood.

Temporary Bridges

- *High-Standard Roadways:* Design temporary bridges to remain stable while experiencing the highest astronomic tide and the 10-year wave height.
- *Low-Standard Roadways:* Design temporary bridges to remain stable while experiencing the highest astronomic tide and the 2-year wave height.

7.6.5.4 Design Criteria

Refer to Section 7.4.3.4. Those criteria apply here, with the following modification to the capacity design criteria:

The reference datum for measuring freeboard is the design storm tide elevation plus the design wave height at the bridge location, on whichever side of the bridge this reference elevation is highest.

7.6.5.5 Design Guidance

Refer to Section 7.4.3.5 for general guidance related to bridge design.

7.6.5.6 Recommended Methods

HEC 25 provides recommended methods for hydraulic and scour analysis of bridges over tidal waterways.

7.6.5.7 Reporting

Section 7.1.11 provides a general list of submittal requirements for hydraulic design projects. Specific additional deliverables for the analysis and design of bridged tidal waterways will include at a minimum:

- For the capacity design event: the water-surface elevation at the bridge; the vertical clearance between the design water surface (storm tide elevation plus wave height) and the lowest point on the low chord; and the percentage of the low chord length that meets the freeboard criterion
- The maximum discharge through the bridge opening for the foundation stability design event
- The maximum velocity through the bridge opening for the foundation stability design event
- The predicted total scour depths and post-scour elevations at each substructure element (shown both graphically and in tabular form)
- Calculations for individual scour components
- Design calculations for any proposed scour countermeasures (i.e. riprap sizing calculations, etc.)
- Design sketches of any proposed scour countermeasures (i.e. abutment riprap, embankment protection, etc.)

7.6.5.8 Plans

Include the following information, as a minimum, in the bridge drawings:

- Location, geometry, and axis alignment of abutments
- Location, geometry, and axis alignment of piers
- Elevations of spread footing bases or pile tips for each abutment and pier
- Existing topography and grading contours in the plan drawing
- The capacity-design water-surface elevation (storm tide elevation plus wave height) in the elevation drawing
- Waterway cross-section geometry in the elevation drawing
- Locations, dimensions, and details for any proposed scour countermeasures
- Magnitude, frequency, and water-surface elevation of overtopping flood or the check flood if overtopping is not possible
- Magnitude, frequency, and water-surface elevation for the 100-year flood if greater than the overtopping flood

7.6.6 ROADWAY EMBANKMENTS

This section provides standards, criteria, and guidance related to the design of roadway embankments parallel and adjacent to coastal shorelines.

7.6.6.1 References

The following references provided source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1. HDS 6 — FHWA HDS 6, *River Engineering for Highway Encroachments*
2. HEC 11 — FHWA HEC 11, *Design of Riprap Revetment*
3. HEC 20 — FHWA HEC 20, *Stream Stability at Highway Structures*
4. HEC 23 — FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
5. HEC 25 — FHWA HEC 25, *Tidal Hydrology, Hydraulics and Scour at Bridges*

6.	AASHTO HDG Vol. XI	AASHTO Highway Drainage Guidelines, *Volume XI – Highways Along Coastal Zones and Lakeshores*
7.	EM 1110-2-1100	U.S. Army Corps of Engineers EM 1110-2-1100, *Coastal Engineering Manual*

7.6.6.2 Standard Practices

7.6.6.2.1 Floodplain Encroachments

Roadway encroachments on 100-year coastal floodplains should be avoided wherever practicable. If a project requires an encroachment on a 100-year coastal floodplain that is regulated by FEMA or potentially creates an adverse impact to private property or insurable buildings along estuaries, reference Sections 7.6.3 and 7.4.1 for appropriate design policy, standards, and criteria, as well as guidance on FEMA coordination, if required.

7.6.6.2.2 Use of Scour and Stream Instability Countermeasures

The stability standards presented in this section will usually be met by using a suitably designed countermeasure to prevent damage to the embankment. Refer to Sections 7.6.7 and 7.4.8 for standards, criteria, and guidance on the design of countermeasures.

7.6.6.3 Design Standards

The standards presented here apply to coastal roadway embankments, with or without retaining walls, for which the profile grade is controlled by tidal water levels and wave heights. Refer to the definition of High- and Low-Standard roadways in Section 7.1.6.

7.6.6.3.1 Capacity Design

- *High-Standard Roadways:* Design coastal roadway embankments to provide adequate freeboard above the 50-year storm-tide elevation plus the 50-year wave height. Freeboard is defined in Section 7.6.6.4.
- *Low-Standard Roadways:* Design coastal roadway embankments with the profile grade above the highest astronomic tide plus the 25-year wave height.

7.6.6.3.2 Stability Design

- *High-Standard Roadways:* Design coastal roadway embankments with protection as needed to remain stable against the 50-year storm surge and 50-year wave attack.
- *Low-Standard Roadways:* Design coastal roadway embankments with protection as needed to remain stable against the 25-year wave attack.

7.6.6.4 Design Criteria

The criteria presented here apply to coastal roadway embankments, with or without retaining walls, for which the profile grade is controlled by tidal water levels and wave heights.

7.6.6.4.1 Capacity Design

- *High-Standard Roadways:* Design coastal roadway embankments with a minimum freeboard of 0.6 m [2.0’]. Freeboard is defined as the vertical distance between the design water surface and the bottom of the aggregate base layer of the pavement structure.

- *Low-Standard Roadways:* No freeboard required since overtopping is a desired and cost-effective mechanism for providing hydraulic relief.

7.6.6.4.2 Stability Design

Demonstrate that the embankment is reasonably expected to remain stable, with or without protection by countermeasures, up through the stability design standard throughout the intended service life of the embankment.

7.6.6.5 Design Guidance

Prominent concerns in the design of coastal highway embankments are:

- Preventing an unacceptable frequency of service interruption by high water (e.g. storm surges) and waves

- Protecting the roadway embankment from destruction by wave attack up through the stability design event.

- Protecting the roadway embankment from scour by adjacent parallel or impinging currents.

It is necessary, therefore, to determine the peak storm tide elevation, the expected significant wave height and the peak velocity of any adjacent parallel or impinging current associated with the design recurrence interval. In some cases it may be necessary to analyze numerous types of events to develop the design parameters. In some locations, for instance, the highest storm tide with a 50-year recurrence interval may be generated by an extratropical storm, such as a Nor’easter, while the 50-year currents and waves may come from a hurricane.

The appropriate level of study to determine the design wave height depends upon several factors, including: the location of the facility; the orientation of the water body with respect to the facility; the straight-line length of the fetch along the anticipated wind direction; the depth of water along the fetch; and the anticipated speed and duration of sustained winds.

7.6.6.6 Recommended Methods

HEC 25 gives detailed guidance for analyzing tide levels, hydraulics, and scour potential in tidal waterways.

U.S. Army Corps of Engineers Manual EM 1110-2-1100 provides guidance on wave height prediction. The methods in EM 1110-2-1100 should generally be used in locations that are not subject to attack from large waves. Examples of appropriate locations include small bays or channels protected by barrier islands, and inland reaches of estuaries. At locations subject to attack by large waves, such as the open ocean coastline, the wave height determination should employ more extensive coastal engineering approaches, including numerical wave modeling.

7.6.6.7 Reporting

Section 7.1.11 provides a general list of submittal requirements for hydraulic design projects. Specific additional deliverables for the analysis and design of coastal roadway embankments will include at a minimum:

- A map or aerial photograph of the affected coastal area showing the embankment location
- If a detailed tidal hydraulic analysis was developed, a map showing the model limits, boundary condition locations, and cross section locations
- A profile drawing showing the design storm tide level and wave height along the embankment
- The predicted total scour depths and post-scour elevations at intervals along the toe of the embankment
- Design calculations for any proposed embankment protection (i.e. riprap sizing calculations)
- Design sketches of any proposed embankment protection, showing longitudinal extent, required thickness of protection, and termination requirements (i.e. toe downs and end terminations)

7.6.6.8 Plans

If protection has been designed for the embankment, then the following must be included on the final design plans:

- Details and dimensions of any required protection
- A profile drawing showing the design storm tide level and wave height along the embankment

7.6.7 SCOUR AND STREAM INSTABILITY COUNTERMEASURES

This section provides standards, criteria, and guidance for the design of countermeasures in coastal areas.

7.6.7.1 References

The following references provide source information for the development of the standards, criteria, and guidance of this subsection (most recent editions apply):

1.	HDS 6	FHWA HDS 6, *River Engineering for Highway Encroachments*
2.	HEC 11	FHWA HEC 11, *Design of Riprap Revetment*
3.	HEC 23	FHWA HEC 23, *Bridge Scour and Stream Instability Countermeasures*
4.	HEC 25	FHWA HEC 25, *Tidal Hydrology, Hydraulics and Scour at Bridges*
5.	AASHTO HDG Vol. VII	AASHTO Highway Drainage Guidelines, *Volume VII – Hydraulic Analysis for the Location and Design of Bridges*
6.	AASHTO HDG Vol. XI	AASHTO Highway Drainage Guidelines, *Volume XI – Highways Along Coastal Zones and Lakeshores*
7.	AASHTO MDM Chap. 17	AASHTO Model Drainage Manual, *Chapter 17 – Bank Protection*
8.	Caltrans Chap. 870	California Department of Transportation Highway Design Manual, Chapter 870 – *Channel and Shore Protection-Erosion Control*
9.	EM 1110-2-1100	U.S. Army Corps of Engineers EM 1110-2-1100, *Coastal Engineering Manual*

7.6.7.2 Standard Practices

The potential for scour and stream instability will be considered when designing highway facilities that interface with shorelines and tidal waterways (see Section 7.6.4). Where it is impracticable or inappropriate to accommodate the estimated scour or stream instability in the design of the facility, countermeasures will be used to mitigate the potential for damage.

7.6.7.2.1 Bridge Piers

New piers will be designed so that they withstand the estimated total scour depth from the design flood or event without the need for countermeasures (see Section 7.4.3). The piers of bridges to be rehabilitated may be protected from scour by countermeasures as appropriate.

7.6.7.3 Design Standards

7.6.7.3.1 Stability Design

Scour and stream instability countermeasures will be designed to meet the appropriate stability standards for the structures they are intended to protect. Specific references to appropriate standards are provided below:

Foundations of Bridge Abutments and Existing Piers

Refer to Section 7.6.5.

Roadway Embankments

Refer to Section 7.6.6.

7.6.7.4 Design Criteria

Refer to Section 7.4.8.4.

7.6.7.5 Design Guidance

Refer to Section 7.4.8.5 and consider additional guidance related specifically to the design of scour and stream instability countermeasures in coastal areas.

7.6.7.5.1 Wave Attack

When designing countermeasure installations in coastal environments that will be subject to wave attack, consider the potential for the countermeasure to be destroyed or compromised by wave attack. The riprap size required to resist wave attack is often larger than that required to withstand the computed current velocity.

7.6.7.5.2 Filter Requirements

Designing riprap countermeasures in tidal waterways can be particularly challenging with respect to filtering, because of the very fine bed sediments that often exist in tidal waterways. The problem is compounded by the relentless pumping action cause by wave impacts. Unless the design adequately prevents the migration of fine sediment through the revetment section, the countermeasure may settle or unravel, thus becoming ineffective. Take special care to provide an adequate filter based on site-specific bed sediment characteristics.

7.6.7.6 Recommended Methods

HEC 25 and the U.S. Army Corps of Engineers' EM 1110-2-1100 provide methods and procedures for determining wave heights and designing countermeasures to withstand wave attack.

7.6.7.7 Reporting

Refer to Section 7.4.8.7.

7.6.7.8 Plans

Refer to Section 7.4.8.8.