CHAPTER 7 - HYDROLOGY/HYDRAULICS

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CHAPTER 7 - HYDROLOGY/HYDRAULICS

7.1 GENERAL

This chapter emphasizes the policies and techniques for investigating and designing highway related water resource systems. To encourage a broader perspective, the term *water resources* is often used in this section instead of *drainage* or *hydraulics* to emphasize that water represents a vital resource having multiple values to both the natural and human environment. Water resource systems and highway systems should, therefore, be designed to complement one another.

Since the technical complexity of water resource analysis covers a broad range of literature, only general guidelines for such analysis are provided. To facilitate implementation of these guidelines, sample checklists and forms are shown as exhibits at the end of the chapter. Each checklist and form is cross-referenced to the appropriate section for explanation and background information.

For an in-depth approach, the designer should consult either the technical references given in the following discussions or a hydraulics engineer. Highway designers should derive enough information from this chapter to properly integrate highway design and water-related systems. Table 7-1 lists the division staff with primary and support responsibilities for design of drainage facilities.

A. Highway Designer's Responsibilities. The designer is responsible for the overall design of the highway and should be familiar with the available guides, the investigation and design processes, the required approvals, and the reporting practices associated with hydrology and hydraulics. The designer should understand the general value of water resource systems such as reservoirs, irrigation canals, potable water supplies, and fish/wildlife habitat.

The designer should acquire available field and aerial surveys for proper hydrologic and hydraulic analysis of the water system, and should investigate the erosion and sediment characteristics of the general highway environment.

The hydrologic process requires the knowledge of the various methods used to determine the volume of water per unit time for a given design situation. The hydraulic process also requires the understanding of open channel flow, culvert design, erosion control methods, and bridge waterways. In hydrologic and hydraulic analyses, the designer must understand the trade-offs involved when weighing risks against total design costs. To develop a scope of water-related responsibilities, the designer should become familiar with the guidelines contained in this chapter.

B. Hydraulic Engineer's Responsibilities. The hydraulics engineer is responsible for providing technical assistance and guidance to the designer for complex drainage problems.

These types of problems include the design of bridge waterways, large culverts, urban drainage systems, and environmental mitigation systems. (Environmental systems include flood protection in urban areas, fish passage designs, water quality, and wetlands replacement.)

The hydraulics engineer is also a source of information concerning applicable water resource laws, FHWA water related policies, AASHTO design policies, and current developments in water resource system designs. The hydraulics engineer should be consulted early during project development for hydrologic input and roadside channel design criteria. The hydraulics engineer should review the final PS&E of all highway designs that have water related systems.

Table 7-1
Staff Design Responsibilities

Drainage System	Division Staff				
	Design	Hydraulics	Bridge	Geotechnical	Environmental
Culverts, 1200 millimeters and smaller	P	S			
Culverts, larger than 1200 millimeters	S	P			
Bridge waterway analysis	S	P	S		
Ditches	P	S			
Small stream channels	S	P			
Urban drainage	S	P			
Environmental mitigation	S	P			S
Underdrain systems	S	S		P	
Erosion protection	S/P	P/S		S	S
Water quality	S	P			S

P = Primary responsibility (performs most of the design)
S = Secondary responsibility (provides consultation and support for the design)

7.2 GUIDANCE AND REFERENCES

This section briefly discusses the laws, regulations, policies, and guidelines for integrating water resource systems with proposed highway systems. To implement these regulations and policies, guidelines (i.e., hydraulic engineering circulars) have been developed. The following information provides the designer with background and reference materials for incorporating these laws, policies, and guidelines into the highway design process.

A. Applicable Laws. Federal, State, and local governments have many laws and regulations related to water resource systems. These will generally affect the design, approval, and construction of water resource designs. The designer should be familiar with these laws and regulations. These laws are summarized in AASHTO's *Highway Drainage Guidelines* - *The Legal Aspects of Highway Drainage,* Volume V.

1. Federal Laws. Federal laws provide protection for water quality, fish and wildlife, flood disasters, navigable and non-navigable waters, and coastal zones. These protective laws are formulated under various legislative acts.

- Federal-aid Highway Act (1970).
- National Environmental Policy Act (1969).
- Flood Disaster Protection Act (1973).
- River and Harbor Act (1899).
- Federal Water Pollution Control Act (1972).
- Fish and Wildlife Coordination Act (1956).
- Tennessee Valley Authority Act (1933).
- Coastal Zone Management Act (1972).

The laws and regulations are numerous, powerful, and complex. The following is a brief summary of the most common and important regulations affecting highway designs and water resources.

a. National Environmental Policy Act (NEPA). This act is the primary Federal regulation for the protection of the environment. (See Chapter 3).

Although the environmental engineer performs the evaluation and preparation of the documents required by NEPA, the designer as well must consider the impacts of highway designs upon water quality and ecological systems such as wildlife, fish, and marine life.

(1) Water Quality. Water quality may be affected by the following:

- Surface runoff, such as paved surfaces replacing natural surfaces.
- Sedimentation and erosion caused by such factors as loss of vegetation due to highway construction.
- Chemical composition, such as pH, turbidity, oxygen demand levels, and minerals (i.e., pavement and soil leaching).
- Ground water quantity and quality.

(2) Ecological Systems. The ecological systems that could be affected are as follows:

- Habitat diversity related to fisheries and wetlands.
- Size and quality of habitat area.
- Interrelationships with water quality factors.

The designer can still strive to integrate highway systems into the natural water resource systems to minimize negative impacts and enhance the positive characteristics of both systems.

b. Flood Disaster Protection Act. Under this act, local communities must have land use controls to qualify for the National Flood Insurance program. This program applies primarily to urban areas that have households located within a designated flood-way area.

This flood-way area is designated by the Federal Emergency Management Administration (FEMA). Generally, the flood-way limits are defined by the 100-year flood plain. Flood insurance maps may be obtained that define the regulated flood ways for a given area. These maps may be acquired from the State or local planning agency that administers the flood insurance program.

If the highway encroaches upon the 100-year flood plain, the increase in water surface elevation must be evaluated for various flood frequencies. If the encroachment area lies within a regulated flood way, then local land-use controls may limit the increase of flood patterns caused by the encroachment (i.e., maximum 300 millimeters increase for the 100-year flood). Flood insurance regulations within the project area may affect the proposed highway hydraulic designs.

c. Navigable and Non-navigable Water Acts. The Federal Water Pollution Control Act and the River and Harbor Act may require permits to be obtained for the following:

- Dredging or filling into navigable or non-navigable waters (including wetlands and lakes).
- Erecting structures over navigable waters.
- Discharging pollutants (i.e., storm sewer systems).

For filling and dredging in lakes and streams, the designer should contact the U.S. Army Corps of Engineers (USACE). For erection of structures over navigable waters, the designer should consult both the U.S. Coast Guard and the U.S. Corps of Engineers. For point discharges of pollutants, the U.S. Environmental Protection Agency should be consulted. These agencies should be contacted for permit approval during the preliminary design stages.

d. Fish and Wildlife Coordination Act. Federal agencies are required by this act to contact the U.S. Fish and Wildlife Service, Department of Interior, when there are plans to modify the waters of any stream or body of water for any purpose. Although the U.S. Fish and Wildlife Service does not have direct permitting authority, the design engineer should be aware that a Memorandum of Understanding (MOU) exists between the U.S. Fish and Wildlife Service and the U.S. Corps of Engineers giving the U.S. Fish and Wildlife Service considerable influence in the issuance of fill and dredging permits.

Fill and dredge permits for such highway features as bridge abutments, roadway embankments, and material sources will be the most common permits obtained for highway activities in water resource systems. The classification of water resource systems is explained in the *Classification of Wetlands and Deep water Habitats of the United States* by the U.S. Fish and Wildlife Service. Also, *A Method for Wetlands Functional Assessment*, Volumes I and II by the Federal Highway Administration should be reviewed.

e. Tennessee Valley Authority (TVA) Act. The TVA is responsible for the conservation and development of the Tennessee River Valley and surrounding area. Highway designs that are located in or affect this region must be approved by the TVA. Approval is based upon impacts on the general quality of the environment, control of pollution, and attainment of State water quality certification. Also, the TVA may require an environmental assessment before issuing the permit.

f. Coastal Zone Management Act. This act encourages State and Federal agencies to develop procedures and programs for managing coastal areas. Although the National Oceanic and Atmosphere Administration is responsible for administering the provisions of this act, the States in cooperation with local governments generally develop and administer coastal zone management programs. These programs define the coastal zone boundaries, define the acceptable land and water uses, and describe the procedures for implementing the coastal zone management program.

If a proposed highway project is located within a coastal management zone, a certificate from the administering agency may be required. The certificate will state that the proposed project is consistent with the local coastal zone management program.

2. State and Local Laws. State laws related to water resources generally are based upon common laws and statutory laws. Common laws are derived from long-standing usages and customs, while statutory laws are developed by legislative governments. Under common law, water-related legalities are based upon a classification system of surface waters, stream waters, flood waters, and groundwaters. Under statutory law, water-related legalities are based upon legislative mandate generally defined by eminent domain, water rights, water resource districts, agriculture drainage, and environmental laws.

At the State level, the most common water-related legal problems involve diversion, collection, concentration, augmentation, obstruction, erosion and sedimentation, and groundwater interference. Since laws related to these problems will vary from State to State, the following is a brief generalization of each problem:

a. Diversion. This relates primarily to the detention, or changing the course, of a stream or drainage way from its natural or existing condition. Depending upon the type or 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 upon property owners upstream, downstream, and adjacent to the project. Any changes in the flow characteristics due to the diversion may require mitigation with the affected property owners. Also, water 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. Basically, diversions of streams or drainage ways should be designed to recreate flow conditions (i.e., depth velocity flowrates, backwater) that are as similar as possible to those that existed before the diversion while still accomplishing the highway design objectives.

b. Collection, Concentration, and Augmentation. Water may be collected and concentrated by a highway drainage system, causing flow rates at the point of discharge to be in excess of those flow rates that would naturally occur without the drainage system. Also, the concentration of flow upstream from a primary culvert may increase the backwater elevations at the culvert inlet. If this occurs, drainage easements and mitigation may be required if the backwater extends beyond the highway right-of-way.

On RRR projects, the flow rates should not be appreciably affected. However, flow rates should be compared in terms of *before* and *after* conditions for proper evaluation of the highway drainage system and protection from potential litigation problems associated with the drainage system.

c. Obstruction. As discussed under collection, concentration, and augmentation, backwater from culverts, and bridges, may require special measures. Culverts and bridges generally affect the flow characteristics within their proposed locations. These structures can cause backwater upstream, increase velocities in the structure area, and affect erosion and sedimentation characteristics downstream. The effects can be significant. Evaluate the effects for various flow conditions and contact impacted individuals and agencies such as adjacent property owners and State water resource agencies. Based upon feedback from the affected parties, the appropriate flow conditions and hydraulic characteristics can be determined.

The evaluation of obstructions may be rather complex and the designer should consult hydraulic and other technical specialists. The designer should ensure that the proper analysis is performed and documented for evidence of compliance with State-related laws.

d. Erosion and Sedimentation. Highways and their structures can have pronounced impacts upon erosion and sedimentation characteristics of a water resource system. If the flow characteristics are significantly changed, then the erosion and sedimentation characteristics will also be changed. Coordinate the design with the appropriate individuals and State agencies that may be affected by a change in these characteristics. Normally, the individuals and State agencies involved in diversions, augmentations, and obstructions will have an interest in highway-affected erosion and sedimentation.

e. Groundwater Interference. Legal problems may occur if proposed highway designs and construction alter the quantity and/or quality of groundwater systems. Legal suits may be initiated by individual owners or public agencies who have groundwater supplies altered by construction operations or encounter groundwater contamination caused by highway runoff. Legally, groundwater is considered to be a part of the real property in which it lies. Therefore, evaluate the potential for groundwater impacts by the proposed highway design. If potential problems do exist, the designer should consult technical experts like the hydraulics engineer or geotechnical engineer.

Local laws, like State laws, may vary considerably from locality to locality. However, most local governments have regulations or statutes that pertain to

- the Flood Disaster Protection Act,
- municipal water supplies, and
- storm water management.

In coastal areas, local governments also participate in the management of coastal zoning.

Because of the variation in local statutes, the designer should consult the county and/or city officials that have jurisdiction over the water resources in the area of the proposed highway project.

B. Federal-Aid Policy Guide (FAPG). The following FHPM sections relate to water resource considerations on FLH projects.

1. Permits. FAPG 650H stipulates that the USACE and U.S. Coast Guard should be contacted for required navigational clearances under bridges. This subsection also discusses the permits required for highway work in or adjacent to streams. While the information contained in this subsection is written primarily for Federal-aid projects, it provides references and historical information related to water resource permits. For additional guidance, see Chapter 3.

2. Erosion and Sediment Control. FAPG 650B specifies that highway projects must be located, designed, constructed and operated according to standards that will minimize erosion and sediment damage to the highway and adjacent properties and prevent pollution of surface and groundwater resources. This subsection provides information for developing erosion and sediment control plans. The erosion and sediment control measures discussed in this subsection include structures for erosion control, detention or sedimentation basins, and soil treatments.

3. Flood plain Encroachments. FAPG 650A specifies that highway projects shall be evaluated for their impacts and costs when they encroach upon flood plains. This evaluation includes an assessment of the capital costs, risks, natural flood plain values, and human flood plain values associated with the encroachment. To complete this evaluation, location studies, hydrologic and hydraulic analysis, and design documentation shall be performed by environmental and design personnel.

The designer must assess the benefits of the highway encroachments against the costs. Both tangible and intangible factors are to be considered. Often, an optimal design can be found that can balance both the inherent risks and costs of the project.

4. Mitigation of Environmental Impacts to Privately Owned Wetlands. FAPG 777 requires State agencies to evaluate and mitigate adverse environmental impacts to privately owned wetlands caused by new construction of Federal-aid highway projects. These laws and regulations also apply to FLH projects that affect privately owned wetlands.

Generally, fish and wildlife officials require some type of mitigation if a highway project encroaches upon a wetlands. The loss or disruption of wetlands can be mitigated by enhancing nonimpacted wetlands, creating new wetlands within the highway right-of-way, or acquiring additional right-of-way for the sole purpose of developing new wetlands. Before wetland losses are mitigated, evaluate the wetland in terms of its vegetative, hydrologic, hydraulic, wildlife, and fish habitat characteristics. Based upon this evaluation, proposed mitigation measures for any adverse wetlands impacts can be developed.

C. FHWA Technical Advisories. FHWA advisories provide technical guidance for the design of highways. In water-related areas, the designer will find few technical advisories. The designer should contact the Hydraulics Engineer for the most current technical advisories.

D. AASHTO Guidelines. The American Association of State Highway and Transportation Officials (AASHTO) published eleven volumes of Highway Drainage Guidelines. AASHTO is currently developing two additional volumes.

These guidelines provide qualitative information on the planning, hydrology, hydraulics, and legal aspects of highway drainage design.

The first seven of these guidelines are summarized as follows:

- Volume I, *Hydraulic Considerations in Highway Planning and Location*. This volume summarizes the general aspects of highway drainage design. The topics include preliminary drainage surveys, flood hazards, location problems, construction and maintenance problems, coordination with water resource systems, permits, legal considerations, reports, and documentation.

- Volume II, *Hydrology*. This volume describes those hydrologic aspects that are most important for determining the design flowrate. The topics discussed include drainage basin characteristics, stream channel characteristics, flood plain characteristics, precipitation, flood history, selection of design flood frequency, and the prediction of flood magnitudes.

- Volume III, *Erosion and Sediment Control in Highway Construction*. This volume discusses erosion and sediment aspects of natural drainage patterns, geology and soils, geometric design, drainage design, construction practices, and maintenance practices.

- Volume IV, *Hydraulic Design of Culverts*. This volume covers the basic highway functions of a culvert and its impact on the surrounding environment. The topics discussed include surveys (i.e., topographic, drainage area, channel characteristics, fish life, high water information, existing structures, field reviews), culvert location, culvert type, culvert hydraulic design, multiple-use culverts (i.e., utilities, stock and wildlife passage, land access, fish passage), irrigation, debris control, service life, safety, design documentation, construction considerations (i.e., temporary erosion control), and maintenance considerations.

- Volume V, *Legal Aspects of Highway Drainage*. This volume provides supplemental information about the legal aspects of highway drainage facilities. The topics described include Federal, State, and local laws (i.e., common and statutory), common drainage complaints, and legal remedies.

- Volume VI, *Hydraulic Analysis and Design of Open Channels.* Open channel hydraulics is a complex and dynamic subject. Simplified design methods and computer programs have evolved over the years that may be used by the designer for the hydraulic analysis of highway systems. However, many of the methods may be inappropriate for certain situations. A culvert may be analyzed with simple charts or calculator programs to determine the general flow characteristics, while a major bridge design across a flood plain will probably require considerable field data and computer analysis for proper determination of the associated flow conditions.

  This volume provides only enough information for familiarity with the subject matter. It provides general information on planning, surveys, hydrology, types of flow conditions, analysis of open channel flow, stream morphology, effects of channel alterations, channel stabilization and bank protection, roadside drainage channels, structural considerations, construction considerations, and maintenance considerations.

- Volume VII, *Hydraulic Analysis for the Location and Design of Bridges.* This volume contains considerable information for the location and hydraulic design of bridges. The primary topics discussed include planning and location, surveys, hydrologic analysis, hydraulics of the stream, stream crossing design, deck drainage, design documentation, construction considerations, and maintenance considerations. Bridges represent an important and expensive part of highway systems. They may also have considerable impact upon their water-related environment.

- Volume VIII, *Hydraulic Aspects in Restoration and Upgrading of Highways.* The other AASHTO guidelines primarily discuss drainage aspects relative to new highway construction. This guideline discusses highway drainage involving existing highways and their drainage structures. The designers should incorporate the information from this volume with the other drainage volumes for highway restoration/upgrade projects.

- Volume IX, *Storm Drain Systems.* This volume provides guidance on the compatible design of storm drain systems and existing drainage impacts to existing drainages, protecting the traveling public from large runoff events (i.e., pavement drainage), and protecting the roadway structure (i.e., embankment, subgrade, pavement) from surface runoff.

- Volume X, *Evaluating Highway Effects on Surface Water Environments*. This volume presents design practices for protecting the surface water environment from highways. The volume discusses the surface water environment in general. It also discusses the effects and their significance of highway drainage designs to the surface environment. Furthermore, it discusses threshold values that the designer and other technical personnel should realize when assessing the interrelationship of highway drainage systems with the surface water environment.

- Volume XI, *Highways Along Coastal Zones and Lake Shores.* This volume discusses the hydraulic aspects of highways in coastal-type environments. These environments include tidal basins, bays and estuaries, large lakes and reservoirs, and the lower reaches of major river systems. These guidelines address the special details of wind, wave, current, and tidal actions upon banks and shores. Such details may significantly affect the design and construction of highways in the coastal environment.

AASHTO is developing additional guidelines, Volume XII on Stormwater Management and Volume XII on Training.

E. Agency Agreements and Standards. Existing statutes, regulations, agency policies, guidelines, and standards influence the integration of highway systems into water resource systems. For this reason, agency project agreements and standards may vary the design criteria for a specific project and the designer should have copies of all such agreements and standards in the project files before proceeding with the drainage design.

F. AASHTO Model Drainage Manual (MDM), 1991. This manual presents 21 chapters on design procedures, example problems, and computer solutions for most aspects of highway hydraulic design. Although the manual is rather generic, it presents large volumes of relevant information for the design of highway drainage systems. Highway designers, hydraulic engineers, and other technical personnel should have access to this important reference manual.

G. Technical References. The publications and computer programs listed in this section provided much of the fundamental source information used in the development of this chapter. While this list is not all inclusive, the publications listed will provide the designer with additional information to supplement this manual. These publications and programs provide the technical assistance for developing and completing highway-related water resource designs.

FHWA publications are generally available from the National Technical Information Service, Springfield, Virginia, 22161.

FHWA Hydraulic Engineering Circulars

HEC No. 1, *Selected Bibliography of Hydraulic and Hydrologic Subjects*. 1983.

HEC No. 9, *Debris-Control Structures.* 1971.

HEC No. 10, *Capacity Charts for the Hydraulic Design of Highway Culverts.* 1972.

HEC No. 11, *Design of Riprap Revetment*. 1989.

HEC No. 12, *Drainage of Highway Pavements*. 1984.

HEC No. 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*. 1983.

HEC No. 15, *Design of Roadside Channels with Flexible Linings*. 1988.

HEC No. 17, *The Design of Encroachments on Flood plains Using Risk Analysis*. 1981.

HEC No. 18, *Evaluating Scour at Bridges.* Edition 2. 1993.

HEC No. 19, *Hydrology*. 1984.

HEC No. 20, *Stream Stability at Highway Structures.* 1991.

HEC No. 21, *Bridge Deck Drainage Systems*. 1993.

FHWA Hydraulic Design Series

HDS No. 1, *Hydraulics of Bridge Waterways.* Second Edition. Revised 1978.

HDS No. 2, *Hydrology*. SI Version (Planned 1994).

HDS No. 3, *Design Charts for Open-channel Flow*. 1961 (reprinted 1973).

HDS No. 4, *Design of Roadside Drainage Channels*. 1965. (SI Version, Planned 1994).

HDS No. 5, *Hydraulic Design of Highway Culverts.* 1985. (SI Version, Planned 1994).

HDS No. 6, *Bridges (WSPRO).* SI Version (Planned 1994).

General Publications

A Guide to Standardized Highway Drainage Products. AASHTO-AGC-ARTBA Joint Committee. 1986.

SCS National Engineering Handbook. Soil Conservation Service. 1985.

Design and Construction of Sanitary and Storm Sewers. Manual No. 37. American Society of Civil Engineers. 1976.

Chow, Ven Te. *Open Channel Hydraulics*. 1959.

Highways in the River Environment. DOT, FHWA. 1990.

Bratner, Ernest F. and King, Horace W. *Handbook of Hydraulics.* 1976.

Viessman Jr, Warren et al. *Introduction to Hydrology*. 1977.

Handbook of Steel Drainage and Highway Construction Products. American Iron and Steel Institute. 1993.

Concrete Pipe Design Manual. American Concrete Pipe Association. June 1987.

Concrete Pipe Handbook. American Concrete Pipe Association. 1988.

Aluminum Drainage Products Manual. Aluminum Association. June 1983.

Highway Runoff Water Quality Training Course, Student Workbook. DOT, FHWA. 1985.

Classification of Wetlands and Deep Water Habitats of the United States. U.S. Fish and Wildlife Service. December 1979.

A Method for Wetlands Functional Assessment. Volumes I and II. DOT, FHWA. March 1983.

Gray, Donald H. and Leiser, Andrew T. *Biotechnical Slope Protection and Erosion Control*. 1982.

Guidelines for Determining Flood Flow Frequency. Bulletin No. 17B. U.S. Department of the Interior, Geological Survey. March 1982.

Computation of Water-Surface Profiles in Open Channels. U.S. Department of the Interior, Geological Survey. 1984.

Highway Water Quality Monitoring Manual. DOT, FHWA. January 1979.

Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method. FHWA-RD-77-159. DOT, FHWA. 1977.

Five to Sixty Minute Precipitation Frequency for Eastern and Central United States. NWS HYDRO-35. U.S. Department of Commerce, National Oceanic and Atmospheric Administration. 1977.

Precipitation-Frequency Atlas for the Western United States. NOAA Atlas 2. U.S. Department of Commerce, National Oceanic and Atmospheric Administration. 1973.

Rainfall Frequency Atlas of the United States. U.S. Weather Bureau TP40. 1961.

Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains. FHWA TS84-204. DOT, FHWA. April 1984.

Design of Urban Highway Drainage. FHWA- TS-79-225. DOT, FHWA. August 1979.

Culvert Inspection Manual. FHWA IP86-2. DOT, FHWA. 1986.

Accuracy of Computed Water Surface Profiles. U.S. Army Corps of Engineers. December 1986.

FHWA Computer Programs

- HY-2 Hydraulic Analysis of Pipe-arch Culverts. 1969 (Mainframe)
- HY-4 Hydraulics of Bridge Waterways. 1969. (Mainframe)
- HY-6 Hydraulic Analysis of Culverts (Box and Circular). 1979. (Mainframe)
- HY-7 Bridge Waterways Analysis Model, WSPRO Research Report. 1986. (Micro)
- HY-8 FHWA *Culvert Analysis.* (Version 4.3). 1994. (Micro)
- HY-9 *Scour at Bridges.* (Version 4.0). 1991.
- HY-10 *BOXCAR.* (Version 1.0). 1989. (Micro)
  PIPECAR (Version 2.1). 1994. (Micro)
  CMPCHECK (Version 1.0). 1989. (Micro)
- HY-11 *Preliminary Analysis System for WSP*. 1989. (Micro)
- HY-12 *FESWMS-2DH.* 1989. (Micro)
- HY-TB *Hydraulic Toolbox* (HEC 12, 14, & 15). 1989. (Micro)
- CANDE *CANDE-89*. 1989. (Micro)
- BRI-STARS. 1994. (Micro)
- HYDRAIN, *Drainage Design System.* (Version 5). 1994. (Micro)

USACE Computer Programs

- HEC-1 Flood Hydrograph Package. 1990. (Micro)
- HEC-2 Water Surface Profile. 1990. (Micro)
- HEC-5 Simulation of Flood Control and Conservation
- HEC-6 Scour and Deposition in Rivers and Reservoirs. 1991. (Micro)
- TR-20 and TR-55, Natural Resource Conservation Service

FHWA Calculator Design Series

- CDS No. 2 Hydraulic Design of Improved Inlets for Culverts Using Programmable Calculators, (HP-65) October 1980
- CDS No. 3 Hydraulic Design of Improved Inlets for Culverts Using Programmable Calculators, (TI-59) January 1981
- CDS No. 5 Hydraulic Design of Stormwater Pumping Stations Using Programmable Calculators, (TI-59) May 1982

7.3 INVESTIGATION PROCESS

In the investigative process, the designer should obtain all available field and aerial survey data, water resources data, geomorphological data, and water quality data that is related to the highway design. This section basically describes the information required for each of these items and the methods for obtaining the required water resource data for highway design purposes. As an aid, check lists for identifying and recording necessary water resource data have been included as Exhibits. When completed, Exhibits 7.1 through 7.5 will provide the designer with a general listing of the critical investigative data necessary for proceeding with the roadway design.

A. Field and Aerial Survey. The designer should strive to obtain accurate and comprehensive field and aerial survey data. Hydrologic and hydraulic analyses are only as good as the data they are based upon. The following is a brief summary of the survey data required for these processes.

1. Existing Surveys. Survey data should be obtained to encompass as much of the proposed highway project and surrounding areas as possible. The United States Geological Survey (USGS) quadrangle maps provide a good starting point for evaluating the highway system and related water resource systems. These maps normally have a plan scale of 1:24 000 to 1:64 000. The USGS maps will show existing highway corridors, natural streams and their tributary systems, manmade structures (i.e., bridges, dams), and topographic features such as lakes, flood plains, stream patterns, and drainage basins with their respective shape, size, and elevations.

Other survey data should be obtained from as many different sources and for as many different time periods as possible. This data can be used to develop a historical perspective of the manmade and natural water resource developments within the project area.

If existing survey data is limited, then the USGS quad maps may indicate other Federal and State agencies that have performed surveys within the area. For example, man-made reservoirs may indicate design involvement by the U.S. Bureau of Reclamation. If so, then the U.S. Bureau of Reclamation may have conducted field, aerial, and hydrologic surveys of the project area. Other agencies that may have conducted surveys include USACE, FS, NPS, U.S. Fish and Wildlife, State and County Highway Departments, and various resource agencies that may have had interests in the project area. If survey data is obtained from outside agencies, then the designer should ensure that this survey data can be correlated. For example, all surveys should be converted to the same elevation datum.

Both aerial and field surveys may be used. Aerial surveys provide a general perspective of the water resource system. Specifically, they can be used to define the flood plain limits of a stream or river. Aerial surveys will only show the water surface elevation of the main stream, but not its channel bottom. Usually field survey data will be required for the channel cross section. The following is a checklist of items that should be evaluated for existing field and/or aerial surveys.

Items to be Evaluated	Type of Survey	
	Field	Aerial[1]
Drainage Areas:		
• Size		X
• Shape		X
• Elevations		X
• Vegetation		X
• Lakes/Reservoirs		X
• Urban Development		X
Natural Streams and their Tributaries:		
• Floodplains		X
• Stream Channels	X[2]	X[3]
• Vegetation	X	X
• Sedimentation	X	
Man-Made Structures:		
• Culverts	X	
• Revetments	X	X
• Bridges	X	X
• Reservoir/Dams	X	X
• Gaging Stations	X	X
• Water Supply and Waste	X	
• Water Treatment Systems	X	X

Note:
[1] Includes USGS topographic maps.
[2] Cross sections
[3] Location and pattern

2. Proposed Surveys. Review all available existing field survey records and determine the extent and type of additional data needed to complete the investigation and analysis of the water-related systems. For determining the extent of additional data, the following is a brief summary of the minimum required survey information.

a. Drainage Areas. Data is obtained for drainage areas primarily to determine the amount of water within a given water resource system. Most of the required data for this process can be obtained from USGS quad maps. The information required for this item includes drainage area size (acres or square miles), drainage area shape (e.g., narrow and deep versus shallow and wide), elevation differences between the drainage area divide and the point at which the volume of flow is to be evaluated (e.g., bridge location), type of vegetation (e.g., percent of forests), urban development (e.g., percent of drainage area), and storage capacity (e.g., lakes and reservoirs for collecting rainfall). If this information cannot be obtained from USGS quad maps, it may be necessary to conduct aerial or field surveys to determine the drainage area characteristics.

b. Natural Streams and Their Tributaries. Data is obtained for natural streams and their tributaries primarily to determine the flow characteristics that occur in them such as depth of flow, velocity of flow, sedimentation, and erosion. Generally, a combination of USGS quad maps, aerial surveys, and field surveys will be required to obtain sufficient analytical data. The information required for this item includes flood plain width, main channel cross sections, cross sections of tributary channels, flood plain vegetation characteristics (i.e., grasses, trees), channel bedding (i.e., sand, clay, gravel, and boulders), channel bed slope, channel water surface slope, flood plain and channel forms (i.e., straight or curved, pools and riffles), and high water marks. Of this information, the channel cross sections are the most costly and difficult to obtain. Also, the exact locations for obtaining channel cross section is primarily judgmental. Basically, channel cross sections should be taken at locations where there is a significant change in channel width, slope, bedding, and/or alignment. In any event, the hydraulics engineer, should be consulted and/or the USGS publication Computation of *Water-Surface Profiles in Open Channels*, and the USACE publication *Accuracy of Computed Water Surface Profiles* reviewed before submitting requests for channel cross sections to the Survey engineer.

c. Manmade Structures. These are normally culverts and bridges. For culverts, data for the existing type, size, shape, length, inlet elevation, outlet elevation, top of roadway elevation, and approaching and existing stream channel cross sections will be required for analyzing the existing hydraulic conditions. If a culvert does not exist at a proposed culvert location, then channel cross sections should be obtained at the proposed culvert inlet and outlet. For bridges, cross sections should be obtained at the proposed bridge site, upstream from the proposed site, and downstream from the proposed site. In addition to this information for culverts and bridges, the survey data required for analyzing natural streams and tributaries should also be obtained within the proposed structure location.

Other hydraulic structures should be surveyed if they have the potential to affect or be affected by the flow characteristics of proposed culverts and bridges. For example, downstream reservoirs may cause backwater through culverts and bridges. Therefore, elevations, and locations of principal reservoir components (i.e., spillways) should be obtained for determining the controlling water surface elevations at the proposed project sites. Also, upstream irrigation systems may cause significant surge flows at the proposed project site during high runoff periods. Therefore, cross sections and location data should be obtained at the controlling irrigation structures.

Gaging stations, water supply and waste water treatment systems, revetments (i.e., riprap stream banks), and storm drain systems should at least be documented in the survey records. The hydraulics engineer should be consulted if additional information is required for hydraulic structures other than culverts and bridges.

B. Water Resource System Characteristics. After the required survey information is obtained, identify those characteristics of the water resource systems that may affect or be affected by the proposed highway project.

The following is a short list of water resource systems and their characteristics that should be investigated.

1. Human Resources. The designer may encounter several water systems associated with human resources. These systems exist for the enhancement and benefit of the human environment. Since these systems may have an impact on the highway design and vice versa, the following is a brief description of systems the designer may encounter during the development of a proposed highway project.

a. Reservoir Systems. If a highway with culverts and/or bridges is located upstream or downstream from a reservoir system, then the hydraulic and hydrologic characteristics of these systems should be investigated. Basically, reservoirs are designed for flood protection, municipal water supply, irrigation water, and recreation. They generally have significant impacts upon the flood peaks, sediment transport, and water surface profiles of natural streams and their tributaries. For example, reservoirs will normally reduce the downstream flood peaks, trap upstream sediment and increase the natural upstream water surface elevations.

If a bridge is located downstream from a reservoir, the designer should contact the U.S. Bureau of Reclamation and U.S. Geological Survey for annual peak flow rates during the period of operation. The water released downstream from the reservoir will be clean due to the reservoir's capacity to trap upstream sediment.

If the bridge is located in erodible material near the reservoir (i.e., alluvial stream), then the clean water will tend to pick-up or erode the existing stream bed material in the vicinity of the bridge.

If a bridge is located just upstream from a reservoir, the natural water surface elevation will be controlled by fluctuations in the reservoir water surface. The fluctuations in the reservoir surface will also influence the sedimentation characteristics near the upstream bridge. For example, if the bridge is located in the vicinity of where the stream meets the reservoir, then the stream will tend to deposit its material at the bridge site location (i.e., alluvial fan). This will tend to increase the water surface elevation in the bridge vicinity. However, if the bridge site water surface is controlled by the reservoir only part of the time, then the streambed may tend to erode from the higher velocities of the natural stream during low water reservoir periods.

If a highway is contiguous to the perimeter of a reservoir, then bank protection may be required between the reservoir and highway. In addition to the fluctuation in reservoir elevations, consider wave action when determining bank protection material and its placement. Also, to minimize sediment entering the reservoir, erosion protection measures should be considered for highway embankments and excavation areas.

Consult the U.S. Geological Survey and U.S. Bureau of Reclamation for information on the following:

- Annual peak flows.
- Upstream and downstream sedimentation characteristics.
- Hydrologic design data of reservoir components such as principal spillway and emergency spillways.
- Reservoir elevations such as normal pool, minimum freeboard, and seasonal elevations.
- Restrictions on turbidity and fish migration in relation to highway projects.

The designer should tie this information to the horizontal and control data of the project. Together, a composite model of the hydrologic and hydraulic characteristics of the reservoir and highway can be developed for analysis.

b. Canal Systems. Canal systems may include irrigation channels, drainage channels, diversion channels, and navigational channels. Since these systems tend to have straighter alignments and less erodible banks than natural stream channels, the hydraulic analysis may be more complicated for bridges and culverts crossing these systems than for natural channels.

For example, a straight channel with nonerodible banks will tend to experience greater erosion of its bed material than a natural meandering stream with erodible slopes. Therefore, bridge foundations may experience scour. However, canals with flat slopes that receive heavy sediment loads from natural streams on steep slopes may have material deposited within their channel sections. This may result in a corresponding increase in water surface elevation at a bridge site or "filling" of a culvert crossing. Also, the effects of highway designs upon sedimentation characteristics of canals should be considered by the designer (i.e., excavation slopes, road ditches).

Canals may also significantly affect the peak flows at highway crossings. For example, most hydrologic methods for predicting peak flows at ungaged sites will not consider the effects of man-made structures such as reservoirs and canals. Therefore, the designer should be aware of the location of the highway crossing relative to structures that regulate the flow through a canal system.

A field survey should locate and document all significant features of the canal system, such as diversion gates.

The U.S. Bureau of Reclamation and local water districts should be contacted for operation and design details of the canal system.

The U.S. Geological Survey should be contacted for hydrologic information related to the canal system, while the U.S. Coast Guard should be contacted for bridge clearances over the canal if it is a navigable waterway.

c. Storm Drainage Systems. New or reconstructed highway drainage may have to connect with an existing storm drainage system. The storm drainage system normally consists of curbs, gutters, inlets (i.e., grates or curb openings), catch basins, manholes, underground pipe systems, and outfall pipes draining into natural streams. Obviously, such drainage systems are normally found in urban areas.

The designer should obtain the available hydrologic and hydraulic information used to design the existing storm drain system from the responsible agency. After this information is obtained, then the proposed highway drainage can be designed to complement the existing storm drain system. For example, if the proposed highway drainage design is downstream from the existing storm drain system, then the proposed drainage design must be adapted for increased runoff or detentions that result from the storm drain system. On the other hand, if the new highway drainage system flows into the existing storm drain system, then the existing system may require modification to handle the additional flow.

Finally, the designer should consult with planning authorities for information on future developments within the project area (i.e., subdivisions). If a new storm drain system is proposed with the highway design, then it should be capable of adapting to new developments within the area.

d. Regulated Flood Ways. As previously discussed under Section 7.2.A.1., the Federal Emergency Management Agency has designated certain flood ways in urban areas for protection under the National Flood Insurance Program. These regulated flood ways are subject to local, State, and Federal restrictions concerning increases in the 100-year water surface elevations due to encroachments.

When a highway design encroaches upon a 100-year flood plain area of residential population, the Federal, State, or local officials should be contacted for Flood Insurance maps of the project area. If these maps show the highway system encroaching within the regulated flood way, then a flood study will be required.

The flood study begins with the methods for obtaining such information as survey data and computer water surface profiles utilized by FEMA for determining the existing 100-year flood plain limits.

If possible, the highway encroachment should be modeled with the existing FEMA information to evaluate the increase in 100-year water surface elevation due to the encroachment. The primary objective is to develop a *before* and *after* comparison so appropriate officials can determine if the highway encroachment is consistent with the National Flood Insurance Act. Since this analysis will normally require considerable survey data and sophisticated computer modeling, the hydraulics engineer should be consulted for proper flood plain evaluation of the project area.

The designer should be primarily concerned with whether or not a regulated flood way exists within the project area and to what extent any highway encroachments must be minimized.

e. Water Supply and Waste Water Treatment System. Highways may influence water supply and treatment systems. For example, highway developments may increase the sediment load to water supply and waste water treatment systems for both short-term and long-term periods. If the sediment load increases are significant, then the water supply and treatment processes may be adversely impacted. For example, increased sediment load may cause premature filling of settling basins. Normally, local governments will try to regulate the amount of sediment and the timing of sediment discharges through a permitting process.

The flows from water treatment plants will generally be small compared to the flow of the streams in which they discharge. If they have any potential impact upon the highway drainage system, however, the collection of flow by both water supply and treatment systems should be considered in the highway design process.

The appropriate local agencies are to be contacted for permits and hydrologic and hydraulic information related to water supply and waste water treatment systems.

2. Fish and Wildlife Resources. Most FLH projects are located in rural and remote areas such as National Parks and National Forests. The U.S. Fish and Wildlife Service has developed a classification system for wetlands and deep water habitats. This classification system provides methods for identifying types of fish and wildlife habitats. It also provides a method for identifying some of the hydrologic and hydraulic characteristics of natural water resource systems.

To aid the development of highway designs, the following classification system similar to the U.S. Fish and Wildlife system is recommended.

a. Riverine System. This system includes practically all moving bodies of fresh water contained within a channel. Given a cross section of a stream channel and its flood plains, the riverine area includes the permanently flooded channel areas and that portion of the flood plain area which is seasonally flooded.

A riverine system may be subclassified as being intermittent, upper perennial, lower perennial, and tidal. Normally, a moving body of water will pass through all four subclassifications.

At the upper part of a drainage basin, the flow in a natural channel will occur only part of the year. This is classified intermittent.

As the channel progresses down the drainage basin, the contributing drainage area increases until the channel can sustain year-round flows, (an upper perennial channel). Both intermittent and upper perennial channels are characterized by (1) steep streambed slopes (greater than 1 percent), (2) high velocities (greater than 2.5 meters per second during flood periods), and (3) very little flood plain development. Since upper perennial streams flow year-round and have high velocities, their stream beds will normally consist of gravel, cobbles, and/or boulders. These gravel-type beds provide excellent spawning areas for fisheries. Also, upper perennial streams provide high levels of dissolved oxygen (an important feature for maintaining good fish habitat) due to the extremely turbulent flow caused by the intermix of high stream velocities and boulders and cobbles. The larger boulders may also create pools that provide feeding and resting areas for fish. An optimum upper perennial stream for fish habitat will have both pools and riffles normally associated with high velocity flows over gravel beds.

A stream classified as lower perennial is located in the lower reaches of the drainage basin. It is primarily characterized by (1) mild stream bed slopes (less than 1 percent), (2) slow velocities (less than 2.5 meters per second during flood stage), and (3) a well developed flood plain. Because lower perennial streams have slower velocities, their ability to transport large bed material is reduced. Also, the upstream sediment particles will be weathered or broken down into smaller sediment particles as they progress downstream.
Therefore, lower perennial stream beds tend to consist of fine sand to silty-clayish materials. Also, the slower velocities and fine stream bed material provide little mixing of air and water. Thus, dissolved oxygen levels will tend to be lower than those found in upper perennial streams. Given the finer stream bed material and reduced dissolved-oxygen levels, fishery spawning habitat will not be as abundant as in upper perennial areas.

Thus, fish passage provided to upper perennial streams may be the primary fishery importance of lower perennial streams.

Tidal riverine subsystems are similar to lower perennial subsystems except that their water surface elevations are influenced by ocean tides. However, the tidal riverine subsystem is still a "fresh" water body with only small amounts of ocean derived salts.

b. Palustrine System. This system consists of flood plain areas with heavy year-round vegetation and isolated bodies of water with less than 8 hectares in surface area. These isolated bodies of water are commonly recognized as marshes, swamps, and ponds.

Although these areas may not present significant hydraulic problems, they may pose serious environmental problems for new or reconstructed highway systems. Since these areas often support diverse species of plant, wildlife, and fisheries, fish and wildlife agencies should be contacted in the early planning and design stages. Basically, the two main concerns for fish and wildlife officials will be the total surface area eliminated by the highway design and the location of the eliminated surface area.

c. Lacustrine System. This system consists of natural lakes and man-made reservoirs with more than 8 hectares in surface area. The section under reservoir systems should be reviewed for hydrologic and hydraulic constraints. Environmentally, the same concerns expressed for palustrine systems will also apply to lacustrine systems.

The above classification systems provide a good starting point for the hydrologic and hydraulic evaluation of highway systems. For example, intermittent and upper perennial streams will have super critical to critical flows, steep hydrographs, very minor and localized backwater from culverts and bridges, and will experience less localized scour at hydraulic structures than those installed in lower perennial streams. On the other hand, lower perennial and tidal streams will tend to have subcritical flows, flatter hydrographs, major and more extensive backwater effects if highways and their hydraulic structures constrict flood plain areas, and will experience considerable localized scour at hydraulic structures. Classification of the particular system by physical observation will aid the designer and/or hydraulic engineer in selecting the appropriate analytical processes for evaluating water-related highway designs.

The classification system will provide information on areas of potential concern to fish and wildlife agencies. For example, upper perennial streams with high velocities and steep gradients present difficulties in the design of culverts for fish passage. However, the good sediment-transporting ability and high dissolved-oxygen levels will minimize the short-term and long-term negative impacts of highway construction within the area. However, lower perennial streams will normally be crossed by highway bridges in lieu of culverts. Thus, fish passage is not a problem. But short-term construction work in streams and long-term runoff from highway pavements and slopes may cause negative impacts that will be more difficult for lower perennial streams to adjust to if these streams have lower velocities and lower dissolved-oxygen levels. Therefore, fish and game officials may request different types of mitigation for different types of situations.

C. Geomorphologic Characteristics. Geomorphology is the geological study of the configuration and evolution of land forms. Along with precipitation and human developments, the hydrologic (drainage basin) and hydraulic (stream channel and flood plains) attributes of the project area are primarily determined by its geomorphology. Geomorphologic items include the following:

1. Drainage Basins. The drainage basin should be evaluated for size, shape, slope, storage, vegetation, and surface infiltration. These characteristics primarily affect the hydrologic runoff to a given hydraulic structure. However, these characteristics may also aid in the hydraulic analysis of such items as sediment runoff versus water surface elevation.

The size of the drainage basin is the most important property affecting runoff. Basically, the runoff will increase as the drainage basin size increases. As mentioned in earlier discussion, a USGS quadrangle map and/or aerial photographs may be used to evaluate the basin size.

The basin shape should be the next characteristic to be investigated. Long narrow drainage basins generally have lower runoff peaks than fan or pear-shaped drainage basins. The shape may be outlined by the main basin channel and its tributaries. The outlined shape allows analysis of preliminary hydraulic flow relationships between the main basin channel and its tributaries, which may also be analyzed for such items as size of tributary versus main channel size and the angle of approach by the tributary to the main channel.

The slope of the drainage basin may be defined as the change in its vertical distance to the change in its horizontal distance. The slope is normally measured at 10 percent and 85 percent of the distance along the main channel from the hydraulic structure to the drainage basin divide. Generally, the runoff will increase as the basin slope increases. Also, the main channel steepness will indicate the existing flow conditions (i.e., rapid or mild) and the sediment transport capabilities within the basin. Multiplying the basin slope by the design runoff or discharge, will provide a relative index for comparing the sediment transport power to other drainage areas where the slope, discharge, and sediment transport characteristics are known.

The storage of the drainage basin is essentially the amount of natural and manmade depressions capable of detaining or delaying runoff to the hydraulic structure location. For determining runoff, most hydrologic methods evaluate only the natural storage such as lakes, ponds, and swamps. The natural storage is normally evaluated as a percent of the total drainage basin. The presence of natural storage of up to 25 percent of the total drainage basin area may reduce the peak runoff by more than 50 percent as compared to a drainage basin with no storage. If manmade storage structures exist within a drainage area, then gaging station data should be used for determining peak runoffs.

The vegetative characteristics of the drainage basin may affect the anticipated runoff. The runoff will decrease as the vegetative percent of drainage basin area increases. When hydrologic equations consider this factor, it is normally evaluated as a percent of forested drainage area.

For example, a drainage basin with 80 percent forest cover may experience a 25 percent or more decrease in runoff as compared to a drainage basin with 25 percent forest cover. Also, the peak runoff at a hydraulic structure will tend to be more stable and longer lasting (i.e., a flat and long hydrograph curve) for a forested area as compared to a nonforested area.

Surface infiltration will affect flow characteristics such as surface flow versus ground water flow within the drainage basin. The surface infiltration within a drainage basin is a function of soil porosity (i.e., gravel versus clay), rainfall intensity, vegetal cover, and the antecedent condition of the basin soils (i.e., percent of saturation). As an example, a paved parking area will pass nearly 90 percent of its rainfall to the area downhill from it, while a porous lawn will absorb nearly 90 percent of the rainfall within its area.

Determine the drainage basin size, shape, slope, storage, and vegetative characteristics from USGS quadrangle maps and/or aerial surveys. Soil infiltration characteristics are normally field investigated. Fortunately, most hydrologic methods will indirectly compensate for soil infiltration and a field investigation may not be necessary. A field investigation of all the drainage basin characteristics is recommended for comparison of the office analysis and with the geomorphologic characteristics of the hydrologic equations.

2. Stream Channels and Flood Plains. Although drainage basins will experience some changes in their form and characteristics during the life of a project, these changes are usually not as pronounced as those that a stream channel with flood plains will experience. For example, a bridge crossing a meandering river in a wide flood plain may be abandoned if a major flood cuts a new channel within the flood plain limits. Also, riprap bank protection may reduce erosion of the riprap area, but increase downstream bank erosion. Further-more, stream channel bottoms may tend to erode downward, thus lowering the water surface elevation for a given flow and possibly causing the stream to abandon its flood plain.

The cross section of a stream channel and its flood plain is a function of the flow, the quantity and character of the sediment in movement through the section, and the character or composition of the materials making up the bed and banks of the channel. These characteristics tend to be dynamic and inter-reactive. Predicting the future shape of a channel or flood plain is beyond the scope of this publication. The following discussion centers upon the classification of stream channels and flood plain development.

Stream channels may be defined as nonalluvial and alluvial. The term alluvial generally applies to deposits of silts, sands, or gravel. Nonalluvial and alluvial may be considered synonymous with nonerodible and erodible, respectively. Since many of the basic hydraulic methods for determining stream depths and velocity are based upon rigid or nonchanging boundaries, the distinction of nonalluvial and alluvial channels is important.

Nonalluvial channels are normally found in rock valleys or in situations where manmade activities have converted the stream channel to essentially a rigid boundary. Also, stream channels near the headwaters area may experience only channel bed erosion without appreciable bank erosion, thus being a hybrid of a nonalluvial and alluvial stream. The methods for determining the hydraulic characteristics of these types of streams will normally be uncomplicated provided the analysis is confined to the rigid boundary portion of the stream and its flood plain.

Alluvial streams normally have a wavy or meandering pattern with well developed flood plains. These streams tend to erode into the outside part of their bends while depositing the eroded material just downstream on the inside of the same bend. Thus, the stream channel flow will tend to be deeper on the outside portion of the bend.

If the outside bank material is erodible, the stream will continue to erode toward the outside. Eventually, the stream pattern will take the form of a well-bent bow. The beginning and ending portions of the stream bend will come closer and closer together. If a major flood occurs and the bank material between the beginning and ending bend portions is erodible, then the flow may break through the plug between these portions and cut-off the bend. This is the typical formation of oxbow lakes. Although the main stream channel will be temporarily straight within this area, a new meander pattern will tend to form due to the stream's tendency to have helical flow.

If the material within the flood plain valley is homogeneous, the river may meander back and forth within the same flood plain area over many decades. Generally, the bank materials will consist of coarse material at the bottom and fine material at the top. This is due to the fact that coarser material is transported in the stream near the bottom and finer material near the top of the flow. If the stream overflows its bank, it may deposit enough fine material on the top of the bank to form natural levees. This will normally occur for a flood frequency comparable to 2 to 5 years.

Another important characteristic of alluvial stream channels is their tendency to have degrading (eroding) or aggrading (depositing) beds. A stream will degrade if its capability to transport material exceeds the supply of material available for transport. However, a stream will aggrade if the material available for transport exceeds the stream's capacity for transporting material. In a given location, a stream may experience both aggradation and degradation over a short period of time.

Although the availability of transportable material is difficult to estimate, the transport capability of a stream can be correlated to its water surface slope and its runoff values (peak discharge). This relationship is most useful in evaluating changes in transport power along the stream channel.

For example, a steep water surface slope is capable of transporting more material than a mild water surface slope. Thus, when the stream slope changes from steep to mild, the stream will tend to deposit material in the mild slope location that was being transported in the steep slope location. As another example, a tributary will generally have a steeper slope than the main channel which it flows into. At the point the tributary meets the main channel, the tributary will deposit its material to form an alluvial fan, assuming that the tributary experiences a backwater effect from the main channel.

An easier method for evaluating changes in sediment transport power is to consider the differences in stream velocities between two closely located stream sections.

For example, a bridge stream section will generally have less area for discharge than the stream section just upstream from the bridge. Since discharge (Q) is a function of stream velocity (V) times the flow area (A) and since the discharge must also remain constant throughout the given stream reach, then the velocities will be greater within the bridge section than those velocities just upstream of the bridge. Thus, the bridge section will have greater stream power for transporting material than the section just upstream of the bridge.

If a bridge section erodes, it is called *general* or *contractual scour*, because its flow area is smaller than the stream's natural cross section. If the bridge piers and/or abutments experience considerable adjacent erosion, it is called *local scour* because the erosion is localized to the vicinity of the obstruction. The term *degradation* applies to a stream reach that erodes over long distances while the term scour applies to more localized erosion.

Several methods should be used in evaluating the alluvial and nonalluvial aspects of streams and their flood plains. These methods include the following:

- Studying historical data (such as quad maps, aerial photos, and old bridge plans).
- Making physical observations such as viewing channel slopes of tributaries at their confluence with main streams (for possible degrading of the main channel) or by identifying a braided stream (for possible aggrading).
- Making numerical evaluations such as solving sediment transport equations (a function of slope, discharge, and sediment size).

Collectively, these methods will provide a composite perspective of the changes that may occur within the project area.

The designer should recognize that stream channels and flood plains are subject to constant change and these changes may profoundly alter the analytic processes. How reliable is the computed depth of flow if the streambed is constantly fluctuating as it degrades and aggrades? The designer should be prepared to evaluate the risks versus costs of designing for the dynamic processes of streams and their flood plains.

D. Water Quality Characteristics. Water quality characteristics include information on the value of the water resource system to humans, fish, and wildlife.

Characteristics such as pH, turbidity, conductivity, dissolved oxygen, water temperature, flow rate, and biological oxygen demand provide information for evaluating the potability of, the suitability of, fish habitat, and the compatibility of hydraulic structures with the water resource system.

Generally, water quality characteristics are only evaluated at the request of other agencies such as the U.S. Environmental Protection Agency, U.S. Fish and Wildlife, and State water resource agencies. However, water quality data should be obtained whenever possible.

The following is a summary of definitions, standards, and plans for evaluating water quality characteristics.

1. Government Water Quality Standards. The State generally is the most responsible for establishing water quality standards within the project area. There are no nationwide water quality standards. Federal laws have set goals for eliminating all point and nonpoint pollution sources. Also, local governments and environmental groups may have considerable regulations covering municipal water supplies and recreational uses of reservoirs. The type of regulations and standards will depend primarily on the type of highway project and its location.

Use the following defined parameters and values in evaluating water quality for highway systems.

a. pH. pH is a measure of the hydrogen ion activity in a water sample. It is an indirect indicator of acidity and alkalinity and can range from 0 to 14 with a value of 7.0 being neutral. If a pH is less than 7, it is considered acidic. If a pH is greater than 7 it is considered alkaline. Most fresh water aquatic life and culvert pipe materials can tolerate a pH range of 6.5 to 9.0.

b. Conductivity. Conductivity is the ability of water to carry electrical current. It is dependent upon the concentration of dissolved ionized substances thus giving an indication of the concentration of dissolved solids in the water.

A body of water that has an electrical conductance greater than 350 mhos/cm will probably be unsafe for drinking. Most freshwater aquatic life should tolerate an electrical conductance as high as 1,000 mmhos/cm.

c. Turbidity. Turbidity is an indication of the amount of suspended solids in the water. Excessive turbidity may reduce the rate of photosynthesis, cause changes in predator relationships, and cause changes in temperature. It is aesthetically displeasing. Generally an acceptable turbidity range is from 0 to 50 NTU (nephelometric turbidity units). Contract specifications may need to give maximum limits by which construction activities can increase water turbidity and temperature.

d. Dissolved Oxygen. Dissolved oxygen is the oxygen dissolved in water. It is a good indicator of the aesthetic and aquatic quality of the water. Most bodies of water having good aquatic environments are characterized by nearly saturated dissolved oxygen levels. However, a euthrophic body of water may exhibit high dissolved oxygen levels during the daytime while reaching nearly zero levels during the night. This phenomenon is due to plant photosynthesis (oxygen producing) taking place during daylight hours only, while plant respiration (oxygen consuming) continues into the night. Most unpolluted streams will show a dissolved oxygen fluctuation of only 1 to 3 mg/L per 24 hours.

While the dissolved oxygen saturation level of water may vary from 6 to 14 mg/L, de-pending upon local temperature and altitude, a range of 5 to 10 mg/L is considered satisfactory for supporting most freshwater aquatic life and producing aesthetically pleasing qualities.

2. Water Quality Data. If water quality standards exist for the project area, then water quality data has probably been collected within the region. This data can be used to develop a base for future water quality monitoring plans.

The National Water Data Exchange (NAWDEX) is a national confederation of water-oriented organizations working together to improve access to water data. It consists of member organizations from all sectors of the water-data community.

NAWDEX provides direct, on-line access to its data bases by its members and other organizations. The program is also authorized to provide limited access to the data bases of the U.S. Geological Survey's National Water Data Storage and Retrieval System (WATSTORE) and the U.S. Environmental Protection Agency's Storage and Retrieval (STORET) System.

The NAWDEX and WATSTORE data bases are maintained on the U.S. Geological Survey's computer system located at its National Center in Reston, VA. The STORET data base is maintained on the U.S. Environmental Protection Agency's computer located in the Research Triangle Park in Raleigh-Durham, NC. Users are provided with complete instruction and user manuals for accessing these data bases.

a. NAWDEX. Two data bases are maintained. The Water Data Sources Directory contains information about organizations that are sources of water or water-related data and services, types of data available, and locations from which data or services may be obtained.

The Master Water Data Index contains information about individual sites at which surface-water or ground water data are collected, locations of the sites, types of data collected, periods of record, frequency of measurements, and identification of the source organizations.

b. WATSTORE. Several files and data bases are maintained that store most hydrologic data collected by the Geological Survey.

The Station Header File contains identification, location, and physical descriptions of sites for which data are stored.

The Daily Values File contains river stages, stream flow values, water temperatures, conductance values, sediment concentrations and discharges, and other parameters that are measured on a daily schedule.

The Peak Flow File contains peak stream flow and stage values for surface-water sites.

The Unit Values File contains stream discharge values, temperatures, and other parameters measured more than once each day.

The Water-Quality File contains the results of chemical, physical, biological, and radiochemical analyses for both surface and ground waters. Access to this file is pending. This data is available through STORET.

The Ground Water Site Inventory File contains inventory information such as site location and identification data, well construction data, geohydrologic characteristics, and other data pertinent to wells, springs, and other sources of ground water.

The Water-Use File contains limited summary data on water use throughout the Nation.

c. STORET. This data base contains water quality data for both surface and ground waters. Data are stored for several Federal agencies and over 40 State agencies. These data include chemical, physical, biological, and radiochemical analyses of water samples and many other types of data.

Contact the hydraulics engineer for more information on how to obtain data from the NAWDEX data bases.

3. Monitoring Plans. The FHWA publications *Highway Water Quality Monitoring Manual and Highway Runoff Water Quality Training Course Workbook* are reference sources suitable for use in developing water quality monitoring plans.

If warranted, water quality monitoring should be conducted 12 months before construction, during construction, and 12 months after construction. Water quality testing should be performed upstream and downstream of the project area. Tests should be performed at least once per month during low flows and twice per month during high flows.

During heavy construction periods, some tests such as turbidity should be performed on a daily basis.

Monitoring should include pH, turbidity, conductivity, and dissolved oxygen. In addition to these tests, the time, date, water temperature, air temperature, and flow rate should be obtained by the monitor.

The collected data should be permanently filed with the USEPA STORET system for future reference by FHWA and other interested agencies.

7.4 DESIGN PROCESSES

Hydrologic analyses and hydraulic designs are necessary whenever water enters or crosses the highway right-of-way. It is essential to estimate the flow and provide a means to safely convey that water. The drainage structure required may be a small roadside ditch or a large bridge.

Once a flood frequency has been selected and flow rates established, the hydraulic analysis or design can be completed. This consists of establishing criteria, developing and evaluating alternatives, and selecting the alternative which best satisfies the established design criteria. Possible damages to the highway, the channel, and surrounding properties must be considered in the analysis.

Provide drainage structures with adequate carrying capacities to minimize damages. In analyzing a structure, examine the channel both upstream and downstream. Culverts should be located and oriented with the stream alignment. A drainage structure that is too small may result in backwater problems with flooding of upstream property or damage to the highway. If the channel is erodible, evaluate scour in the vicinity of the structure to determine if a lining is necessary.

This section provides guidelines and design criteria for culverts, open channels and stream crossings.

A. Criterion for Analyses and Designs. The design of a drainage system begins with the selection of the appropriate standards and the choosing of a design flood frequency. Once the design flood frequency is chosen, freeboard and other special configurations or restrictions can be established. The following standards are provided to assist in the selection process for determining the appropriate design criterion.

1. Floods. Adequate drainage designs provide facilities with capabilities of withstanding certain flood intensities without sustaining major damage. The minimum design frequencies in Table 7-2 are recommended for rural roads. See Section 7.4.B for the procedures to determine a project's design flood frequency.

A minimum design flood frequency of at least 50 years shall be used for all bridges. See Chapter 10, Section 10.3.B.

A minimum design flood frequency of 10 years should be used for all urban drainage systems (including ditches). Urban drainage systems should not cause flooding of more than one-half of a single travel lane width for the selected frequency. It may be impractical to design for less frequent flooding of these drainage systems (because of right of way considerations, costs, etc.).

Design temporary detour structures using a flood frequency of at least 10 years. However, detour structures to be used for 6 months or less may be designed on a lesser flood frequency if site specific circumstances permit. Major detour structures may require design flood frequencies substantially greater than 10 years.

Table 7-2
Flood Frequencies for Rural Roads

Projected ADT	Minimum Design Flood Frequencies (years)				
	Curbs and Ditches	Road Culverts and Embankment Protection	Small Streams Creeks ($Q_{10} < 1.5$ m^3/s)	Large Streams Rivers ($Q_{10} > 1.5$ m^3/s)	Bridges[1]
≤400	5	25	10	50	50
>400	10	50	25	50	50

Note: [1]See FAPG 650A. Check for 100 year flood.

2. Freeboard. Freeboard is the vertical distance between a water surface at design flow and the low point of a bridge beam or the top of a road surface. Overtopping of bridges should not be permitted unless the bridge is specifically designed to withstand the overtopping. Based on the design flood frequencies established from the guidelines set forth herein, the following freeboard criteria shown in Table 7-3 should be used.

3. Other Design Components. While freeboard may be one of the primary considerations in developing the design of a drainage structure, there are many other integral parts to be considered and evaluated in the total design of a facility.

a. Flow Rates. Culverts are generally capable of handling flow rates of less than 30 m^3/s. Bridges should be considered for stream crossings with a design flood exceeding 30 m^3/s.

Table 7-3
Freeboard Criteria[1]

Drainage Feature	Freeboard[2] (meters)
Culvert[3], Ditches, Slope protection[4], etc.	None[5]
Bridges (including long span culverts)	0.4[5,6,7]
Low water crossings	None

Notes:
[1]Floodplain ordinances or other legislative mandates may limit allowable backwater or encroachment on the floodplain. Social considerations including the importance of the facility as an emergency evacuation route or as a National defense access road should be considered.

[2]Ecological considerations and geological or geomorphic conditions may affect freeboard selections.

[3]Headwater at culvert inlets should not exceed 2.4 meters. When this occurs, the designer should consult the Hydraulics Engineer.

[4]Channel stability considerations may limit velocity or the amount of constriction.

[5]If the design flood has an inaccuracy of more than 10%, allow at least 300 millimeters of freeboard between the top of subgrade and the top of the design flood. Increase the freeboard for bridges to 600 millimeters. Freeboard for bridges is measured from the lowest point of the lowest member of the superstructure to the top of the design flood.

[6]When wooded debris is a factor to be considered, bridges should have a freeboard of 1 to 1.5 meters. When ice flows become a factor, freeboard should be from 1.5 to 3 meters.

[7]Special clearances may be required for bridges over a navigable body of water.

b. Headwater. The headwater to diameter ratio for culverts 1200 millimeters in diameter and smaller should not exceed 2.0. For larger culverts, the ratio should not be greater than 1.5.

c. Size. See Section 7.4.D for size selection criteria.

d. Gradients. Culverts should not be installed on grades steeper than 10 percent without being anchored. The minimum gradient for all minor channels and ditches shall not be less than 0.5 percent.

e. End Treatments. Concrete headwalls should be installed on the inlets of culverts 2400 millimeters in diameter and larger. On culverts 1200 millimeters in diameter or larger but less than 2400 millimeters, special end treatments such as special end sections and sloped or beveled ends should be considered. End sections should be used on culverts smaller than 1200 millimeters in diameter when the invert is within 1.5 meters or less of the shoulder elevation. This criterion may be waived when fill slopes are 1:2 and steeper or where the end of the culvert is in back of guardrail. See Section 7.4.D.11.

f. Fish Passage. Culvert barrel velocities should not exceed 1.2 meters per second for a 2-year flood.

g. Scour. When velocities at a pipe outlet exceed 2 m/s, erosion protection measures shall be considered. See Section 7.4.C.3.

h. Culvert Materials. Culvert materials and their approved applications are set forth in Section 7.4.D.

B. Hydrology. A hydrologic analysis must be performed before the hydraulic design of a drainage structure can be initiated. This process, utilizing flood discharge and frequency data, enables the designer to make a rational decision in selecting the proper drainage facilities for a functional highway.

Begin by determining the peak discharge for the site or develop a design hydrograph. The peak discharge is the maximum flow for a given storm event. A hydrograph is a graph of water flow versus time from which the maximum or peak flow of a flood and its time distribution can be determined.

Annual peak flows should be used to develop flood discharge and frequency relationships for use in economic analysis and design of drainage structures. The flood discharge and frequency relationship is best shown with a logarithmic plot of a flood magnitude versus return period.

Hydrographs may be used to estimate the length of time a road will be flooded or to evaluate the temporary upstream storage of flood waters when determining the water surface elevation for a culvert.

1. Flood Design Frequency. One of the first steps in making a hydrologic analysis is the selection of a design flood frequency. See Section 7.4.A.1 and Table 7-2. When determining flood frequency, give consideration to all significant impacts and risks involved. This determination is the responsibility of the designer. The selection of the design flood frequency shall be documented.

Design factors to be considered and the degree of documentation required depends on the individual structure and site characteristics. The hydraulic design must be such that risks to traffic, potential property damage and failure from floods are consistent with good engineering practice and economics. Recognizing that floods can not be precisely predicted and that it is seldom feasible to design for the very rare flood, all designs should be reviewed for the extent of probable damage, should the design flood be exceeded.

Design headwater/backwater and flood frequency criteria should be evaluated relative to the following:

- Damage to adjacent property.
- Damage to the structure and roadway.
- Traffic interruptions.
- Hazard to human life.
- Damage to stream and flood plain environment.

The potential damage to adjacent property or inconvenience to owners should be of major concern in the design of all hydraulic structures.

2. Estimating Peak Flow. For any given site, there are several methods available for estimating peak flows and their return periods. No single method is applicable to all watersheds. Engineering judgement 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 (size and type), availability of data, the validity of the method for the site, land use, and the degree of accuracy desired. Several methods should be used and the results compared. Hopefully, the results obtained from one method will supplement the results of another method.

There are many methods available for estimating peak flows at sites without gages. These methods include the rational formula, U.S. Geological Survey (USGS) regression equations, Federal Highway Administration (FHWA) regression equations, Soil Conservation Service (SCS) methods, USGS Index Flood method, and other local methods. Most of these methods and techniques are discussed in detail in Hydraulic Engineering Circular No. 19.

a. Rational Formula. The rational formula is one of the most commonly used equations for estimating peak flows from urban, rural, or combined areas for *watersheds smaller than 100 hectares*. This formula presents a relationship between rainfall intensity and peak runoff.

$$Q = CiA/360$$

where:

- Q = Peak flow in cubic meters per second.
- C = A dimensionless runoff coefficient assumed to be a function of the cover of the watershed.
- I = Rainfall intensity in millimeters per hour, for the selected frequency and for a duration equal to the time of concentration.
- A = The drainage area in hectares.

Some values of the runoff coefficient, C, are reported by a joint committee of the American Society of Civil Engineers and the Water Pollution Control Federation. These values are considered to be applicable for storms of 5- to 10-year frequencies. Storms with greater recurrence intervals require the use of higher coefficients because infiltration and other abstractions have a proportionally smaller effect on peak runoff.

The rainfall intensity (i) can be obtained from an Intensity-Duration-Frequency (I-D-F) curve for the local area. In the eastern United States this curve can be developed from information in NOAA Technical Memorandum NWS HYDRO-35, *5 to 60- Minute Precipitation Frequency for Eastern and Central United States*. HYDRO-35 contains precipitation and frequency information for durations of 60 minutes and less. For durations of greater than 60 minutes, rainfall intensity-frequency data are obtained from

Weather Bureau Atlas Technical Paper 40, *Rainfall Frequency Atlas of the United States*. In the western 11 States, isopluvials for 2-year and 100-year frequencies and 6- hour and 24-hour durations are provided in the 11 volumes of NOAA Atlas 2.

The time of concentration (t_c) for overland flow can be determined by various methods. Nomographs are presented in HDS No. 4 and various Natural Resources Conservation Service publications. Mathematical methods are also available as presented in HEC No. 19 and HEC No. 12. The design engineer needs to be aware of the assumptions implicit in these methods.

When using the rational formula, make the following assumptions:

- The peak flow occurs when the entire drainage area is contributing flow.
- The rainfall intensity is uniform over a duration of time equal to or greater than the time of concentration.
- The time of concentration is the time required for the runoff to flow from the most remote point in the watershed to the point of interest.
- The frequency of the peak flow is equal to the frequency of the rainfall intensity.

b. Natural Resources Conservation Service (NRCS) Methods.
The SCS has published a method of calculating peak flows for *watersheds with areas smaller than 800 hectares*. The method uses charts and tables for a generalized 24 hour storm. The graphs relate discharge to precipitation and drainage area for various soil curve numbers and for flat, moderate, and steep slopes. This method was developed for rolling agricultural and rolling undeveloped land.

A detailed explanation of this method can be found in Section 4 of the NRCS *National Engineering Handbook* and in HEC No. 19.

The NRCS has also published a tabular method of estimating peak discharges and a crude hydrograph for use on *watersheds up to 50* square kilometers in area. This method is used to estimate the effect of:

- changes and development in the watershed, and
- detention storage for small watersheds.

Experience is necessary in determining the various parameters required in the NRCS methods.

c. Regional Regression Equations. Regression equations are one of the most commonly accepted methods for estimating peak flows at sites without gages or sites with insufficient data. Multiple regression analysis techniques are used to determine the relation of basin characteristics and climatic conditions to flood peaks of selected recurrence intervals. Information from gaged basins with similar drainage patterns and climatological characteristics are used to develop regression equations. Studies throughout the country have established peak flow equations based on multiple linear regression techniques. The equations are of the form

$$Q_T = a\ A^{b}\ B^{c} \ldots M^{n}$$

Where:

Q_T = flood magnitude having a recurrence interval T.

a = Regression constant.

A, B, C,...M = basin and climatic parameters.

b, c, d,...n = regression exponents.

The primary basin characteristic is the drainage area. Almost all regression equations include drainage areas above the point of interest as an independent variable. Other parameters typically required are precipitation, channel slope, basin elevation, etc.

The advantages of using a multiple regression analysis are as follows:

- Provides a mathematical relation between a dependent variable (Q) and independent variables (A, B, C, etc.).
- Provides an evaluation of the independent variables that best define the dependent variable.
- Provides a measure of the accuracy of the equation (standard error of estimate) and tests the significance of the coefficients of each independent variable.
- Evaluates relative significance of each independent variable by indicating those variables that have a coefficient that is significantly different from zero at a particular percent confidence level.

The U.S. Geological Survey, in cooperation with the States, has developed regression equations for most of the United States. The equations are contained in individual State reports. These regression equations permit peak flows to be estimated for recurrence intervals ranging from 2 to 100 years for natural streams. Regression equations are developed using independent variables (basin characteristics) within given ranges for each State and hydrologic region. The equations should be applied within the range of independent variables utilized in the development of the equations since the relationships of the equations outside these ranges is not known.

Whenever possible, peak flows obtained from regression equations should be compared to flow frequencies at nearby gaging stations to evaluate validity. Regression equations may give better estimates of peak flows of various frequencies than formal statistical analyses, especially since the regression equations more nearly predict the potential of the watershed to experience a peak flow of a given magnitude whereas a frequency analysis is biased by what has been recorded at the site.

The FHWA method for estimating peak flows from small rural watersheds relates the peak flow to easily determined hydrophysiographic parameters. A multiple regression approach was used to develop equations for each of 24 hydrophysiographic zones identified for the United States and Puerto Rico. The rainfall isoerodent factor, watershed area, and difference in elevation are the independent variables required for predicting the 10-year discharge with the FHWA 3-parameter method. Five- and seven-parameter methods, as well as an all-zone technique, are also available. Adjustment to the 10-year equations must be made for basins with surface storage greater than 4 percent. Discharges for other return periods are determined as a function of the storage-adjusted 10-year event.

The FHWA method is intended for use on watersheds smaller than 130 square kilometers but may be used on areas up to 250 square kilometers. The FHWA method is outlined in the report, *Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method*. This method was developed from a smaller data base than USGS regression equations, but does provide the designer with an alternate method for estimating peak flows. This method should not be used if the area of interest is covered by USGS regression equations.

A multiple regression analysis method (channel geometry method) exists that uses channel characteristics in the estimating procedure. This method is primarily used in some of the western States with bankfull width as the primary factor. Considerable difficulty can be experienced when determining channel characteristics for use with this method.

Generally, the more comprehensive the area over which the regression equations are developed, the less reliable the results. Each of the regression equations has been developed for a specific range of variables.

The designer needs to fully understand the limitations and applications for each of the regression analysis techniques to most effectively estimate a peak flow.

d. Flood Frequency Analysis Using Recorded Data. When a sufficient period of record is available, a more desirable method for determining the peak flow may be 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 one-half the frequency of the design flow. Unfortunately, flow records are generally insufficient or do not exist for the site of interest. This is usually true for locations where most Direct Federal drainage structures are required. This method assumes that there have been no significant changes in the characteristics of the drainage area or climatological patterns that will alter the ability of the basin to produce runoff.

Several of the more popular techniques include Log-Pearson Type III, Normal and Log-Normal, and Gumbel Extreme Value Distributions. Refer to HEC No. 19 for analysis methods of gaged data.

Regional equations may improve peak flow estimate at gaged sites by weighting the statistical analysis estimate with the regression estimate.

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

e. Verification of Flow by Historical Observations. Data can sometimes be obtained that can be used to estimate the discharge of historical floods through stage-discharge relationships or open-channel flow techniques. Useful information might include:

- highwater marks, and

- eyewitness reports of overtopping depths of highways and bridges.

Flows determined by historical observations should be used only as a check on other methods. Because of the small number of observations or inherent inaccuracies, flood-frequency curves should not be developed solely from this method.

3. Hydrographs. To estimate time of peak flows for a drainage basin, time of water overtopping a road, or the effect of upstream storage on peak discharge, a plot of flow versus time (hydrograph) is necessary. The method used to develop the hydrograph depends on the data available for the drainage area, the validity of the method for the site, and the experience of the designer with the method.

One of the following methods, whichever is most appropriate, should be used for the site under consideration.

a. Unit hydrographs. A unit hydrograph is defined as the direct runoff resulting from a rainfall event, which has a specific distribution in time and space and lasts for a unit duration of time. Unit hydrographics are most accurate when based on continuous readings from stream and rainfall gages. The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area. Using a unit hydrograph, a hydrograph for other rainfall events can be developed based on differing amounts of precipitation.

b. Synthetic unit hydrographs. When gage data is not available for stream crossings, one of the following methods for synthetic unit hydrographs shall be used.

(1) Snyder and NRCS synthetic unit hydrographs. These are two of the most common synthetic unit hydrograph methods. Use the HEC-1 computer program to develop unit hydrographs, using either of these methods.

(2) USGS synthetic unit hydrographs. The USGS is developing reports for each State that will give hydrographs for a range of return periods. This method uses USGS procedures for estimating peak flows to find the hydrograph peak. The shape of the hydrograph depends on basin characteristics.

Stream gage data is the basis for most flow prediction methods. Gages can give continuous flow measurements or they may give only the peak flow that occurred between readings. Continuous reading gages give the entire hydrograph of a flood.

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. Gaging station records of similar streams in the region should be used as a guide in making this adjustment.

C. Open Channels. Open channels may be natural or manmade channels in which water flows with a free surface, and are the primary facilities for conveying surface runoff.

Channels in roadway design include the following types:

- Roadside ditches in cut sections.
- Gutters in curbed sections.
- Toe-of-slope ditches.
- Interceptor ditches placed back of the top of cut slopes.
- Inlet and outlet channels for culverts.
- Channel changes of existing streams.
- Rivers and streams parallel to the roadway.
- Stream channels under bridges.

The analysis and understanding of open channel hydraulics is used to evaluate or design channels for roadway hydraulic structures.

In the analysis which establishes the best hydraulic design criteria, factors such as capital investment and probable future costs including maintenance and flood damage to properties, traffic service requirements, and the stream must be evaluated. The hydraulic design of open channels consists of establishing the criteria, developing and evaluating alternatives, and selecting the alternative which best satisfies the established criteria. The detail in which risks are considered should be commensurate with the flood hazard at the site, economics, and current engineering practices.

The adequacy of channels under bridges, and those channels immediately upstream and downstream from bridges, as well as their stabilization, are the responsibility and concern of the hydraulics engineer and the bridge design staff. However, the designer (concerned with the location and alignment of bridges) is responsible for providing a vicinity map to the bridge design staff for each bridge site.

1. Hydraulics of Open Channels. This section contains a general discussion of the fundamentals of open channel hydraulics and the application of the methods and procedures for analysis and design of channels.

The following are the major objectives of open channel hydraulic analysis:

- Documentation of existing conditions.

- Analysis and documentation of the effects that alternate design will have on existing conditions.

- The design of a proposed facility.

The water surface profiles, velocity and flow distribution are of primary concern in achieving these objectives.

a. Types of Flow. Open channel flow is usually classified as uniform or nonuniform; steady or unsteady; and subcritical, critical, or supercritical. Of these, nonuniform, unsteady, subcritical flow is the most common type of flow in open channels. Due to the complexity and difficulty involved in the analysis of nonuniform unsteady flow, most hydraulic computations are made with certain simplified assumptions that allow the application of steady, uniform, or gradually varied flow principals and methods of analysis.

Steady flow methods assume that the discharge at a point does not change with time. Uniform flow methods assume that there is no change in velocity, magnitude, or direction with distance along a stream line. Therefore, steady uniform flow assures constant velocity and flow rate from section to section along the channel.

Steady uniform flow is an idealized concept of open channel flow that seldom occurs in natural channels and is difficult to obtain even in model channels. *For most practical highway applications the flow can be assumed to be steady and changes in width, depth, or direction are sufficiently small that flow can be considered uniform.* The changes in channel characteristics occur over a long distance such that flow gradually varies. For these reasons use of the uniform flow theory is usually within acceptable degrees of accuracy.

By plotting specific energy head against depth of flow for a constant discharge, a specific energy diagram is obtained. When specific energy is minimal, the corresponding depth is critical depth. Flow depths less than critical are termed supercritical flow, and depths greater than the critical depth are termed subcritical flow.

The distinction between subcritical and supercritical flow is important in the analysis of open channel flow. Supercritical flow is often characterized as rapid or shooting with flow depths less than critical depth, whereas subcritical flow is tranquil and slow with depths greater than critical. The location of control sections, and the method of analysis will depend upon which type of flow prevails within the channel reach being studied. The Froude number uniquely describes these flow regimes, with the Froude number of critical flow being equal to one. Values greater than one indicate supercritical flow, and values less than one indicate subcritical flow.

In open channel flow, critical depth is the flow depth in which the specific energy is a minimum. The specific energy is expressed as:

$$H_E = d + \frac{V^2}{2g}$$

Where:

HE = specific energy (meters)

d = depth of flow (meters)

V = velocity of flow (m/s)

g = acceleration constant (m/s^2)

b. Equations. The following equations are most commonly used to analyze open channel flow.

Manning's Equation:

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

Energy Equation:

$$d_1 + Z_1 + \frac{(V_1)^2}{2g} = d_2 + Z_2 = \frac{(V_2)^2}{2g} + H_L$$

Continuity Equation:

$$Q = AV$$

Conveyance Equation:

$$k = \frac{Q}{S^{1/2}} = \frac{1}{n} AR^{2/3}$$

where:

Q = Discharge. (m^3/s)
A = Cross section of flow area. (m^2)

$$R = \frac{A}{WP} \; (hydraulic \;\; radius)$$

WP = Wetted perimeter of flow area. (m)
V = Mean velocity. (m/s)
S = Slope of energy grade line. (m/m)
n = Manning's roughness coefficient.
d = Depth of flow at a point. (m)
Z = Elevation or height above some datum in meters.
H_L = Energy head loss. (m)
K = Conveyance

There is no exact method for selection of "n" values in Manning's equation as this coefficient expresses the resistance of flow which consists of many variables. The factors affecting Manning's "n" values include the following:

- Surface roughness.
- Vegetation.
- Channel irregularity.
- Channel alignment.
- Scour and sedimentation.
- Obstructions.
- Size and shape of channel.
- Flow depth and discharge.
- Seasonal changes in vegetation.
- Sedimentation bedload and forms.

Some typical Manning's "n" values can be found in Hydraulic Design Series No. 3, *Design Charts for Open Channel Flow*.

In addition, the FHWA manual titled *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains and Roughness Characteristics of Natural Channels*, WSP 1849 list procedures for the determination of Manning's roughness coefficient for channels and flood plains. Photographs of flood plain segments where "n" values have been verified are presented as a comparison standard to aid in assigning "n" values to flood plains.

Manning's equation is used for open channel analysis where uniform flow exists or can be reasonably assumed to exist. Nonuniform or varied flow requires the use of methods other than, or in addition to, Manning's equation. The energy equation is used to analyze flow where changes in the flow resistance, size, shape, or slope of channel occur (gradually varied flow). The energy balance concept of the energy equation is especially useful for computing water surface profiles.

The conveyance equation is a convenient method for analyzing the flow velocity and distribution where the cross section consists of multiple subdivisions each with a different "n" value or geometric character. The continuity equation and Manning's equation are used to compute channel discharges directly for a given or assumed depth of flow.

c. Flow Depths. There are various flow depths that are used in a hydraulic analysis.
Some of the more common are the following:

(1) Normal depth. Depth at which uniform flow will occur for a given discharge and channel.

(2) Critical depth. Depth for which specific energy is a minimum for a given discharge.

(3) Mean depth. Depth determined by dividing the area of flow in a given channel by the width of flow at the surface. This is sometimes referred to as the hydraulic depth.

(4) Equivalent depth. Depth computed by converting the area of flow to a rectangular flow area with the width of the rectangle twice the depth (equivalent depth) of the rectangle.

2. Analysis of Open Channel Flow. Channel design is a process of establishing criteria, analyzing existing conditions, and using trial and error solutions to develop a design that meets the established criteria.

The following are factors to consider when analyzing open channel flow:

- Stage and depth.
- Channel roughness, geometry and alignment.
- Waterway area.
- Conveyance.
- Energy gradeline and water surface slopes.
- Discharge.
- Velocity.
- Flow distribution.
- Drift and debris.

The stage-discharge relationship is one of the more important factors considered in analysis and design. The total discharge of the stream may be computed for various depths. The data plotted in graphic form (sometimes termed a rating curve) gives the designer a visual display of the relationship.

For channel design, an accurate stage-discharge relationship is necessary to evaluate the interrelationships of flow characteristics (such as width and depth of flow) with certain design characteristics (such as freeboard, conveyance capacity, and type and degree of channel stabilization). The stage-discharge relationship can be used to evaluate a range of conditions.

The stage-discharge relationship may be estimated by several techniques. A single section analysis may be used with limited data for preliminary analysis or for situations where the basic assumptions in a single section analysis are reasonably applicable. A more accurate but more complex method of estimating stage-discharge relations involves the use of water surface profile computations. The method to be used will depend upon the accuracy required, the risk involved, the cost of the study, and the validity of the basic assumptions of a single section analysis.

a. Single Section Analysis. The single section analysis method of establishing a stage-discharge relationship is based on certain simplifying assumptions that must reasonably apply to the actual channel conditions if the relationship is to be used for other than preliminary studies. The analysis can be used for design studies or in a control section, to identify a starting elevation for a water surface profile, or for preliminary studies. This type of stage-discharge analysis is approximate and may be subject to large error if the assumptions implicit in single-section analysis are not reasonably applicable.

The single section method assumes uniform discharge, cross section, slope, and "n" values. These values must be reasonably representative of the average channel characteristics within a uniform cross section. Computations involve designating subsections of the cross section according to geometric and roughness characteristics and computing the conveyance of each subsection for various depths of flow. The total conveyance of the section at any given stage is equal to the sum of all subsection conveyances.

The conveyance equation is used to compute conveyance in subsections. The total discharge is equal to the sum of all subsection conveyances across the channel section multiplied by the slope factor ($S^{0.5}$). The slope used should be the water surface slope. However, the channel slope is assumed to be parallel to the water surface, for uniform flow and if a one section analysis is valid, S must also be equal to S_o.

When estimating a stage-discharge relation by the single section method, it is desirable to have at least one reference point of known stage-discharge data. With this information, the known point can be compared with the computed rating curve and if necessary, adjustments made to roughness or slope values to obtain satisfactory correlation. The computed stage is very sensitive to the estimated slope, and small adjustments within the range of accuracy of estimation of the water surface slope are not contrary to the assumption that $S = S_o$.

b. Water Surface Profiles. The use of a water surface profile is a more accurate method of establishing the stage-discharge relationship for open channels. This method should be used in critical areas and for final studies where uniform steady flow cannot be assumed to be reasonably representative of actual flow conditions.

Water surface profile computations take into account the many variables and controls that influence the stage-discharge relationship. The computation procedures permit taking into account changes in cross section, roughness, or slope along the stream. Rapidly varied flow conditions such as hydraulic jumps, drawdowns and abrupt transitions must be computed individually and integrated into the profile analysis.

The energy equation should be used to compute water surface profiles. The energy regime (subcritical or supercritical) of the channel, as well as the type of channel, (natural and irregular or uniform prismatic), will determine which of the following procedures is to be used. When flow is subcritical, the hydraulic control is downstream and the analysis must begin a sufficient distance downstream of the channel reach in question and proceed upstream. In supercritical flow, the control section is located upstream and the profile computations must begin at the control and proceed downstream. The control is critical depth where flow passes from subcritical to supercritical. Additional discussion of control sections is contained in the next section. For natural or irregular channels, use is made of an energy balancing technique, usually the standard Step Method or some variation. Cross sections and channel roughness descriptions are required at each location along the stream where changes in section, slope, and roughness occur.

When the water surface profile computation involves a channel of uniform cross section and roughness but with reaches of different slopes, a direct step method is normally employed. Chow's *Open Channel Hydraulics* and the U.S. Geological Survey's *Computation of Water-Surface Profiles in Open Channels* include discussions of theory and computational procedures for these methods. These methods require iterative computations but various computer programs are available that carry out these computations.

WSPRO/HY-7 is the water surface profile program developed for the analysis of bridge waterways and the program recommended. An alternative program is the U.S. Army Corps of Engineers HEC-2 program.

c. Control Sections. Sections of stream channel where the geometric and physical characteristics control the depth of flow (a stable stage-discharge relationship) are known as control sections. In stable channels with subcritical flow, this relationship is controlled by a section or reach downstream of the site known as section control or channel control, respectively.

Section controls may be either natural or manmade and may consist of a dam, a ledge or rock outcrop, a boulder-covered riffle, a roadway or railroad embankment, a constriction at a bridge crossing, or other topographic feature. Section controls are frequently effective only for low flows, becoming completely submerged and thus ineffective at medium and high stages. Control sections of this type are most easily identified by field observation.

Channel control consists of all the channel's physical features which, for a given discharge, will determine the stage at a site. These features include waterway area and geometry of channel cross sections; roughness characteristics of the channel bed, banks and flood plain; and channel and flood plain alignment. If channel control is known or suspected to govern the stage-discharge relationship through a channel reach, the water surface profile calculation should begin downstream of that reach with two or more alternate starting water surface elevations and separate profiles computed upstream. If sufficient distance and sections are employed in the Standard Step Method between the starting point and the channel section for which the stage-discharge relationship is desired, the separate profiles will tend to converge on the same water surface elevation.

The stage-discharge relationship for channels where flow is supercritical is controlled by features located upstream of the site. This feature could be a change from a mild or flat slope to a steep slope, a constricted section, a weir, an overflow dam, or other feature. Water surface profile computations must begin at the control section and proceed downstream through the site to the next control section in order to determine if supercritical flow at the site will be submerged by subcritical flow downstream. Water surface profile computations will be grossly in error if the computations are carried upstream through a channel reach subject to upstream control.

3. Channel Stabilization. Design channels to proper size and type to convey the runoff. They must be stable so that excessive erosion and/or scour does not occur along the channel bottom or channel banks. This may require protecting the channel with some form of lining.

For roadside drainage channels and the smaller streams, rock riprap, concrete, and vegetation may be used as means for channel lining. Procedures for the design of roadside drainage channels are contained in Hydraulic Engineering Circular No. 15, *Design of Stable Channels with Flexible Linings*.

For the larger streams and rivers, rock riprap or some other means of stabilization may be necessary. The design procedure of rock riprap for the larger streams and rivers is included in Hydraulic Engineering Circular No. 11.

The small channels normally associated with the typical section of the roadway may be minor drainage channels and include the following:

- Roadway ditches in cut sections.
- Gutters in curbed sections.
- Toe-of-fill and top-of-cut ditches.
- Furrow ditches.
- Small channels formed by berms.
- Spillways and waterways.
- Inlet and outlet ditches for small culverts.

The cross sections of these channels are generally based on minimum standard dimensions to permit easy construction and maintenance with highway equipment. These channels normally have adequate capacity as long as they are properly maintained; have sufficient grade; and erosion, siltation, slides, or plugged culverts are not a problem. Consider the spacing of culverts or inlets to prevent excessive buildup of water in the ditch or gutter when designing roadway cut ditches and curbed gutters. Minor drainage channels may have vee, trapezoidal, rectangular, or triangular shapes.

In areas where it will provide adequate protection, vegetation is generally the most economical lining. Fiberglass roving, jute or other organic netting, or a type of synthetic fabric may be needed to establish a vegetative lining in some ditches.

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 provisions must often be made at the outlet of a smooth lined channel to dissipate the energy of high velocity flow in order to avoid undue scour. This is generally the case at the bottom of spillways and waterways down cut and fill slopes. If high velocity flow (supercritical velocity) is developed in a channel because of smooth lining or steep grades, a special channel design may be required.

If a minor drainage channel is to carry more than the normal runoff from the highway cross section or if its grade will be very flat, the channel shall be designed by the hydraulics engineer. The designer should recognize the conditions that may require this additional work and bring them to the attention of the Hydraulics engineer.

The minimum grade for minor channels should desirably be 0.5 percent. Absolute minimum grades are 0.3 percent for both curbed pavement sections and "v" ditches. In urban areas, curbed sections may have a minimum grade of 0.15 percent. Flat-bottom ditches, 2.5 - 3.0 meters wide, may have a minimum grade of 0.5 percent.

At the tops of crest-vertical curves and the bottoms of sag-vertical curves, there will be substantial lengths of ditch or gutter grade that are flatter than the absolute values above. This is generally not a serious problem on crests, but additional culverts may be needed in sags. In the bottom of sag curbed sections, place an inlet at the bottom and another inlet at a slightly higher elevation to provide protection should the bottom inlet become plugged.

It is not necessary that the ditch grade follow that of the roadbed. Also, it is not necessary for the roadside ditch to be standardized for any length of highway. Wider, deeper, or flat-bottom ditches may be used as required to meet the different amounts of runoff, slopes of channels, type of lining, and distances between points of discharge into culvert inlets. Ditches that follow the roadway can be included in the computer earthwork design by use of the *Special Ditch Grade and Slope Selection* forms.

Riprap usually consists of a layer of loose rock placed on embankments along streams, at inlets and outlets of culverts, and on open drainage channel sides and bottoms. The purpose of such riprap is to prevent scour and washouts that would damage a drainage facility, the highway, or adjacent property. Loose rock riprap may also be used as an erosion control method on slopes highly subject to erosion.

Riprap may be in forms other than loose rock, including keyed rock, hand-placed rock, mesh-enclosed rock, grouted rock, concrete in bags, concrete slabs poured in place, or precast concrete blocks.

Keyed riprap is loose rock riprap that is slapped with a piece of heavy steel plating after placement, in order to key the rocks together and provide a smoother surface. Keying also increases stability and allows the use of less rock or a smaller size rock.

a. Riprap Design. The design of riprap includes a determination of the depth of water and the velocity of flow that will impinge upon the rock for the design flood, and the determination of the class and thickness of the riprap layer that will withstand the water forces.

These determinations are the responsibility of the hydraulics engineer. (See Section 7.1.B)

There are five classes of riprap that are generally used along streams for low, intermediate, and high velocity forces. These classes and their gradations are specified in the current FP, which makes the following recommendations for their use:

- Class 1 to protect areas from minor erosion.
- Class 2 as a ditch lining in areas exposed to significant scour.
- Class 3 for scour protection at the outlets of culvert pipes smaller than 1200 millimeters in diameter.
- Class 4 for scour protection at the outlets of culvert pipe larger than 1200 millimeters in diameter and for protection of roadway embankments susceptible to scour.
- Class 5 and 6 for protection at bridge abutments, piers, or at other critical areas susceptible to erosion.

The plans shall show one or more typical sections with dimensions for the types of riprap specified and indicate the estimated quantities at each riprap site.

Two methods of constructing riprap blankets are using a foundation trench or using a thickened toe. The foundation trench is preferred where it is feasible. The foundation trench should be dug to hard rock, keyed into soft rock, or dug below the depth of scour. If the scour depth is questionable, the thickened toe may be used. The thickened toe will adjust itself should scour occur and thus provide deeper support. A foundation trench will not normally be required in solid rock. However, if the riprap will be founded on a steep rock slope above the stream bottom, a trench or a bench should be provided at the base of the riprap, or some other means used to retain the riprap.

The ends of riprap sections (particularly the upstream end) are often not given due consideration in design or during construction. Indicate on the plans or specify in the specifications that the ends of riprap sections are to be keyed into the natural ground. If this is applicable, allow a sufficient keyed depth to prevent undermining and migration of the rocks at the ends.

Ordinarily, riprap shall be placed outside the normal fill slope. However, in restricted areas, riprap may be incorporated in the normal fill slope.

Gabion mattress (See HEC No. 15) stability is similar to that of riprap but because the stones are bound by a mesh, they tend to act as a single unit. Movement of stones within a gabion is negligible. This permits use of smaller stone sizes compared to those required for loose riprap. Of course the stability of gabions depends on the integrity of the mesh. In streams with high sediment concentrations or with rocks moving along the bed of the channel, the containment mesh may be abraded and eventually fail. Under these conditions the gabion will no longer behave as a single unit but rather the stones will behave individually. Applications of gabion mattresses and baskets under these conditions should be avoided.

b. Filter Material. Many riprap failures have been the result of fine soil from the embankment being washed away through the openings in the riprap. The riprap then loses some of its stability and is easily moved by the force of the water during flood conditions. For channels protected with rock riprap or gabions, the need for a filter blanket must be analyzed.

The filter blanket may be either a granular filter blanket or a filter fabric. Filter fabric is a woven pervious synthetic cloth placed on the slope under the riprap blanket which prevents fine soil from washing out of the embankment. Specify filter fabric for all slopes along a stream where riprap is placed, except on solid rock or where the hydraulic engineer indicates that it is not necessary. Both HEC No. 11 and HEC No. 15 provide design guidance for filter blankets.

4. Channel Changes. Alterations or relocations of existing stream channels should be avoided wherever possible without incurring excessive costs to the project. Consider alternate solutions to prevent unnecessary disruption of natural channels. If a channel change is necessary, the concurrence of all agencies having legal responsibility for the waterway shall be obtained before the change is incorporated in the final plans.

Properly designed channel changes can reduce the hazard of flood damage to a highway crossing by reducing skew and curvature, and by sometimes providing a larger main channel. Unfortunately, there are nearly always unwanted side effects that damage fish habitat and generally reduce the quality of the streamside environment. Velocity of flow is almost always increased over that of the natural channel because of a reduction of channel roughness and an increase in the channel slope. Erosion and downstream siltation problems may result from a channel change that may last over a long period of time. The removal of stream bank vegetation, the raw cut slopes, and the channel lining are detrimental to the fish carrying capacity of the stream. These modifications are also visually unattractive.

When a channel change is necessary, design the channel with stable streambed and banks capable of carrying the design flood. Make special efforts to improve fish habitat and visual appearance.

a. Design. Where practical, the channel change should duplicate the existing stream characteristics including stream width, depth, slope, sinuosity, bank cover, side slopes, and flow and velocity distribution. Complete a hydraulic analysis to determine the adequacy of the channel change and whether a lining is needed.

b. Linings. In order to retain stable channels, channel changes are generally lined with riprap. The banks, and very often the bottom also, may require a lining. Riprap linings shall be designed using procedures in HEC No. 11.

c. Enhancement for Fish Habitat and Aesthetics. The following additional steps should be considered to improve on a channel change:

- Require that the clearing limits not extend outside the slope stakes.

- Design channel sections that approximate the natural channel. Such a channel will be deepest on the outside of bends. Avoid the conventional neat lines of slope and gradient. Attempt to make the new channel fit into the surroundings. An on-site review or aerial photos may be helpful here.

- Excavate all cut slopes above the waterline with rough surfaces and revegetate to stabilize the soil and provide shade.

- Do not permit riprap to extend above a natural channel bank even if overflow is anticipated. Such riprap serves little, if any, purpose and is not pleasing in appearance.

- If space is available, riprap should be topped with a layer of graded rock and then topsoil and planted with native vegetation to provide shade and help restore a natural appearance.

- Boulders may be installed in the channel bottom to improve fish habitat. This is required by some State fish and game agencies. As a rule of thumb, "fish boulders" should be large enough to withstand the force of the current, placed low enough so that the mean annual flood will inundate them and wash away accumulated debris, and be small enough so that a single boulder or cluster of boulders will occupy no more than 25 percent of the channel bottom width. Fish boulders should be located in a random manner.

If the section of existing channel that is being relocated has suitable boulders in it, it is preferable that these be used in the channel change rather than using fractured rock. This principle applies also to cobbles and gravel in the existing streambed which, if suitable, may be used to line the bottom of the channel change.

- Alternating rock jetties (groins) may be constructed to control velocity and improve fish habitat. Jetty rock should be of a larger size than those used for riprap on the banks of the channel change.

- Gabion "ribs" extending across the channel may be used as control structures or weirs and also to improve fish habitat.

d. Plans. Drawings should be prepared for all alterations, relocations, and encroachments of waterways, except for very minor encroachments. The plans should show the plan and profile for the new channel, provide an adequate number of cross sections so the channel can be constructed as designed, and include details of additional measures to restore fish habitat and a natural appearance such as boulders, weirs, groins, and plantings.

D. Culverts. The function of a culvert is to convey surface water across or from the highway right-of-way. In addition to the hydraulic function, the culvert must carry construction and highway traffic and earth loads. Therefore, culvert design involves both hydraulic and structural design considerations. Hydraulic aspects of culvert design are set forth in this section.

Culverts are available in a variety of sizes, shapes, and materials. These factors, along with several others, affect their capacity and overall performance. Sizes may vary from 300 millimeter circular pipes to extremely large arch sections that are sometimes used in place of bridges.

The most commonly used culvert shape is circular, but arches, boxes, and elliptical shapes are sometimes used. Pipe arch and elliptical shapes are generally used in lieu of circular pipe where there is limited cover. Arch culverts have application in locations where less obstruction to a waterway is a desirable feature, and where foundations are adequate for structural support. Box culverts can be designed to pass large flows and to fit nearly any site condition. A box or rectangular culvert lends itself more readily than other shapes to low allowable headwater situations since the height may be decreased and the span increased to satisfy the location requirements.

The material selected for a culvert is dependent upon several factors, such as durability, structural strength, roughness, bedding condition, abrasion and corrosion resistance, and water tightness. The more common culvert materials used are concrete, steel (smooth and corrugated), and corrugated aluminum.

Another factor that significantly affects the performance of a culvert is the culvert inlet configuration. The culvert inlet may consist of a culvert barrel projecting from the roadway fill or mitred to the embankment slope. Other inlets have headwalls, wingwalls, and apron slabs or standard end sections of concrete or metal.

1. Design. The hydraulic design of a culvert consists of an analysis of the performance of the culvert conveying flow from one side of the roadway to the other. The designer must select a design flood frequency, estimate the design discharge for that frequency, and set an allowable headwater elevation based on the selected design flood and headwater considerations. The culvert size and type can be selected after the design discharge, controlling design headwater, tailwater, and allowable outlet velocity have been determined.

The design of a culvert includes the determination of the following:

- Line, grade, and length of invert.
- Size, type, end treatment, headwater, and outlet velocity.
- Amount and type of cover.
- Type of acceptable materials and thicknesses acceptable.
- Type of coating (if required).
- Need for fish passage measures.
- Need for protective measures against abrasion and corrosion.
- Need for special designed inlets or outlets.

In addition, at critical installations, the design may have to consider special foundation work and backfill procedures.

a. Discharge. The discharge used in culvert design is usually estimated on the basis of a preselected recurrence interval and the culvert is designed to operate in a manner that is in acceptable limits or risk at that flow rate.

The rate of flow of water to be conveyed by the facility must be determined. This rate of flow (commonly known as the discharge and measured in cubic meters per second) is designated by the letter "Q" in hydraulic formulas. The design discharge is an estimated rate of flow that will occur in a flood of a preselected frequency or recurrence interval. The recurrence interval for a flood that has a 1 in 100 (1 percent) chance of happening in any 1 year is 100 years. Similarly, for a flood that has a 1 in 50 (2 percent) chance or a 1 in 25 (4 percent) chance of happening in any 1 year, the recurrence intervals are 50 or 25 years respectively. Such floods are commonly called 100, 50, or 25 year floods. A 50-year flood is substantially greater in magnitude than a 25 year flood, and a 100 year flood is substantially greater in magnitude than a 50 year flood.

b. Headwater. Culverts generally constrict the natural stream flow which causes a rise in the upstream water surface. The elevation of this water surface is termed *headwater elevation* or the total flow depth in the stream measured from the culvert inlet invert is termed *headwater depth.*

In selecting the design headwater elevation, the designer should consider the following:

- Upstream property damage.
- Damage to the culvert and the roadway.
- Traffic interruption.
- Hazard to human life.
- Headwater/Culvert Depth (HW/D) ratio.
- Low point in the roadway grade line.
- Roadway elevation above the structure.
- Elevation at which water will flow to the next cross drainage.

The headwater elevation for the design discharge should be consistent with the freeboard and overtopping criteria in Section 7.4.A. The designer should verify that the watershed divides are higher than the design headwater elevations. In flat terrain, drainage divides are often undefined or nonexistent and culverts should be located and designed for least disruption of the existing flow distribution.

c. Tailwater. Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

A field inspection of the downstream channel should be made to determine whether there are obstructions that will influence the tailwater depth. Tailwater depth may be controlled by the stage in a contributing stream, headwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

d. Outlet Velocity. The outlet velocity of a highway culvert is the velocity measured at the downstream end of the culvert, and it is usually higher than the maximum natural stream velocity. This higher velocity can cause stream bed scour and bank erosion for a limited distance downstream from the culvert outlet. Permissible velocities at the outlet will depend upon stream bed type.

If the outlet velocity of a culvert is too high, it may be reduced by changing the barrel roughness. If this does not give a satisfactory reduction, it may be necessary to use some type of outlet protection or energy dissipation device.

Variation in shape and size of a culvert seldom have a significant effect on the outlet velocity. Slope and roughness of the culvert barrel are the principle factors affecting the outlet velocity.

2. Flow Characteristics. There are two major types of flow characteristics that are described as culverts with inlet control and culverts with outlet control.

For each type of control a different combination of factors is used to determine the hydraulic capacity of a culvert. The determination of actual flow conditions can be difficult; therefore, the designer must check for both types of control and design for the most adverse condition.

a. Inlet control. A culvert operates with inlet control when the flow capacity is controlled at the entrance by

- depth of headwater,
- cross sectional area,
- inlet edge configuration, or
- barrel shape.

Sketches to illustrate inlet control flow for unsubmerged and submerged entrances are shown in Figure 7-1.

When a culvert operates in inlet control, headwater depth and the inlet edge configuration determine the culvert capacity with the culvert barrel usually flowing only partially full.

For a culvert operating with inlet control, the roughness, slope, and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance.

b. Outlet Control. In outlet control, culvert hydraulic performance is determined by

- depth of headwater,
- cross sectional area,
- inlet edge configuration,
- culvert shape,
- barrel slope,
- barrel length,
- barrel roughness, or
- depth of tailwater.

Culverts operating in outlet control may flow full or partly full depending on various combinations of the above factors. In outlet control, factors that may affect performance appreciably for a given culvert size and headwater are barrel length and roughness, and tailwater depth. Typical types of outlet control flow are shown in Figure 7-2.

3. Performance Curves. Performance curves are point plots of discharge versus culvert headwater or elevation. See Figure 7-3. The curves aid in the selection of culvert type, size, shape, material, and inlet geometry that fulfills site requirements.

The curves plotted in Figure 7-3 apply only to the applicable specified criteria. If any data is revised or modified, the performance curves will change. The performance curves shown are for a 1200-millimeter round corrugated metal pipe with a beveled inlet. The pipe is on a 3 percent grade and 49 meters in length. The curves cross at approximately 3.3 m^3/s.

This is the point where inlet control changes to outlet control. Under outlet control, more energy (headwater) is required to get the flow through the barrel than to get the flow past the inlet.

Performance curves should be developed for both inlet and outlet control conditions for all culverts 1200 millimeters in diameter and larger. Performance curves should also be developed for smaller cross drain culverts that are placed in locations considered to be critical or sensitive.

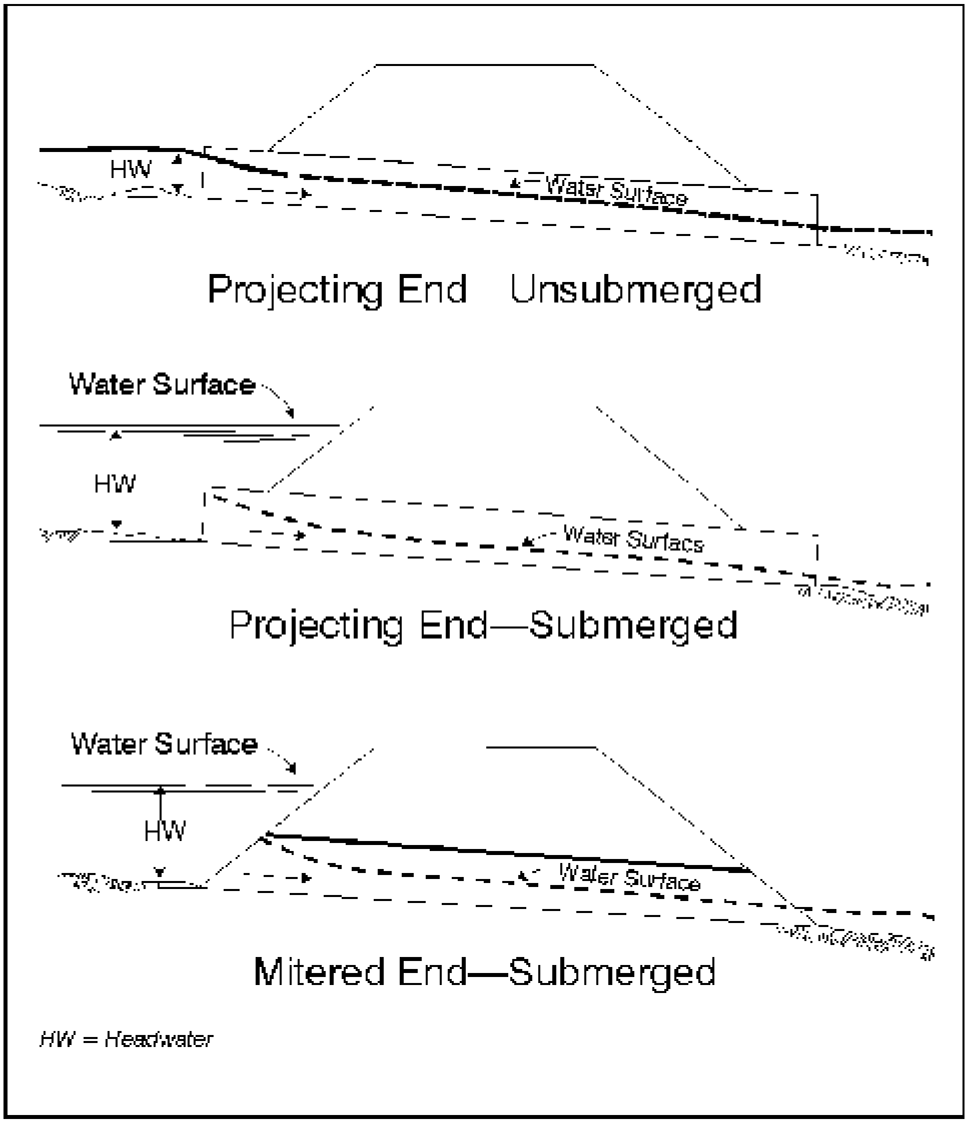


Figure 7-1
Typical Culvert Inlet Control Sections

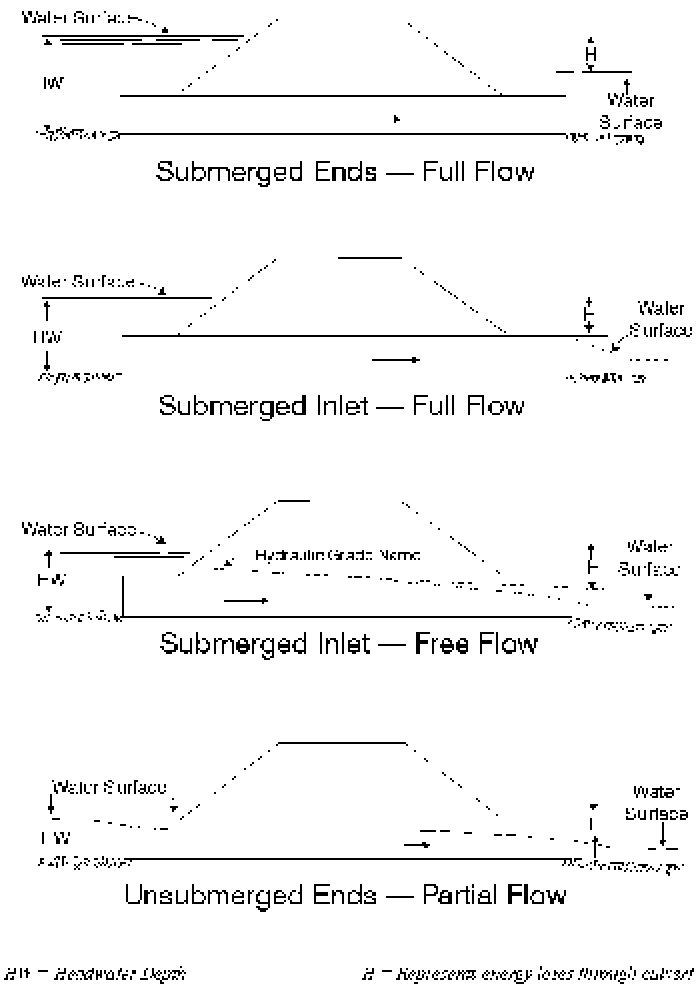


Figure 37 **Figure 7-2**
Typical Culvert Outlet Control Sections

4. Size Selection. (See Section 7.4.A.3) Hydraulic Design Series No. 5, *Hydraulic Design of Highway Culverts*, is the text to be used for the design and selection of drainage culverts. This circular explains inlet and outlet control and the procedure for designing culverts.

Culvert design basically involves the trial and error method:

(1) Select a culvert shape, type, and size with a particular inlet end treatment.

(2) Determine a headwater depth (the depth of water backed up at the inlet end of the culvert) from the charts in the circular for both inlet and outlet control for the design discharge, the grade and length of culvert, and the depth of water at the outlet (tailwater).

(3) Compare the largest depth of headwater (as determined from either inlet or outlet control) to the design criteria. If the design criteria is not met, continue trying other culvert configurations until one or more configurations are found to satisfy the design parameters.

(4) Estimate the culvert outlet velocity and determine if there is a need for any special features such as energy dissipators, riprap protection, fish passage, etc. See Exhibit 7.6 for a standard culvert design worksheet format.

Critical areas shall be checked by the designer by plotting the headwater surface elevation on the detail map. If the headwater surface extends beyond the right-of-way, the designer should evaluate alternative culvert design instead of obtaining more right-of-way or a permanent easement.

For roadside ditch drainage across road approaches, the minimum culvert size is 300 millimeters in diameter. For deeper ditches, an 450-millimeter culvert is preferable.

The minimum size culvert for crossing a highway should be a 600-millimeter diameter or equivalent. In unusual circumstances, an 450-millimeter culvert may be used in place of a 600-millimeter culvert when the length of culvert to be installed is relatively short and ditch erosion upstream of the culvert or culvert plugging is not critical.

When a culvert is connected to a catch basin or drop inlet, or is part of a storm sewer system, the minimum size is 300 millimeters in diameter.

Although the hydraulic engineer is responsible for the design of larger culverts (larger than 1200 millimeters), the designer must furnish data on the location, grade, and length of culverts; the grade of the road; and any site conditions that may be helpful to the hydraulics engineer. Once the culverts are designed, the designer must ensure that all necessary data is shown on the plans. Should it be necessary to change the location, grade, or length of a culvert, the designer should check with the hydraulic engineer to determine if the original designed culvert is still adequate.

5. Materials. The durability and service life of a drainage pipe installation is directly related to the environmental conditions encountered at the site and the type of materials and coatings from which the culvert was fabricated. Two principal causes of early failure in drainage pipe materials are corrosion and abrasion. The environmental damage caused by corrosion and abrasion can be delayed by the type of materials, coatings, and invert protection.

It is Federal Lands Highway (FLH) policy to specify alternative drainage pipe materials on all projects where feasible and to comply with the provisions of the Federal-Aid Policy Guide Section 635.411(d). All permanent drainage pipe installations shall be designed for a minimum of 50 years maintenance-free service life. A shorter service life may be used for temporary installations, and a longer service life may be considered in unusual situations.

All suitable pipe materials, including reinforced concrete, steel, aluminum, and plastic pipe shall be considered as alternatives on FLH projects. The portion of this pipe selection criteria covering metal pipe complies with the guidance contained in Federal Highway Administration (FHWA) Technical Advisory T 5040.12 dated October 22, 1979, and incorporates information contained in FHWA-FLP-91-006, Durability of Special Coatings for Corrugated Steel Pipe.

a. Definitions of Corrosion and Abrasion:

Corrosion: Alkalinity/Acidity (pH) and Resistivity: Determinations of pH and resistivity are required at each pipe location in order to specify pipe materials capable of providing a maintenance free service life. The samples shall be 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 insure 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 2 days following flood flows to insure 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 soil side conditions control.

Abrasion: An estimate of the potential for abrasion is required at each pipe location 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 meters per second 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.

■Level 4, **severe abrasive** conditions exist in areas of heavy bed loads of sand, gravel, and rock and velocities exceeding 4.5 m/s.

These definitions of abrasion levels are intended as guidance to help the designer consider the impacts of bedload 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 and not a 10 or 50-year design flood.

b. Alternatives to be Considered:

(1) **Reinforced Concrete**

A reinforced concrete pipe (AASHTO M 170M) shall typically be specified as an alternative whenever environmental conditions permit. The appropriate pipe Class shall be determined from approved FLH fill height standard drawings. It is FLH policy to use Class II as the minimum for all reinforced concrete pipes. Within the following limits of corrosion and abrasion, reinforced concrete pipe can be assumed to have a service life of a minimum of 50 years. Typically a minimum service life of 75 years can be assumed for reinforced concrete pipe.

Corrosion and abrasion effects on concrete pipes:

■**Corrosion:** Reinforced concrete pipe should not be specified for **extremely corrosive** conditions where the pH is less than 3 and the resistivity is less than 300 ohm centimeters ($\Omega \cdot cm$). Where the pH is less than 4, or the pipe is exposed to wetting and drying in a salt or brackish water environment, protective coatings, such as epoxy resin mortars or poly vinyl chloride sheets should be used. When the sulfate concentration is greater than 0.2 percent in the soil or 2000 parts per million in the water, Type V cement should be specified. When the sulfate concentration is greater than 1.5 percent in the soil or 15,000 parts per million in the water, Type V cement should be used with a sulfate resistant pozzolan, or a higher cement ratio may be used such as in an AASHTO Class V pipe design.

■**Abrasion:** On installations in **severe abrasive** environments, consider using 7 or 8 sack concrete or increasing the cover over the reinforcing steel.

(2) **Steel**

Steel pipe will typically be specified as an alternative when the environmental conditions permit. The appropriate minimum structural metal thickness shall be determined from approved FLH fill height tables. The minimum wall thickness for all steel pipes used on FLH projects is 1.63 mm, as shown in the FLH fill-height tables.

It is FLH policy that steel pipe provides a useful, maintenance-free service life for a period of time beyond the point of first perforation. This assumes an acceptable risk for most FLH 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 combine, 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.

Steel pipe is divided into 2 categories; steel pipe with metallic coatings, and steel pipe with non-metallic coatings.

Steel Pipe with Metallic Coatings. Under **non-abrasive** and **low-abrasive** conditions, the service life of steel pipe with metallic coatings may be determined based upon corrosion (pH and resistivity) from Figure 7-4. Figure 7-4 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 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.

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

- **Galvanized steel (AASHTO M 218).**

- **Aluminum/zinc alloy coated steel (galvalume) (AASHTO M 289).**

- **Aluminum coated steel sheets (Type 2) (AASHTO M 274).**

Steel Pipe with Non-Metallic Protective Coatings. Protective coatings may be used to protect against corrosion or abrasion. Coatings to protect against corrosion may only be used in **non-abrasive** and **low abrasive** environments, except that bituminous coated pipe should not be used in **low abrasive** environments. Under **moderate abrasive** conditions, bituminous paved pipe and concrete lined pipe may be used for invert protection *where corrosion protection is not required*.

The additional service life noted below in *italics* 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 shall be considered as alternatives on Federal Lands Highway projects:

- **Bituminous coated (AASHTO M 190).** Bituminous coatings 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 Figure 7-4.

■**Bituminous coated and paved (AASHTO M 190).** Bituminous paved invert with bituminous coatings can be expected to add *25 years* life to water side locations.

■**Concrete lined (ASTM A 849).** Concrete lining can be expected to add *25 years* of service life. Due to the natural cracking of the concrete, the concrete lining should be applied over an asphalt coating if corrosion protection is needed.

■**Polymer coated (AASHTO M 245M). Ethylene Acrylic Acid Film** coatings should provide an additional *30 years* service life with a 0.25 mm thickness.

Only limited data is available for the service life of **Aramid fiber bonded coated (ASTM A 885) and Epoxy coated** pipes and no additional service life is currently credited with this policy.

Corrosion and abrasion effects on steel pipes:

Within the corrosion and abrasion limits noted below, the average service life of steel pipe can be predicted. Detailed examples are included in Exhibit 7.10 which includes Figure 7.4 and Figure 7.5 for determining service life.

■**Corrosion:**Under **nonabrasive** and **low abrasive** conditions, the metal thickness of galvanized, galvalume, and aluminum coated steel (Type 2) alternatives should be determined from Figure 7-4 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 5-9 pH and resistivity equal to or greater than 1500 Ω·cm, aluminum coated steel (Type 2) can be expected to give a service life of twice that of plain galvanized pipe. The National Corrugated Steel Pipe Association has recommended that steel pipe not be used below a pH of 4 when the resistivity is below 1500 Ω·cm.

Figure 7-4 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 Figure 7-4, 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 from that determined from Figure 7-4, *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 from that determined from Figure 7-4 *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 abrasion** locations. *Metal pipes should not be specified in **moderate** and **severe abrasive** environments where coatings are required to protect against water side corrosion.*

(3) **Aluminum Alloy**

Aluminum alloy pipe (AASHTO M 196M) will typically be specified as an alternative when environmental conditions permit. The appropriate minimum structural metal thickness shall be determined from approved FLH fill height tables. It is FLH policy to use a minimum metal thickness for all aluminum alloy pipes of 1.52 mm as shown in the FLH standard 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 and abrasion effects on aluminum alloy pipes:

■**Corrosion:** An aluminum alloy should be allowed if the pH is between 4 and 9 and the resistivity is greater than 500 Ω·cm. 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 **moderate 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.

(4) **Plastic**

Polyethylene and polyvinyl chloride plastic pipe may be specified as alternatives for pipe diameters and minimum resin cell classifications shown in the American Associations of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges, Division I Design, Section 18, Soil Thermoplastic Pipe Interaction Systems. The thickness of the plastic alternatives shall meet the structural requirements of the AASHTO Section 18. 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 shall be 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).**

Corrosion and abrasion effects on plastic pipes:

■**Corrosion:** Plastic alternatives may be specified without regard to the resistivity and pH of the site.

■**Abrasion:** 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.

Industry representatives have pointed out that a conflict exists between the resin cell classification requirements of AASHTO Highway Bridge Specifications Section 18 and AASHTO M 294 for corrugated polyethylene pipe. Until AASHTO resolves this conflict, corrugated polyethylene pipe may also be permitted if it meets the materials requirements of AASHTO M 294. The allowable fill heights for this material are based upon permitted usage in states and is included in the FLH standard fill-height table.

The locations selected for use of plastic pipes should address owner agency concerns of possible damage due to fire, ultraviolet sunlight, and rodents. Until more experience is gained by FLH, plastic pipe will be permitted under mainline routes only after careful analyses by the designer and concurrence of the land owning agencies.

6. Culvert Location. Culvert location is an integral part of the total design. Its main purpose is to convey drainage water across the roadway section expeditiously and effectively. The designer should identify all live stream crossings, springs, low areas, gullies, and impoundment areas created by the new roadway embankment for possible culvert locations. See Sections 7.4.D.1, 7.4.D.12 and in Chapter 9, see Section 9.4.E.

a. Alignment. Locate culverts on existing stream alignments. Align the culvert to give the stream a direct entrance and a direct exit. Abrupt changes in direction at either end may retard the flow and make a larger structure necessary. If necessary, a direct inlet and outlet may be obtained by means of a channel change, skewing the culvert, or a combination of these. The choice of alignment should be based on economy, environmental concerns, hydraulic performance, and/or maintenance considerations.

Reasonable precautions should be taken to prevent the stream from changing its course near the ends of the culvert including the use of riprap, sod, paving, or metal end sections. For certain site conditions, a curved alignment may be appropriate to limit structure excavation.

b. Gradient. The grade chosen for a particular culvert will depend on the site conditions. For highway purposes, culverts may be considered to be in one of two categories:

(1) Minimum sized culverts used for cross drains to carry away intermittent roadside ditch water or water from very small drainage areas. These culverts seldom are required to carry their maximum discharge.

The grade for ditch cross drains should not be flatter than 0.5 percent nor steeper than 10 percent. See Section 7.4.A.3.

(2) Culverts sized to carry live streams and large runoffs. These culverts are individually designed to carry the design discharge for the drainage area they serve without excessively backing up water.

When placing culverts in live streams design the culvert invert to coincide with the average streambed flow line. Deviations are acceptable under the following conditions:

- When headroom is limited. The culvert inlet may be lowered below the streambed in streams where sedimentation will not be a problem, in which case the inlet basin may have to be stabilized with riprap or by other means. Alternatives to lowering the inlet include the use of a pipe arch, an oversize round pipe with the entire inlet buried below the streambed, multiple culverts, a drop inlet, or raising the road grade.

- Under high fills. The culvert should be designed with sufficient camber to allow for settlement as indicated on the plans.

- When ponding is permissible. Sometimes it may not be necessary to place the culvert at the streambed grade. If ponding is permissible or a random fill is placed on the inlet end of the pipe, the inlet can be raised. If the outlet can be placed on nonerodible soil or the pipe can be made to spill on riprap or rock, the outlet can be raised and the length of culvert reduced. If fish passage is a requirement, these methods can not be used.

- In steep sloping areas. The culvert need not always be placed on the same steep grade. It may be skewed and put on any desired grade with a spillway or cutoff wall constructed at the outlet, or a down drain may be used down the fill slope.

- When fish passage is required. The culvert will usually have to be placed on a very flat grade (less than 0.5 percent), or an oversize culvert placed with either the invert below streambed grade or fish baffles installed in the invert.

7. Inlet Protection. Inlets on culverts, especially on culverts to be installed in live streams, should be evaluated relative to debris control and buoyancy.

a. Debris Control. Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. The consequences may be damages from inundation of the road and upstream property.

The designer has three options for coping with the debris problem:

(1) Retain the debris upstream of the culvert.

(2) Attempt to pass the debris through the culvert.

(3) Install a bridge.

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. If the debris is to be passed through the structure or retained at the inlet, a relief opening should be considered either in the form of a vertical riser or a relief culvert placed higher in the embankment. It is often more economical to construct debris control structures after problems develop, since debris problems do not occur at all suspected locations.

Precede the design of the debris control structure with a thorough study of the debris problem.

The following are among the factors to be considered in a debris study:

- Type of debris.
- Quantity of debris.
- Expected changes in type and quantity of debris due to future land use.
- Stream flow velocity in the vicinity of culvert entrance.
- Maintenance access requirements.
- Availability of storage.
- Maintenance plan for debris removal.
- Assessment of damage due to debris clogging, if protection is not provided.

Hydraulic Engineering Circular No. 9, *Debris Control Structures*, should be used when designing debris control structures.

b. Buoyancy. The forces acting on a culvert inlet during flows are variable and indeterminate. When a culvert is functioning with inlet control, an air pocket begins just inside the inlet that creates a buoyant effect when the inlet is submerged. The buoyancy forces increase with an increase in headwater depth. These forces, along with vortexes and eddy currents, can cause scour, undermine culvert inlets, and erode embankment slopes thereby making the inlet vulnerable to failure especially with deep headwater.

In general, installing a culvert in a natural stream channel constricts the normal flow. The constriction is accentuated when the capacity of the culvert is impaired by debris or damage.

The large unequal pressures resulting from inlet constriction are in effect buoyant forces that can cause entrance failures, particularly on corrugated metal pipe with mitred, skewed, or projecting ends. The failure potential will increase with steepness of the culvert slope, depth of the potential headwater, flatness of the fill slope over the upstream end of the culvert, and the height of the fill.

Anchorage at the culvert entrance helps to protect against these failures by increasing the deadload on the end of the culvert, protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. Provide a standard concrete headwall or endwall to counteract the hydrostatic uplift and to prevent failure due to buoyancy.

8. Sedimentation. Deposits usually occur within the culvert barrels at flow rates smaller than the design flow. The deposits may be removed during larger floods dependent upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert, and other factors.

Culvert location in both plan and profile is of particular importance to the maintenance of sediment-free culvert barrels. Deposits occur in culverts because the sediment transport capacity of flow within the culvert is often less than in the stream.

Deposits in culverts may also occur due to the following conditions:

- At moderate flow rates the culvert cross section is larger than that of the stream, so the flow depth and sediment transport capacity is reduced.

- Point bars form on the inside of stream bends. Culvert inlets placed at bends in the stream will be subjected to deposition in the same manner. This effect is most pronounced in multiple-barrel culverts with the barrel on the inside of the curve often becoming almost totally plugged with sediment deposits.

- Abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce sedimentation. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

9. Fish Passage. At some culvert locations, the ability of the structure to accommodate migrating fish is an important design consideration. For such sites, State fish and wildlife agencies should be consulted early in the roadway planning process. Some situations may require the construction of a bridge to span the natural stream. However, culvert modifications can often be constructed to meet the design criteria established by the fish and wildlife agencies.

Early in the planning process, fish migration data should be collected including pertinent field data. If the stream crossing is located on a known, suspected, or potential fish migration route, the following data is desirable:

- Species of migrating fish.

- Size and swimming speed of fish.

- Locations of spawning beds, rearing habitat, and food producing areas upstream and downstream of the site.

- Description of fish habitat at the proposed crossing.

- Dates of start, peak, and end of migration.

- Average flow depths during periods of migration.

An understanding of design inadequacies that will inhibit natural migration patterns is desirable. Excessive velocities and shallow depths inside or adjacent to the culvert should be avoided. High outlet elevations, often resulting from the formation of a scour hole, may prevent fish from entering the culvert. High outlet velocities also dislodge sediment which fills in small pools further downstream, smothering eggs and food-producing areas in the process. High upstream invert elevations produce a large unnatural pool above the culvert that will trap sediment. Depressing the upstream invert elevation is also harmful.

Simulating the natural stream bottom conditions in a culvert is the most desirable design option to accommodate fish passage. Open bottom culverts, such as arches, have obvious advantages if adequate foundation support exists for the culvert. Oversized culverts with buried inverts have the advantage of a natural bottom while overcoming the problem of poor foundation material. However, on steep slopes, provisions may be necessary to hold bottom material in place. Another option is to construct baffles in the bottom of culverts to help simulate natural conditions.

When the simulation of natural stream bottom conditions is unrealistic or unnecessary, criteria for maintaining minimum depths and maximum velocities is most important. The high roughness coefficient of corrugated metal may be all that is required at some locations to maintain desirable depths and velocities. When maintaining a minimum depth in a culvert is a problem, downstream weirs can be constructed. However, provisions must be made for fish to bypass the weirs.

A popular method of providing for fish passage is to provide dual culverts, one culvert designed for hydraulic capacity and one culvert designed for fish passage. The latter culvert would have a flatter slope, higher roughness, and could contain fish baffles. In this case, the hydraulically efficient barrel would convey most of the flow. To design parallel, dissimilar culverts, it is necessary to construct separate performance curves (elevation versus discharge) for each culvert. The two performance curves are added together at equal elevations to obtain the combined performance curve.

The hydraulic design of culverts with fish baffles is accomplished by modifying the friction resistance of the barrel in outlet control to account for the high resistance imposed by the baffles. For inlet control, only the reduced area of the entrance due to the baffles and any edge modifications need to be considered in the procedure. See HDS No. 5 for the design procedures.

Outlet and barrel velocities are also critical for fish passage. Velocities in culvert barrels should be 1.2 meters per second or less for the flow conditions expected during fish migration periods. If these velocities cannot be obtained because of the required slope of the culvert, one of the following alternative methods may be used to provide fish passage:

- Use an oversize culvert with the invert placed below the streambed (0.3 - 1.0 meters) and backfilled with cobbles and gravel to the average streambed grade.

- Use an oversize culvert with baffles in the bottom to provide fish resting pools.

- Construct weirs downstream of the culvert to raise the water level in the culvert barrel and to slow down the velocity.

10. Outlet Protection. Scour at culvert outlets is a common occurrence. The natural channel flow is usually confined to a lesser width and greater depth as it passes through a culvert barrel. An increased velocity results with potentially erosive capabilities as it exits the barrel. Turbulence and erosive eddies form as the flow expands to conform to the natural channel. However, the velocity and depth of flow at the culvert outlet and the velocity distribution upon reentering the natural channel are not the only factors that need consideration.

The characteristics of the channel bed and bank material, velocity and depth of flow in the channel at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Due to the variation in expected flows and the difficulty in evaluating some of these factors, scour prediction is subjective.

Scour in the vicinity of a culvert outlet can be classified into two separate types called local scour and general stream degradation:

a. Local Scour. Local scour is typified by a scour hole produced at the culvert outlet. This is the result of high exit velocities, and the effects extend only a limited distance downstream.

Coarse material scoured from the circular or elongated hole is deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow. Methods for predicting scour hole dimensions are found in HEC No. 14.

b. General Stream Degradation. General stream degradation is a phenomenon that is independent of culvert performance. Natural causes produce a lowering of the stream bed over time. The identification of a degrading stream is an essential part of the original site investigation.

Both types of scour can occur simultaneously at a culvert outlet.

Since prediction of scour at culvert outlets is difficult, and protection is expensive, one approach involves providing a minimum amount of protection followed by periodic site inspection. As part of the field investigation, scour and outlet protection at similar culverts in the vicinity will provide guidance. The initial level of protection should be sufficient to withstand extensive damage from one storm event. Once the initial minimum outlet protection is constructed, an assessment of its performance after a number of storm events should be evaluated and reviewed. If the outlet protection is insufficient, additional protection should be provided. If the outlet protection is sufficient, inspection is required only after larger storm events.

Protection against scour at culvert outlets varies from limited riprap placement to complex and expensive energy dissipation devices. At some locations, 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 stream bed.

Riprapped channel expansions and concrete aprons protect the channel and redistribute or spread the flow. Barrel outlet expansions operate in a similar manner. Headwalls and cutoff walls protect the integrity of the fill. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices. These include hydraulic jump basins, impact basins, drop structures, and stilling wells. Design information for the general types of energy dissipators is provided in HEC No. 14.

11. Culvert Lengths and End Treatments. The lengths of culverts are generally determined by the use of plotted cross sections. If a pipe is on a skew, the pipe section may have to be detailed using map contours.

An end section should be used on a small diameter culvert when the culvert invert grade at the inlet (or outlet) is within approximately 1.5 meters of the shoulder elevation, except on fill slopes with a 1:2 ratio and steeper, or where the end of culvert is in back of guardrail.

End sections improve the appearance of a highway; prevent run-off-the-road vehicles from striking the end of a culvert (thus eliminating a very severe hazard); prevent scour, undermining, and seepage at culvert ends; and increase the hydraulic efficiency of the culvert over that of a thin-edge projecting inlet.

When an end section is not used nor the culvert end shielded by a barrier, the end should extend beyond the clear zone for run-off-the-road vehicle recovery.

When fill slopes are 1:2 or steeper, the culvert should extend 0.3 to 0.6 meters beyond the toe of fill to prevent the end from being buried by a sloughing embankment and so that the culvert may be easily located by maintenance crews.

When fill slopes are flatter than 1:2, the culvert may end slightly inside the toe of fill and the fill slope may be steepened around the culvert end to reduce the length of exposed pipe.

12. Plans. The highway designer is responsible for showing all details on the plans for culvert installations to include the following:

- Size.
- Location.
- Alignment.
- Maximum depth of cover.
- Estimated structure excavation.
- Acceptable materials (including any specical coatings).
- End treatment.
- Allowable alternates (if any).

In addition, the information placed on the plans for culvert pipe 1200 millimeters and larger shall include a typical cross section showing length, grade, design headwater and design discharge, and any special foundation work or end treatment.

Headwalls, energy dissipator, or riprap must be shown and standard drawings or special drawings included in the plans for their construction or placement.

a. Plan and Profile. Culvert location and alignment are shown on the plans with an arrow crossing the highway alignment, the point of the arrow being the direction of flow. The length of the arrow should be the approximate length of culvert and begin and end at the proposed culvert inlet and outlet.
(See Exhibit 9.21 in Chapter 9.)

The designer should lay out culverts on plotted cross sections in order to determine culvert lengths, grades, maximum cover, and structure excavation. It is preferable that a separate culvert roll be plotted for each project and made available to the project engineer, particularly on projects where culverts may operate under outlet control or where culverts are placed under high fills. The grade and length of culvert may significantly affect the headwater depth if these are changed during construction from what was designed; and the depth of cover, if changed during construction, may require a different wall thickness than that specified in design. The designer should provide the project engineer with data relative to all such critical culverts.

b. Drainage Summary. The designer shall complete a drainage summary sheet for the plans. Show maximum pipe cover, structure excavation, type of pipe (wall thickness, size, length, etc.), and acceptable alternate pipe materials. Other items associated with culverts are also listed on this sheet. See Exhibit 9.26 in Chapter 9 for an example of a drainage summary.

Generally, corrugated steel pipe or pipe arch, with the minimum standard size corrugations and thickness suitable for the pipe cover, are specified as a pay item on the plans, and allowable alternatives to these are listed on the drainage summary. The alternatives may include steel with larger corrugations, structural plate metal, aluminum in different sizes of corrugations, and concrete. When structural plate is required, then structural plate steel pipe or pipe arch is designated as the pay item, and aluminum or concrete alternates are allowed if these are appropriate.

The designer should use the following procedure for listing culvert quantities and alternates:

(1) Lay out culverts on plotted cross section (or pipe sections), compute structure excavation, and determine maximum cover to subgrade. These two values are listed on the drainage summary, along with the station, for each culvert. It should be noted that there are minimum depths of cover shown on the standard drawings, and these must be reflected in the culvert layouts.

(2) From the standard drawings, determine the pipe wall thickness for metal pipe (or pipe arch), for the maximum depth of cover as determined above, and for the minimum size corrugations listed. The design length of culvert is then listed on the drainage summary under the item for the respective size of culvert and wall thickness. When a project has culverts of the same size, some of which require different wall thicknesses, then more than one pay item for that size of pipe should be specified.

(3) Select alternate culverts from the standard drawings and list them on the drainage summary. For metal pipe alternates, the wall thickness is indicated under the metal kind (steel or aluminum), and the size of corrugations is shown on the standard drainage summary form. For concrete alternates, the size and reinforcing class are indicated. (For concrete culverts 450 millimeters and less in diameter, which do not cross the highway, the "non-reinforced" column is checked if the pipe cover is within the limits shown on the standard drawing.)

The designer should ensure that the proposed culvert installation will meet the selected design criteria regardless of which alternative is selected. For example, the designer should realize that concrete pipe is smoother than the corrugated metal pipe, and will pass the design discharge at a greater velocity than the corrugated metal pipe. Thus, the culvert outlet pipe may require more riprap or additional fish passage features than that required for a corrugated metal pipe. Also, if concrete pipe is allowed as an alternative, the equivalent culvert size must be shown on the drainage summary for all culverts over 600 millimeters in diameter. A smaller size concrete pipe may be able to carry the same discharge as the designed corrugated metal pipe, due to its smoother barrel.

E. Roadway Drainage. Effective drainage of the roadway is essential to maintenance of the service level of the roadway and to traffic safety. Water on the roadway slows traffic and contributes to traffic safety.

The purpose of a drainage system is to remove the storm water from the surface of the road. Design of the surface drainage system is particularly important at locations where ponding can occur. If too much spread is allowed, the basic purpose of the drainage system is defeated. The designer should adopt a width of spread and storm frequency that meets the need of the project.

On curbed and uncurbed roadways it is important to maintain a minimum longitudinal slope in order to avoid undue spread of flow on the roadway.

1. Design. The flow of water in the gutter should be restricted to a depth, and corresponding width, that will not severely obstruct or cause a hazard to traffic. This flow is a function of the quantity of water, gutter gradient, roughness of pavement where the flow is contained, and cross section shape of the flow area.

Inlets shall be placed at sufficient intervals to limit the spread to 2.5 - 3 meters from the curb or one-half of the travel way in each direction, whichever is less.

Storm sewers should generally be designed to convey the 10-year runoff without surcharge. However, the system should be checked for the 50-year runoff in situations where it would be necessary to prevent flooding of major highways, underpasses, or other depressed roadways where ponded water can only be removed through the storm sewer system. Matching pipe crown lines at junctions is preferable to matching inverts.

Storm sewers should be designed with slopes sufficient to develop a self-cleaning velocity of one meter per second when flowing full.

The computed hydraulic grade line (water surface profile), predicated on the selected design flow, shall not exceed the top of any inlets, manholes, junctions, etc., to an extent that would cause unacceptable inundation of the travel way and/or adjoining property. If the computations indicate that such would be the case, the storm sewer design shall be revised as necessary so subsequent hydraulic grade line computations will indicate that escaping flow has been reduced to acceptable levels. In high risk locations, such as underpasses or other depressed roadways, the hydraulic grade should be checked using a 50-year storm.

2. Flow in Gutters. A roadway gutter is defined as the section of roadway next to the curb which conveys water during a storm runoff event. The gutter, in this sense, would include a portion or all or a travel lane. 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.

Modification of Manning's equation is necessary for use in computing flow in triangular channels because the hydraulic radius in the equation does not adequately describe the gutter cross section. To compute gutter flow, Manning's equation is integrated for an increment of width across the section.

The resulting equation in terms of cross slope and spread on the roadway is:

$$Q = \frac{K}{n} S_x^{5/3} S^{1/2} T^{8/3}$$

Where:

- Q = flow rate (m^3/s)
- K = 0.38
- n = Mannings coefficient of roughness
- T = width of flow (spread) (m)
- S_x = cross slope (m/m)
- s = longitudinal (gutter) slope (m/m)

The above equation is applicable only to sections having a uniform cross slope. It neglects the resistance of the curb face, but this resistance is negligible from a practical point of view if the cross slope is 10 percent or less.

Spread on the pavement and flow depth at the curb are used as criteria for spacing roadway inlets. Charts in Hydraulic Engineering Circular No. 12, *Drainage of Highway Pavements* provide solutions for the equation. The charts can be used to determine either discharge (Q) where depth (d) and spread (T) are known or depth and spread where the discharge is known. Charts are also available to determine spread or flow in composite sections where the cross slope of the gutter adjacent to the curb is different from the cross slope of the roadway pavement.

3. Inlet Spacing. Inlets should be spaced so as to limit the spread on the roadway to a prescribed amount during the design storm. The allowable spread is part of the design criteria. The spacing of the inlets will be affected by the capacity of the individual inlets.

Where an inlet is located on a continuous grade, flow that is bypassing that inlet must be included in the total gutter flow contributing to the next inlet downstream unless it is carried off on a side road or otherwise intercepted.

Where an inlet is located at the bottom of a sag vertical curve (referred to as a sump or low point) all of the flow must go into the inlet unless it would be acceptable for some of the flow to overtop the curb or crown of the pavement.

To properly drain sag vertical curves, it is often desirable to place three inlets in each curve: one inlet at the low point and one flanking inlet on each side of the low point. The flanking inlets should be placed so that they will limit the spread in the low (flatter) gradient approaches to the sag point and will act in relief of the sag inlet if it should become clogged. The use of flanking inlets will also serve to reduce the deposition of sediment on the roadway.

The determination of the maximum spread on the roadway approaching the sag point should be based on the flattest grade in the section just upstream of the sag inlet. For most highways, the vertical curve has sufficient length to result in a gutter section whose effective gradient is 0.1 percent. There are cases where special treatment of the gutter gradient is provided and, in these instances, the flattest slope that will occur should be used instead of the customary 0.1 percent.

Large quantities of runoff from areas off the project that would normally enter the project from side roads should, wherever possible, be picked up on the side road before the runoff reaches the project.

Roadway inlets are, at best, inefficient devices for intercepting water. 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 along the shoulder. This reduces the amount of water that has to be picked up by the inlets and prevents mud and debris from being carried onto the pavement.

4. Inlet Types and Design. Either grate inlets, curb opening inlets, a combination of curb opening and grate inlets, or slotted drain pipe inlets may be used for intercepting runoff.

Curb opening inlets are preferred on grades 3 percent or less because of their self-cleaning ability. Grate inlets clog easily. Sometimes the use of grates will be found necessary either with or without curb opening inlets. When grates are used, hydraulically efficient and safe grates should be used in areas where pedestrian and bicycle traffic is a factor. Grate bars parallel to the direction of flow of water are more effective than bars perpendicular to the flow. If clogging is a factor, the grates should be considered only partially effective.

a. Grate Inlets. Grate inlets on a continuous grade will intercept all or nearly all of the gutter flow passing over the grate--or the frontal flow, if the grate is sufficiently long and the gutter flow velocity is low. Only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

A grate inlet in a sag location operates as a weir at shallow depths and as an orifice at greater depths. Grates of larger dimension and grates with more open area (i.e., with less space occupied by lateral and longitudinal bars) will operate as weirs to greater depths than smaller grates or grates with less open area.

Design curves for grate inlets are provided in HEC No. 12. *Drainage of Highway Pavements*.

b. Curb Inlets. The interception capacity of curb-opening inlets is largely dependent on flow depth at the curb and curb opening length. Effective flow depth at the curb and consequently, curb-opening inlet interception capacity and efficiency, is increased by the use of a gutter depression at the curb opening or a depressed gutter to increase the proportion of the total flow adjacent to the curb. (Top slab supports placed flush with the curb line can substantially reduce the interception capacity of curb openings. If intermediate top slab supports are used, they should be recessed several inches from the curb line and rounded in shape.)

Curb-opening inlets in continuous grade situations are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. (This type of inlet is preferred over grate inlets in locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.)

For determining the length, efficiency, and equivalent cross slope for curb opening inlet systems, see HEC No. 12.

A curb-opening inlet in a sag operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The weir location for a curb-opening inlet having a local depression is at the edge of the gutter. The effective weir length is the width of the locally depressed gutter section and the length of the curb opening.

c. Combination Inlets. Performance data on combined curb opening and grate inlets is limited. The capacity of a combination inlet on a continuous grade is not appreciably greater than with the grate inlet alone. Combination inlets are typically used in a sag location. The curb-opening provides a relief opening if the grate should become clogged.

d. Slotted Drain Inlets. Slotted inlets function in essentially the same manner as curb opening inlets (i.e., as weirs with flow entering from the side). Interception capacity is dependent on flow depth and inlet length. Efficiency is dependent on flow depth, inlet length, and total gutter flow.

On continuous grade, flow interception by slotted drain inlets is subjected to lateral acceleration due to the cross slope of the pavement. Research results indicate that for slotted inlets with slot widths greater than or equal to 45 millimeters, the length of slotted inlet required for total interception can be computed by the same method as for curb opening inlets.

Slotted drain inlets located in sags perform as weirs to depths of about 60 millimeters, depending on slot width and length. At depths greater than about 120 millimeters, they perform as orifices. Between these depths, flow is in a transition stage.

e. Median and Roadside Ditch Inlets. Medians and roadside ditches may be drained by grate inlets similar to those used for pavement drainage. 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 helps prevent erosion and may increase the interception capacity of the inlet marginally by acceleration of the flow.

Small dikes placed downstream of median or ditch inlets ensure complete interception of the flow. The dikes usually need not be high and should have safe slopes. The height of the dike required for complete interception on continuous grades or the depth of ponding in sag vertical curves can be computed. The effective perimeter of a grate in an open channel with a dike should be taken as 2(L+W) since one side of the grate is not adjacent to a curb.

f. Bridge Deck Inlets. Bridge decks are most effectively drained where the gradient is sufficient to convey accumulated water off the deck for interception. Such practice, of course, would be dependent on keeping accumulated spread within acceptable limits. It is frequently possible to eliminate or at least minimize deck drainage by intercepting accumulated flow upgrade of the bridge.

The principles of inlet interception on bridge decks are the same as for roadway inlets. However, specific requirements for a bridge deck drainage system usually differ from those of an approach roadway drainage system in the following respects:

- Total or near total interception is usually desirable upgrade of expansion joints.
- Deck drainage is highly susceptible to clogging.
- Inlet spacing is often restricted by bent spacing.
- Inlet sizes are often restricted by structural considerations.

The design of bridge deck drainage systems should be a combined and cooperative effort by the hydraulic and structural designers.

It should be noted that small size inlets operate as orifices at lesser depths than inlets of larger dimensions. Experiments with 100 millimeter diameter deck drains have indicated that this size range would operate as an orifice at depths of less than 30 millimeters on continuous grades. Interception capacities of small deck drains are extremely low.

Use of a safety factor should be considered in computing the interception capacity of bridge deck drains since they tend to clog. The most practical way of applying a safety factor is with inlet spacing since structural constraints would usually limit the size of inlets.

5. Catch Basins. Concrete catch basins with grated tops are used to collect water from a gutter line or ditch and drain it into a culvert or storm sewer. In curbed sections of highway where a substantial concrete curb or curb and gutter is constructed (usually in urban areas), catch basins are installed to drain the pavement. In these cases the catch basins may be part of a storm drain (or storm sewer) system, or they may be connected to individual outlet pipes. The catch basins should be spaced so that water from the design storm does not spread excessively onto the traffic lanes.

Catch basins may also be installed on highway cross pipes in narrow medians between parallel highways. The locations of catch basins and the size of their grates should be determined by the hydraulic engineer along with minimum pipe sizes and minimum grades for storm drain systems.

A catch basin may be required at the lower ends of bridges where the runoff from the bridge will be sufficient to erode the fill at the corners of the bridge. It is the designer's responsibility to provide for bridge end catch basins on the plans.

6. Storm Sewer Design. The system which carries the storm water runoff, consisting of one or more pipes connecting two or more drop inlets, is referred to as the storm sewer system. The purpose of the storm sewer system is to collect storm water runoff from the roadway and convey it to an outfall.

A properly designed storm sewer system will effectively transport water for the recurrence interval for which it was intended. The system should be self-cleaning without creating unacceptable high water velocities. Storm sewers may be designed as either open channels (where there is a free water surface) or for pressure flow conditions. When the storm sewer system is to be designed as pressure flow it should be assured that the hydraulic grade line does not exceed an elevation that may create unacceptable flooding conditions. Storm drains systems may be designed either way provided the designer is consistent in the design methods used. There will be little differences in the systems if good design practices are followed.

The basic concepts of storm drain design are as follows:

- Do not use pipe sizes less than 450-millimeter diameter except in special cases.

- To determine minimum pipe size relative to flow, use critical depth of flow (circular) with a d_c to D ratio equal to 0.8.

Where:

d_c = critical depth (m).
D = diameter (m).

- If the pipe gradient is fixed (i.e., set by physical controls), determine pipe size for full flow provided the size of the pipe selected is not less than the minimum size permitted for critical depth of flow.

- At changes in size of pipe or box, always place soffits or top inside surfaces of the two pipes at the same level rather than placing flow lines at the same level. This technique will help prevent backwater profiles from rising and upstream velocities from decreasing. Naturally this rule cannot be followed in every instance, but it should be adhered to where practical.

- Do not discharge the contents of a larger pipe or box into a smaller one even though the capacity of the smaller pipe or box may be greater due to a steep slope. Special consideration should be given to available outlets of an existing system.

- Calculate the hydraulic gradient of each storm sewer run to determine water surface elevations at critical points, such as junction structures and drop inlets.

All pipes or boxes should be designed such that velocities of flow will not be less than 600 millimeters per second at design flow or lower. For very flat flow lines, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm system should have flatter slopes than in the lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deters settling of particles due to steadily increasing flow streams.

A complete and proper hydraulic analysis should be performed to ensure that the system is efficient and cost effective. A simplistic approach to the design can result in flooding or failure of the system.

The design of storm drains is discussed in the FHWA design manual, *Design of Urban Highway Drainage*. Another good reference is *Modern Sewer Design*, by the American Iron and Steel Institute.

Inlet computations (Exhibit 7.7) and storm sewer computations (Exhibit 7.8) should be performed and recorded on the appropriate worksheets. These completed worksheets should be included in the project design file. Hydraulic grade line computations (Exhibit 7.9) will be performed by the hydraulics engineer and the completed forms are to be furnished to the designer.

F. Bridge Waterways. The hydraulic requirements of stream crossings must be recognized and considered in the analysis and design of bridges. Features which are important to the hydraulic performance of a bridge include the following:

- The approach fill alignment, skew and profile.

- Bridge location, skew and length.

- Span lengths for bent and pier location and design.

- Foundation design.

- Superstructure configuration and elevations.

The hydraulics of bridge waterways will include consideration of the total crossing including approach embankments and structures on the flood plains. The hydraulic analysis involves determining the backwater associated with each alternative profile and water opening, the effects on flow distribution and velocities, and the scour potential.

1. Design. Piers and abutments should be designed to minimize flow disruption and potential scour. Piers located in any channel should be limited to a practical minimum, and piers should not be located in the channel of small streams. Piers properly oriented with the flow do not contribute significantly to bridge backwater. The type of abutment used has little effect on the total backwater except where the flow section is severely contracted by a short bridge. Orientation of abutments is usually the same as for piers in adjacent bents.

The hydraulic analysis of stream crossings should consider the following factors:

a. Backwater. No installation should create a backwater that would significantly increase flood damage to properties upstream of the crossing. The amount of backwater permissible will vary with the site, depending upon the flood conditions existing at the specific site and the damages to upstream properties. The designer may use the 2 percent (50-year) flood with backwater limited to 300 millimeters as an aid in selecting the waterway opening and crossing profile.

b. Velocity. The waterway should be designed so the velocity of water through the structure will not damage either the highway facility or increase damages to downstream property. The acceptable average velocity should be based on the characteristics of the individual site. These characteristics include the following:

- Natural stream velocity.
- Bed materials.
- Soil types.
- Foundation materials.
- Risk considerations from backwater and scour.

c. Flow. Retain existing flow distribution to the extent practicable. This will help minimize damage to property by either excessive backwater or high local velocities and will avoid concentrating flow in areas which were not subjected to concentrated flow prior to construction.

d. Freeboard. Provide adequate freeboard at structure and design pier to pass anticipated debris. 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.

Flood magnitudes ranging from the mean annual, 2.33-year, through the 100-year flood or the flood of record, whichever is larger, shall be analyzed and the results compared to the minimum flood frequency criteria. See Table 7-2.

2. Hydrologic Analysis. In order to proceed with the hydraulic design of a structure for a waterway crossing, the designer must first estimate flood frequencies. Data needs for the hydrologic analysis are largely dependent upon the methods used to estimate flood flows. Information on flood flows, drainage basin characteristics, highwater during past floods, flood history at existing structures, channel geometry, and precipitation are commonly needed hydrologic data.

3. Bridge Versus Culvert. The waterway crossing may be of such size that the waterway opening can be provided for by either culverts or a bridge. Estimates of costs and damages associated with each will indicate which structure alternative should be selected.

a. Bridges have the following advantages:

- Less susceptibility to clogging with drift and debris.
- Waterway increases with rising water surface until water begins to submerge superstructure.
- Scour increases waterway opening.
- Widening does not usually affect hydraulic capacity.

b. Culverts have the following advantages:

- Require less structural maintenance than bridges.
- Capacity increases with stage.
- Capacity can sometimes be increased by installing improved inlets.
- Usually easier and quicker to build than bridges.
- Scour is localized, more predictable and easier to control.
- Storage can be used to reduce peak discharge.
- Grade raises and widening projects sometimes can be accommodated by extending culvert ends.

4. Hydraulic Analysis. The hydraulic design of a stream crossing is a trial and error process in which alternative waterway opening designs are tried for each profile alternative. There is a variety of information required and steps to be taken in the hydraulic analysis.

Cross sections of the stream channel and flood plains are required to establish the stage discharge relationships and conveyance. Sufficient cross sections should be obtained to provide an accurate representation of the channel and flood plains. If a stream control section (such as a constriction, confluence, or dense vegetal cover) exists downstream of the crossing site, cross sections should be computed beginning at the control section.

Extend cross sections laterally to include the total flood plain for the design and larger floods. The cross section should be normal to expected flood flow directions and not necessarily normal to the stream channel. The number of sections required is dependent upon flow conditions at the site. Guidance should be sought from the hydraulic engineer who will be responsible for the analysis.

Data on land use and ground cover should be obtained for assessing roughness characteristics in conveyance computations.

Photographs (ground or aerial) of the channel and flood plains and descriptions are necessary for use in the analysis. A site inspection by the designer may be necessary to ensure a good estimate of roughness coefficients. General characteristics helpful in making design decisions should be noted; these include soil types in the stream bed, banks, and over-bank areas and stream bed material gradation if possible. Evidence of drift and debris (size and volume), bank caving, headcuts, and other conditions which could affect abutment and pier location, orientation, and type, should be recorded. Photographs of the channel and stream bed, preferably in color, can be a valuable aid to the designer and can serve as excellent documentation of existing conditions.

Water surface elevations of the stream at the crossing must be computed, and a stage discharge curve must be developed showing how the normal water surface elevation of the stream varies with the discharge. Stage discharge relationships shall be established for a range of flood magnitudes, usually the mean annual flood through the 1 percent (100-year flood) or flood of record, whichever is larger.

Stage discharge relationships are commonly estimated by either water surface profile computations (preferred) or by single section analysis.

Single section analysis assumes the following:

- Steady uniform flow in the stream.

- Cross sections, slopes, and "n" values that are reasonably representative of the stream characteristics for the reach under study.

A more reliable method of establishing stage discharge relationships is by water surface profile computations that account for variations in cross section, roughness, and channel slope.

Stage discharge computations are discussed in Section 7.4.B. This stage discharge relationship is usually presented in a stage discharge curve. From this curve the designer can determine the elevation of the water surface for various flood discharges.

The flow distribution must be analyzed. Flow distribution refers to the proportion of the total flow in the stream that is conveyed by each of the various subsections of the cross section. The analysis of flow distribution will reveal sections where the flow rates are relatively high and sections which are relatively ineffective in conveying flow. This information is necessary to properly locate bridges or other openings on the flood plains, determine bridge lengths, locate overflow sections and approach roadways, and evaluate the need for and location of spur dikes and other protective and preventive features to be incorporated in the design.

Flow distribution usually will change with changes in stage and discharge and should be determined for the various flow rates of interest in the design of the crossing. Flow distribution is determined by converting the conveyance of each subsection to discharge. The results should be carefully examined and when possible compared with observed flows to determine whether the computed flow distribution is reasonable.

Proposed structure alternatives shall be analyzed. In cooperation with the bridge engineer, the hydraulics engineer helps select span lengths, pier type and size, abutment type, and roadway profile. The location and orientation of piers and abutments must be selected. If a proposed design does not satisfy the criteria as outlined or the existing conditions are adversely affected, another alternative shall be analyzed.

Backwater calculations shall be made for a range of discharges and for each profile and structure alternative. Backwater estimates are an aid in selecting the waterway opening and crossing profile.

There are several methods currently used to calculate backwater at bridge constrictions. Most of these methods have been computerized since the equations used are complex and often require trial and error solutions. HDS No. 1, *Hydraulics of Bridge Waterways* contains a hand procedure for calculating bridge backwater.

G. Encroachments on Flood Plains. Encroachments into streams are to be avoided wherever possible and encroachments onto flood plains shall be minimized to the fullest extent practical. See FAPG 650A relative to policies and procedures for the location and hydraulic design of highway encroachments onto flood plains. Exhibit 7.4 is a sample worksheet to assist in evaluating proposed flood plain encroachments.

Procedures for the design of encroachments on flood plains are provided in HEC No. 17, *The Design of Encroachments on Flood Plains Using Risk Analysis*. The application of risk analysis to the design of drainage structures allows the designer to select the design that will result in the least total expected cost to the public.

Additional information on procedures and guidelines applicable to flood plain encroachments may be found in a paper titled *Guidelines for the Evaluation of Highway Encroachments on Flood Plains*, published in 1982 by the Office of Engineering, FHWA, Washington, DC. Also see Chapter 3 relative to consulting and coordinating the protection of flood plains with USACE and FEMA.

H. Erosion Control. Erosion and sediment control are important considerations in the development of a highway facility. It is the policy of FLHO that highways be designed and constructed to standards that will minimize erosion and sediment damage to the highway and adjacent properties.

Both temporary and permanent erosion control measures shall be considered during the design of a highway and all necessary features incorporated into the contract plans and specifications.

Temporary control measures are those features temporarily installed for use during construction activities and upon completion of the project are generally removed and disposed. The design for temporary control measures such as silt fences, brush barriers, diversion channels, sediment traps, check dams, slope drains, berms, etc. are contained in the manual *Best Management Practices for Erosion and Sediment Control*, published in 1995 by FHWA, Federal Lands Highway Office.

Standard plans detailing the more common temporary control devices are available and should be included in every set of project plans containing construction activities that could possibly affect soil degradation or water quality.

Permanent control measures are those features installed as part of the highway to minimize scour, sedimentation, erosion, etc. during the life of the facility.

Curbs, gutters and downdrains are used for controlling drainage from a highway, especially in embankment areas under certain conditions. The use of curbs (mountable type) should be limited to drainage control along roadway embankments with highly erodible soils, along cut sections where a ditch is not to be constructed, and along narrow medians. The curb should be offset 300 to 600 millimeters outside the normal shoulder line. Curbs used in conjunction with guardrail should not project inside the face of the rail. The use of curb with guardrail should be avoided. Refer to Chapter 8, Section 8.4 for precautions in using curbs.

Road inlets are often needed to collect runoff from pavements to prevent erosion of embankment slopes or to intercept water at bridges. Inlets used for these purposes differ from other uses because:

- economies that are achieved by system design are often not possible since a series of inlets is not used;

- total or near total interception is sometimes necessary to limit bypass flow from running onto bridge deck;

- a closed storm drainage system is often not available for disposal of the intercepted flow and special methods of disposal must be provided at each inlet.

Downdrains or chutes are generally used to convey the discharge water from the inlets to the embankment toe. Buried pipe downdrains are preferable because the flow is confined, erosion along the embankment slope is minimized, interference with maintenance functions is reduced, and they are aesthetically pleasing.

Erosion at the outlets of downdrains must be controlled or minimized. At locations of minimal erosion, the designer should consider placing graded gravel or a rock at the outfall to control scour. Areas where significant scour is anticipated will require a special design with specific details included in the contract.

See HEC No. 12 for details and information relative to permanent inlets, downdrains, grates, curbs, gutters, and other similar roadway drainage designs useful in controlling erosion. Section 7.2 lists additional sources of information applicable to the design of debris control structures, riprap or slope protection installations, energy dissipator systems, ditch and/or channel linings, and similar structures.

7.5 APPROVALS

Section 7.2 discusses various laws and requirements that may involve review and approval of hydraulics and related features. Approvals may be in the form of special permits or actual PS&E approvals by cooperating or client agencies. Permit requirements, clearance procedures, and approvals are also outlined in Chapter 3.

It is important to establish contact and coordination with any involved agencies early in the project development process. This will help to identify requirements, design criteria, and permits or other approvals that will be needed. Hydrologic and hydraulic analyses and designs must be developed in a manner that will facilitate necessary approvals and project clearances. Special approval actions will vary with each project and may occur at various stages of design.

The hydraulics engineer should review all drainage designs included in the plans. This review should be performed during the early development of the project prior to finalizing the design to provide time for the designer to evaluate the hydraulic engineer's recommendations and if necessary make the appropriate modifications. The review should include a check of both the hydrologic and hydraulic computations for channels, culverts, inlets, bridge waterways, and all other drainage features in the proposed work.

Proposed bridge sites and corresponding waterways should be reviewed by the hydraulics engineer during the location phase to permit a hydrologic and hydraulic analyses evaluation before the layout is finalized. This input could benefit the designer during the development of the plans and specifications of the project's hydraulic features, such as channel alterations, slope stabilization, and temporary and permanent erosion controls.

Major revisions or modifications to a designed drainage facility or feature should be reviewed by the Hydraulics engineer.

7.6 REPORTING AND DOCUMENTATION

The reports and design computations prepared for a project are just as important as the field survey and final plans. Every effort should be made to insure that data is clear, accurate, complete, and contains all information necessary for layout and design. Copies of all reports and support documents shall be placed in the project design file.

The hydrologic analysis performed on each project may be compiled in a hydrologic report or otherwise incorporated into the design file. Document all the pertinent information on which the hydrologic decision was based.
Documents normally will contain the following:

- Relevant maps.
- Drainage area data.
- Field survey information.
- Photographs.
- Hydrologic computations.
- Flood frequency analysis.
- Stage discharge data.
- Flood history (including information from local residents who witnessed or had knowledge of unusual events).

The documentation usually consists of two parts. A data section containing the hydrologic background information and an analysis section containing the design computations. The comprehensiveness of the documentation will depend on the nature of the stream crossed or flood plain encroached upon, cost of proposed structure, and the class of highway.

The preparation of the reports on large and complex drainage designs (flood plain encroachments, etc.) is the responsibility of the hydraulics engineer. Computations and documentation of routine drainage facilities (such as pipe cross drains and ditches) are the responsibility of the designer.

A. Open Channels. Documentation on the design for the protection of stream banks and open channels shall contain the following minimum data:

- Project identification.
- Location of proposed work.
- Design discharge and frequency.
- Channel cross section and gradient.
- Type of lining (size and gradation of riprap, filter blanket, etc.) including design computations.
- Other pertinent data.

B. Culverts. Documentation on the design of culverts should contain the following minimum data:

- 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 (foundations, pH factors, water and soil resistivity, etc.).
- Hydrologic design computations.
- Hydraulic design calculations and culvert performance curves.
- Economic analysis.
- Other pertinent data.

Exhibit 7.6 is an example of a culvert design worksheet for use in recording the data.

C. Roadway Drainage. The design of a roadway drainage facility should be supported by documentation containing the minimum following information:

- Project identification.
- Location of proposed installation.
- Roadway gradient and applicable cross section.
- Design discharge and frequency.
- Manning's roughness coefficient (n).
- Type and size of inlets.
- Inlet efficiency computations.
- Data on intercepted and by-pass flows.
- Other pertinent data.

Exhibits 7.7, 7.8, and 7.9 are examples of roadway drainage worksheets for recording data and computations on inlets, storm sewer systems, and hydraulic grade lines, respectively.

D. Bridge Waterways. The bridge waterway analysis shall be documented and shall include the following:

- Project identification.
- Location of proposed bridge site.
- Drainage area map and site topography.
- Information on existing structures.
- $Q_{overtopping}$ and/or Q_{100}.
- Design flood frequency.
- Design discharge (include hydrologic method used).
- Design highwater and elevation versus discharge curve based upon natural stream cross section and natural or existing conditions (indicate how highwater was determined).
- Historical data (recorded highwater with dates, etc.).
- Cross sections including a cross section of the entire flood plain. Include Manning's *n* values for the various subareas, natural channel velocity, and backwaters created by the proposed structure.
- Velocity through the structure area below design highwater on a plane normal to stream flow at flood stage within the limits of the bridge.

Include a summary of any investigations accomplished for environmental review of flood plain impacts with the documentation. Computations supporting the design of riprap, spur dike placement, and for scour prevention at piers should also be included in the documentation.

7.7 DIVISION PROCEDURES

Reserved for Federal Lands Highway Division office use in supplementing the policy and guidelines set forth in this chapter with appropriate Division procedures and direction.

LIST OF EXHIBITS

Hydraulic Survey Data

Date of Survey: ____________________

By: ____________________

Project Identification: ____________________

Route No.: ____________________ Site Location: ____________________

State: ____________________ National Forest/Park: ____________________

County: ____________________

Nearest Community: ____________________

Plane coordinates or Lat. and Long. from a Highway Map: ____________________

____

1. **Existing Structures** *(any structure at the site, or upstream or downstream from the proposed site having a comparable drainage area)*

- Date of original construction: ____________________
- Was present bridge in place at time of extreme high water? ____________________
- Has bridge ever been washed out? ____________ Date: ____________________

  Describe what portion of bridge or approaches have been washed out: ____________________

  ____
- Elevation of maximum high water:

  Upstream side of existing structure ____________________

  Downstream side of existing structure ____________________

  ____________ m upstream of existing structure ____________________

  ____________ m downstream of existing structure ____________________

  ____________ m at other locations on the flood plain (describe) ____________________

  ____

  ____

  Date of maximum high water: ____________ Source of information: ____________________
- Other relevant information: ____________________

  ____

2. Stream Flow Data at Proposed Site

- Elevation of maximum high water of this stream at proposed location if different from data for existing site:

  ____________ m on upstream side of proposed ______________________________

  ____________ m on downstream side of proposed ______________________________

  ____________ m at other locations on the flood plain (describe) ______________________________

  ____

  Date: ______________________ Source of information ______________________________

  ____

- Elevations of highest backwater caused by another stream ______________________________

  Date ______________________ Stream Name ______________________________

  Source of information ____

- Elev. of normal water ____________ (Average) Elev. of extreme low water ____________________

  Date: ______________________

  Source of information ____

- Velocity of current at high water: ____________ m/s
- Velocity of current at normal water: ____________ m/s
- Other relevant information: ____

  ____

3. Site Conditions

- Amount and character of drift during a freshet or flood:

  ____

- Do banks or bed show scour? ____

  Description and location of scour: ____

  ____

- Describe streambed material (mud, silt, clay, sand, gravel, cobbles, boulders, soft solid rock, stratified rock, hard rock, silt sedimentation, deposition of large stones, etc.) is material loose or well compacted?

  ____

  ____

- Comment on stream ecology and wildlife habitat: ______________________________

  ____

Hydraulic Survey Data
(page 3 of 3)

4. Influence and Control of Site

- Location and condition of dams upstream or downstream that will affect high water discharge at this site: ______

  ______

- Location and description of any water-gaging stations in the immediate vicinity: ______

  ______

  ______

  Elevation ______ on gage corresponds to elevation ______ on survey datum.

- Extent to which sink-holes affect runoff, etc.: ______

  ______

- Brief description of usage of stream for navigational purposes by small boats, etc. ______

  ______

  ______

  ______

  ______

- Other relevant information: ______

  ______

5. General Remarks *(List any unusual conditions or features not covered or considered significant.)*

______

______

______

______

______

______

______

______

Hydraulic Design Check List*

*See HEC No. 19

Project Identification: ____________________

Route No. __________ **National Forest/Park:** ____________________

State: ____________________

County: ____________________ **Prepared by:** ____________________

Date: ____________________

Reference Data

Maps:	
USGS Quad. Scale Date	
USGS 1:250 000	
Other	
Local Zoning Maps	
Flood Hazard Delineation (Quad.)	
Flood Plain Delineation (HUD)	

Aerial Photos Scale Date	

Studies by External Agencies:	
USACE Flood Plain Inform. Report	
SCS Watershed Studies	
Local Watershed Studies	
USGS Gages & Studies	
TVA Flood Studies	
Interim Flood Plain Studies	
Water Resource Agency Data	
Regional Planning Data	
Forest Service	
Utility Company Plans	
State ( )	

Studies by Internal Sources:	
Bridge Inspection Reports	
Hydraulics Sect. Records	
Drainage Records	
Flood Records (High Water, Newspaper)	

Hydrology

Technical Resources:	
Highway Engineering Circular	
Research Reports	
Textbooks	
FHWA Directives	

Discharge Calculations:	
Drainage Areas	
Rational Formula	
USGS Regression Equation	
FHWA Regression Equation	
Regional Flood Analysis	
SCS Method	
Area-Discharge Curves	
Log-Person Type III Gage Rating	

High Water Elevations:	
Field Surveys	
External Sources	
Personal Reconnaissance	

Flood History:	
External Sources	
Personal Reconnaissance	
Maintenance Records	

Hydraulic Design

Calibration of High Water Data:	
Discharge and Frequency of H.W. el.	
Influences Responsible for H.W. el.	
Anal. Hydraulic Perform. of Existing Facility for Min. Flow thru 100 Yr.	
Anal. Hydraulic Perform. of Prop. Facility for Min. Flow thru 100 Yr.	

Design Appurtenances:	
Dissipators	
Rip Rap	
Erosion & Sediment Control	
Fish & Wildlife Protection	

Technical Aids:	
Design Manual	
Technical Library	

Computer Programs:	
FHWA Culvert Analysis (HY-8)	
FHWA Bridge Waterways (HY-7)	
HEC-2 Water Surface Profile	
QUICK HEC-12 Drop Inlet Design	
Log-Pearson Type III Analysis	

EXHIBIT 7.2
Hydraulic Design Check List

Project Identification: ______________________________

Route No. ____________ **National Forest/Park:** ______________________________

State: ______________________________

County: ______________________________ **Prepared by:** ______________________________

Date: ______________________________

Note: Attach site photographs with appropriate identification using separate blank pages.

Design Site Data

1. Existing Structure(s)—Provide Sketch of Culvert/Structure with Dimensions and Brief Description.

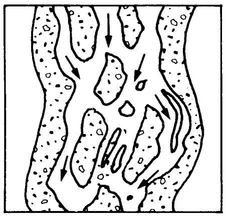
Comments: Include structure or culvert type and condition. Note particularly any scour adjacent to abutments or at culvert outlet and the presence of debris or sediment. Also note the location of any utilities in the area of the crossing.

2. High Water Marks - Describe the nature and location of any apparent high water marks and relate to a date of occurrence, if possible.

Stream Channel and Related Aspects

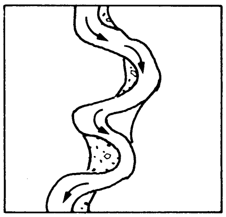
1. Stream Morphology

Braided



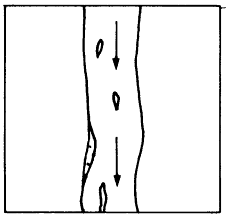
☐

Meandering



☐

Straight



☐

Is the Stream Channel - **Stable** ☐ <u>Check Appropriate Type</u>

Aggrading ☐

Degrading ☐

Comments:

____

2. Channel and Overbank Roughness Coefficients *(Check appropriate box).*

a. Basic Channel Description

☐ Channel in Earth
☐ Channel Cut into Rock
☐ Channel Fine Gravel
☐ Channel Coarse Gravel

b. Surface Irregularity of Channel.

☐ Smooth - Best obtainable section for materials involved.
☐ Minor - Slightly eroded or scoured side slopes.
☐ Moderate - Moderately sloughed or eroded side slopes.
☐ Severe - Badly sloughed banks of natural channels or badly eroded sides of man-made channels-jagged and irregular sides or bottom sections of channels in rock.

c. Variations in Shape and Size of Cross-Sections.

☐ Changes in size or shape occurring gradually.
☐ Large and small sections alternating occasionally or shape changes causing occasional shifting of main flow from side to side.
☐ Large and small sections alternating frequently or shape changes causing frequent shifting of main flow from side to side.

d. Channel Obstructions - (Judge the relative effect of obstructions - consider the degree to which the obstructions reduce the average cross-sectional area, character of obstructions, and location and spacing of obstructions). Smooth or rounded objects create less turbulence than sharp, angular objects.

The effect of obstructions is:

☐ Negligible
☐ Minor
☐ Appreciable
☐ Severe

e. Degree of Vegetation - *(Note - Amount and character of foliage).*

The effect of vegetative growth upon flow condition is:

☐ Low- Dense growths of flexible turf grasses where average depth of flow is 2 to 3 times the height of vegetation. Supple seedling tree switches where the average depth of flow is 3 to 4 times the height of vegetation.

☐ Medium - Turf grasses where the average depth of flow is 1 to 2 times the height of vegetation. Stemmy grasses, weeds, or tree seedlings (moderate cover), average depth of flow 2 to 3 times the height of vegetation. Bushy growths (moderately dense), along channel side slopes with no significant vegetation along channel bottom.

☐ High - Turf grasses where average height is about equal to the average depth of flow. Trees 8 to 10 years old with some weeds or brush. Bushy growths about 1 year old with some weeds. No significant vegetation along channel bottom.

☐ Very high - Turf grasses where average depth of flow is less than 1/2 the height of vegetation. Bushy growths about 1 year old intergrown with weeds. Dense growth of cattails along channel bottom. Tree intergrown with weeds and brush (Thick growth).

Additional Comments:

____

____

____

____

____

____

3. Requirements for Erosion Protection or Flow Control. Comment on the need (if any) for training walls, cutoff walls or special slope or channel protection.

____

____

____

____

____

Peripheral Site Data

1. Hydraulic Control - Note location and description.

____

____

____

____

____

2. Upstream and Downstream Structures - Provide sketches and brief descriptions of existing bridges/culverts. Include dimensions.

3. Maximum Allowable Headwater - Describe the nature of the apparent controlling feature and note its location.

4. Fish Passage Requirements - Comment on the apparent need for fish passage or impediments to same; such as, dams or restrictive crossings in the area.

5. Watershed Area - Check watershed boundaries for accuracy - Note current land uses within watershed.

6. Flow Control Structures Within Watershed - Note the location and type of all significant flow control structures (dams, etc.) within the watershed. Provide sketches with dimensions as required.

Risk Assessment for Encroachments

Project Identification: ____________________

Route No. __________ **National Forest/Park:** ____________________

State: __________ **County:** ____________________

General Location: ____________________

____

General Requirements

1. **Location of Crossing:** ____________________

2. **Name of Stream:** ____________________

3. **Current ADT:** __________ **Projected ADT:** __________

4. **Hydraulic Data:**

   Q_2 = __________ Stage Elevation = __________

   Q_5 = __________ Stage Elevation = __________

   Q_{10} = __________ Stage Elevation = __________

   Q_{25} = __________ Stage Elevation = __________

   Q_{50} = __________ Stage Elevation = __________

   Q_{100} = __________ Stage Elevation = __________

   $Q_{overtopping}$ = __________

   Design Flood Frequency ____________________

   Selecting Factor ____________________

5. **Site Data:**

   a) Low member elevation: ____________________

   b) Minimum roadway overflow elevation: ____________________

   c) Elevations of high risk property:

   Residences: __________

   Other Buildings: __________

   d) Elevations of prime farm land: __________

Bridge Site Analysis

1. **Backwater Damage**

   (Major flood damage in this context refers to buildings, housing developments, etc.)

   a. Is there flood damage potential to the residence(s) or other buildings during a 100 year flood?

      Yes ☐ No ☐ (Go to 2)

   b. Would this flood damage occur even if the roadway crossing was not there?

      Yes ☐ No ☐ (Go to d)

   c. Could the stream crossing be designed in such a manner so as to minimize this potential flood damage?

      Yes ☐ No ☐ (Go to 2)

   d. Analyze using Least Total Expected Cost (LTEC) design procedures. (See HEC No. 17).

2. **Traffic Related Losses**

   a. Is a practical detour available? (i.e., emergency vehicle access, evacuation route, school bus; mail delivery, etc.)

      Yes ☐ No ☐ (Go to d)

   b. Does the ADT exceed 50 vehicles per day?

      Yes ☐ No ☐ (Go to 3)

   c. Would the duration of road closure in days multiplied by the length of detour minus the length of normal route in kilometers exceed 30?

      Yes ☐ No ☐ (Go to 3)

   d. Analyze using Least Total Expected Cost (LTEC) design procedures. (See HEC No. 17.)

3. **Roadway and/or Structural Repair Costs**

   a. Is there danger of structural or roadway failure due to scour, ice, debris, or other means?

      Yes ☐ No ☐ (Go to 4)

   b. Will the cost of protecting the site from damage exceed the cost of providing additional waterway capacity?

      Yes ☐ No ☐ (Go to 4)

   c. Analyze using Least Total Expected Cost (LTEC) design procedures. (See HEC No. 17.)

4. **Capital Costs**

   a. Will the capital cost of the structure exceed $1,000,000?

      Yes ☐ No ☐ (Go to 5)

   b. Analyze using Least Total Expected Cost (LTEC) design procedures. (See HEC No. 17.)

5. If an LTEC design is not required and there are no other factors requiring further study, proceed with the design, selecting the lowest acceptable grade line and the smallest waterway opening consistent with the constraints imposed on the project.

Risk Management

- ☐ The risk assessment has demonstrated that potential flood damage costs, traffic related coats, roadway and/or structure repair costs are minor and therefore disregarded for this project.
- ☐ The risk assessment has indicated the need for further analysis in one or more of the categories examined (specify).

____

____

Risk Assessment by: ______________________

Date: ______________________

Flood Damage Report for Bridges

Location of Bridge: ______________________________

Bridge Identification: ______________________________

Highway Route No. ______________ **State:** ______________

County: ______________

Report Prepared By: ______________ **Date:** ______________

1. Extent of Damage/Repair Required

(A) Approach Roadways

- Were approaches overtopped? Yes ☐; No ☐ *(Go to B)*
- Approximately length of overtopped pavement section ________ m
- Estimated maximum depth of water over road ________ m
- Rough estimate of damage. $ ________

  Square meters of pavement to be replaced ________

  Cubic meters of roadway fill needed ________
- Comments: ______________________________

____

____

____

(B) Superstructure

- Was bridge deck damaged? Yes ☐ No ☐ *(Go to C)*

Damage was:

Minor ☐ Major ☐ Replacement required ________

- Was low bridge member inundated? Yes ☐ No ☐
- Was bridge deck inundated? Yes ☐ No ☐ (maximum depth ________)
- Effect of debris (clogging of waterway opening, etc.) on superstructure damage.

____

- Was bridge deck tied down? Yes ☐ No ☐
- Comments: ______________________________

____

____

____

____

EXHIBIT 7.5
Flood Damage Report-Bridge Sites
(page 1 of 2)

(C) Substructure

Was substructure damaged? Yes ☐ No ☐ *(Go to 2)*

	Repairs Minor	Repairs Major	Replacement Required
Abutment Damage	☐	☐	☐
Pier Damage	☐	☐	☐
Riprap Damage	☐	*☐	*☐

*Estimated cubic meters of riprap needed ____________________

Comments on extent of scour at site: ____

____

____

Effects of debris on substructure damage: ____

____

____

2. **In your opinion, what were major factor(s) contributing to bridge damage/failure.** ____________________

____

3. **Length of time water flowed over road** . ________________ **hours. How long was (or is) road expected to be closed to traffic?** ____________________

4. **Is this a good site for a more detailed study by others? Yes** ☐ **No** ☐

- Is hydrologic and hydraulic data available? Yes ☐ No ☐ Don't know ☐
- Is this a "typical" bridge crossing or is the hydraulics complicated by other features (upstream/downstream bridges, flood plain development, etc.). Typical ☐ Not Typical ☐

- Comments: ____

____

____

____

PROJECT : ______	STATION: ______	**Culvert Design Work Sheet**
______	SHEET ______ OF ______	DESIGNER/DATE: ______ / ______ REVIEWER/DATE: ______ / ______

SEE ADD'L SHTS.

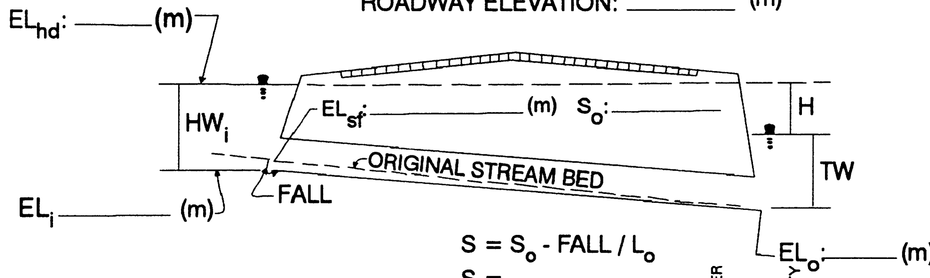
HYDROLOGICAL DATA

- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ______
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOWS (m^3/S)	TW (m)

ROADWAY ELEVATION: ______ (m)



$S = S_o$ - FALL / L_o

S = ______

L_o = ______

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (m^3/S)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	**COMMENTS**

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW_i/D = HW/D OR HW_t/D FROM DESIGN CHARTS

(3) FALL = HW_i - (EL_{hd} - EL_{sf}); FALL IS ZERO FOR CULVERTS ON GRADE

(4) EL_{hi} = HW_i + EL_i (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW OR $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (19.6n^2L)/R^{1.33}]\ V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS/DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

EXHIBIT 7.6 Sample of Culvert Design Work Sheet

(Page 1 of 2)

PROJECT: ID FH 27-1(I) Sunrise River Road	STATION: 7 + 246 SHEET 1 OF 1	**Culvert Design Work Sheet** DESIGNER/DATE: FLH / 1-1-95 REVIEWER/DATE: /

SEE ADD'L SHTS.

HYDROLOGICAL DATA

☐ METHOD: RATIONAL

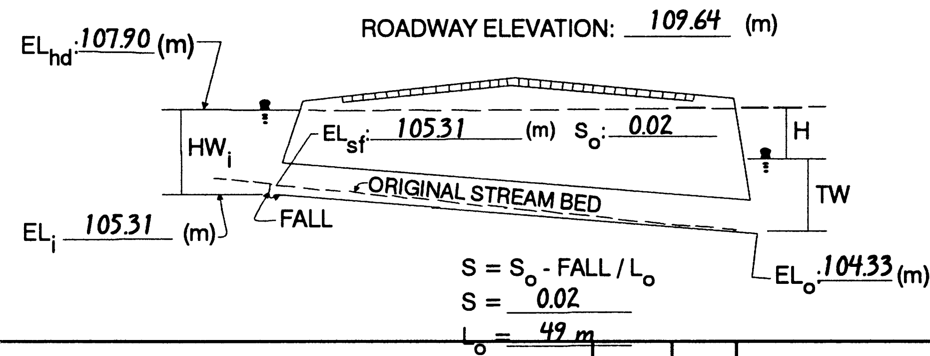
☐ DRAINAGE AREA: 21 ha ☐ STREAM SLOPE: 0.02

☐ CHANNEL SHAPE: TRAPEZOIDAL B=1.6m Z=0.6m

☐ ROUTING: ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOWS (m^3/S)	TW (m)
50	3.40	0.49



CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE		TOTAL FLOW Q (m^3/S)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)			COMMENTS
CMP 1050 mm	HEADWALL w/1:1 BEVELS	3.40	3.40	2.25	2.41	0	107.72	0.49	1.04	1.07	1.07	0.2	3.83	108.76	108.78	3.84	OUTLET CNTRL EL^{HO} > 107.90
CMP 1200 mm	HEADWALL w/1:1 BEVELS	3.40	3.40	1.5	1.83	0	107.14	0.49	0.98	1.10	1.10	0.2	1.74	107.17	107.17	3.38	OUTLET CONTROL
RCP 1050 mm	GROVED END IN HEADWALL	3.40	3.40	2.1	2.25	0	107.56	0.49	1.04	1.07	1.07	0.2	1.46	106.86	107.56	5.18	INLET CONTROL

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_t/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ OR $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (19.6n^2L)/R^{1.33}] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS/DISCUSSION:

BOTH A 1200 mm CMP WITH 1:1 BEVELS AND 1050 mm RCP WITH GROOVED END MEET DESIGN CRITERIA.

CULVERT BARREL SELECTED:

SIZE: 1200 mm

SHAPE: CIRCULAR

MATERIAL: CMP n 0.024

ENTRANCE: HEADWALL w/1:1 BVL

EXHIBIT 7.6

(Page 2 of 2)

Inlet Design

Project: ____ **Designer:** ______________________________

____ **Date:** ________________ , **19** ____________

Comments: ____

____

Drainage Area No. (1)	Inlets: Number (2)	Inlets: Location (3)	Runoff at Inlet (4)	By-pass Flow (5)	Total (6)	t_c (7)	i (8)	Q_t (9)	Gutters: S_x or S_e (10)	Gutters: S_o (11)	Gutters: d (12)	Gutters: T (13)	Inlets: Type (14)	Inlets: L (15)	Inlets: E (16)	Inlets: Q_i (17)	Q_b (18)
			CA Values			Min.	In./Hr.	CFS	Ft./Ft.	Ft./Ft.	Feet			Feet		CFS	

EXHIBIT 7.7 Inlet Design Worksheet

Instructions - Inlet Design

Summarize the computations involved in runoff and determination of inlet capacities.

Column	Description	Column	Description
1.	Identify drainage area by assigning a number, letter, etc.	16.	Efficiency of inlet (E). See HEC No. 12. Record as a decimal value to nearest hundredth.
2.	Identify inlets by a numbering system.	17.	Record flow intercepted by inlet (Q_i). Multiply value in column 16 by the value in column 9.
3.	Record location of inlet (centerline station, lt,rt,etc.)	18.	By pass flow (Q_b). Determined by subtracting the value in column 17 for the value in column 9.
4.	Compute and record CA for runoff at inlet. Multiply runoff coefficient (C) by the drainage area (A) in hectares.		
5.	Record CA for by-pass flow. Divide the value in column 18 for the previous inlet by the value in column 8 for the same previous inlet. For first inlet in a run, record zero.		
6.	Add values in columns 4 and 5 and record.		
7.	Time of concentration (t_c) for drainage area.		
8.	Rainfall intensity (i) based on t_c determined in column 7.		
9.	Total flow at inlet (Q_t). Multiply values from columns 6 and 8 and divide by 360.		
10.	Pavement cross slope (S_x) or equivalent (S_e).		
11.	Roadway grade (S_o).		
12.	Calculate depth of flow (d) at the curb for discharge. Use values from columns 9, 10, and 11.		
13.	Calculate width of spread on the pavement (T) for discharge. Use values from columns 9, 10, and 11.		
14.	Identify inlet type (grate type, curb opening, etc.)		
15.	Length of inlet (L).		

EXHIBIT 7.7 Inlet Design Worksheet

Storm Sewer Design

Project: ____________________ **Designer:** ____________________

____________________ **Date:** ____________________

Comments: ____________________

Inlet No.	Location	Inlet	Contributing Flows	Structure No.	Total	t_c			Intensity (i)	Total Discharbe Through Pipe (Q)	Pipe Design						Pipe Inverts		
						Overland Flow	Flow In Pipe	Flow Design Value			Diameter	Slope	Capacity	Velocity	Lenght	t_c In Pipe	Inlet Elevation	Total Fall	Outlet Elevation
		(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
(1)	(2)	CA Values				Minutes			mm/Hr.	m^3/s	mm	m/m	m^2/s	m/s	m	Min.	Meters		

EXHIBIT 7.8 Storm Sewer Design Worksheet

(page 1 of 2)

Instructions - Storm Sewer Design

Summarize the computations involved in sizing runs of sewer pipe.

Column	Description	Column	Description
1.	Identify inlet. Use identification number assigned to the inlet. (Column 2, on the Inlet design worksheet)	12.	Record diameter of storm sewer pipe.
2.	Record location of inlet (centerline station, lt,rt, etc.)	13.	Record slope (gradient) of pipe.
3.	Record inlet CA value. Determine CA value by dividing the value in column 17 by the value in column 8 on the inlet design worksheet.	14.	Calculate capacity of pipe using Manning's formula and record. (Note: This capacity must be approximately equal to or greater than the value for Q in column 11. If value is less, the hydraulic grade line shall be checked to determine if the HGL is lower than the gutter flow line for the run.)
4.	Determine and record CA values of flows for any contributing upstream structures.	15.	Calculate velocity of flow in the pipe using Manning's formula and record.
5.	Assign identification number to inflowing structure recorded in column 4.	16.	Record length of each run. Measure length from center to center of inlets or junctions.
6.	Add values in columns 3 and 4 and record.	17.	Record time of concentration in the pipe being evaluated. Compute the t_c by dividing the length of run (column 16) by the velocity of flow (column 15) and converting the quotient to minutes by dividing by 60.
7.	Time of concentration (t_c) for overland flow.	18.	Record inlet invert elevation.
8.	Time required for flow in the pipe.	19.	Compute and record total fall (difference in elevation between inlet and outlet inverts). Multiply value in column 16 by the value recorded in column 13.
9.	Time of concentration used for the design. For the first run, time of concentration is the inlet time for the first inlet. For all succeeding runs, time of concentration may be either the time as computed along the sewer line or the inlet time of the inlet at the beginning of the run under consideration, depending upon which of these two periods is longer.	20.	Record outlet invert elevation. Calculate by subtracting the value in column 19 from the value in column 18.
10.	Record rainfall intensity for t_c value in column 9.		
11.	Compute total discharge (Q) and record. Multiply value from column 6 by the value in column 10 and divide by 360.		

EXHIBIT 7.8 Storm Sewer Design Worksheet

Hydraulic Grade Line

Project: ____________________

____

Designer: ____________________

Date: ____________________

Comments: ____________________

Inlet Station	Outlet Water Surface Elevation	D_o	Q_o	L_o	S_{fo}	H_f	V_o	Q_i	V_i	Head Losses				H_t	HGL Elevation	Flow Line Elevation
										H_{tm}	H_e	H_j	H_b			
	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
(1)	Meters	mm	m^3/s	m	m/m	m	m/s	m^3/s	m/s	m	m	m	m	m	m	m

EXHIBIT 7.9 Hydraulic Grade Line Worksheet

(page 1 of 2)

Instructions - Hydraulic Grade Line Worksheet

The Hydraulic Grade Line (HGL) should be computed for all storm sewer systems. The purpose is to determine the elevation under design conditions to which water will rise in various inlets, manholes, junctions, etc. Computations are to be summarized on the worksheet. The HGL elevation should always be lower than the gutter flow line elevation.

Column	Description
1.	Record the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.
2.	Enter the outlet water surface elevation if the outlet will be submerged during the design storm or enter 0.8 of the pipe diameter added to the invert elevation of the outflow pipe, whichever is greater.
3.	Record the outflow pipe diameter (D_o).
4.	Record the design discharge of outflow pipe (Q_o).
5.	Record the length of outflow pipe (L_o).
6.	Compute and record the friction slope in m/m of the outflow pipe using the formula: $$S_{fo} = \left(\frac{Qn}{AR^{2/3}}\right)^2$$ Where: Q = Discharge (m^3/s) A = Cross-sectional area of pipe (m^2) R = Hydraulic Radius (m) n = Manning's roughness coefficient
7.	Compute friction loss by multiplying the value in column 5 by the value in column 6 and record. (H_f)
8.	Record flow velocity of outflow pipe. (V_o)
9.	Record design discharges (Q_1, Q_2, Q_3 . . .) for each pipe flowing into the junction. (Q_i)
10.	Record velocity (V_1, V_2, V_3 . . .) for each pipe flowing into the junction. (V_i)
11.	Compute and record the terminal junction losses (H_{tm}) for the upper reach of each storm sewer run.*
12.	Compute and record the pipe entrance losses (H_e) for the upper reach of each storm sewer run.*
13.	Compute and record junction losses (H_j) for each junction.*
14.	Compute and record bend losses (changes in direction of flow) for each inflowing pipe.*
15.	Summarize and record total head losses (H_t). Determined by adding the values in columns 7, 11, 12, 13, and 14.
16.	Record elevation of hydraulic grade line. Determined by adding the values in columns 2 and 15.
17.	Record elevation of gutter flow line.

* See Design of Urban Drainage. FHWA. 1979.

EXHIBIT 7.9 Hydraulic Grade Line Worksheet

Estimating Steel Pipe Service Life

The following procedure may be used for determining the expected service life of steel pipe. Examples using this procedure are shown below and in Part VI of FHWA-FLP-91-006. The "Scaling Tendency" portion of the procedure in FHWA-FLP-91-006 is not included in these guidelines.

At each pipe location estimate the abrasion level, and determine soil and water pH and resistivity. Using the fill height and pipe diameter needed, determine the minimum metal thickness due to structural needs from the fill-height tables (“structural” metal thickness). Use this information in the Service Life Estimation Chart (page 3 of 3) to determine the service life for the most critical (soil or water side) pH and resistivity. Note that the graphical information in the chart is for a plain galvanized steel with a metal thickness of 1.63 mm. The service life for 1.63 mm steel plate is referred to below and in the Figures as the "base" service life.

If the Service Life Estimation Chart is used, record the "base" service life (years), and adjust the service life for the required minimum fill height thickness by multiplying the base service life by the appropriate “structural” metal thickness factor from the table. This is referred to here and in the chart as the "structural" service life.

If the "structural" service life is greater than 50 years, check the pipe to determine if abrasion protection is needed. At **non-abrasive** or **low abrasive** sites, no additional protection is needed. At sites that are **moderately abrasive**, increase the "structural" steel thickness by one standard thickness (unless it is already the maximum thickness) or add paved invert protection. At **severely abrasive** sites, increase "structural" steel thickness by one standard thickness and add paved invert protection. If the steel plate thickness is already the maximum thickness, do not use steel pipe in severely abrasive environments. Otherwise, specify metallic coated steel pipe for use at this site.

If the "structural" service life determined above is less than 50 years, adjust the service life depending upon the abrasion level.

If the site has a **non-abrasive** or **low abrasive** environment, determine the added life for the larger steel thicknesses (multiply the "base" service by the factors in the table), and/or add the life for protective coatings as needed to reach the 50-year service life. Aluminum coated steel (Type 2), in the pH range (between 5 and 9), and the resistivity (greater than 1500 Ω-cm), has a service life which is the base service life multiplied by a factor of 2.0. Specify all combinations which provide a 50-year service life that are determined to be economically competitive. *If no metal thickness and protective coating combinations will achieve a life of 50 years, do not use steel pipe at this site.*

If the site has a **moderate** or **severely abrasive** environment, check the service life for the larger metal thicknesses until a service life of 50 years is achieved. *If no metal thickness will achieve a life of 50 years, do not use steel pipe at this site.* If additional metal thickness will achieve a 50 years service life, and the site is **moderately abrasive**, increase the metal thickness by one standard thickness (if the required metal thickness is less than 4.27 mm) or add invert protection. If the site is **severely abrasive**, and the required metal thickness is less than 4.27 mm, increase the metal thickness by one standard thickness and add invert protection. Specify all steel pipe thickness/protective coating combinations achieving a 50 year service life which are determined to be economically competitive.

EXHIBIT 7.10 Estimating Steel Pipe Service Life

A spreadsheet has also been developed as an alternate to the Service Life Estimation Chart. The spreadsheet uses the mathematical relationships and adds the service lives for various protective coatings. In addition to showing the possible service life combinations, the user is prompted to increase pipe thickness and/or add invert protection to provide needed abrasion protection. The spreadsheet computes the service life for each plate thickness based upon pH and resistivity and adds the service life for protective coatings. In making the service life determinations, do *not* reduce the metal thickness below the “structural” metal thickness needed for the fill height.

Examples of Steel Pipe Service Life Prediction:

Example PROJECT: NTP 1A STATION: 12+908

DATA:

Diameter:	900 mm	Fill Height:	15.0 m
Abrasion:	Level 2, Low-Abrasive		

	Water Side	Soil Side
pH:	5.5	5.5
Resistivity	18,000 Ω·cm	18,000 Ω·cm

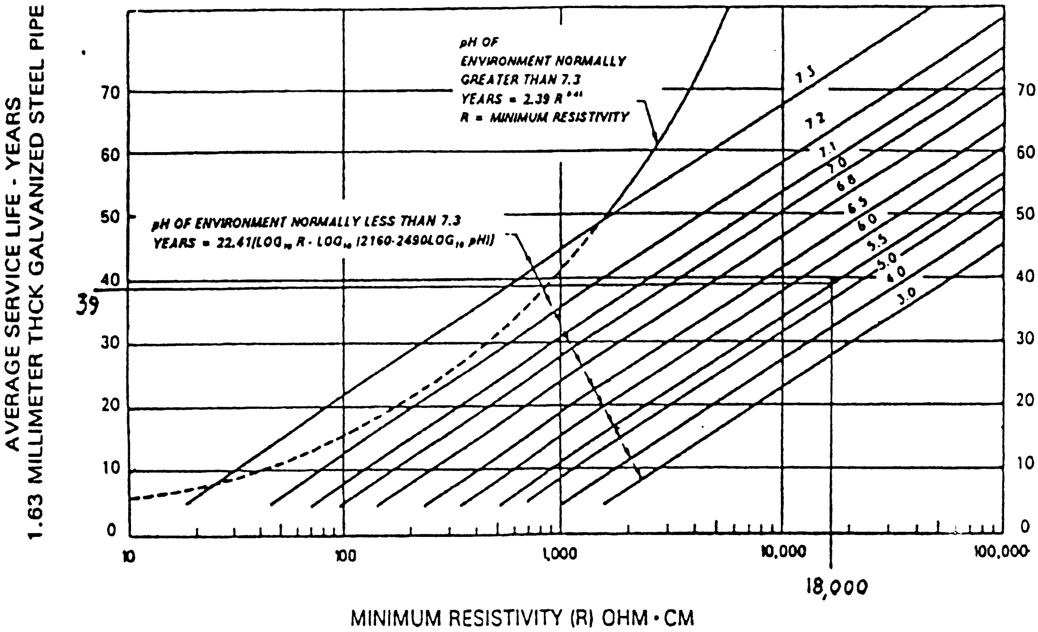
From FLH Standard Drawing M 602-1, the fill height of 15 m permits a minimum steel plate thickness for structural strength of 1.63 mm (using 68 mm x 13 mm corrugations). From the equation or graph in Exhibit 7.10 (3 of 3), the "base" service life with this pH and Resistivity is **39** years for 1.63 mm thick metal pipe. Multiply the “base” service life by the thickness factors in Exhibit 7.10 (3 of 3) to determine the "structural" unprotected (plain galvanized) metal thickness providing a minimum 50-year service life.

Abrasion levels 1 and 2 do not require adjustments for abrasion. In addition to plain galvanized pipe, the added life for protective coatings yields the following alternatives for a minimum 50-year service life:

1. 2.77 mm metal thickness for plain galvanized coated pipes - (**39** x 1.7 = 67 years)
2. 1.63 mm metal thickness for aluminum coated (Type 2) - (**39** x 1.0 x 2 = 78 years)
3. 2.01 mm metal thickness for all plain galvanized coated pipes with bituminous coating - (**39** x 1.2 + 10 = 57 years)
4. 1.63 mm metal thickness for all plain galvanized coated pipe with bituminous paved invert - (**39** + 25 = 64 years)
5. 1.63 mm metal thickness for all plain galvanzied coated pipes with concrete lining - (**39** + 25 = 64 years)
6. 1.63 mm metal thickness for all plain galvanized pipes with polymer coating - (**39** + 30 = 69 years)

All of the alternatives shown above are acceptable. The estimated service life is shown in parentheses. Note that the spread sheet shows similar values. The pipes which have an acceptable service life have been shaded on the spread sheet examples for ease of comparison.

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	2.8

* 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 ≥ 1500, the average service life determined for plain galvanized culverts should be multiplied by 2.0 for Aluminum coated steel, (Type 2).

EXHIBIT 7.10 Estimating Steel Pipe Service Life
(page 3 of 3)