CHAPTER 10
BRIDGE HYDRAULICS

10.0 INTRODUCTION TO BRIDGES

A. General - Bridges. Bridges serve a variety of highway purposes including the elimination of conflicts with traffic and other modes of transportation, such as rail, marine, air and pedestrian. Bridges enable watercourses to maintain the natural function of flow conveyance and sustain aquatic life. Bridges are also important and expensive highway-hydraulic structures and are vulnerable to failure from flood-related causes. In order to minimize the risk of failure, the hydraulic requirements of stream crossings must be recognized and considered carefully.

Features which are important to the hydraulic performance of a bridge include the approach fill alignment, skew and profile; bridge location, skew and length; span lengths; bent and pier location and design; and foundation and superstructure configuration and elevations. These features of a highway-stream crossing are usually the responsibility of location, design and bridge engineers; however, the integrity and safety of the facility are often as dependent upon competent hydraulic design as on competent structural and geometric design.

Only structures designed hydraulically as bridges are treated in this chapter, regardless of length. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the flood plains.

B. Design Requirements. Hydrologic and hydraulic analyses are required for all bridge projects over waterways that may adversely affect the floodplain even if no structural modifications are necessary. Typically, this should include an estimate of peak discharge (sometimes complete runoff hydrographs), comparing water surface profiles for existing and proposed conditions, consideration of potential stream stability problems, and consideration of scour potential. Hydrologic and hydraulic analyses generally should not be required for minor modifications such as bridge widening that does not affect the water surface profile.

10.1 PLANNING AND LOCATION CONSIDERATIONS

A. Overview of Planning and Location Considerations. Generally, a stream crossing location is selected during the planning of the corridor alignment phases of a highway project. The final location should be selected only after detailed survey information has been obtained and preliminary hydraulic studies have been completed. Although they are not the sole consideration in bridge location, hydraulic aspects should receive major attention in the initial planning of a highway alignment. The location and alignment of the highway can either magnify or eliminate hydraulic problems at the crossing.

National objectives, such as U.S. Coast Guard navigation clearance requirements for marine traffic, also must be considered in site selection. Another national objective is to reduce the rate of annual increase in flood damage losses by restricting the use of flood plains. This means that highways in the vicinity of streams must conform with the Federal Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP) requirements (E.O.11988). Additionally, the preservation of wetlands must be considered in the selection of stream crossings during preliminary design.

B. Location and Orientation. Some of the factors to consider in the selection and orientation of bridge alignments are as follows:

- The safety of the highway user.
- Vertical and horizontal highway profiles.
- Hydraulic performance.
- Construction and maintenance costs.
- Foundation conditions.
- Highway capacity.
- Navigation requirements.
- Stream regime.
• Environmental considerations.
• Flood hazard and risk.
• Structural efficiency and performance.

Obviously, the most favorable hydraulic conditions cannot always be achieved when other considerations are in conflict. Increases in additional construction and maintenance costs are sometimes necessary to achieve a satisfactory balance among competing factors including hydraulic performance. However, the selected design always should ensure that the integrity of the hydraulic design and the safety of the bridge are not compromised.

Hydraulic considerations in site selection are numerous because of the many possible flow conditions that may be encountered at the crossing and because of the many water-related environmental factors. Flow may be in an incised stream channel, or the stream may have floodplains which are several kilometers (miles) wide. Floodplains may be clear or heavily vegetated, symmetrical about the stream channel or highly eccentric, clearly defined by natural topography or man-made levees, or indeterminate. Flow may be uniformly distributed across the floodplains or concentrated in swales in the overbank areas. Flow direction often varies with the return period of the flow so that a bridge substructure oriented for one flow would be incorrectly oriented for another. Flow direction in overbank areas is often unrelated to that in the main or low flow channel. In some instances, the floodplains convey a large proportion of the total flow during extreme floods and the stream channel conveys only a small proportion.

Not all of the above will apply to each stream crossing or bridge location, but many of the most important site considerations are hydraulic or water-related. Crossing location alternatives often do not include the most desirable site from the hydraulic design viewpoint, but the difficulties involved often can be reduced by careful hydraulic analysis.

Incorporation of roadway approaches which will accommodate overflow may be necessary for some configurations. Such overflow reduces the threat to the bridge structure itself. Of course, the flow of traffic is interrupted, and the potential costs associated with such interruption and potential damage to the roadway embankment and bridge integrity should be considered by the designer.

C. Structure Type. The final selection of structure type should not be made prior to the completion of detailed surveys and necessary scour, hydrologic and hydraulic studies. Where final structure-type selection is necessary to satisfy the requirements of the environmental assessment, public hearings or right-of-way acquisition, all available information should be furnished to the engineer and the decision made in consideration of all factors which influence the selection. Selection of the final type, size, and location of structures is at the sole discretion of the engineer.

D. Environmental Considerations. This chapter does not include a definitive discussion of the environmental considerations in site selection and bridge design. Because of the many and complex considerations involved, discussion here is limited to a broad approach regarding the environmental concerns that must be addressed in the selection and design of a stream crossing. A Field Checklist for the Preliminary Design Permit Coordination was developed by a joint taskforce of PennDOT, PA DEP, and PFBC is included in Appendix 10A, Field Checklist for the Preliminary Design Permit Coordination. The use of the checklist is highly encouraged for PennDOT projects. The checklist was developed to facilitate early coordination and documentation for projects to help streamline and expedite the permitting process.

The environmental effects of construction activities may be classified as the hydrologic and hydraulic, physical, chemical, cultural, historical, archeological, aesthetic and biological aspects.

Hydrology at a crossing is unlikely to be a factor in site selection, but the hydraulics may be an overriding concern. Hydraulic considerations include the effects of the crossing on velocities, water surface profiles, velocity and flow distribution, scour, bank stability, sediment transport, aggradation and degradation of the channel, and the supply of sediment to the stream or water body. The engineer must evaluate the potential effect of the crossing on these factors as well as the potential effects of the crossing site on the environment.

Effects of a highway on the chemical quality of surface waters are not ordinarily a major consideration in site selection, though it is possible that contaminants in the form of minerals or from a sanitary landfill would be exposed in one location and not at an alternate site. There is some concern for chemical quality at crossing sites, particularly near public water supply intakes, due to the risk of toxic material spills. The probability of such spills
should be considered in weighing factors that influence site selection. The use of deicing salts, fertilizers, growth inhibitors and other chemicals usually would not vary appreciably among alternate sites, but the adverse effects of normal usage of such materials may vary among alternatives and, therefore, may be a factor in site selection. A formal evaluation of these issues will occur during the Water Quality Certification under Section 401 of the Clean Water Act.

Aesthetic considerations include effects on the visual, odor and taste qualities of surface waters. The aesthetic quality of surface waters should be considered in site selection where potable water supplies, water contact sports and fisheries are involved. The visual quality most often affected by highways is temporary turbidity during construction.

Biological considerations in site selection include the effects on habitat and ecosystems in the floodplain, and aquatic ecosystems in the stream and associated wetlands. It is advisable that biologists or environmental managers assess this aspect of site selection, but much of the information necessary for a valid assessment of the biological effects and the alternatives available for mitigation must come from the designer. This data evaluation includes the following:

- Alternatives to avoid, minimize, or mitigate impact to wetlands.
- Alternatives to avoid, minimize, or mitigate impacts to cultural resources.
- Alternatives to avoid or minimize encroaching on or crossing streams.
- Effects of circulation of fresh or brackish water in marshes and estuaries.
- Feasibility of providing mitigating measures for the loss of invertebrate population.
- Effects on shaded areas and resting areas for fish.

E. Stream Characteristics. All streams change with time, and the rate and manner in which they will change can be estimated. Planning engineers should be very conscious of stream morphology and be aware that methods are available for quantifying natural changes and changes that can occur as the result of stream encroachments and crossings. Chapter 8, Open Channels, includes an introduction to the concepts of stream morphology. Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels (AASHTO, 2000d), contains more comprehensive discussion and is recommended as a convenient summary of stream response to highway construction. HDS-6, River Engineering for Highway Environment, (FHWA, 2001c) is a document on river mechanics, containing valuable material on the qualitative and quantitative analysis of river system response to natural and construction-induced change. The designer should attempt to ensure that highway work within a stream environment does not induce significant change in the stream morphology.

F. Replacement, Repair and Rehabilitation. The decision to replace, repair, or rehabilitate a bridge often is made in the planning and location phase of highway project development. Bridges may be replaced for a number of reasons, including the following:

- Functional obsolescence.
- Structural inadequacies or deterioration.
- Structural damage from collision.
- Alignment and geometric inadequacies.
- Flood-related damage, damage due to scour and debris impact.
- Inadequate clearances for navigation.
- Plans for water resources projects.
- Flood control.
- Inadequate hydraulic capacity.

The hydraulic adequacy of an existing crossing should be examined critically before a decision is made to replace the bridge in-kind, widen or undertake major rehabilitation. The purpose of the examination is to determine if the existing crossing will provide adequately for changed traffic service requirements, to reevaluate flood hazards and risks, and to reexamine the hydrology and hydraulics. Flood experience at an existing crossing is extremely important in analyzing the hydraulic adequacy of stream crossings. The performance of existing crossings during floods is valuable input to the analysis, and it constitutes information that usually is not available at new crossings.
Some problems experienced at a crossing may be due to the occurrence of rare floods rather than the hydraulic inadequacy of the existing crossing. On the other hand, a stream crossing that has served well over a long period of time does not assure its hydraulic adequacy. The odds are 2-to-1 that a 20-year-old bridge has not experienced a 2% exceedance probability flood event and over 4-to-1 that a 1% exceedance probability flood has not occurred during the existence of the bridge. Hydrologic changes may be due to flood control by reservoirs, channelization or levees, development in the watershed which increases the runoff and peak flows, changes in land use practices, and other causes. The hydraulics of the stream may have been changed by channelization, levees, development in the floodplain, different land use in the floodplain, changes in the stream regime, and other causes.

The stability of the stream itself can change with time from natural or construction-induced causes. A stream which was relatively stable at the time a bridge was constructed may be highly unstable when the bridge needs to be rehabilitated or reconstructed. Aggradation, degradation, or lateral instability should be considered when a decision is made to rehabilitate or reconstruct a bridge.

Methods used to analyze the hydrology and hydraulics at bridge sites continue to improve. The adequacy of the analysis for the original crossing design should be examined before undertaking major reconstruction or replacement. In many cases the method used in the original analysis is no longer an appropriate method. The analysis for these crossings should be recalculated using an appropriate method. Additionally, the risk of failure of the existing structure should be reconsidered, including the following:

- Increased traffic volumes.
- Changed traffic service requirements.
- Increased highway construction and maintenance costs.
- Liability for damages to property that could be attributed to the highway crossing.

10.2 COORDINATION WITH OTHER AGENCIES

A. General - Coordination with Other Agencies. Numerous local, state, and federal agencies have vested interests in surface waters. These agencies represent interests in the following areas:

- Water rights.
- Flood control.
- Drainage.
- Natural resource conservation.
- Navigation and maintenance of channels.
- Recreation.
- Flood plain management.
- Safety of flood plain occupancy.
- Aquatic habitat for fish and wildlife.
- Preservation of wetlands.
- Regulation of construction for the protection of environmental values.
- Gaging stations and apparatus for environmental data collection.

Other local, state, and federal agencies have vested interests in historic and archaeological preservation of cultural resources which include an interest in historic bridge structures and archaeological resources. Early coordination with these agencies will reveal areas of mutual interest and offer opportunities to conserve public funds and to resolve conflicts between PennDOT's plans, plans for water resource development, and requirements for resource protection and preservation.

The following is a list of agencies commonly involved with bridge planning and location:

- U.S. Army Corps of Engineers (USACE).
- Conservation Districts.
- River Basin Commissions.
- Pennsylvania Department of Environmental Protection (PA DEP).
B. Water Resource Development Projects. Water resource development projects often require the relocation or reconstruction of existing highways and can interfere with the location or design of proposed highway-stream crossings. Many water resource development projects are planned or authorized for periods of years or even decades before construction begins. Others are never built and may even be permanently stopped by court decisions or regulatory agency actions.

Where stream crossing locations are chosen to take advantage of, or to accommodate, planned water resource development projects (such as reservoirs or stream channel modifications), it should be recognized that the water resource agency's plans may never come to fruition and that the highway facility must be designed for both existing and future site conditions. Planning and constructing a highway facility at a future water resource project site must be studied carefully. The excess cost of building the facility to accommodate a planned water resource project must be considered in selecting the stream crossing site. Some alternatives available to the department include:

- Cost-sharing with the water resource agency.
- Constructing the highway and stream crossing without consideration of the planned water resource development project.
- Choosing an alternate location, if practicable, which would not be impacted by the planned water resource development project.

C. FEMA Designated Flood Plains. Many highway bridge crossings involve flood plains in the NFIP administrated by FEMA. FEMA criteria often influence the design of a bridge over a waterway; therefore, it is important that FEMA requirements are considered in the planning phase of a project and accommodated in the design. Early coordination with the community's NFIP administrator is essential to identify and avert potential problems.

10.3 DATA COLLECTION

For purpose of this section, site information is broadly classified as data collection. Sources of data include aerial and field surveys; interviews; water resources, fish and wildlife, and planning agencies; newspapers; and flood hazard delineation studies. Complete and accurate survey information is necessary to design a crossing which will meet the requirements of the site. The individual in charge of the field hydraulics survey should have a general knowledge of drainage design and coordinate data collection with the hydraulics engineer. The amount of survey data collected and the detail of the data should be commensurate with the complexity of the hydraulics, the severity of stream stability problems, the importance and cost of the structure, and the risk of damage to the highway and of causing damage to other properties and values.

The bridge hydraulics data collection in this section should be coordinated with data collection efforts discussed in Chapter 6, Data Collection.

A. Topographic Features. The survey data collected should provide sufficient information for structural and hydraulic engineers to select the location of the crossing, make trial layouts, and conduct hydraulic studies. All significant physical features and cultural resources in the vicinity of the proposed crossing site should be identified and located, particularly those features which could be adversely affected. Features such as residences, commercial and industrial establishments, croplands, wetlands, roadways, railroads, utilities, wells and other facilities can influence design. Their locations and elevations should be established by the survey.
The extent of survey coverage required for the hydraulic design of a highway-stream crossing is related to the topography and stream slope. Backwater above bridges may extend a considerable distance upstream in streams with relatively flat slopes, and features which may be affected by the backwater should be located and identified.

B. Land Use and Development Resources. Present and future land use and cultural resources in the vicinity of a stream affect both the hydraulics of the stream and the design of the highway-stream crossing. Flow distribution, velocities, and the stage-discharge relationships are influenced by development and land use. The potential for floating debris (such as lumber, house trailers or debris from timbering operations) is largely dependent on land use in the watershed and development in the floodplains.

One of the objectives in the location and design of highway-stream crossings is the avoidance of damage to private property from highway-caused flooding. The practicability of avoidance generally can be evaluated on the basis of the probability of the highway causing incremental flood damage, the economic cost of the probable damage, and the cost to avoid and/or mitigate the damage should be considered. Information on land use, such as crops, parks and recreational development, and the elevation, use and value of structures that may be affected should be collected.

C. Hydrologic Data. Data requirements for hydrologic analyses are largely dependent upon the methods used to estimate flood flows. Commonly needed hydrological data include information on flood flows, drainage basin characteristics, highwater during past floods, flood history at existing structures, channel geometry, and precipitation. A more detailed and complete discussion of data needs for hydrologic analysis are contained in Chapter 7, Hydrology.

D. Flood Data. Flood flow data and streamflow records are available for many locations on streams in the US. Data collected by the USGS and other agencies are published periodically in the surface water records which are available at local offices of the USGS. Data also are available from other sources, such as universities and local and state governmental agencies. Railroad and state highway maintenance files often contain valuable information on flood stages. Newspaper and magazine accounts of floods may contain information from which stages and water surface profiles can be reconstructed. Flood marks and other positive evidence of unusual flood events are valuable data, especially where no gaging station records are available or records are short. The USGS publishes open file reports which document unusual flood events. Flood hazard reports and flood insurance studies compiled for FEMA may contain information on floods that have 10-, 50-, 100-, and 500-year return periods.

The USGS has established a service, the National Water Information System (NWIS), to provide assistance in identifying, locating, and acquiring available water data through a national network of local assistance centers. These centers provide access by computer terminal to information about the availability of streamflow data, groundwater levels, sediment discharges, and the quality of surface and groundwaters from stations operated by more than 300 agencies. Information regarding NWIS can be obtained from local offices of the USGS.

Records should be examined for inconsistencies and evidence of changes in the stream hydrology. Discrepancies in records obtained from different sources may be the result of inadequate accuracy standards of one source. Changes in the hydrology of a stream may be the result of changes in land use or urbanization in the watershed, channelization, flood control, or dam projects. A chronological plot of annual maximum flood peaks may reveal trends in the hydrology of a stream that would not otherwise be detected. Records which show some changes in the stream hydrology are said to lack stationarity. Stationarity should exist in the data used for the hydrologic analysis, and conditions should be reasonably representative of existing or present watershed conditions. For assistance with evaluating the stationarity of a stream gage record, contact the Pennsylvania District of the USGS.

A considerable period of time usually elapses between the conception of a highway project and its construction. During this period, important information can be collected if flood events occur. Correlation of stage at the bridge site with a stream gaging station upstream or downstream will help to verify the stage-discharge relationship by providing one known point on a curve that is otherwise based entirely on computations. If there is no gaging station on the stream, stage and discharge data should be collected to aid in both the hydrologic and hydraulic analyses. This requires preparation in advance so that resources can be quickly mobilized when a flood occurs. An agreement can be made with the USGS, or another similarly qualified agency, to collect flood data at bridge sites. This data often can be used to improve designs.
E. Highwater Information. Reliable highwater data can provide invaluable information for establishing the stage and discharge of past floods, for locating existing hydraulic controls, and for establishing highway profiles. Obtaining more than one highwater mark and its exact location for the same flood event is recommended. Several dependable highwater marks are required to compute flood discharge by the slope-area method.

It is extremely important that experienced personnel be used to identify and evaluate highwater marks because the apparent evidence of highwater can be deceiving to the uninitiated. Highwater marks should be flagged and surveyed as soon as practicable after a flood because they may disappear within weeks in heavily vegetated areas. If an unusual flood has not occurred for several years, the highwater marks located by even experienced personnel are likely to represent a relatively small flood. Highwater stages may be misleading since they are sometimes caused by ice, log jams, confluences, or land use which has subsequently disappeared. Such stages may be on the order of a meter (3 feet) off the normal stage for the same discharge. Examination of aerial photographs taken during the flood, or more than one indirect measurement taken at reaches some distance apart, can assist in identifying these stages as abnormal.

Information on highwater elevations can be obtained by observing debris and mud lines on tree trunks and bridge abutments, wash-lines and fine-debris lines on banks and bridge approach fills, wisps of grass or hay lodged in tree limbs and fences, and evidence of erosion and scour. Interviews with residents, commercial and school bus drivers, mail carriers, law enforcement officers, highway and railroad maintenance personnel, and others who might have an opportunity to observe unusual floods may yield additional information. The date of the flood occurrence, the name and address of the observer, and the stage and location of the observation should be recorded. The observed frequency of occurrence should be noted since reliable information that a stream reaches a certain elevation every two or three years provides important frequency information for the designer. A few hours spent in interviewing several people who are familiar with the flood history of a stream can result in substantial savings in construction, liability, and future maintenance, and can lead to improvements in the design.

Important highwater information includes:

- Major flood events since construction and dates of occurrence.
- Flood heights upstream and downstream of the bridge (flood stages within bridge openings generally provide little useful information).
- Observed differences in water surface elevations upstream and downstream of the structure at as many locations as the information is available; dates and flood magnitudes should be included, as available.
- Observation on flow distribution, overbank flow, flow directions, and velocities.
- Direction of flow relative to piers and relative to the low-water channel.
- Observed drift size and quantities.
- Clearance and freeboard.
- Duration of flooding.
- Damage to the highway, slope protection, stream control measures, bridge and other property.
- Magnitude of flood relative to other notable floods.
- Photographs of the structure, flood events, stream and any other feature that will aid in the design of the proposed bridge.
- Bridge design details including deck profile, superstructure design, pier design and orientation, and bridge rail design.

F. Existing Structures. Structures in the vicinity of a proposed bridge may have experienced unusual floods or floods which were sufficiently large to provide useful information. Information on floods which have occurred since the construction of bridges may be obtained from highway maintenance files, from residents of the area, and from highwater marks.

Data at existing structures should include the highwater information noted above and as much of the following as is available or is practical to obtain:

- Date of construction.
- Location relative to proposed structure.
- Hydrologic and hydraulic design data, assumptions, and calculations.
- Cross section under bridge from as-built plans.
• Present cross section under bridge.
• Type and size of materials in stream bed and banks.
• Condition of structure.
• Sediment deposits, scour and erosion.
• Evidence of headcutting in stream.
• Roadway profile extending to the extremities of the floodplain.

G. Channel Characteristics. Survey data are required to analyze the streamflow characteristics and stream morphology at bridge sites. In both analyses, aerial photographs are useful in identifying types of vegetation, sizes and locations of sandbars, thalwegs, stream controls, geological formations, existing stream bank protective works, and old meander channels. A series of aerial photographs taken over a period of years can be used to determine the pattern and estimate the rate of movement of meander bends. Aerial photographs also can be used to determine whether the stream is straight, meandering or braided, and to detect evidence of stream degradation or aggradation.

Cross sections of the stream channel and floodplains are required to establish the stage-discharge relationship and conveyance. Sufficient cross sections should be obtained to provide an accurate representation of the channel and floodplains. If a stream control section, such as constriction or confluence of dense vegetal cover, exists downstream of the crossing site, cross sections should be obtained so that a water surface profile can be computed beginning at the control section. Cross sections should be extended laterally to include the total floodplain for the design and larger floods. The cross sections 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.

Data on land use, vegetative cover, and stream bed material should be obtained to assess roughness characteristics for use in conveyance computations. Photographs of the channel and floodplains and descriptions are necessary for use in the analysis, and a site inspection by the designer may be necessary to ensure a good estimate of roughness coefficients.

Other characteristics necessary to make design decisions should be noted. These include soil types in the stream bed, banks and overbank areas, and stream bed material gradation, if possible. Features such as rock outcrops and meander plugs, or deposits of cohesive materials in old channel bendways due to a cutoff should be noted (AASHTO, 2000b). Evidence of drift and debris size and volume, ice conditions, bank caving, waterfalls, headcuts, and other conditions which would affect abutment and pier location, orientation, and type should be recorded. Photographs of the channel and stream bed, preferably in color, can be valuable aids to the designer and serve as excellent documentation of existing conditions.

H. Environmental Data. The need for environmental data in the engineering analysis of a stream crossing stems from the obligation to investigate possible impacts due to specific design configurations. In those cases where an environmental assessment has been completed earlier in project development, part or all of this evaluation already may have been accomplished. Where an environmental assessment has not been made, the data developed for planning and location of the crossing is often of value in the engineering-environmental analysis.

The engineer and environmental specialist working as a team need information on water quality standards for the stream. Some of this information is available in the water quality standards and criteria published in Chapter 93, Water Quality Standards by the PA DEP. Physical, chemical and biological data for many streams also are available from state and federal water pollution control agencies, USGS, and from municipalities, water districts and industries which use the stream as a supply source.

A description of existing water circulation patterns and definition of the types and extent of potentially affected wetlands are necessary for the team to assess the effects of each bridge-fill configuration. Data on circulation, tides, water velocity, water quality, and wetlands may be available from the USGS, U.S. Fish and Wildlife Service, the USACE, universities, and marine institutes, as well as other state, federal and local agencies and organizations.

Information on fish and fish habitat often is necessary in order to evaluate proposed channel modifications and to design replacement habitat. Fish and fish habitat information is available from state and federal fish and game agencies, such as the Pennsylvania Fish and Boat Commission, the U.S. Fish and Wildlife Service, and the Pennsylvania Game Commission. An analysis of the material in the stream bed and banks, as well as proposed fill materials, may provide essential data for projects in critical water-use areas, such as near municipal or industrial water supply intakes.
It may be necessary for the highway agency to collect data at critical sites, if the required information is unavailable from other sources.

Data needs may be summarized as follows:

- Information necessary to define the environmental sensitivity of the crossing; e.g., water use, water quality and wetlands information.
- Information necessary to determine the most practicable, environmentally compatible design; e.g., circulation patterns and sediment transport data.
- Information necessary to define the need for mitigation measures; e.g., fish habitat, sediment analysis, water use, and quality standards, and to design these measures if necessary.

1. **Site Plan.** A general site plan should be prepared to show the physical features of the site. Although a contour map of a larger area generally is required for these purposes, a contour map of the site is helpful in defining flow directions, and in making decisions on span lengths and abutment and pier locations and orientation. The site plan should be prepared as noted in Publication 13M, Design Manual, Part 2, *Highway Design*, Section 10.7.

J. **Field Reviews.** Field reviews are highly desirable in order for the designer to become familiar with the site. The most complete survey data cannot adequately depict all site conditions or substitute for personal inspection. A field review also is useful in confirming that additional site data are necessary. The selection of roughness coefficients, the evaluation of apparent flow directions and concentrations, and first-hand observation of land use and floodplain development are the factors that most often need to be confirmed by field inspection. Consultation with construction and maintenance engineers regarding site conditions often will provide information regarding factors that they have found to be important to their responsibilities.

**10.4 BRIDGE HYDRAULIC CONSIDERATIONS**

A. **General - Bridge Hydraulic Considerations.** Design criteria are the real means for placing accepted policies into action. These policies become the basis for the selection of the final design configuration of the stream-crossing system. After establishing the flood frequency and the stage-discharge curve according to the principles described in Chapter 7, *Hydrology* and Chapter 8, *Open Channels*, the type of cross-drainage facility can be chosen. Usually this choice is between a bridge or a culvert.

In some cases the choice may not be especially clear. It may be useful to evaluate both types of facilities and to make a choice based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be necessary. The hydraulic analysis of a highway stream crossing for a particular flood frequency involves the following general considerations related to the hydraulic analyses for the location and design of bridges:

- Backwater associated with each alternative vertical profile and waterway opening should not significantly increase flood damage to property upstream of the crossing.
- Effects on flow distribution and velocities; the velocities through the structure(s) should not damage either the highway facility or increase damages to adjacent property.
- Existing flow distribution should be maintained to the extent practicable.
- Pier spacing and orientation, and abutment designed to minimize flow disruption and potential scour.
- Foundation design and/or scour countermeasures to avoid failure by scour.
- Freeboard at structure(s) designed to pass anticipated debris and ice.
- Risks of damage.
- Stream instability countermeasures.
- Minimal disruption of ecosystems and values unique to the floodplain and stream.
- Highway level of service compatible with that commonly expected for the class of highway.
- Design choices should support costs for construction, maintenance and operation, including probable repair and reconstruction and potential liability that are affordable.
B. Design Storm. The flood-frequency analysis for a stream crossing is discussed in Chapter 7, *Hydrology*. For purposes of this section, the hydrologic analysis consists of establishing peak flow-frequency relationships for the crossing and flow-duration hydrographs as necessary for the site analysis. Stage-discharge relationships are a part of the hydraulic analysis, and the hydraulic analysis for design includes an evaluation of the effects of the highway crossing for a range of flow rates.

The flood selected for design may differ from the flood of a preselected exceedance probability. Relatively rare floods (i.e., floods larger than the design flood) may need to be considered in the hydraulic analysis. For these reasons, and because of uncertainties in the hydrologic analysis, the hydrologic analysis should define peak flow-frequency relationships for a wide range of events for use in the subsequent hydraulic analysis of the crossing. The hydraulic analysis may need to consider the ordinary highwater and the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year flood events. It is easier to develop the discharges for all of these at one time during the hydrologic phase of the analysis.

C. Flow Near Bridges. When flood flow encounters a restriction in the natural stream, adjustments take place in the flow regime in the vicinity of the restriction. Flow is contracted through the bridge and then must expand as it exits the bridge. An exchange of energy between potential energy and kinetic energy is required to maintain the contraction and expansion of flow. In addition, energy is required to overcome friction and disturbances associated with piers and abutments. This exchange of energy is reflected by an increase in the depth of flow upstream of the encroachment, termed backwater, as shown in Figure 10.1.

In subcritical flow conditions, the backwater tails off upstream until the undisturbed water surface elevation is reached. The distance upstream over which backwater occurs is dependent on the channel conditions and flow conditions, see Chapter 8, *Open Channels*. The maximum backwater tends to occur a short distance upstream of the bridge, as shown in Figure 10.1. The relatively steep water surface gradient between the maximum backwater and the opening is termed the drawdown area.

In a stream channel with supercritical flow conditions, it is possible to have a constriction, such as a bridge, that does not affect the upstream flow conditions. However, if the constriction is severe enough, it could cause a change in flow regime such that a subcritical flow backwater occurs upstream of the bridge with a hydraulic jump forming upstream of the bridge at the transition from supercritical to subcritical flow.

Figure 10.1 Backwater at a Stream Crossing

As subcritical flow moves toward the bridge opening, the velocity increases as the flow lines begin to converge and become constricted between the bridge abutments, see Figure 10.2. This increase in velocity can result in scour along the embankment and through the bridge. At the bridge abutments, the high velocities can cause severe
turbulence and eddies. Piers in the waterway create additional local turbulence and vortices. Turbulence, eddying, and vortices often result in scour.

The hydraulic design of a bridge over a waterway involves establishing a location, bridge length, orientation, roadway profile, and bridge profile such that the risks associated with backwater and increased velocities are not excessive.

D. Allowable Backwater Due to Bridges. For design storm conditions, the allowable backwater should be established by the designer based on consideration of design policies, regulatory requirements, and the risk associated with potential flood-related damage to the highway and adjacent properties.

Figure 10.2 Typical Flow Directions through Bridge Opening
Analysis of the backwater associated with the 100-year event must conform with FEMA's NFIP requirements, where applicable. Other considerations include (a) the Federal Aid Policy Guide 23 CFR 650, (b) 25 PA Code § 105 and § 106, and (c) PennDOT policy regarding mitigation of occasional inundation. Generally, if the risk associated with the backwater is deemed to be excessive, the designer should consider one or more of the following actions:

- Consider other design alternatives to reduce the backwater.
- Perform a detailed economic analysis of various design options.
- Design channel improvements.
- Determine the incremental area of high risk that will be inundated and arrange purchase of flood easements or acquisition of the affected property.

E. Flow Distribution. Flow distribution is the proportion of the total flow in the stream that is conveyed by each of the various portions of the cross section. Velocity distribution is the average velocity in each of the subsections and may not be indicative of locally high or low velocities. Hydraulic computations; however, are usually sufficiently accurate for design purposes. Figure 10.3 shows the flow distribution for the main channel and floodplains of a stream at a given discharge. Flow distribution usually will change with changes in stage and discharge and should be estimated for the various flow rates of interest in the design of the crossing.

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 floodplains, determine bridge lengths, locate overflow sections in approach roadways, and to evaluate the need for and location of spur dikes and other protective and preventive features to be incorporated into the design.

Flow distribution will be disturbed during some floods by any stream crossing that uses a combination of fill and bridge within the flood plain; however, flow distribution should be preserved to the extent practicable in order to:

- Avoid disruption of the stream-side environment.
- Preserve local drainage patterns.
- Minimize damage to property by either excessive backwater or high local velocities.
- Avoid concentrating flow in areas that were not subjected to concentrated flow prior to construction of the highway facility.
- Avoid diversions for long distances along the roadway embankment.

Flow distribution is determined by converting the conveyance of each subsection to discharge. The results should be examined carefully and, when possible, compared with observed floods to determine whether the computed flow
distribution is reasonable. There may be topographic features, vegetative cover, development or other physical features upstream or downstream of the cross section which would make the computations invalid.

Generally, the disturbance of flow distribution can be minimized by establishing bridge openings at the areas of high conveyance. For many situations, one-dimensional analysis techniques will suffice for determining optimum bridge locations. Complex sites, such as those at a bend, and skewed crossings can be analyzed with one-dimensional models only by using a great deal of intuition, experience, and engineering judgment to supplement the quantitative analysis. Unfortunately, complex sites are encountered frequently in stream crossing design. The development of two-dimensional methods of analysis greatly enhances the capabilities of hydraulic engineers to deal with these complex sites.

F. Bridge Scour and Stream Degradation. A scour analysis is required for all new bridges, replacements, and widenings. Scour analysis procedures are discussed in Section 10.9.

If a scour analysis indicates high depths of potential contraction scour, it may be more cost effective to provide a wider structure than that required by the amount of flow through the structure. On the other hand, it may be more cost effective to design foundations and armoring to withstand local scour depths. The designer must decide which alternative is more desirable.

Bridge foundations must be designed to adequately withstand anticipated scour as noted in Publication 15M, Design Manual, Part 4, Structures, Policies and Procedures, Chapter 7.

Stream stability issues, such as potential vertical and horizontal degradation, may warrant accommodations in the design of the bridge. If the channel is vertically degrading, it is likely that as the channel deepens the channel banks will slough, resulting in widening. Also, where significant meandering is occurring, meanders tend to migrate downstream and increase in amplitude. For possible measures, such as river training techniques, see HEC-20, Stream Stability at Highway Structures (FHWA, 2001d), and HEC-23, Stream Instability Counter Measures (FHWA, 2001a).

G. Freeboard and Low Chord. The lowest level of the bridge superstructure is called the low chord elevation. Navigational clearance and other reasons notwithstanding, the low chord elevation is equal to the sum of the design water surface elevation (highwater) and the freeboard.

If you do not have constraints that limit the height of your profile, consider adding freeboard for passage of debris. Generally for bridge replacement structures, the low chord should not be lower than that of the structure being replaced unless geometric constraints exist. The bridge opening generally should not be reduced where hydraulics is a controlling factor.

10.5 STREAM CROSSING DESIGN

The process by which a highway-stream crossing design is developed has changed significantly over the last few years. Fortunately, hydraulic engineering technology is available to meet the challenge of changed construction economics as well as the needs created by increased traffic demands and the concerns for highway user safety, the environment, and increasing flood losses caused by floodplain development.

Procedures for the design of a stream crossing generally should follow the sequence indicated below:

1. Hydrologic Analysis. Estimate of the stream discharge at the site for a range of flood exceedance probabilities. All existing flood control measures, land use and anticipated changes in the watershed should be considered in the hydrologic analysis.

2. Stream and Floodplain Analysis. The estimate of streamflow characteristics at the crossing site for the range of discharges being considered. These estimates include stages, flow distributions, velocities, stream morphology, sediment transport and the influence of land use and development.

3. Identification of Criteria for Design. Certain criteria or standards, by which a design is judged to meet the objectives of a crossing design. These criteria may override risk and economic considerations in some
crossing designs, and may include standards imposed by legislation, design policy, or the importance of the highway (e.g., national defense or for emergency vehicle access), and preclude politically, environmentally or socially unacceptable solutions such as placing an embankment in a significant marsh or wetland, or leaving a community isolated during floods.

4. Analysis of Design Alternatives. The analysis of a highway-stream crossing involves an engineering, environmental, and economic evaluation of various design alternatives. The objective of the analysis is to achieve a design which will have the least expected cost to society considering expected losses and capital costs within the constraints imposed by the criteria for design.

The hydraulic analysis of the stream is discussed in Section 10.4. This section deals primarily with the design procedure beginning with the identification of criteria for design.

A. Design Criteria for Highway-Stream Crossing System. Hydraulic engineers and other highway engineering and environmental personnel are encouraged to think of a highway-stream crossing as a system consisting of the stream and its floodplains, the bridge(s) provided to pass floods of an estimated exceedance probability, and the roadways on the floodplains which may be overtopped when that flood is exceeded.

It is customary to arrive at the design of the highway components of the stream-crossing system individually based on input from the several disciplines involved. The common objective of all disciplines, however, must be to provide a safe facility for traffic, prudent expenditure of public funds, and a design which minimizes damage to property and the environment, to the extent practicable.

First and foremost, all floods which occur during the life of the facility will pass the crossing site. With unlimited funds for construction, any crossing can be designed for a very small probability of damage or traffic interruption. It is good economics and good engineering, however, to weigh the capital costs of measures taken to avoid damage and traffic interruption against the probability of future costs. In order to assess the future costs, risks associated with floods that are smaller than, as well as in excess of, the customary design flood must be evaluated, including the largest flood that must pass through the highway structure(s) and larger floods that will overtop the highway. Thus, all components of the highway-stream crossing are a part of the system that must be considered in the hydraulic design.

The hydraulics of the stream component of the highway-stream crossing system are discussed in Section 10.4. Highway and bridge components are discussed in the following sections.

1. Design Strategies/Alternatives. Highway profile and alignment ordinarily are considered somewhat independently of bridge waterways except for the consideration given to providing some elevation differential between the conventional design highwater and the roadway surface. As part of a highway-stream crossing system, both profile and alignment are controls on the magnitude of the maximum flood that will pass through the waterway opening(s) provided. The bridge waterway(s) and the roadway alignment and profile together determine the capacity of the system to pass floods without overtopping of one or both components. The effects of the roadway profile on the adequacy of the waterway opening(s) should be considered in establishing the profile and in designing the bridge waterway(s).

The stage-discharge relationship for the stream and backwater associated with the design alternative are the hydraulic considerations for establishing the highway profile. The horizontal and vertical alignment of the highway are factors that must be considered in establishing the waterway design. The vertical alignment will be considered first in this discussion.

Several profile alternatives are available for consideration, dependent upon site topography, traffic requirements and flood damage potential. The alternatives range from crossings which are designed to overtop frequently to crossings which are designed to overtop rarely or never. The alternative selected will depend upon the criteria established for the design, construction costs, risks, stage-discharge-duration relations, flow distribution and scour consideration.

In Figure 10.4, vertical sag curve profile, the bridge is near the low point in a sag-vertical curve profile. Examples of the use of this profile configuration are the use of low bridges in rolling terrain for low traffic roads which are frequently overtopped and high bridges in rugged terrain which probably will never be
threatened by flood. A distinctive feature of this profile is the certainty that the bridge structure will be submerged when any overflow of the roadway occurs. If possible, bridges on vertical sag curves should be avoided because drift can accumulate in the superstructure and on the bends and accentuate general and local scour. Also, large drift can produce high impact forces on the structure, possibly causing structural failure, especially if scour has affected the foundations. If a vertical sag curve cannot be avoided, and there is even a small probability of overtopping, it is advisable to avoid curbs and use open-type railing with this profile in order to minimize damage for high velocity flow around the ends of parapets. Bridges on vertical sag curves also are undesirable as far as deck drainage is concerned because ponding will occur on the deck if inlets have insufficient capacity to intercept the flow or become clogged with ice or debris.

Figure 10.5, crest-vertical curve profile, illustrates a profile which may be used where the valley width is sufficient to utilize a profile which allows the roadway to be overtopped without the superstructure of the bridge being submerged. Variations of this profile may be used in locations where the stream channel is located on one side of the floodplain, i.e., an eccentric crossing, and the profile allows overtopping of the approach roadway on only one side. The difference between the low point in the roadway profile and the low chord in the bridge superstructure can be varied, within geometric constraints, to meet requirements for maintaining free surface flow and to accommodate passage of ice, debris and drift.

A third profile alternative is shown in Figure 10.6, level profile. Variations in this profile include a slight crest-vertical curve on the bridge to establish a camber in the superstructure. With this profile, all floods with stages below the profile elevation of the roadway and bridge deck will pass through the waterway opening provided. The disadvantages of a sag-vertical curve profile are applicable, and the same cautions should be exercised. With either profile configuration, severe contraction scour will be likely to occur under the bridge and downstream for a short distance when the superstructure is partially or totally submerged. The velocity of flow and depth of the superstructure may impose large hydraulic forces on the bridge superstructure. The accumulation of debris or ice on the upstream side of the structure can increase the effective depth of the superstructure, impose larger hydraulic forces on the bridge superstructure, and increase scour depths. Since no relief from these forces is afforded, crossings on zero gradients and in sag-vertical curves are more vulnerable than those with profiles which provide an alternative to forcing all water through the bridge waterway.
The horizontal alignment of a highway at a stream crossing also must be considered in selecting waterway opening location(s) and design, and the crossing profile. Potential lateral migration of the stream could threaten the stability of the structure, see Figure 10.7.
Water surface elevations during peak flow along a skewed highway-stream crossing are not equal because of the gradient in the water surface profile. The elevations upstream of the crossing will differ, sometimes considerably, depending upon the water surface gradient, land use, the floodplain width and the severity of the crossing skew. Water surface differentials between the upstream side of the crossing and the downstream side will be much greater at points A and B than the backwater above the bridge, as illustrated, in Figure 10.8.

The apparent effect of the crossing on water surface elevations is sometimes highly exaggerated because of the difference in water surface elevations from one side of the road to the other. Figure 10.9 is an example in which the roadway was cut in a location equivalent to point A in Figure 10.8, because of the differential across the road during an extreme flood. It may be advisable, in many locations, to provide waterway opening(s) at locations A and/or B, Figure 10.8, in order to avoid large head differences from the upstream to the downstream side of the roadway. Again, this is dependent upon several considerations including the land use upstream and downstream, water surface gradient, the severity of the skew, backwater from the highway crossing, and the length of the roadway encroachment in the floodplain.

Large head differentials from the upstream side to downstream side of skewed crossings can be explained by referring to Figure 10.8. Flood waters upstream of the embankment near location B will be diverted toward the structure since there will be a gradient from B. Downstream, water must flow from the bridge toward B in order to fill the area downstream of the roadway on the left floodplain. Thus, the water surface elevation upstream at B will be higher than at the bridge and downstream at B will usually be lower than downstream of the bridge.
Figure 10.8  Upstream and Downstream Water Surface Elevations at a Skewed Crossing

Figure 10.9  Roadway Cut through the Embankment Caused by Head Differentials Across the Road at a Skewed Crossing
Similarly, the water surface elevation on the upstream right floodplain near location A will be approximately equal to the water surface elevation at the bridge plus the velocity head at the bridge. Thus, there will be no flow from A to the bridge along the roadway embankment. On the downstream side, flood waters will flow from the bridge toward A and, dependent on the velocity head at the bridge, will be lower downstream of A than downstream of the bridge. Therefore, the water surface differential from upstream to downstream will be greater at both A and B than at the bridge.

One-dimensional methods (step backwater) may be inadequate to provide a quantitative analysis of water surface elevations up- and downstream of a skewed highway crossing of a stream. Finite element and finite difference models are two-dimensional methods that can be applied in some complex situations. These models enable designers to study the water surface elevations in cross sections rather than in profile only and can identify locations where undesirable head differentials could occur. These are more complex models which require more site information and a greater length of time to use. The accuracy provided by the two-dimensional model is not justified in most cases; thus a one-dimensional model is used, tempered with sound engineering judgment.

Discussion of the hydraulic analysis of stream crossings is provided in Section 10.5.B. Structural features that can have a significant effect on the performance of the system are discussed later in this section.

2. Waterway Openings. The design flood for a PennDOT structure is based on the roadway classification. The design flood may be adjusted based on horizontal and vertical alignment of the approach roadways, preservation of the natural flow distribution of the stream, environmental impacts, and/or avoidance on creating undue hazards to the highway and other properties.

Concerns other than hydraulic requirements which influence waterway opening location and size include clearances for navigation, roadway geometries, terrain, soil stability at abutments, access to adjacent properties, intersections and interchanges with other roads, separations for other roads or railroads, wetlands, economics, and numerous others. Discussion here will be limited to hydraulic and economic considerations in locating and sizing waterway openings.

(a) Location. The choice of location of a waterway opening at a stream crossing site with limited floodplain widths is not difficult because it is readily apparent that one opening will suffice or that it is not physically practicable to use more than one opening. Similarly, when approach roadways are not significantly higher than the floodplains, auxiliary waterway openings on the floodplains are either unnecessary or culvert-size openings for local drainage are all that is necessary.

The location of waterway openings in the highway-stream crossing system is more complex for designs for rare floods and at sites with extensive floodplains. In this discussion, it is assumed that an opening will be provided at the principal channel of the stream and that available options include a wider opening at that location and auxiliary opening(s) on the floodplain or some combination of these options.

Several factors influence decisions on the location of waterway opening(s) to provide for flood passage. Basic objectives in choosing the location(s) of auxiliary opening(s) include maintenance of flow distribution and flow directions to the extent practicable, provision for relatively large flow concentrations in the floodplains, avoidance of diversion of floodplain flow along roadway embankments for long distances, and considerations of backwater and scour damage to the highway and other property. Site conditions, economics, budgetary constraints, and the horizontal alignment of the highway limit the extent to which these objectives can be met. The objectives are complementary in that the purpose in maintaining flow distribution and direction is to minimize damage to the floodplain environment and to avoid excessive backwater and scour. Providing for large flow concentrations achieves similar purposes, as does avoiding long distance diversion along roadway embankments.

Other site-specific factors which influence opening location are local drainage, the possibility of causing a cutoff in a meander bend, other transportation facilities in the vicinity, floodplain use, and the horizontal and vertical alignment of the highway.

The need to provide for local drainage occurs where an area on the floodplain will not drain after the highway is constructed and where the highway alignment intersects a tributary stream upstream of its
confluence with the main stream (Figure 10.10). In either case, diversion along the highway fill to the principal stream channel or providing an auxiliary opening on the floodplain is possible. Diversion along the highway embankment can create maintenance problems by increasing the gradient in the tributary channel and by providing a more efficient channel for floodplain flow at the toe of the highway embankment. Headcutting can occur which will endanger the embankment and upstream property, and a delta can form in the principal stream channel, causing it to change course (Figure 10.11).

An opening on the floodplain for a tributary stream or local drainage is the most desirable alternative if additional costs for construction are warranted. Other factors, such as flow concentration in the floodplain or diversion of floodplain flow for a long distance along the highway embankment, may influence the decision as well. Although the location of the opening on the floodplain will be influenced by the need to accommodate for local drainage or a tributary, any opening on the floodplain will be subjected to flood flows of the principal stream. The size of the opening may be influenced more by the amount of floodplain flow from the large stream than by flow from the tributary.

A highway located in the bend of a river, as in Figure 10.12, presents a particularly difficult problem in the location of auxiliary openings. Flow in the floodplain is likely to be concentrated across the mouth of the bend, as illustrated. Because the distance across the bend is shorter than in the channel, the water surface slope is steeper and conveyance in the floodplain is increased relative to conveyance in the channel. An auxiliary opening at this location which severely constricts the flow will cause general scour which could result in a cutoff of the bend, depending upon the width across the bend, the length of the bend, land use and soils. If an opening is necessary, the design should be conservative to guard against the possibility of scouring velocities. Other measures also may be advisable. Spur dikes can make the opening more hydraulically efficient, and armoring the flow line may protect against scour in some instances. Additional auxiliary openings can be used in a long bend to reduce the degree of flow constriction, and the opening can be located away from the mouth of the bend in some instances. Also, elimination of the bend by channel realignment may be possible at some locations. Where the effects of an auxiliary opening at a bend could be especially damaging, physical modeling may be warranted.

Figure 10.10 Diversion and Bridge Alternatives for Tributary Stream in the Floodplain
Figure 10.11 Delta Formed in Principal Stream by Divided Tributary

Figure 10.12 Stream Crossing in a Bend
Often, there are existing crossings in the vicinity of the proposed crossing which have altered the flow distribution. In order to keep the effects of a new crossing on existing flow distribution to a reasonable minimum, considerable weight should be given to the location of existing opening(s), but the size of proposed opening(s) should be based on hydraulic requirements. Where existing auxiliary opening(s) do not accommodate significant flow, decisions on opening(s) in the new crossing should be based on the hydraulic requirements of the crossing system.

Other transportation facilities on the floodplain may require grade separation. This opening on the floodplain will pass flood waters, and roadway elevations required for separation may significantly alter the magnitude of the flood that must be passed through the proposed highway openings. Consideration should be given to the flow constriction at the separation which may cause damage to the other facility (Figure 10.13).

Floodplain uses which can influence opening location include development and the need for access across the highway right-of-way. Auxiliary waterway openings may need to be located to accommodate these uses and in recognition of the effects of development on flow distribution and tolerable backwater.

Auxiliary waterway opening(s), or relatively low roadway profiles in bridge approach areas, may be necessary in order to avoid large differences in water surface elevations between the upstream and downstream side of skewed highway crossings of streams with wide floodplains. An opening located at either A or B in Figure 10.8 should be sized so that head differentials will be minimized and severe scour will not develop.

Figure 10.13 Grade Separation for another Transportation Facility in the Floodplain Will Serve as an Auxiliary Waterway Opening
(b) Size. The size of a waterway opening is limited by the boundaries comprised of the stream bed and/or floodplain, the roadway embankment ends at each side, and the superstructure of the bridge at the top. Many characteristics of a crossing site influence the selection of waterway opening size. These characteristics are used in defining criteria for judging the acceptability of alternative designs of the crossing system as discussed in the introduction of this section. It is possible that a multitude of roadway profile and waterway opening alternatives would satisfy the criteria established from characteristics of the site. As an example, if criteria for the crossing include a severe limitation on backwater, this limitation can be satisfied either by using a small opening and a low roadway profile or by bridging all of the stream cross section. It is probable, however, that in most locations, alternatives which are somewhere between these two extremes will also satisfy the criteria established for the site and will prove to be more prudent insofar as the expenditure of public funds is concerned.

The performance of a waterway opening is dependent not only on the boundaries defined by the terrain, the bridge superstructure, and the embankment at each end of the bridge, but also by water surface elevations. The flood which will flow through the opening without disrupting traffic is determined by the above physical boundaries of the opening and the profile of the crossing.

Both alternative roadway profiles and waterway openings are practicable for many highway-stream crossings. Where this freedom is available, the probability of overtopping is a design decision which can be made considering the economic consequences of the decision. Bridge structural components, foundations, waterway opening size, and approach roadways should be designed so that the selected stream crossing system results in optimal or near-optimal use of public funds. Capital costs for construction; risks of damage to approach roadways; risks of damage to the bridge from buoyancy, drag, impact loads and scour; the costs of traffic interruption; and risks to other properties should be considered in determining the economic consequences of selecting a design from available alternatives.

The design of many other stream crossing systems is constrained by social, political, or environmental concerns; engineering considerations, such as geometries; multiple-use purposes, such as navigation, livestock passage and land access; economic considerations other than optimal use of public funds; policy decisions, such as minimum standards; and topographic controls. Where such constraints are imposed, the alternative crossing design which meets the constraints at the least cost in public funds should be selected. Economic concerns must be considered in selecting an alternative. In either case, foundations, bridge structural components, the size of the waterway opening, protective and preventive measures, and the stream crossing profile should be predicated on capital costs and risk costs for all floods which substantially contribute to those costs.

Waterway openings are most severely taxed by the largest flow that must pass through the opening. This flow rate is approximated at incipient overtopping of the highway-stream crossing system; therefore, waterway openings should be sized considering the probability of such an occurrence and the associated risks of damage.

(c) Auxiliary Openings. The need for auxiliary waterway openings, or relief openings, arises on streams with floodplains. The terminology adopted here, i.e., auxiliary openings, is intended to be consistent with the concept of a highway-stream crossing system in which each component has a specific role. The purpose of openings on the floodplain is to pass a portion of the flood flow in the floodplain when the stream reaches a certain stage. It does not provide relief for the principal waterway opening in the sense that an emergency spillway at a dam does, but it has predictable capacity during flood events.

The location of auxiliary openings was discussed above. The required size of auxiliary openings is not extensively researched, although an effort has been undertaken by the State of Mississippi to study the hydraulic performance of existing multiple bridge systems.

A method currently in use for determining the size of auxiliary openings is described here. While some accuracy may be lost due to the use of one-dimensional models to analyze two-dimensional flow, the results are sufficient for most applications.

Auxiliary openings on the floodplain generally are assigned a portion of the total streamflow based on conveyance calculations. Conceptually, the flow will separate at an assumed or a real divide and
continue to the appropriate opening. For a normal crossing system at a straight reach of the stream, this flow divide can be accomplished approximately as assumed by sizing all waterway openings so that backwater above each is approximately the same. If any opening is sized so as to create more backwater than another, the divide will not be as planned because the highwater surface at that opening will cause diversion toward another opening.

The complexity in analysis with one-dimensional models comes with crossings which are not normal to the flow direction, with bends and sinuosity in the stream system, and with flow directions which vary along the crossing and with stages in the stream. For this reason, it has been recommended (above) that auxiliary openings in skewed crossings at either location A or B, Figure 10.8, should be conservatively sized as judged by the best available method of analysis. Use of the Surface Water Modeling System should be considered for the evaluation of such crossings.

(d) Replacement Bridges. Investments in replacement bridges constitute an increasing proportion of capital expenditures for highway construction. A wealth of experience may be available at the site of existing bridges relative to the hydrology and morphology of the stream. This experience and modern hydrologic, hydraulic and economic analysis technology should be fully exploited when the replacement crossing system is selected. In particular, floodplain usage may have changed near the crossing site since the original construction indicating a need to reevaluate the risk to private property. Traffic volume increases and changes in traffic character may indicate a need to reassess traffic service requirements. Changes in the relative costs of construction, maintenance and flood damage repair of various components of the crossing system may indicate that an alternative that differs from the existing crossing system should be selected.

Many existing bridges have withstood substantial floods, and studies may indicate that no change in the stream crossing system is warranted. In such cases, the fact should not be overlooked that the replacement of short spans with longer spans or a truss with a girder design will result in a reduction in the waterway opening if the replacement structure has a deeper superstructure and the same vertical alignment). As a result of the replacement, the risk of backwater damage will be increased, the probability of overtopping will be increased, and there will be a greater hazard of ice and debris damage. If the existing level of traffic service is to be maintained, insofar as interruption by flooding is concerned, and the risks of flood damage to the highway and other property are not to be increased, the grade of the bridge deck should be raised to compensate for the deeper superstructure of the replacement bridge.

3. Structural Alternatives. A myriad of structural alternatives is available for use in a highway-stream crossing system when all of the possible combinations of bridge lengths, spans, pier types and orientation, geometries, parapet designs, and superstructure designs are considered. In addition, at many crossings, multiple bridges or a single bridge may be viable alternatives, or large culverts may be used in lieu of one or more bridges.

The hydraulics of the highway-stream crossing system should be given considerable study in choosing the preferred design from the long list of available alternatives.

(a) Bridge or Culvert. Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Estimates of costs and risks associated with each will indicate which structural alternative should be selected on the basis of economics. Other considerations which may influence structure-type selection are listed in Table 10.1 and discussed in subsequent sections.

Table 10.1 lists many of the advantages and disadvantages of bridges and culverts. Those considerations which are associated with the use of culverts are discussed in Highway Drainage Guidelines: Hydraulic Design of Culverts (AASHTO, 2000b). Culvert(s) in combination with bridge(s) are used in numerous highway-stream crossings, either to pass flow in a floodplain or to provide for local drainage in the floodplain. Where culverts or small bridges are used in the floodplain in conjunction with a bridge, the potential scour as a result of the head differential from upstream to downstream and the long duration of the hydrograph should be considered, see Figure 10.14. As an example, a culvert at location A or B in Figure 10.8 would have a high outlet velocity because of the large head differential across the roadway, and severe scour could occur at the outlet.
Table 10.1 Bridge or Culvert

<table>
<thead>
<tr>
<th>Bridges</th>
<th>Disadvantages</th>
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<tbody>
<tr>
<td><strong>Advantages</strong></td>
<td><strong>Disadvantages</strong></td>
</tr>
<tr>
<td>• Less susceptible to clogging with drift, ice and debris</td>
<td>• Require more structural maintenance than culverts</td>
</tr>
<tr>
<td>• Waterway increases with rising water surface until water begins to submerge superstructure</td>
<td>• Spill slopes susceptible to erosion and scour damage</td>
</tr>
<tr>
<td>• Scour increases waterway opening</td>
<td>• Piers and abutments susceptible to failure from scour</td>
</tr>
<tr>
<td>• Flowline is flexible</td>
<td>• Susceptible to ice and frost formation on deck</td>
</tr>
<tr>
<td>• Minimal impact on aquatic environment and wetlands</td>
<td>• Bridge railing and parapets hazardous as compared to recovery areas</td>
</tr>
<tr>
<td>• Widening does not usually affect hydraulic capacity</td>
<td>• Deck drainage may require frequent maintenance cleanout</td>
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<table>
<thead>
<tr>
<th>Culverts</th>
<th>Disadvantages</th>
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<tbody>
<tr>
<td><strong>Advantages</strong></td>
<td><strong>Disadvantages</strong></td>
</tr>
<tr>
<td>• Provides an uninterrupted view of the road</td>
<td>• Silting in multiple barrel culverts may require periodic cleanout</td>
</tr>
<tr>
<td>• Roadside recovery area can be provided</td>
<td>• No increase in waterway as stage rises above soffit</td>
</tr>
<tr>
<td>• Grade raises and widening projects sometimes can be accommodated by extending culvert ends</td>
<td>• May clog with drift, debris or ice</td>
</tr>
<tr>
<td>• Require less structural maintenance than bridges</td>
<td>• Possible barrier to fish passage</td>
</tr>
<tr>
<td>• Frost and ice usually do not form above the culvert before other roadway areas</td>
<td>• Susceptible to erosion of fill slopes and scour at outlets</td>
</tr>
<tr>
<td>• Capacity increases with stage</td>
<td>• Susceptible to abrasion and corrosion damage</td>
</tr>
<tr>
<td>• Capacity can sometimes be increased by installing improved inlets</td>
<td>• Extension may reduce hydraulic capacity</td>
</tr>
<tr>
<td>• Usually, easier and quicker to build than bridges</td>
<td>• Inlets of flexible culverts susceptible to failure by buoyancy</td>
</tr>
<tr>
<td>• Scour is localized, more predictable and easier to control</td>
<td>• Rigid culverts susceptible to separation at joints</td>
</tr>
<tr>
<td>• Can be used to arrest headcutting</td>
<td>• Susceptible to failure by piping and/or infiltration</td>
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<tr>
<td>• Storage can be utilized to reduce peak discharge</td>
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Piers. Economy of construction usually plays a large role in the determination of spans, pier locations and orientation, and substructure and superstructure design. It is necessary that construction costs always be a factor in the structural design of a bridge in order to make use of economically available structural materials, but the cost of construction is only one part of the total economic cost of a stream crossing system. Hydraulic considerations, maintenance costs, and expected cost of failure also should be considered in making decisions on the type, size, and location of the system.

The number of piers in any channel should be limited to a practicable minimum, and piers in the channel of small streams should be avoided, if practical. Piers properly oriented with the flow do not make large contributions to bridge backwater. But, they do contribute to general scour. In some instances, severe scour has developed immediately downstream of bridges because of eddy currents and because piers oriented improperly occupy a significant area in the channel. Lateral as well as vertical scour also occurs at some locations.

Piers should be aligned with flow direction at flood stage in order to minimize the opportunity for drift to be caught in piling or columns, to reduce the contraction effect of piers in the waterways, to minimize ice forces and the possibility of ice dams forming at the bridge, and to minimize backwater and local scour (Highway Research Board, 1970). Pier orientation is difficult where flow direction changes with stage or time. Circular piers, or some variation thereof, are probably the best alternative if orientation at other than flood stage is critical.

Piers located on a bank or in the stream channel near the bank are likely to cause lateral scouring of the bank. Piers located near the stream bank in the floodplain are vulnerable because they can cause bank scour. They also are vulnerable to failure from undermining by meander migration. Piers which must be placed in locations where they will be especially vulnerable to scour damage should be founded at elevations safe from undermining (Figure 10.15).
Pier shape is also a factor in local scour (Highway Research Board, 1970). A solid pier will not collect as much debris as a pier 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.

Where ice is a problem, piers can be battered and armored to facilitate breaking up ice flows which otherwise would crush against the leading edge of the pier (Watt & Podolny, 1976). Armoring the leading edge is also a necessary countermeasure in many streams that transport large material as bed load (Figure 10.16).
(c) Abutments. Abutments may be of vertical walls or the fill slope may spill through the end span forming a "spill-through" abutment, or a combination of these types of abutments may be used. The type of abutment used has little effect on total backwater except where the flow section is severely contracted by a short bridge. Selection of abutment type is based largely on cost, the type of foundation material, and stability considerations.

The principal concerns of hydraulic engineers for abutments are orientation and security from scour-related failure. Concerns for security from scour usually are resolved by protective and preventive measures which are discussed in Section 10.5.C. Orientation is usually the same as for piers in adjacent bents; the effects of skew are discussed elsewhere in this section.

(d) Foundations. The foundation is one of the elements of a bridge which is most vulnerable to attack by floods. Examination of individual boring logs and plots of the profiles of various subsurface materials are important to the prediction of potential scour depths as well as to estimation of the bearing capacity of the soils.

Five basic types of foundations are used for stream crossings: (1) spread footings, (2) pile or drilled shaft trestle bents, (3) footings on piles or drilled shafts, (4) pedestals, and (5) caissons, see Figures 10.17 to 10.20. Spread footings are dependent upon the bearing capacity of the material in which they are founded for support. They may be used if founded below the level of anticipated scour or where sound rock is relatively shallow.
Figure 10.17 Spread Footings  
(after Highway Research Board, 1970)

Figure 10.18 Drilled Shaft Foundation  
(after Highway Research Board, 1970)

Figure 10.19 Typical Pile Foundation  
(after Highway Research Board, 1970)
Piling or drilled shafts, whether in a trestle-type bent or under a footing, usually depend upon the surrounding material for skin friction and lateral stability. In some instances, they can be carried to rock or other dense material for load-carrying capacity. Tip elevations for piling or drilled shafts should be based on estimates of potential scour depths as well as bearing in order to avoid losing lateral support and load-carrying capacity during floods. Pile bearing capacity derived from driving records has little validity during floods if the material through which the piles were driven is scoured away.

Caissons are used in large rivers and usually are sunk to dense material by excavation inside the caisson. Founding depths are such that scour usually is not a problem at the pier after construction is completed. Severe contraction scour has developed downstream of some such bridges, however, because of contraction of the flow from the large piers.

Attention should be given to potential scour and the possibility of channel shifts in designing foundations for bridges on floodplains and spans approaching the stream channel. The thalweg in the channel should not be considered to be in a fixed location when establishing founding elevations. The history of a stream and a study of how active it has been can be very useful in making decisions on pile and drilled shaft tip elevations (Figure 10.21).
(e) Superstructures. Hydraulic forces that should be considered in the design of a bridge superstructure include buoyancy, drag, and impact from ice and debris. Decisions on superstructure type should be influenced by the profile configuration, the probability of submergence, expected problems with ice and debris, flow velocities, and the usual economic, structural and geometric considerations.

(1) Buoyancy. When the superstructure of a bridge is submerged or partially submerged, the new effective weight of the superstructure is the weight of the structure less the weight of the volume of water displaced. This will differ from the submerged weight of the superstructure if air is trapped under the deck or if the superstructure is of box girder design. Where solid parapets are used, the large volume above the deck may displace water if the bridge is at the bottom of a sag-vertical curve, or on a small gradient on the approach roadways. In some cases, this effect can be reduced if drain slots are provided in the parapets.

The effect of trapped air under the bridge deck can be reduced by providing holes through the deck between each girder. Superstructures are usually anchored to piers to counter buoyant forces and resist drag forces created by flowing water.

(2) Drag Forces. The drag forces on a submerged or partially submerged superstructure may be calculated by the following equation.

(Equation 10.1)

\[
F_d = C_d \rho H \frac{v^2}{2}
\]

where:  
\( F_d \) = drag force per unit of bridge length, N/m (lb/ft)  
\( C_d \) = coefficient of drag  
\( \rho \) = density of water, kg/m\(^3\) (slug/ft\(^3\))  
\( H \) = depth of submergence of superstructure, m (ft)  
\( v \) = velocity of flow, m/s (ft/s)  
Note: Slugs = 32.2 pounds mass

The coefficient of drag, \( C_d \), can be taken as 2.0 to 2.2 based on usual Reynolds numbers in natural streams and the usual shape of bridge superstructures (Streeter, 1971).

The density of fresh water is usually taken as:

\[
\rho = 1000 \text{ kg/m}^3 = (1.94 \text{ slugs/ft}^3)
\]

It is apparent from Figure 10.22 that combined buoyant and drag forces need to be dealt with in design. Structural and hydraulic engineers should be aware of these forces in the selection of profile alternatives, and of the probability of overtopping and the implications of parapet and superstructure alternatives. If warranted by the probability of submergence, superstructures should be adequately anchored and vented.
(3) Ice Forces. Ice forces are considered in the design of bridge piers according to procedures described in Section 3.9.2 of the LRFD, Specifications for Highway Bridges (AASHTO, 1998 and subsequent). Inherent in this section of the bridge specifications is an assumption that the bridge superstructure will not be subjected to impact forces from floating ice, static pressure from thermal movements or from ice jams, or uplift from adhering ice in water of fluctuating levels. It is reasonable that bridge superstructures would rarely be subjected to uplift from adhering ice or pressure from thermal movements since these forces normally are associated with relatively large bodies of water and superstructures normally should be high enough to be unaffected.

Ice jams and floating ice forces can be imposed on superstructures depending upon the highway-stream crossing profile configuration and the probability of overtopping during the spring breakup of ice on the stream.

The bridge specifications recommend that static ice pressures shall be given consideration, but a method for calculating forces associated with ice jams is not prescribed or recommended. A method for computing dynamic forces on piers from floating ice is included by values used in the equation developed from data at piers and may not be applicable for computing forces on superstructures.

Figures 10.23 and 10.24 are examples of ice damage to bridge superstructures. Research is needed to determine the static and dynamic loads that can be expected from ice loads on superstructures under various conditions of ice strength and streamflow. The probability of ice flows occurring simultaneously with streamflow at stages which would impose ice forces on the bridge superstructure is probably small in many locations.
Figure 10.23  Span Displaced by Force of Ice

Figure 10.24  Girder Twisted by Force of Ice
In addition to the forces imposed on bridge substructures and superstructures by ice loads, there are other hydraulic implications of ice flows. As an example, an ice jam at a bridge in or near a community could cause flooding of the community by water backed up behind the ice dam.

Figure 10.25 is an example in which the sluicing action of streamflow under an ice jam caused a foundation failure. It is clear that the ice jam was the cause of the failure since the scour occurred under the ice at the upstream side of the bridge. Streamflow during the event which caused failure had a probability of exceedance in any one-year period of about 0.20. The joint probability of the ice flow and streamflow that occurred during the event is likely to be much smaller than 0.20 unless significant ice forms on the stream every year and the annual maximum streamflow occurs every year with the spring breakup of ice. Even though the joint probability may be small, the relatively small additional cost of riprap scour protection, a deep foundation and/or a longer opening to protect against such a failure could be a good investment at many locations.

Ice Engineering (USACE, 1980) is a good source of information on how ice jams are formed, where they are likely to form, how to prevent them, and how to combat them.

(4) Debris Forces. Floating debris may consist of logs, trees, house trailers, automobiles, storage tanks, lumber, houses, and many other items representative of floodplain usage upstream of the bridge. Because of the variety of debris involved, it would be necessary to compute impact forces on a worst case basis for the debris that could originate in the basin using an assumption for the negative acceleration of the debris on impact in the equation:

\[ F = M \frac{dv}{dt} = \frac{Mv^2}{2S} \]

where:  
\( F \) = the impact imparted by the debris, N (lb)  
\( M \) = the mass of the debris, kg (slug)  
\( dv/dt \) = the change in velocity of the debris with respect to time  
\( v \) = velocity of the debris on impact, m/s (ft/sec)  
\( S \) = stopping distance, m (ft)

Instantaneous stopping of the debris would indicate an infinite impact force; therefore, it is apparent that yielding of both the debris and the bridge component is possible. For research on the maximum loads for debris loading, see NCHRP Report 445, Debris Forces on Bridges (Parola, A., et. al., 2000).

Drag forces on the superstructure also can be increased significantly by a buildup of debris. The buildup increases not only the effective depth of the superstructure, but the drag coefficient also may be increased because of the increased resistance to flow.

In addition to forces imposed on bridges by debris, perhaps the most common hazard is partial or total clogging of the waterway opening. Partial clogging can result in a channel shift or a sluicing action similar to that from the ice in Figure 10.25. Total clogging can result in failure of the bridge or a shift in the channel location from under the bridge.
4. Channel Modification. A primary objective in the design of a highway-stream crossing system should be to disturb the stream as little as is practical. Channel modifications should be made only where modification is necessary to achieve compatibility between the highway and stream and to accommodate streamflow with a minimum of interruption to the stream and its environment.

Highway-associated channel modifications involve only short reaches of a stream in almost all instances. For an in-depth discussion of channel modification, refer to Chapter 8, Open Channels.

Modifications for the purpose of increasing channel capacity are not commonly encountered at highway-stream crossings because of the limited effectiveness of such modifications. Channel capacity improvements usually must be carried long distances downstream if the stage-discharge relationship at the site is to be altered. Because of flow controls downstream, the stage-discharge relationship at a crossing will not be altered a discernible amount by a short reach of modified channel.

Stream channels are sometimes widened through the waterway opening of a bridge. As indicated, this essentially will have no effect on the natural stage-discharge relationship of the stream, but by increasing the waterway opening under the bridge, backwater at high flood stages may be decreased. However, short reaches of stream channels which have been enlarged may return to the natural cross section by deposition within the enlarged reach unless it is regularly maintained. At many locations, it may be advisable to consider additional bridge length rather than to rely on an enlarged channel section under the bridge. When considering widening a stream reach, any environmental effects must also be taken into consideration.

An enlarged section through the bridge may be more successful if excavated so that the bed load will not be deposited in the excavated area and the enlarged area will be available when needed to convey flow through the bridge (Figure 10.26). However, concerns for the stream and floodplain environment and for bank stability may preclude the use of such an enlarged section on some streams. When existing fishable streams must be widened to provide additional conveyance capacity, widening should occur above the normal flow elevation within the stream and be confined to the floodplain above the normal water surface elevation within the stream. For a complete description of recommended alterations to fishable streams please refer to Publication 13M, Design Manual, Part 2, Highway Design, Chapter 10.
In some circumstances, it may be advantageous to realign a channel in order to create better flow alignment with a waterway opening. These circumstances might arise at crossing sites where other controls make it difficult or impractical to select an alternative site or different highway alignment (Figure 10.27). Modification of the channel alignment usually can be accomplished successfully if some general principles and guidelines are followed even though generalized criteria applicable to all streams are difficult to formulate. It should be noted that mitigation and/or enhancement measures such as those discussed in *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d) should be considered in developing the design of any channel modification. It also should be noted that the channel modification as illustrated in Figure 10.27 does not resolve any problems with the skew of the highway crossing with flow direction in the floodplain.

Figure 10.26 Channel Enlargement which Preserves the Section of the Low Flow Channel
The general rules which should be followed in making channel alignment modifications in alluvial channels pertain to the radii of bends, sinuosity, slope, system response and stream power. The radii of bends in the realigned channel should be made about equal to the mean radii of bends in an extensive reach of the river. The angle between a line drawn tangent to the inside of two successive bends and the bank line between bends should be about 20 degrees. This allows sufficient crossing length for the thalweg to move from one side of the channel to the other. The sinuosity, \( P \), multiplied by the channel slope will be unchanged by the alignment modification. Thus, if subscript 1 represents the original channel and subscript 2 the modified channel, then:

\[
P_1 S_1 = P_2 S_2
\]

(Equation 10.3)

The slope, \( S_2 \), in the modified channel should be chosen to satisfy Equation 10.4 where \( S_2 \) is in m/m (ft/ft) and \( Q \) is in m\(^3\)/s (cfs).

\[
S_2 Q^{1/4} < 0.0007 \quad (0.0017 \text{ in U.S. Customary Units})
\]

(Equation 10.4)

If this equation is not satisfied, there is a possibility of the channel becoming braided. In order to avoid causing a braided channel, the sinuosity, \( P_2 \), should be increased in order to reduce the slope, \( S_2 \).

Stream power in the new channel should be about the same as in the old channel (AASHTO, 2000d). Equating stream power in the old and new channel provides guidance for establishing the width, \( W_2 \), in the channel modification in the form of the equation:

\[
W_2 S_1 = S_2 W_1
\]

(Equation 10.5)

and

\[
\frac{W}{S} \approx \text{const}
\]

(Equation 10.6)
Thus, if the slope $S_2 > S_1$, then $W_2$ can be greater than $W_1$ so long as the increase in width is moderate.

Maintenance of the realigned channel may require riprap protection on the outside bank at bends and removal of deposits after floods so that new meander patterns will not form. For a more detailed discussions of channel modifications, see *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d). In addition, *Restoration of Fish Habitat in Relocated Streams* (FHWA, 1979a) contains excellent guidance for the design and construction of relocated channels and describes measures which will lead to rapid recovery of aesthetic and fishery values in new channels.

### B. Analysis of the Stream Crossing System.

Analysis of the stream crossing system involves the use of hydraulic engineering principles and techniques and engineering economics to select an alternative design. The design must provide the traffic service required at a minimum cost and meet the criteria established for protection of the stream environment, as well as social, political, and other economic and engineering concerns. This section addresses the hydraulic, environmental and economic analyses of stream crossings.

The hydraulic analysis of a highway-stream crossing system involves determining the backwater associated with each alternative profile and waterway opening(s), the effects on flow distribution and velocities, and estimating the scour potential. The environmental analysis involves an assessment of the impacts of the crossing alternative on the stream environment and the consideration of provisions for practicable mitigation and enhancement measures.

The hydraulic, environmental and economic analyses must be of the total crossing facility including all roadway items, all waterway openings, and the environmental implications and risks associated with each alternative.

1. **Hydraulic Performance of the Crossing System.** The hydraulic performance and backwater at various stream stages generally are the first measures used to judge the acceptability of an alternative highway-stream crossing system design.

The introduction of a constriction such as a highway crossing in a stream channel with supercritical flow conditions will not cause backwater above the constriction. Constriction results in the conversion of potential energy to kinetic energy, and, therefore, there is no rise to the water surface elevation upstream of the introduced constriction as long as the constriction is minor and does not cause a hydraulic jump to occur. However, there will be a rise in the water surface elevation through the constriction.

A highway facility which constricts a stream with subcritical flow conditions will cause higher water surface elevations upstream of the crossing than prevailed during floods prior to construction of the highway facility. The higher water surface elevations represent the amount of kinetic energy converted from potential energy to overcome losses comprised principally of contraction and expansion losses. Other losses include those from piers, abutments, eccentricity, superstructures, if submerged, and friction from longer flow paths. Friction losses can be significant if the floodplain constriction is relatively severe and the resistance to flow in the floodplain is high, as in wooded areas (Schneider et al., 1977).

The increase in water surface elevation upstream of the highway facility is referred to as backwater (AASHTO, 2000a). It is measured upstream of the waterway opening above the theoretical water surface elevation prior to construction of the crossing (Figure 10.28). Backwater above a highway crossing will cause incremental depth and duration of flooding and an increased area of inundation for a given flood magnitude. The incremental flooding associated with various flood magnitudes should be considered in evaluating the risks associated with alternative designs of the stream crossing system (Schneider & Wilson, 1980; Corry et al., 1980).

The hydraulic performance of a stream crossing system, such as the one illustrated in Figure 10.29, will be similar to that illustrated in Figure 10.28. The performance of a crossing system which includes an auxiliary opening is also illustrated. Figure 10.29 is schematic only and is not intended to represent the relative increase in hydraulic performance provided by an auxiliary opening or the relative decrease in backwater that would be provided.

The flow through waterway openings in a highway-stream crossing system is greatly influenced by the profile elevation of the roadway (Schneider & Wilson, 1980). If the backwater at a bridge is relatively small, the capacity of the waterway opening at incipient overtopping is relatively insensitive to changes in opening size for practicable alternatives in bridge length. Conversely, practicable alternative roadway profiles may
substantially change the flow rate which must pass through the waterway opening at incipient overtopping. However, the total performance of a highway-stream crossing system is sensitive to both waterway opening and profile elevation. The effects on velocities, flow distribution, stream environment, scour and construction costs and risk costs are highly responsive to the total design of the crossing.

The hydraulic design of a highway-stream crossing system is an iterative process in which alternative waterway opening designs are tried for each profile alternative. Each alternative opening and profile will have an associated hydraulic capacity and probability that the opening capacity will be exceeded.

2. Backwater. Backwater, or the increment of increased flood depth upstream of a highway crossing of a stream, often is used as the sole criteria for judging the adequacy of an alternative waterway opening. The amount of backwater permissible is sometimes established for application statewide without regard for risks to property, which are site specific, and differences in flow conditions at various crossing sites. The advent of the NFIP of the FEMA resulted in widespread adoption by various jurisdictions of such standard measures of acceptability. Some jurisdictions have established rules that require waterway openings adequate for floods with an exceedance probability of 1% in any period of one year with backwater not in excess of a stated amount. Such rules require the same standards of stream crossing design regardless of the warranted level of traffic service or the risk of property damage.

Figure 10.28 Backwater at a Highway-Stream Crossing
Backwater should not be used as the sole criterion for judging the acceptability of an alternative design. Instead, it is an aid that can be used in selecting the waterway opening and crossing profile, and to assess the risk costs of incremental flooding caused by the crossing facility.


3. Flow Distribution. Flow distribution will be disturbed during some floods by any highway-stream crossing system which utilizes a combination of fill and bridge within the floodplain. Flow distribution should be preserved to the extent practical, however, in order to avoid disruption of the streamside environment (U.S. Water Resources Council, 1978), to preserve local drainage patterns, to minimize damage to property by either excessive backwater or high local velocities, and to avoid concentrating flow in areas which were not subjected to concentrated flow prior to construction of the highway facility. Many crossing sites will present other good reasons for minimizing redistribution of existing flow, such as those at which an auxiliary opening on the floodplain will be more hydraulically efficient than an extension of the opening at the stream channel.

Flow distribution is a consideration at crossing sites where significant flow is distributed over a relatively wide floodplain during most floods. Minimal disruption of flow distribution can be achieved by providing openings to avoid diversions for long distances along the roadway embankment, at locations where relatively large flow concentrations occur, and by providing a crossing stream profile which will be overtopped at appropriate locations. An opening which is provided to maintain flow distribution must be sized to avoid diversion caused by backwater creating a hydraulic gradient toward another opening. This can be accomplished where the
crossing is normal to flow direction by adjusting opening sizes so that backwater above each opening is approximately equal. Complex sites, such as those at a bend, as in Figure 10.13, and skewed crossing, as in Figure 10.14, can be analyzed with one-dimensional models only by using a large measure of intuition, experience and engineering judgment to supplement the quantitative analysis. Unfortunately, complex sites are encountered frequently in stream crossing design. Development of two-dimensional methods of analysis greatly enhances the capabilities of hydraulic designers to deal with these complex sites.

4. Velocity. Velocities in the waterway opening(s) of most stream crossings are higher than velocities in the natural stream because of the contraction of the flow by approach fills, bridge piers and, in some cases, bridge superstructures. In some instances, flow will pass through critical in the contraction and return to subcritical downstream if the contraction is severe or velocities in the stream are near critical.

The use of an average velocity for all waterway openings as a criterion for design is not advisable. The acceptability of the average velocity in a waterway opening is dependent upon site characteristics such as natural stream velocity, bed materials, soil types, foundation materials, and risk considerations from backwater and scour as well as practical countermeasures to reduce the risk of damage. Other site-specific considerations, such as existing crossings, also may influence decisions regarding acceptable average velocities.

Scour computations completed with average velocity may result in under estimation of the scour that occurs at a particular location. This is because the velocity within a channel in the area where scour occurs may be higher than the average velocity across the entire channel. As the selection of the appropriate design velocity varies depending on the design objective, designers are encouraged to reference HEC-18, *Evaluating Scour at Bridges, Fourth Edition* (FHWA, 2001b) for selection of appropriate design velocities for scour computations and reference HEC-23, *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Second Edition* (FHWA, 2001a) to select appropriate velocities for countermeasure design.

5. Scour. Potential scour can be a most significant criterion in the analysis of a stream crossing system. The design of a crossing system involves an acceptable balance between (1) a waterway opening that will not create undue damage by backwater or suffer undue damage from scour, and (2) a crossing profile sufficiently high to provide the required traffic service.

Scour at waterway openings occurs as natural scour, contraction scour and local scour.

Natural Scour:
- Is a long-term aggradation and degradation of the stream bed.
- May be lateral migration.
- May be the natural trend of the stream or the result of some modification to the stream or watershed.

Contraction Scour:
- Occurs when the flow area of a stream at flood stage is decreased from normal, either by a natural contraction or by a bridge.
- Occurs due to increased velocities in the contracted section.

Local Scour:
- Involves removal of material from around piers, abutments, spurs and embankments.
- Occurs due to high velocities and flow disturbances such as eddies and vortices (Figures 10.30 and 10.31).
The effects of natural, contraction, and local scour are generally additive where they occur at the same location, except that the existence of one may affect the magnitude of another (Figure 10.32). General degradation in the stream also may be occurring naturally or as a result of development. Aggradation and degradation should be considered in assessing the potential effects of natural, general and local scour because changes in flow conditions, sediment transport or debris can significantly change the scour potential at the water opening.
The rates of scour in different materials and under different flow conditions depend on erosive power in the flow, erosional resistance of the material, and a balance between sediment transported into and out of the section. In erosion-resistant materials, final, worst case, or equilibrium scour may not be reached in any one flood event but may develop over a series of events.

The resistance of fine-grained, cohesive materials to erosion depends on a number of physiochemical and environmental factors, since bonding between particles must break down before erosion can occur. Weak sandstones and weakly cemented sands and gravels may become cohesionless because the cementing material may dissolve. Laminated materials, such as hard shales, may tend to delaminate during floods (Neill, 1973).

A channel bed may be underlain by strata of varying resistance to scour. Where a comparatively resistant layer overlies more easily eroded material, it may be advisable to take precautions against scour penetration by careful design and construction of piers, by adoption of protective measures, and by avoiding severe contraction of the flow. Erosion-resistant strata may limit the potential depth of scour, and careful evaluation of bed stratigraphy may reveal opportunities for savings in foundation costs if the structure is designed and constructed to preserve the integrity of the resistant strata.

(a) Contraction Scour. The depth and area of contraction, or general scour at a waterway opening may be affected by any or all of the following factors (Neill, 1973):

- Slope, natural alignment and shifting of the channel.
- Type and amount of bed material in transport.
- Nature and occurrence of flood events.
- Accumulations of ice or other debris.
- Construction and/or realignment of flow due to the stream crossing.
- Layout and geometry of training works.
- Geometry and orientation of piers.
- Classification, stratification and consolidation of bed and subbed materials.
- Placement or loss of riprap and other protective materials.
- Natural or man-made changes in flow or sediment regimes.
- Failures, such as collapse of a nearby structure.
- Growth of vegetation in the channel or floodplain.

Countermeasures: Experience, Selection, and Design Guidance, Second Edition (FHWA, 2001a), and HDS-6, River Engineering for Highway Encroachments (FHWA, 2001c) contain the state-of-knowledge and practice for dealing with scour at highway bridges and the procedures for designing new, replacement and rehabilitated bridges to resist scour.

Numerous studies of scour at bridge waterways have been conducted, and, as yet, no definitive work applicable to all situations is advisable. The peculiarities and applicability of each method should be understood when scour potential is estimated.

General scour occurs at contractions because of increases in velocity and bed shear stress and a corresponding increase in stream power. Scour continues so long as more bed material is transported from the contracted section than is transported into the section.

For the case where flow is confined to the stream channel, Figure 10.33 can be used to illustrate the equilibrium condition. At equilibrium, the total sediment transport through the contracted section, \( Q_{S2} \), is equal to the total sediment transported into section 2 from section 1, \( Q_{S1} \) (Highway Research Board, 1970).

Equilibrium can be reached by erosion of the banks at section 2, by scour in the stream bed, or by a combination of stream bed and bank erosion. The stream will attempt to reach equilibrium by bank erosion so that \( W_1 = W_2 \) where piers or embankment ends form a contracted section in the channel of a stream which transports heavy bed loads or has an erosion-resistant bed.

Where stream banks are armored, as with riprap, the equilibrium area of general scour at a contracted section can be conservatively estimated by assuming stream bed scour and a trapezoidal cross section that will make the average velocity through the waterway opening equal to the estimated average channel velocity outside the constriction (Neill, 1973). It should be noted, however, that in bends the scour may assume a triangular rather than trapezoidal shape with the deepest scour near the outer bank. In some cases, it may be desirable to assume an envelope of worst scour, assuming that the deepest scour can shift (Figure 10.34).
An equation for computing contraction scour where all overbank flow is forced into the channel section by the contraction and the channel itself is contracted, either by piers or by the embankment ends, is provided in *Synthesis of Highway Practice 5, Scour at Bridge Waterways* (Highway Research Board, 1970) and *Scour at Bridge Crossings* (Laursen, 1958). The equation is based on a balance of sediment supply and sediment transport capacity. It will not be quoted here because the applicability can best be assessed by study of material presented in the above references. Inherent in the procedure is an assumption that scour will be limited to the stream bed. Experience with such contractions indicates, however, that some of the general scour will occur as bank erosion when the stream channel is constricted by embankment or piers.

General scour does not occur as a result of contraction by embankment ends where abutments are set back from the stream banks a sufficient distance to avoid diverting overbank flow into the channel. However, general scour can occur at such waterway openings if piers occupy a significant portion of the channel.

General scour at auxiliary openings on floodplains differs from scour in the channel because the sediment transport from upstream is usually small (Laursen, 1962 & 1963).

(b) Local Scour. Local scour occurs at embankment ends, piers, spur dikes, and similar obstructions in the flow. It is caused by the vortex of fluid which results from the pileup of water on the upstream edge and subsequent acceleration of flow around the obstruction. Scour continues so long as sediment transported out of the scour hole exceeds the sediment transported into the hole. As the depth of the hole is increased, the strength of the vortex is reduced and scouring ceases as sediment transport rates become equal.

Local scour at abutment ends and at piers located in the floodplain away from the stream bank should be considered clearwater scour because no bed load is present. The depth of maximum scour at piers located in a stream will depend on the presence or absence of dunes. If dunes are present, the depth of scour will vary with time. Sediment transport in the trough of a dune is small, and local scour will be greater than when the rest of the dune passes the local scour hole and sediment supply is large. Maximum scour depths, as determined from laboratory tests, are commonly 30% greater than equilibrium scour.

The shape of the upstream part of a local scour hole at piers is a truncated cone with the cone angle approximating the angle of repose of the sediment. Downstream slopes are flatter where the roller mixes with other flow and a bar is formed downstream of the hole. The lateral extent of the scour hole can be estimated from the angle of repose of the material and the depth of scour.

recommended. It also should be noted that appropriate adjustments to computed scour depths should be made for pier shape.


(c) Natural Scour. A long reach of a stream in flood can be considered to be a series of contractions and expansions. During the rising flood, scour occurs in the contractions, deposition occurs in the expansions; during the falling stages, the opposite occurs. Thus, the stream bed can be extremely erratic and changeable from natural scour and filling in the channel.

Natural scour also occurs as meander bends progress downstream. As a meander moves, the outside bank and bed at the bend is scoured and filling occurs on the inside of the bend. Significant scour occurs at the outside of bends during floods even where the bend can be considered to be in a relatively stable location. Even in straight reaches, natural scour occurs with the migration of bed forms and shifting of the thalweg.

Wherever possible, estimates of maximum natural scour should be based on soundings at the site or at comparable sites on the same stream during floods which result in representative scour. Such measurements are usually difficult to obtain because of hazards associated with making soundings during floods and because the floods of interest are relatively rare occurrences. Natural scour may be estimated at existing bridge sites by reviewing stream bed elevation changes found in bi-annual inspection reports.

Borings, density logging and other geotechnical methods of investigating the sub-bed stratigraphy have been suggested as methods of inferring past scour levels. The lowest level of recent scouring may be detected by differences in density, the line of demarcation between the recently deposited material and the older bed material. Since the interpretation of geotechnical data is necessary, the estimation of maximum scour by these methods is subject to considerable uncertainty (Neill, 1973).


(d) In-Stream Borrow Areas, Commercial Mining, and Dredging for Navigation and Flood Control. In-stream borrow is often used as a source of good quality fill material by highway contractors. Commercial mining of sands and gravels in streams also is common because the material is clean, often well-graded, and the supply is replenished by the stream. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause scour at the bridge (Figure 10.35). Scour occurs upstream of the borrow because of the increased gradient of the stream bed. The bed load of the stream will be deposited in the borrow area, and scour occurs downstream as the stream regains its bed load.

In-stream borrow may not pose a scour hazard to a bridge, depending upon the amount of material removed from the stream, the effects of the borrow area on flow directions, the location of the borrow area, the size of the stream, and the sediment transport capacity of the stream. Many borrow areas have been filled without detriment to nearby bridges during a moderate rise in large streams which carry a large sediment load (Figure 10.36).
In-stream mining for aggregates and dredging for navigation and flood control can be extremely damaging where so much material is removed from the stream bed that all of the incoming sediment supply is trapped and degradation of long reaches occurs (Figures 10.37(a) and 10.37(b)). At some locations, dredging may be necessary, or commercial mining cannot be terminated either by legal action or by purchase. In these cases, measures to stabilize the stream bed elevation and the stream bank may be necessary, or pier and abutment foundations must be set below the expected future elevation of the stream bed.

\textbf{(e) Combined Effects of Natural, Local, and Contraction Scour.} Two methods for estimating total scour at a crossing site are provided in Section 10.9.C. and by Chapter 10 of the \textit{Model Drainage Manual} (AASHTO, 2005). Method 1 is used when stream bed armoring is of concern, more accurate contraction scour estimates are necessary, or deposition is expected and is a primary concern. Method 2 is used when armoring is not a concern or insufficient information is available to permit evaluation or where more accurate estimates are not necessary.
There are large uncertainties in scour estimates. The following factors should be taken into account when using the final estimate of total scour for the design of piers and abutments (Neill, 1973):

- The reliability of hydrologic, hydraulic and geotechnical data.
- The probability of exceedance of design floods.
- The foundation type and its susceptibility to failure in the event that estimated scour is exceeded.
- The consequences of total or partial pier failure.
• Agency experience on the same or comparable streams.


Hydraulic engineers most commonly have been concerned with flood flows in the design of stream crossings. The importance of low flows and the effects of alterations to low flows have been recognized in recent years. Restoration of Fish Habitat in Relocated Streams (FHWA, 1979a) and HDS-6, Highways in the River Environment (FHWA, 2001c) contain a wealth of information on methods of stream rehabilitation.

Wetlands often are associated with floodplains, and special attention is warranted so as to cause minimal effects on the functions of wetlands. Ecological Effects of Highway Fills on Wetlands: Research Report (Transportation Research Board, 1979b) and Ecological Effects of Highway Fills on Wetlands: User's Manual (Transportation Research Board, 1979c) are suggested as guides for use in analyzing the effects on wetlands associated with stream crossings.

Sections 10.1.D. and 10.3.H. contain information on environmental planning and data acquisition which is valuable in establishing criteria for designs to protect the water environment.

7. Economic Analysis. The objective in the design of a stream crossing system is to select the alternative which satisfies all engineering and environmental criteria and has the least total cost to construct, maintain and operate. This objective also is applicable to other components of a highway. It presumes, however, that least total cost design also will satisfy political and social considerations and emergency vehicle and emergency evacuation needs, and that budget allocations are unlimited. In the real world, political and social considerations and emergency needs affect decisions daily. Budgetary constraints exist in that all highway needs cannot be met instantly, and every decision regarding the selection of an alternative design affects the timing of projects to fill needs of lesser priority. Alternative selection also has budgetary implications in that the highway agency accepts the maintenance and operation costs, repair and reconstruction risks, and liability associated with the alternative. The decision, therefore, should be based on the best information that can be developed regarding capital costs and probable future costs. Where this information is available, the cost to the public of choosing other than an optimum design can be identified and dealt with rationally.

When standard recurrence intervals are prescribed for design, the engineering and economic analyses consist of selecting the profile and waterway opening alternative which will meet that standard at the least total expected cost and conducting studies to determine whether a higher standard is warranted for that particular crossing. It also may be desirable to extend the analyses to other exceedance probabilities to provide information on the incremental costs of designing for a different probability of exceedance.

Freeboard above stated recurrence interval floods and fixed crossing profiles both require determination of the largest flood that will flow through the waterway opening(s). This is necessary in order to be able to estimate expected future costs of flood damages to the highway, incremental damages to other property from backwater, and the expected future costs of traffic interruption and detours. The economic analysis consists of selecting the alternative which meets the constraint and results in the least total economic cost. Documentation of the analysis should include the economic costs of designing with the constraints imposed by the fixed gradeline and freeboard.

Hydraulic Design of Bridges with Risk Analysis (Schneider & Wilson, 1980) and HEC-17, The Design of Encroachments on Flood Plains Using Risk Analysis (Corry et al., 1980) are recommended for techniques of
economic analysis of stream crossing systems. Selected references listed in HEC-17, *The Design of Encroachments on Flood Plains Using Risk Analysis* (Corry et al., 1980) are recommended for further reading and study.

C. **Protective and Preventive Measures.** Numerous measures are used at stream crossings to protect the highway facility from flood damages. These may be categorized as measures intended to prevent such damages, such as riprap to prevent or inhibit stream bank erosion, and measures intended to protect the highway facility from flood damages, such as deep pier foundations to protect a bridge against severe scour. Measures generally in use are more easily explained by discussing elements of the crossing rather than by the categories of protective and preventive measures.

The principles of economic analysis can be extended to measures used to protect the crossing system to the extent that damages are predictable. For example, in the study of a crossing system, a question may arise regarding the economic feasibility of providing slope protection for the downstream side of the highway fill. Considering capital costs for slope revetment, the cost to replace lost pavement and embankment, and the cost of traffic delays while repairs are made, revetment may prove to be justified for the alternative having a 10% probability of suffering severe damage and not justified for the alternative with a 4% probability of suffering extensive damage from overtopping. Similar logic can be applied to slope protection at embankment ends, the use of spur dikes, stream bank protection, pier foundations (see the following sections), and numerous other appurtenances and components of the stream crossing system.

Actions of alluvial streams often are unpredictable, especially regarding the time which will pass before an anticipated event will occur, such as the passage of a meander bend. Considering the large investments that usually are involved in river training and control, it is sometimes prudent to adopt a "wait-and-see" strategy. Knowledge of the stream and its response to floods is the best guide for determining when protective works should be installed. When protection is needed, whether at the time of construction of the crossing system or at a later date, the cost of providing the control measures should be compared with the costs of traffic delays and repairing damages that would be incurred.

1. **Pier Foundations.** Preventive and protective measures both are used at piers to avoid damage by scour, which is the primary flood-related concern at piers. Preventive measures generally are intended to minimize the flow disturbance caused by the pier and thus to minimize local scour at the pier, or they are intended to inhibit or arrest local scour at the pier at an acceptable depth.

Protective measures consist of various types of foundations at depths secure from failure by scour. Protective and preventive measures both are used at some locations, particularly where cofferdams are used and scour during construction could greatly exceed anticipated scour after the cofferdams are removed.

Stream degradation and general and natural scour at the crossing site should not be overlooked whether preventive, protective or both measures are used to secure the pier against scour. The effect of piers in contracting the flow and causing general scour should be considered in selecting span lengths. Debris and ice accumulations upstream of a bridge can cause great exaggeration of general scour even during relatively minor floods (see Figure 10.15). Submergence of the bridge superstructure also will contract flow and cause increased general scour. The depth of any bridge superstructure, including parapets, that can become submerged at any time is very important to the security of the bridge. Open rails, no curbs, and a substructure that is as slender as possible should be used where a chance of submergence exists.

While it may not be possible to subject the design of pier footings to a rigorous economic analysis because of uncertainties in scour predictions, a good case can be made for extreme conservatism in foundation design (Highway Research Board, 1970; Laursen, 1970). The capital costs of providing a foundation secure against scour usually will be small when compared to the risk costs of scour-related failure.

(a) **Preventive Measures at Piers.** The preventive measures most commonly recommended to minimize flow disturbance and resultant scour are circular piers and a circular nose on an elongated pier oriented with the flow direction. *Synthesis of Highway Practice 5, Scour at Bridge Waterways* (Highway Research Board, 1970), and *Scour at Bridge Crossings* (Laursen, 1958) provide information on the effects of pier shape and pier orientation on local scour depths.
Pier foundations also may be considered in the category of measures used to minimize flow disturbances and resultant local scour. Figure 10.38 illustrates three foundation treatments that are intended to suppress the vortex and thereby reduce the sediment transport capacity out of the scour hole (Neill, 1973). An increased width of pier projected normal to flow direction in the stream tends to increase local scour; therefore, measures illustrated in Figure 10.38 should be below the expected general scour level. The foundation or riprap provided to suppress the vortex should be extended to the expected limits of the vortex action in order to minimize the chance of undermining. *Guide to Bridge Hydraulics* (Neill, 1973) suggests that the measure used to suppress the vortex should extend out from the pier for a distance at least equal to the diameter or width of the pier shaft, and up to twice this distance is acceptable.

A very conservative approach is recommended in estimating the extent of general or natural scour for the use of vortex suppression measures. A conservative approach also is recommended in the design of the suppression measure and judging its probable effectiveness. These measures will not control degradation, general or natural scour, and may fail to control local scour. They may even make it worse if the enlarged section of the pier is exposed to the main current of the stream.
Other preventive measures that have been used usually are intended to minimize the effects of general or natural scour at the bridge and to protect the bridge from progressive stream degradation. These measures usually consist of hardening the perimeter of the waterway at the bridge with riprap, soil cement or concrete. At some locations, drop structures or grade-control structures of concrete, sheet piling, gabions or grouted riprap are used in or downstream of the bridge waterway opening to halt the progression of stream degradation. All of these measures are susceptible to failure by piping and by stream degradation which exceeds anticipated degradation to the extent that the downstream side of the structure is undermined. Water bypassing the structure because of erosion of the stream banks into which the structure is tied also can cause failure of grade-controlled structures.
Generally, flexible drop structures are less susceptible to failure from undermining and piping because of their self-healing qualities. Rock must be large enough to prevent displacement by high velocity flow or wire enclosed to form an articulated structure. The effects of piping can be controlled with gravel or geotextile filters. An apron should be provided downstream of the structure to prevent undermining by scour. Structures intended to protect bridges from damage by degradation often involve substantial capital investment, and they should be designed to withstand the static and dynamic hydraulic forces that will be imposed on them. Failure of the drop structure could involve the loss of investment in the bridge it was intended to protect as well as the investment in the drop structure. Physical modeling of design alternatives may be warranted in many instances. Figure 10.39 shows the construction of a drop structure of grouted riprap between concrete walls. Figure 10.40 shows a more conventional structure of sheet piling and rock riprap.

Caissons are used in large alluvial rivers such as the Mississippi, Missouri, Arkansas and Red Rivers, where extreme scour is expected. In many cases, woven mattresses of willow or other materials are used for scour prevention during and after construction. The rock used on the woven mattress must be large enough to resist rolling during extreme floods if the mattress is intended to afford protection after construction is completed.

(b) Protective Measures at Piers. Several foundation types are used at locations where protection against scour is provided by deep foundations. Spread footings located below anticipated general and local scour are illustrated in Figure 10.17. These generally are used in relatively stable streams whose beds are naturally armored with boulders and cobbles, and in streams where deeper foundations are extremely difficult to construct because of boulders below the stream bed. Spread footings also are used where the pier can be founded on sound rock.

Serious problems and failures have occurred at numerous locations where spread footings were used in seemingly stable streams and the footings supposedly were placed at secure depths, or the difficulties in driving piles or using drilled shaft foundations were seemingly insurmountable. Figure 10.18 illustrates such a location.
Figure 10.39  Drop Structure of Concrete Walls and Grouted Riprap

Figure 10.40  Grade Control Structure of Sheet Piling and Rock Riprap
Shales, sandstones, and other soft rocks can be eroded, go into solution, or delaminate when exposed. Problems have developed at bridges founded on these erodible rocks. Drilled shafts or drilled piers offer a practical solution to the provision of a foundation secure against scour in these rocks as well as in most gravels and clays where piling cannot be driven. Figure 10.18 shows typical drilled shaft designs.

Piling driven deep below anticipated scour depths affords protection against scour where the piles are of sufficient length to support the structure after scour. The piling should have reserve bending strength, or battered piles should be utilized to resist the forces of ice and debris in the scoured condition. Load-bearing capacity based on skin friction will be different after scour occurs. Large piling sizes or concrete filled pipe piles which depend on point-bearing capacity will increase the ability of the piling to withstand scour in many cases. Figure 10.19 shows typical pile foundation designs.

Some protection of pile footings may be provided by sheet piling surrounding the footing. The pile cap should be located below expected natural or general scour or the enlarged section will cause exaggerated local scour because of the large area subjected to the main current of the stream. The pile cap should be extended to suppress the vortex action at the pier; allowing the sheet pile to project above the cap also may help to suppress the vortex. Figure 10.20 shows a sheet pile-protected pile footing.

2. Abutments and Approach Fills. Embankments projecting into floodplains are susceptible to scour damage from flow concentrations at the abutments, along the embankment where efficient channels are formed by right-of-way clearing and borrow excavation, and by flow over the roadway. Measures commonly used to inhibit scour or protect the embankment from damage by scour include revetments of concrete, rock, sacked cement and sand or soil cement, sheet pile retaining walls, spur dikes, toe dikes, guide banks, and other dikes of various descriptions used to impede flow along the projecting embankment.

(a) Protective and Preventive Measures Along Embankments. Embankments that encroach on flood plains are most commonly subjected to scour and erosion damage by overflow and by flow diverted along the embankment to waterway openings. Damage also can occur from the redistribution of flow in the floodplain downstream of the waterway.

The incidence of damage from flow along approach fills probably is highest in wooded floodplains where the rights-of-way are cleared of all trees and where borrow areas are established upstream of the embankment. Damage to approach fills usually is not severe, but scour from the flow contraction at the abutment will be greater than would otherwise occur. Preventive measures along the embankment, therefore, usually are intended to provide for security at the abutment rather than for the approach fill.

The most commonly recommended preventive measures are abstention from right-of-way clearing beyond the fill slope and use of borrow spaces which will not cause damage to the highway facility during floods. Other measures which have been used are pervious dikes of timber or finger dikes of earthen material spaced along and normal to the approach fill to impede flow along the embankment. Spur dikes and toe dikes at the abutment also serve this purpose. In wooded floodplains, dikes used to impede flow or align flow with the waterway opening should extend into wooded areas. Riprap or other revetment is not usually necessary to protect the fill from lateral flow except where the fill is of cohesionless material or the crossing is on a relatively severe skew with the floodplain. Where there is adequate rainfall, plating the fill slope with topsoil and a good vegetative cover usually will provide adequate protection for fills of cohesionless material.

The measure commonly used to protect fills projecting into floodplains against erosion and scour from overtopping is to establish a "freeboard" in the crossing profile above design flood elevations. There is a probability, however, that the design will be exceeded and a probability that the highway facility will be overtopped. The practice of ignoring these probabilities may lead to design imbalance in that floods which exceed the design flood are forced to pass through the waterway opening. The frequent consequences of floods significantly in excess of the design flood being forced through the waterway opening are failure of abutments and end spans of bridges from scour and destruction of approach fills at the abutment. The alternative to "freeboard" is a crossing profile which has an acknowledged probability of overtopping and waterway opening designed considering the risk of damage from the flood that must flow through the opening. If practicable, overflow should occur at a location or locations removed from the bridge in order to minimize the chance of damage to the bridge. Where the probability of overtopping
is acknowledged and planned for, documented justification of money committed to construction and flood damage repair can be provided.

Preventive measures used to protect approach fills from erosion and scour from overflow are revetments usually of rock, wire-enclosed rock, or concrete. Rock riprap should be of adequate size to prevent displacement, dumped rock is preferred over hand-placed riprap, and a filter of granular material or filter cloth should be provided. Economic justification can be established by a comparison of the capital cost of construction with the expected annual traffic costs and repair costs without revetment.

Preventive measures also are needed at some crossings to protect the embankment against wave action, especially at reservoirs. Riprap of durable, hard rock usually is used at these locations. The top elevation of the rock required is dependent upon storage and flood elevations in the reservoir, and wave height, which can be computed using wind velocities and the reservoir fetch. The *Model Drainage Manual* (AASHTO, 2005) is recommended for design guidance.

(b) Protective and Preventive Measures at Abutments. Scour at abutments usually is caused by turbulence from diversion of floodplain flow into the waterway opening by a fill embankment. For abutments on a fill slope, the scour usually occurs at the upstream corner of the embankment and, dependent upon the degree of contraction, flow depths and volume flow rate in the floodplain may extend to the first or second pier of the bridge (Figure 10.41). Protective measures used at abutments consist of vertical abutment walls, sheet pile toe walls, and deep foundations of piles or drilled shafts. Vertical abutment walls founded below anticipated scour depths will protect bridge ends and the embankment if the walls are extended around the fill slopes to the depth of anticipated scour.

Figure 10.41 Scour Abutments and Adjacent Pier (after Highway Research Board, 1970)

Sheet pile toe walls sometimes are installed to repair scour damage after a flood. They commonly are used where rock is not available or access for placing rock is difficult. Figure 10.42 illustrates the use of deadmen for anchoring to provide an effective structural system. Other measures such as a structural cap or steel H-piles also may be necessary.

Preventive measures commonly used are revetments of rigid or articulated concrete, sacked concrete, or spur dikes or guide banks to align flow with the waterway opening and move scour away from the bridge end, and toe dikes to prevent lateral flow from eroding the corner of the downstream side of the embankment.

Revetment at the abutment usually is placed on the slopes under the bridge end and around the corners of the embankment to guard against progressive erosion of the embankment. Revetment on the fill slope does not inhibit scour from the flow contraction and, therefore, is susceptible to failure from undermining. The revetment should be continued down below the level of expected scour to protect it and the embankment from failure (Figure 10.43). An alternative that can be used on cohesionless soils is
a flexible apron extended to the limits of the expected scour (Figure 10.44). The apron tends to be self-healing since it will settle into any area that scours and inhibit further scour. Flexible aprons may not work as well on cohesive materials because the steep faces of failures are not protected by the material of the apron (Neill, 1973). Materials commonly used for flexible aprons are rock riprap, articulated concrete and wire-enclosed rock.

Figure 10.42  Sheet Pile Toe Wall (after Highway Research Board, 1970)

Figure 10.43  Cellular Concrete Revetment on Filter Cloth Revetment Toed in to Prevent Undermining
Sheet piling also is used at the revetment toe in lieu of flexible aprons or extending the revetment below the expected scour. This would appear to have merit where rigid revetment is used, severe scour is expected, and materials are not available for a flexible apron. Measures are necessary to ensure the structural integrity of the sheet-piling structure after scour.

Guide banks are appendages to the highway embankment at the bridge abutment. They usually are smooth extensions of the fill slope on the upstream side. The twofold purpose of guide banks is to align flow from the floodplain with the waterway opening and to minimize scour at the abutment by moving the scour-causing turbulence upstream to the end of the guide bank (Figure 10.45). Guide banks are usually of earthen embankment but are sometimes of rock where an excess of this material is available. Revetment is advisable for protection of the dike where scour is expected to occur, although a failure at the upstream end of a guide bank usually does not immediately threaten the bridge end. Clearing around the end of the dike should be kept to a minimum in wooded floodplains in order to enhance the effectiveness of a guide bank in reducing turbulence. A small culvert through the dike in lieu of a drainage channel around the end to provide for local drainage also serves to minimize the turbulence of mixing flows from different directions.

The suggested shape of Guide banks is elliptical with a major to minor axis ratio of 2.5:1 (Bradley, 1978). The length suggested varies with the ratio of flow diverted from the floodplain to flow in the first 30 meters (100 ft) of waterway under the bridge. The shape suggested is based on laboratory experiments and the length on model and field data (Bradley, 1978). Optimum shape and length undoubtedly will differ for each site and possibly for each flood at a site. Field experience has shown that the recommended elliptical shape is usually quite effective in reducing turbulence. If practical reasons require the use of another shape such as a straight dike, more scour should be expected at the upstream end of the dike.

Toe dikes sometimes also are needed downstream of the bridge end to guide flow away from the structure so that redistribution in the floodplain will not cause erosion damage to the embankment downstream at the ridge end. This need usually occurs in wooded floodplains where the right-of-way is cleared of trees, especially if some portion of the floodplain at some distance removed from the waterway opening has been cleared of trees (Figure 10.46). Toe dikes similar to those described for use upstream of the waterway opening are appropriate, but the shape is of less importance. The length should be sufficient to force flow into the forested area downstream, if present, or long enough to move scour away from the bridge end if the floodplain is not wooded. In some instances, downstream clearing or borrow excavation within the right-of-way has caused erosion downstream at the bridge end and changed the distribution of flow downstream of the highway facility. Imagine a natural ridge in the wooded area in Figure 10.46 between the stream and the cleared field. Prior to construction of the highway facility, the cleared field would not experience floods very often. Use of the right-of-way for borrow could divert flow to the field.
during any flood event that is out-of-banks. Restoration of the divide within the right-of-way would alleviate flooding caused by the borrow.

Figure 10.45  Guide Bank (after Highway Research Board, 1970)

3. Bank Stabilization and River Training. Bank stabilization and river training devices are preventive measures installed to inhibit erosion and movement of stream banks. The measures may be necessary as defense against actions of the stream which threaten the highway crossing or to protect the stream banks and the highway from an anticipated response to construction of the highway. Table 1 in *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d) is convenient for making qualitative assessments of stream responses to highway crossings.

Various materials and devices are used for bank stabilization and river training, including rock riprap, concrete lining, wood, steel or rock jetties, steel or concrete jack fields, wire fences, timber bulkheads, articulated concrete mattresses, and guide banks, dikes and spurs usually of earth and rock. The choice of the appropriate device or devices for use depends on the total action of the river for a distance upstream and downstream. Study of long reaches will help to avoid futile attempts at localized control where the river is in the midst of changes that will bypass the control measure or make it unnecessary. Regardless of the size of the stream and the control measure used, stream response to the installation of the measure must be considered. For instance, bank stabilization at a crossing can cause scour in the bed of the channel or redirect the current of the stream toward an otherwise stable bank downstream of the stabilized reach.
Bank stabilization and river training is a very specialized field which requires intimate knowledge of the stream and its propensity to change, knowledge of the bed load and debris-carrying characteristics of the stream, and experience and experimentation at similar sites on the same or similar streams. Design is, to a large extent, an art, and many questions concerning the relative merits of various measures have not been answered definitively (AASHTO, 2000d; ASCE, 1965; FHWA, 1991; California Division of Highways 1960; Neill, 1973; USACE, 1969, HEC-18, HEC-20, HEC-23, and HDS-6 are recommended reading).

The following general principles for the design and construction of bank protection and training works are adapted from *Guide to Bridge Hydraulics* (Neill, 1973):

- **a.** The cost of the protective measures should not exceed the cost of the consequences of the anticipated action of the stream.

- **b.** Designs should be based on studies of channel trends and processes and on experience with comparable situations; the ultimate effects of the works on the natural channel both upstream and downstream should be considered.

- **c.** Site reconnaissance is imperative; reconnaissance may be by on-site inspection, aerial reconnaissance, and/or aerial photographs taken over a period of years.

- **d.** The possibility of using physical model studies should be considered at an early stage.

- **e.** The works should be inspected periodically after construction with the aid of surveys to check results and modify the design, if necessary.

- **f.** In lieu of maintaining an existing bridge in trouble, consideration should be given to an alternate location away from the river hazard.

The "principle of expendability," paraphrased from *Bank and Shore Protection in California Highway Practice* (California Division of Highways, 1960), can be added to the above list.
g. Don't lose sight of the fact that the objective in installing bank stabilization and river training measures is to protect the highway. The protective measures themselves are expendable.

The effectiveness of protective and training measures in many alluvial streams, and indeed, the need for the measures, may be short-term because the stream will move to attack at another location or even outflank the installation. Extensive works which would be required for long-term effectiveness would violate the first principle above; therefore, they usually are not considered. The alternative to extensive training works is a continuing effort to protect the highway by successive installations intended to counter the most recent actions of the stream. Each successive installation usually is tested against the first principle considering the ability of the highway at that point in time to provide the desired traffic service. In effect, this is an application of the watchful waiting suggested in Section 10.5.C., but the implications are greater than the anticipated future need of a "one-time" installation. In some cases, if the need, the time of need, and the cost of future installations could be anticipated, a cost analysis might show that a bridge long enough to allow the stream to follow natural processes would be in the public interest. Planned-for extension of the bridge after (and if) the stream destroys an approach roadway might be a viable alternative to a long bridge initially.

Where measures are needed to protect a highway facility from anticipated actions by a stream, the possibility of a cooperative project with another governmental agency, particularly the USACE, should be investigated. Other agencies such as NRCS, U.S. Fish and Wildlife, and PA DEP, have responsibilities and authority to undertake stream stabilization efforts, and mutually beneficial projects may be possible.

4. Buoyant, Drag, Debris and Ice Forces on Bridge Superstructures. The first line of defense against hydraulic forces imposed on a submerged or partially submerged bridge superstructure is to locate the bridge at an elevation where the probability of submergence is small. Obviously, this is not economically or physically practical at many locations.

The second line of defense is to make the superstructure as shallow as possible. Box girders which would displace great volumes of water and have a relatively small weight compared to the weight of the water displaced are not a good design alternative unless the probability of submergence is very small. Solid parapets and curbs which increase the effective depth of the superstructure can give increased buoyancy over that of open rail designs. If submerged, the increased effective depth of the superstructure will cause increased general scour, and drag forces on the substructure will be much greater than with open rails.

The third line of defense is to provide a roadway approach profile that will be overtopped prior to bridge superstructure submerging. This measure will reduce the probability of submergence for a bridge at a given elevation. It also will afford relief against bridge end scour, general scour and local scour which are accentuated by superstructure submergence, and debris clogging the waterway openings.

Where there is even a small probability of total or partial submergence, superstructures should be securely anchored to the substructure to resist buoyant, drag, debris and ice forces. Air holes also should be provided through each span and between each girder to reduce the volume of displaced water by release of air that would otherwise be trapped under the superstructure.

D. Dolphins and Fender Systems. Dolphins and fender systems are two slightly different structural systems with the same purpose. In reference to bridges, this purpose is to protect piers, bents and other bridge structural members from damage due to collision by marine traffic. Dolphin types range from simple pile clusters to massive concrete structures. Fender system types are less variable, consisting usually of pile-supported stringers, as shown in Figure 10.47.

The need for dolphins and fender systems often can be eliminated by spanning smaller rivers or by judicious pier placement. Construction costs of long spans may be economically unattractive when compared with shorter spans; however, when all costs are considered, both construction- and maintenance-related, the long span solution may be the most attractive. The bridge designer should receive guidance from the engineer in the form of estimated depths of scour for the dolphin and fender systems. This information influences fender and dolphin pile lengths, diameters and spacing, thereby affecting cost comparisons. Also, estimated debris removal costs may be a factor. The bridge designer should consider these factors, as well as maintenance costs due to collisions, when making decisions on span lengths.
Dolphin and pier scour may combine to cause deeper scour than either would cause separately. In some cases, fender systems may "shield" bridge piers, reducing velocities and scour at the pier; however, this shielding effect can vanish, or be modified, if the fender system is lost due to collision or unforeseen scour problems. Pier and fender systems introduced into relatively narrow rivers may cause general scour between the fender systems. This scour is usually greatest near the downstream end of the system.

10.6 HYDRAULICS OF BRIDGE OPENINGS

A. Hydraulic Performance of Bridges. The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities should be analyzed using a computer program such as HEC-RAS, HEC-2, or WSPRO. Alternative methods of analyzing bridge hydraulics are discussed in this section, but emphasis is placed on the use of HEC-RAS.
The hydraulic variables and flow types as defined by FHWA are defined in Figures 10.48 and 10.49:

- Backwater \( (h_1) \) is measured relative to the normal water surface (NORMAL W.S. or N.W.S.) elevation without the effect of the bridge at the approach cross section (Section 4). It is the result of contraction and re-expansion head losses and head losses due to drag from bridge piers. Backwater also can be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 10.49.

- Type I flow consists of subcritical flow throughout the approach, bridge and exit cross sections and is the most common condition encountered in practice.

- Types IIA and IIB both represent subcritical approach flows which have been choked by the contraction resulting in the occurrence of critical depth in the bridge opening. In Type IIA, the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In Type IIB, the critical water surface elevation in the bridge span is higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible.

- Type III flow is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.
Figure 10.48 Bridge Hydraulics Definition Sketch

- **A**: Profile on Stream C
- **B**: W.S. with Backwater
  - **Q_c**: Normal W.S.
  - **Q_a**: Actual W.S. on C

- **C**: Section 3
  - W.S. with Backwater
  - **Q_c**, **Q_a**, **Q_{b,c}**

- **D**: Plan at Bridge
  - Flow through sections 1 to 4
B. Methodologies. No single method is ideally suited for all situations. If a satisfactory computation cannot be achieved with a given method, an alternate method should be attempted. However, it has been found that, with careful attention to the setup requirements of each method, essentially duplicate results usually can be achieved using both momentum and energy methods.

For the purpose of presentation, the computational methodologies used in the Hydrologic Engineering Center's HEC-RAS computer program will be utilized to describe the bridge hydraulic computations.
Chapter 10 - Bridge Hydraulics

1. Two-Dimensional Modeling. The water surface profile and velocities in a section of river are often predicted using a computer model. In practice, most analyses are performed using one-dimensional methods such as the standard step method found in HEC-RAS, HEC-2, or WSPRO. While one-dimensional methods are adequate for many applications, these methods cannot provide a detailed determination of the cross-stream water surface elevations, flow velocities or flow distribution.

Two-dimensional models are required when the basic assumption that the water surface elevation and energy grade line elevation are constant across the cross sections is no longer valid, or when it is anticipated that flow velocities in the y and z directions may influence the hydraulic analysis. In one-dimensional modeling, flow velocities are assumed to be parallel in the x direction at any given cross section. Two-dimensional models are more complex and require more time to set up and calibrate. They may require more field data then a one-dimensional model and, depending on complexity, may require a little more computer time.

"Bri-Stars" is a quasi-two-dimensional model capable of computing alluvial scour/deposition through subcritical, supercritical and a combination of both flow conditions involving hydraulic jumps. It is capable of simulating channel widening/narrowing as well as local scour due to highway encroachments. "Bri-Stars" has a bridge component which allows the computation of hydraulic flow variables and the resulting scour. It also includes a companion expert system program which allows the classification of streams by their morphological properties.

The USGS has developed a two-dimensional finite element model for the FHWA that is designated FESWMS. This model has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist. This two-dimensional modeling system is flexible and may be applied to many types of steady and unsteady flow problems including multiple opening bridge crossings, guide banks, floodplain encroachments, multiple channels, flow around islands and flow in estuaries. Where the flow is essentially two-dimensional in the horizontal plane, a one-dimensional analysis may lead to costly over-design or possibly improper design of hydraulic structures and improvements.

SMS is a surface water modeling system that provides a windows interface to use FESWMS, RMA2 and other two-dimensional models.

2. Physical Modeling. Complex hydrodynamic situations may require physical models when:

- Hydraulic performance data are needed that cannot be reliably obtained from mathematical modeling.
- Risk of failure or excessive over-design is unacceptable.
- Research is needed.

The constraints on physical modeling are:

- Size (scale).
- Cost.
- Time.

C. Bridge Modeling Methods. There are numerous methods for estimating the hydraulic impact of bridge openings on water surface profiles. PennDOT recommends and anticipates that computer programs will be employed to perform such estimates. Generally, the designer should refer to the documentation of the specific computer program for the theory employed and operating instructions.

1. HEC-RAS Modeling. The following sections provide a general discussion of recommended modeling approaches for the USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS) computer program. The HEC-RAS computer model is capable of performing one-dimensional water surface profile calculations for steady and unsteady, gradually varied flow in natural or constructed channels. For illustration purposes, the minimum number of cross sections needed for HEC-RAS to run are shown in Figure 10.50. Additional requirements for cross section locations are described in Publication 13M, Design Manual, Part 2, Highway Design, Chapter 10. The following sections have been excerpted from the HEC-RAS Hydraulic Reference Manual, Version 3.1. For additional information and guidance, the HEC-RAS help manuals should be consulted.
Cross section 1 is located sufficiently downstream from the structure so that the flow is not affected by the structure (i.e., the flow has fully expanded). This distance (the expansion reach length, \( L_e \)) generally should be determined by field investigation during high flows. If field investigation is not possible, then alternative criteria for locating the downstream section must be used. The USGS has established criteria for locating cross section 1 a distance downstream from the bridge equal to one times the bridge opening width (the distance between points B and C on Figure 10.50, (WSPRO User Manual)). The USACE locates the downstream cross section four times the average length of the side constriction caused by the structure abutments (the average of the distance from A to B and C to D on Figure 10.50 (see HEC-2 User Manual, USACE, 1983). The expansion distance will vary depending upon the degree of constriction, the shape of the constriction, the magnitude of the flow, and the velocity of the flow.

A detailed study was completed by the Hydrologic Engineering Center entitled RE-42, Flow Transitions in Bridge Backwater Analysis (USACE, 1995a). The purpose of this study was to provide better guidance to hydraulic engineers performing water surface profile computations through bridges. Specifically, the study focused on determining the expansion reach length, \( L_e \); the contraction reach length, \( L_c \); the expansion energy loss coefficient, \( C_e \); and the contraction energy loss coefficient, \( C_c \). A summary of this research, and the final recommendations, can be found in RE-45, Hydraulic Reference Manual for HEC-RAS (USACE, 1995b).

The user should not allow the distance between cross sections 1 and 2 to become so great that friction losses will not be adequately modeled. If warning messages indicate that additional cross sections are needed between cross sections 1 and 2, then intermediate cross sections may be placed within the expansion reach in order to adequately model friction losses. The ineffective flow option can be used to limit the effective flow area of the intermediate cross sections in the expansion reach.

Cross section 2 is located a short distance downstream from the bridge (i.e., commonly placed at the downstream toe of the road embankment). This cross section should represent the effective flow area just outside the bridge.
Cross section 3 should be located a short distance upstream from the bridge (commonly placed at the upstream toe of the road embankment). The distance between cross section 3 and the bridge should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening. Cross section 3 represents the effective flow area just upstream of the bridge. Note that both cross sections 2 and 3 will have ineffective flow areas to either side of the bridge opening during low flow and pressure flow profiles. In order to model only the effective flow areas at these two sections, the modeler should use the ineffective flow area option at both of these cross sections.

Cross section 4 is an upstream cross section where the flow lines are approximately parallel and the cross section is fully effective. In general, flow contractions occur over a shorter distance than flow expansions. The distance between cross section 3 and 4 (the contraction reach length, \( L_c \)) generally should be determined by field investigation during high flows. The USGS has established a criterion for locating cross section 4 a distance upstream from the bridge equal to one times the bridge opening width (the distance between points B and C on Figure 10.50). The USACE used a criterion to locate the upstream cross section one times the average length of the side constriction caused by the structure abutments (the average of the distance from A to B and C to D on Figure 10.50). The USACE has performed a detailed study, RE-42, Flow Transitions in Bridge Backwater Analysis (USACE, 1995a). From that publication it is recommended that cross section 4 be located at one times the average length of the side constriction upstream from cross section 3.

During the hydraulic computations, the program automatically formulates two additional cross sections inside of the bridge structure. The geometry inside of the bridge is a combination of the bounding cross sections (sections 2 and 3) and the bridge geometry. The bridge geometry consists of the bridge deck and roadway, abutments, and piers, if any. The user can specify different bridge geometry for the upstream and downstream sides of the structure if necessary. Cross section 2 and the structure information on the downstream side of the bridge are used as the geometry just inside the structure at the downstream end. Cross section 3 and the upstream structure information are used as the bridge geometry just inside the structure at the upstream end.

2. HEC-2 Modeling. The water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the caused by the constriction. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge using the special bridge option are shown in Figure 10.51.

Energy losses caused by structures such as bridges and culverts are computed in two parts. First, the losses due to expansion and contraction of the cross section on the upstream and downstream sides of the structure are computed in the standard step calculations. Secondly, the loss through the structure itself is computed by either the normal bridge or the special bridge methods.

The user's instructional manual for HEC-2 should serve as a source for more detailed information for using the computer model. Numerous input/output examples are provided.
HEC-2 has its own data creation package -- COED -- that assists the user with preparing/editing input data and includes powerful online help features. A separate, stand-alone data editing program which checks for input/modeling errors, EDIT2, also is provided. Cross section, water surface profile and rating curve viewing/plotting/printing are provided using the PLOT2 program.

The normal bridge method handles the cross section at the bridge just as it would any river cross section with the exception that the area of the bridge below the water surface is subtracted from the total area and the wetted perimeter is increased where the water surface elevation exceeds the low chord. The normal bridge method is particularly applicable for bridges without piers, bridges under high submergence and for low flow through circular and arch culverts. Whenever flow crosses critical depth in a structure, the special bridge method should be used. The normal bridge method is automatically used by the computer for bridges without piers and under low flow control, even if data were provided for the special bridge method.

The special bridge method can be used for any bridge, but should be used for bridges with piers where low flow controls, for pressure flow and whenever flow passes through critical depth when going through the structure. The special bridge method computes losses through the structure for low flow, weir flow and pressure flow or for any combination of these.

A series of program capabilities is available to restrict flow to the effective flow areas of cross sections. Among these capabilities are options to simulate sediment deposition, to confine flows to leveed channels, to block out road fills and bridge decks, and to analyze floodplain encroachments.

Cross sections with low overbank areas or levees require special consideration in computing water surface profiles because of possible overflow into areas outside the main channel. Consult the User's Instruction Manual for HEC-2 for more detailed information on using this computer model. HEC-2 is considered legacy software, has been replaced by HEC-RAS since HEC-RAS has more robust algorithms in it than HEC-2. The USACE no longer supports it.

3. **WSPRO Modeling.** As for HEC-2, the water surface profile should extend upstream and downstream to points beyond the influence of the bridge. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge without guide banks are shown in Figure 10.52. The additional cross sections that are necessary for computing the entire profile are not shown in this figure. Cross sections 1, 3 and 4 are required for a Type I flow analysis and are referred to as the approach section, bridge section and exit section, respectively. In addition, cross section 3F, which is called the full-valley section, is needed for the water surface profile computation without the presence of the bridge contraction. Cross section 2 is used as a control point in Type II flow but requires no input data. Two more cross sections must be defined if a guide bank and roadway profile are specified, one at the upstream end of the guide bank, and one immediately before the defining approach cross section.
Pressure flow through the bridge opening is assumed to occur when the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening. The flow is then calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined. WSPRO can also simultaneously consider embankment overflow as a weir discharge. This leads to flow Classes 1 through 6 as given in Table 10.2.

In free-surface flow, there is no contact between the water surface and the low-girder elevation of the bridge. In orifice flow, only the upstream girder is submerged; while in submerged orifice flow, both the upstream and downstream girders are submerged. A total of four different bridge types can be treated. WSPRO is no longer supported by the USGS. The user's instruction manual for WSPRO should be used for more detailed information on using the WSPRO computer model.

<table>
<thead>
<tr>
<th>Flow Through Bridge Opening Only</th>
<th>Flow Through and Over Road Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1 - Free surface flow</td>
<td>Class 4 - Free surface flow</td>
</tr>
<tr>
<td>Class 2 - Orifice flow</td>
<td>Class 5 - Orifice flow</td>
</tr>
<tr>
<td>Class 3 - Submerged orifice flow</td>
<td>Class 6 - Submerged orifice flow</td>
</tr>
</tbody>
</table>
D. Defining Ineffective Flow Areas. A basic problem in defining the bridge data is the definition of ineffective flow areas near the bridge structure. Referring to Figure 10.50, the dashed lines represent the boundary of effective flow for low flow and pressure flow conditions. Therefore, for cross sections 2 and 3, ineffective flow areas to either side of the bridge opening (along distance AB and CD) should not be included as part of the active flow area for low flow or pressure flow.

The bridge example shown in Figure 10.53 is a typical situation where the bridge spans the entire floodway and its abutments obstruct the natural floodplain. This situation is similar to the one shown in plan view in Figure 10.50.
The elimination of the ineffective overbank areas can be accomplished by redefining the geometry at cross sections 2 and 3 or by using the natural ground profile and requesting the program's ineffective area option to eliminate the use of the overbank area (as shown in part C of Figure 10.53). Also, for high flows (flows overtopping the bridge deck), the area outside of the main bridge opening may no longer be ineffective, and will need to be included as active flow area. If the modeler chooses to redefine the cross section, a fixed boundary is used at the sides of the cross section to contain the flow, when in fact a solid boundary is not physically present. The use of the ineffective area option is more appropriate and it does not add wetted perimeter to the active flow boundary above the given ground profile.

The ineffective area option is used at cross sections 2 and 3 to keep all the active flow in the area of the bridge opening until the elevations associated with the left and/or right ineffective flow areas are exceeded by the computed water surface elevation. The program allows the stations and controlling elevations of the left and right ineffective flow areas to be specified by the user. Also, the stations of the ineffective flow areas do not have to coincide with stations of the ground profile; the program will interpolate the ground station. The ineffective flow areas should be set at stations that will adequately describe the active flow area at cross sections 2 and 3. In general, these stations should be placed outside the edges of the bridge opening to allow for the contraction and expansion of flow that occurs in the immediate vicinity of the bridge. On the upstream side of the bridge (cross section 3) the flow is contracting rapidly. A practical method for placing the stations of the ineffective flow areas is to assume a 1:1 contraction rate in the immediate vicinity of the bridge. In other words, if cross section 3 is 3 meters (~10 feet) from the upstream bridge face, the ineffective flow areas should be placed 3 meters (~10 feet) away from each side of the bridge opening. On the downstream side of the bridge (cross section 2), a similar assumption can be applied. The active flow area on the downstream side of the bridge may be less than, equal to, or greater than the width of the bridge opening. As flow converges into the bridge opening, depending on the abruptness of the abutments, the active flow area may constrict to be less than the bridge opening. As the flow passes through and out of the bridge, it begins to expand. Because of this phenomenon, estimating the stationing of the ineffective flow areas at cross section 2 can be very difficult. In general, the user should make the active flow area equal to the width of the bridge.
opening or wider (to account for flow expanding), unless the bridge abutments are very abrupt (vertical wall abutments).

The elevations specified for ineffective flow should correspond to elevations where significant weir flow passes over the bridge. For the downstream cross sections, the threshold water surface elevation for weir flow usually is not known on the initial run, so an estimate must be made. An elevation below the minimum top-of-road, such as an average between the low chord and minimum top-of-road, can be used as a first estimate.

Using the ineffective area option to define the ineffective flow areas allows the overbank areas to become effective as soon as the ineffective area elevations are exceeded. The assumption is that under weir flow conditions, the water can generally flow across the whole bridge length and the entire overbank in the vicinity of the bridge would be effectively carrying flow up to and over the bridge. If it is more reasonable to assume only part of the overbank is effective for carrying flow when the bridge is under weir flow, then the overbank n values can be increased to reduce the amount of conveyance in the overbank areas under weir flow conditions.

Cross section 3, just upstream from the bridge, is usually defined in the same manner as cross section 2. In many cases the cross sections are identical. Generally, the only differences are the stations and elevations used for the ineffective area option. For the upstream cross section, the elevation initially should be set to the low point of the top of road. When this is done, the user possibly could obtain a solution where the bridge hydraulics are computing weir flow, but the upstream water surface elevation is lower than the top of road. Both the weir flow and pressure flow equations are based on the energy grade line in the upstream cross section. Once an upstream energy is computed from the bridge hydraulics, the program tries to compute a water surface elevation in the upstream cross section that corresponds to that energy. Occasionally the program may get a water surface that is confined by the ineffective flow areas and lower than the minimum top of road. When this happens, the user should decrease the elevations of the upstream ineffective flow areas in order to get them to turn off. Once they turn off, the computed water surface elevation will be closer to the computed energy gradeline (which is higher than the minimum high chord of the bridge).

Using the ineffective area option in the manner just described for the two cross sections on either side of the bridge provides for a constricted section when all of the flow is going under the bridge. When the water surface is higher than the control elevations used, the entire cross section is used. The program user should check the computed solutions on either side of the bridge section to ensure they are consistent with the type of flow. That is, for low flow or pressure flow solutions, the output should show the effective area restricted to the bridge opening. When the bridge output indicates weir flow, the solution should show that the entire cross section is effective. During overflow situations, the modeler should ensure that the overbank flow around the bridge is consistent with the weir flow.

The maximum effect of the bridge on WSE is expected to occur at cross section 4; this section is sometimes referred to as the approach section. However, in order to determine the extent of the impact, it is recommended that profile computations continue upstream until the water surface does not differ significantly from the estimated pre-construction conditions.

E. Contraction and Expansion Losses. Losses due to contraction and expansion of flow between cross sections are determined during the standard step profile calculations. Manning's equation is used to calculate friction losses, and all other losses are described in terms of a coefficient times the absolute value of the change in velocity head between adjacent cross sections. When the velocity head increases in the downstream direction, a contraction coefficient is used; when the velocity head decreases, an expansion coefficient is used. As shown in Figure 10.50 the flow contraction occurs between cross sections 4 and 3, while the flow expansion occurs between cross sections 2 and 1. The contraction and expansion coefficients are used to compute energy losses associated with changes in the shape of river cross sections (or effective flow areas). The loss due to expansion of flow usually is larger than the contraction loss, and losses from short abrupt transitions are larger than losses from gradual transitions. Typical values for contraction and expansion coefficients under subcritical flow conditions are shown in Table 10.3.
Table 10.3 Subcritical Flow Contraction and Expansion Coefficients

<table>
<thead>
<tr>
<th></th>
<th>Contraction</th>
<th>Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>No transition loss computed</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Graduate transitions</td>
<td>0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>Typical bridge sections</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>Abrupt transitions</td>
<td>0.6</td>
<td>0.8</td>
</tr>
</tbody>
</table>

In general, contraction and expansion coefficients for supercritical flow should be lower than subcritical flow. For typical bridges that are under Class C flow conditions (totally supercritical flow), the contraction and expansion coefficients should be around 0.05 and 0.1, respectively. For abrupt bridge transitions under Class C flow, values of 0.1 and 0.2 may be more appropriate.

F. Hydraulic Computations through the Bridge. Flow through a bridge can be classified as low flow (Class A, B, C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), pressure and weir flow, and highly submerged flows. The determination of which flow type(s) exist(s) can be complicated. For this reason, it is recommended that the bridge modeling be used to analyze flow through a bridge. This section describes in detail how HEC-RAS models each flow type.

1. Low Flow Computations. Low flow exists when the flow going through the bridge opening is open channel flow (water surface below the highest point on the low chord of the bridge opening). For low flow computations, the program first uses the momentum equation to identify the class of flow. This is accomplished by first calculating the momentum at critical depth inside the bridge at the upstream and downstream ends. The end with the higher momentum (and therefore the most constricted section) will be the controlling section in the bridge. If the two sections are identical, the program selects the upstream bridge section as the controlling section. The momentum at critical depth in the controlling section is then compared to the momentum of the flow downstream of the bridge when performing a subcritical profile (upstream of the bridge for a supercritical profile). If the momentum downstream is greater than the critical depth momentum inside the bridge, the class of flow is considered to be completely subcritical (i.e., Class A low flow). If the momentum downstream is less than the momentum at critical depth in the controlling bridge section, then it is assumed that the constriction will cause the flow to pass through critical depth and a hydraulic jump will occur at some distance downstream (i.e., Class B low flow). If the profile is completely supercritical through the bridge, then this is considered Class C low flow.

2. Class A Low Flow. Class A low flow exists when the water surface through the bridge is completely subcritical (i.e., above critical depth). Energy losses through the expansion (cross sections 2 to 1) are calculated as friction losses and expansion losses. Friction losses are based on a weighted friction slope times a weighted reach length between cross sections 1 and 2. The weighted friction slope is based on one of the four available alternatives in the HEC-RAS, with the average-conveyance method being the default. This option is user selectable. The average length used in the calculation is based on a discharge-weighted reach length. Energy losses through the contraction (cross sections 3 to 4) are calculated as friction losses and contraction losses. Friction and contraction losses between cross sections 3 and 4 are calculated in the same way as friction and expansion losses between cross sections 1 and 2.

Four methods are available for computing losses through the bridge (cross sections 2 to 3):

- Energy Equation (Standard Step Method).
- Momentum Balance.
- Yarnell Equation.
- FHWA WSPRO method.

The user can select any or all of these methods to be computed. This allows the modeler to compare the answers from several techniques all in a single execution of the program. If more than one method is selected, the user must choose either a single method as the final solution or direct the program to use the method that computes the greatest energy loss through the bridge at cross section 3 as the final solution. Minimal results
are available for all the methods computed, but detailed results are available for the method that is selected as the final answer. A detailed discussion of each method follows.

(a) Energy Equation (Standard Step Method). The energy based method treats a bridge in the same manner as a natural river cross section, except the area of the bridge structure (i.e., piers and abutments) below the water surface is subtracted from the total area, and the wetted perimeter is increased where the water is in contact with the bridge structure. As described previously, the program formulates two cross sections inside the bridge by combining the ground information of sections 2 and 3 with the bridge geometry. As shown in Figure 10.54, for the purposes of discussion, these cross sections will be referred to as sections BD (Bridge Downstream) and BU (Bridge Upstream).

The sequence of calculations starts with a standard step calculation from just downstream of the bridge (cross section 2) to just inside of the bridge (cross section BD) at the downstream end. The program then performs a standard step through the bridge (from cross section BD to BU). The last calculation is to step out of the bridge (from cross section BU to cross section 3).

The energy based method requires Manning's n values for friction losses and contraction and expansion coefficients for transition losses. The estimate of Manning's n values is well documented in many hydraulics textbooks, as well as several research studies. Basic guidance for estimating roughness coefficients is provided in Chapter 2 of RE-45, Hydraulic Reference Manual for HEC-RAS (USACE, 1995b). Contraction and expansion coefficients are also provided in Chapter 3, as well as parts of Chapter 5 of RE-45, Hydraulic Reference Manual for HEC-RAS (USACE, 1995b). Detailed output is available for cross sections inside the bridge (cross sections BD and BU) as well as the user-entered cross sections (2 and 3).

(b) Momentum Balance Method. The momentum method is based on performing a momentum balance from cross section 2 to cross section 3. This method can be difficult and time consuming to use by hand, so its use should be restricted to the aid of the computer modeling system HEC-RAS, and its application will not be discussed here. For a detailed discussion of the momentum method, refer to RE-45, Hydraulic Reference Manual for HEC-RAS (USACE, 1995b).

(c) Yarnell Equation. The Yarnell equation is an empirical equation that is used to predict the change in water surface from just downstream of the bridge (cross section 2 of Figure 10.50) to just upstream of the bridge (cross section 3). The equation is based on approximately 2600 lab experiments in which the researchers varied the shape of the piers, the width, the length, the angle, and the flow rate. The Yarnell equation is as follows (Yarnell, 1934):

Figure 10.54 Cross Sections Near and Inside the Bridge
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\[ H_{3-2} = 2K(K + 10\omega - 0.6)(\alpha + 15\alpha^4) \frac{V_2^2}{2g} \]

where:
- \( H_{3-2} \) = drop in water surface elevation from section 3 to 2
- \( K \) = Yarnell's pier shape coefficient
- \( \omega \) = ratio of velocity head to depth at section 2
- \( \alpha \) = obstructed area of the piers divided by the total unobstructed area at cross section 2
- \( V_2 \) = velocity downstream at cross section 2

The computed upstream water surface elevation (cross section 3) is simply the downstream water surface elevation plus \( H_{3-2} \). With the upstream water surface known, the program computes the corresponding velocity head and energy elevation for the upstream cross section (cross section 3). When the Yarnell method is used, hydraulic information is only provided at cross sections 2 and 3 (no information is provided for sections BU and BD).

The Yarnell equation is sensitive to the pier shape (K coefficient), the pier obstructed area, and the velocity of the water. The method is not sensitive to the shape of the bridge opening, the shape of the abutments, or the width of the bridge. Because of these limitations, the Yarnell method should be used only at bridges where the majority of the energy losses are associated with the piers. When Yarnell's equation is used for computing the change in water surface through the bridge, the user must supply the Yarnell pier shape coefficient, K. Table 10.4 gives values for Yarnell's pier coefficient, K, for various pier shapes.

<table>
<thead>
<tr>
<th>Pier Shape</th>
<th>Yarnell K Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Semi-circular nose and tail</td>
<td>0.90</td>
</tr>
<tr>
<td>Twin-cylinder piers with connecting diaphragm</td>
<td>0.95</td>
</tr>
<tr>
<td>Twin-cylinder piers without diaphragm</td>
<td>1.05</td>
</tr>
<tr>
<td>90 degree triangular nose and tail</td>
<td>1.05</td>
</tr>
<tr>
<td>Square nose and tail</td>
<td>1.25</td>
</tr>
<tr>
<td>Ten pile trestle bent</td>
<td>2.50</td>
</tr>
</tbody>
</table>

The delivered WSPRO Contracted Opening Method. The low flow hydraulic computations of FHWA's WSPRO computer program has been adapted as an option for low flow hydraulics in HEC-RAS. The WSPRO methodology had to be modified slightly in order to fit into the HEC-RAS concept of cross section locations around and through a bridge.

The WSPRO method computes the water surface profile through a bridge by solving the energy equation. The method is an iterative solution performed from cross section 1 to cross section 4 of Figure 10.50. The energy balance is performed in steps from cross section (1) to the cross section just downstream of the bridge (2), from just downstream of the bridge (2) to inside of the bridge at the downstream end (BD), from inside of the bridge at the downstream end (BD) to inside of the bridge at the upstream end (BU), from inside of the bridge at the upstream end (BU) to just upstream of the bridge (3), and from just upstream of the bridge (3) to the approach cross section (4). A general energy balance equation from the exit section to the approach section can be written as follows:

\[ h_i + \frac{\alpha_i V_i^2}{2g} = h_i + \frac{\alpha_i V_i^2}{2g} + h_{i(4-i)} \]

where:
- \( h_i \) = water surface elevation at cross section i.
- \( V_i \) = velocity at cross section i.
- \( h_{i(4-i)} \) = water surface elevation at cross section 4.

(Equation 10.8)
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\[ V_4 = \text{velocity at cross section } 4. \]

\[ h_{L(4-1)} = \text{energy losses from cross section } 4 \text{ to } 1. \]

The incremental energy losses from cross section 4 to cross section 1 are calculated as follows:

**From Section 1 to 2**

Losses from cross section 1 to cross section 2 are based on friction losses and an expansion loss. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between cross sections 1 and 2. The following equation is used for friction losses from cross sections 1 to 2:

\[
h_{f(1-2)} = \frac{BQ^2}{K_2K_1}
\]

where \( B \) is the flow weighted distance between cross sections 1 and 2, and \( K_1 \) and \( K_2 \) are the total conveyance at cross sections 1 and 2, respectively. The expansion loss from cross section 2 to cross section 1 is computed by the following equation:

\[
h_e = \frac{Q^2}{2gA_1^2} \left[ 2\beta_1 - \alpha_1 - 2\beta_2 \left( \frac{A_1}{A_2} \right) + \alpha_2 \left( \frac{A_1}{A_2} \right)^2 \right]
\]

where \( \alpha_1 \) and \( \beta_1 \) are energy and momentum correction factors for nonuniform flow. \( \alpha_1 \) and \( \beta_1 \) are computed as follows:

\[
\alpha_1 = \frac{\sum (K_i^2/A_i)}{K_f^2/A_f}
\]

\( \alpha_1 \) and \( \beta_1 \) are related to the bridge geometry and are defined as follows:

\[
\beta_1 = \frac{\sum (K_i^2/A_i)}{K_f^2/A_f}
\]

\[
\alpha_1 = \frac{1}{C^2}
\]

where \( C \) is an empirical discharge coefficient for the bridge, which was originally developed as part of the Contracted Opening method by Kindswater, Carter, and Tracy (USGS, 1953), and subsequently modified by Matthai (USGS, 1968).

**From Section 2 to 3**

Losses from cross section 2 to cross section 3 are based on friction losses only. The energy balance is performed in three steps: from cross section 2 to BD, BD to BU, and BU to cross section 3. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between cross sections. The following equation is used for friction losses from BD to BU:

\[
\frac{A_1}{A_2}
\]

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\[ h_{f(BU\text{--}BD)} = \frac{L_{B}Q^{2}}{K_{BU}K_{BD}} \]

where \( K_{BU} \) and \( K_{BD} \) are the total conveyance at cross sections BU and BD, respectively, and \( L_{B} \) is the length through the bridge. Similar equations are used for the friction losses from cross section 2 to BD and BU to cross section 3.

**From Section 3 to 4**

Energy losses from cross sections 3 to 4 are based on friction losses only. The equation for computing the friction loss is as follows:

\[ h_{f(3\text{--}4)} = \frac{L_{av}Q^{2}}{K_{3}K_{4}} \]

where \( L_{av} \) is the effective flow length in the approach reach, and \( K_{3} \) and \( K_{4} \) are the total conveyances at cross sections 3 and 4. The effective flow length is computed as the average length of 20 equal conveyance stream tubes (WSPRO, 1986).

3. **Class B Low Flow.** Class B low flow can exist for either subcritical or supercritical profiles. For either profile, Class B flow occurs when the profile passes through critical depth in the bridge constriction. For a subcritical profile, the momentum equation is used to compute an upstream water surface (cross section 3 of Figure 10.50) above critical depth and a downstream water surface (cross section 2) below critical depth. For a supercritical profile, the bridge is acting as a control and is causing the upstream water surface elevation to be above critical depth. Momentum is used to calculate an upstream water surface above critical depth and a downstream water surface below critical depth. If for some reason the momentum equation fails to converge on an answer during the Class B flow computations, the program will automatically switch to an energy-based method for calculating the Class B profile through the bridge.

Whenever Class B flow is found to exist, the user should run the program in a mixed flow regime mode. If the user is running a mixed flow regime profile, the program will proceed with backwater calculations upstream, and later with forewater calculations downstream from the bridge. Also, any hydraulic jumps that may occur upstream and downstream of the bridge can be located if they exist.

4. **Class C Low Flow.** Class C low flow exists when the water surface through the bridge is completely supercritical. The program can use either the energy equation or the momentum equation to compute the water surface through the bridge for this class of flow. A summary of the low flow classes is presented in Table 10.5.

<table>
<thead>
<tr>
<th>Type Designation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I or Class A</td>
<td>Subcritical flow through Zones 1, 2 and 3</td>
</tr>
<tr>
<td>II A or Class B</td>
<td>Subcritical flow Zones 1 and 3, flow through critical depth Zone 2</td>
</tr>
<tr>
<td>II B or Class B</td>
<td>Subcritical Zone 3, flow through critical Zone 2, hydraulic jump Zone 1</td>
</tr>
<tr>
<td>III or Class C</td>
<td>Supercritical flow through Zones 1, 2 and 3</td>
</tr>
</tbody>
</table>

5. **High Flow.** High flow refers to conditions in which the water surface impinges on the bridge superstructure. When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate. At tailwaters which submerge the low chord but do not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow. If the tailwater...
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exceeds critical depth over the road, neither sluice gate flow nor orifice flow are reasonable. Head losses under such high tailwater conditions usually are estimated as friction and turbulence losses using the energy balance method.

The HEC-RAS program has the ability to compute high flows (flows that come into contact with the maximum low chord of the bridge deck) by either the Energy equation (standard step method) or by using separate hydraulic equations for pressure and/or weir flow. The two methodologies are explained below.

(a) Energy Equation (Standard Step Method). The energy-based method is applied to high flows in the same manner as it is applied to low flows. Computations are based on balancing the energy equation in three steps through the bridge. Energy losses are based on friction, contraction, and expansion losses. Output from this method is available at the cross sections inside the bridge as well as outside.

As mentioned previously, friction losses are based on the use of Manning's equation. Guidance for selecting Manning's n values is provided in Chapter 8, Open Channels. Contraction and expansion losses are based on a coefficient multiplied by the difference in velocity head between adjacent cross sections.

The energy-based method performs all computations as though they are open channel flow. At the cross sections inside the bridge, the area obstructed by the bridge piers, abutments, and deck is subtracted from the total flow area and additional wetted perimeter is added. Occasionally the Hydraulic Grade Line (HGL) inside the bridge (between sections BU and BD) are elevations that would be inside of the bridge deck. The HGL shown as water surface by HEC-RAS inside of the bridge reflects the hydraulic grade line elevations, not necessarily the actual water surface elevations. Additionally, the active flow area is limited to the open bridge area.

(b) Pressure and Weir Flow Method. A second approach for the computation of high flows is to utilize separate hydraulic equations to compute the flow as pressure and/or weir flow. These two types of flow are presented below.

(1) Pressure Flow Computations. Pressure flow occurs when the flow comes into contact with the entire low chord of the bridge. Once the flow contacts the full length of the low chord on the upstream side of the bridge, a backwater occurs and orifice flow is established. HEC-RAS will handle two cases of orifice flow; the first is when only the upstream side of the bridge is in contact with the water, and the second is when the bridge opening is flowing completely full. The HEC-RAS program will automatically select the appropriate equation, depending upon the flow situation. For the first case (see Figure 10.55), a sluice gate type of equation is used (FHWA, 1978):
\[ Q = C_d A_{BU} \left[ 2g \left( Y_3 - \frac{Z}{2} + \frac{\alpha V_3^2}{2g} \right) \right]^\frac{1}{2} \]

where:
- \( Q \) = total discharge through the bridge opening
- \( C_d \) = coefficient of discharge for pressure flow
- \( A_{BU} \) = net area of the bridge opening at section BU
- \( Y_3 \) = hydraulic depth at section 3
- \( Z \) = vertical distance from maximum bridge low chord to the mean river bed elevation at section BU

The discharge coefficient \( C_d \), can vary depending upon the depth of water upstream. Values for \( C_d \) range from 0.27 to 0.5, with a typical value of 0.5 commonly used in practice. The user can enter a fixed value for this coefficient or the program will compute one based on the amount that the inlet is submerged. A diagram relating \( C_d \) to \( Y_3/Z \) is shown in Figure 10.56. As shown in Figure 10.56, the limiting value of \( Y_3/Z \) is 1.1. There is a transition zone somewhere between \( Y_3/Z = 1.0 \) and 1.1 where free surface flow changes to orifice flow. The type of flow in this range is unpredictable, and Equation 10.17 is not applicable.

In the second case, when both the upstream and downstream side of the bridge are submerged, the standard full flowing orifice equation is used (see Figure 10.57). This equation is as follows:

\[ Q = CA\sqrt{2gH} \]

where:
- \( C \) = coefficient of discharge for fully submerged pressure flow; typical value of \( C \) is 0.8.
- \( H \) = the difference between the energy grade elevation upstream and the water surface elevation downstream
- \( A \) = net area of the bridge opening

Typical values for the full-flow discharge coefficient \( C \) range from 0.7 to 0.9, with a value of 0.8 commonly used for most bridges. The user must enter a value for \( C \) whenever the pressure flow method is selected.
Figure 10.56 Coefficient of Discharge for Sluice Gate Type Flow

Figure 10.57 Example of a Bridge under Fully Submerged Pressure Flow
The discharge coefficient $C$ can be related to the total loss coefficient, which comes from the form of the orifice equation that is used in the HEC-2 computer program:

\[ Q = A \sqrt[2]{\frac{2gH}{K}} \]

(Equation 10.19)

where $K = \text{total loss coefficient}$. The conversion from $K$ to $C$ is as follows:

\[ C = \sqrt[2]{\frac{1}{K}} \]

(Equation 10.20)

The program will begin checking for the possibility of pressure flow when the computed low flow energy grade line is above the maximum low chord elevation at the upstream side of the bridge. Once pressure flow is computed, the pressure flow answer is compared to the low flow answer, and the higher of the two is used. The user has the option to tell the program to use the water surface, instead of energy, to trigger the pressure flow calculation.

(2) Weir Flow Computations. Flow over the bridge, and the roadway approaching the bridge, is calculated using the standard weir equation (see Figure 10.20):

\[ Q = CLH^{3/2} \]

(Equation 10.21)

where:

- $Q$ = total flow over the weir
- $C$ = coefficient of discharge for weir flow
- $L$ = effective length of the weir
- $H$ = difference between energy upstream and road crest

The approach velocity is included by using the energy grade line elevation in lieu of the upstream water surface elevation for computing the head, $H$.

Under free flow conditions (discharge independent of tailwater) the coefficient of discharge $C$, ranges from 2.5 to 3.1 (1.4 - 1.7 for metric units) for broad crested weirs depending primarily upon the gross head on the crest ($C$ increases with head). Increased resistance to flow caused by obstructions such as trash on bridge railings, curbs, and other barriers would decrease the value of $C$. 
Tables of weir coefficients, C, are given for broad-crested weirs in King’s Handbook (King, 1963), with the value of C varying with measured head H and breadth of weir. For rectangular weirs with a breadth of 4.6 meters (~15 feet) and an H of 0.3 meters (~1 foot) or more, the given value is 1.45 (2.63 for U.S. Customary Units). Trapezoidal weirs generally have a larger coefficient with typical values ranging from 1.5 to 1.7 (2.7 to 3.08 for U.S. Customary Units).

*Hydraulics of Bridge Waterways* (Bradley, 1978) provides a curve of C versus the head on the roadway. The roadway section is shown as a trapezoid and the coefficient rapidly changes from 1.5 for a very small H to 1.07 for H = 0.2 meters (2.9 for a very small H to 3.03 for H ≈ 0.6 ft). From there, the curve levels off near a value of 1.69 (3.05 for U.S. Customary).

According to the data available, for application of weir flow computations the assumption of a rectangular weir for flow over the bridge deck (assuming the bridge can withstand the forces) and a coefficient of 1.44 (2.6 for U.S. Customary Units) is acceptable. If the weir flow is over the roadway approaches to the bridge, a value of 1.66 (3.0 for U.S. Customary) should be used. If weir flow occurs as a combination of bridge and roadway overflow, then an average coefficient (weighted by weir length) should be used.

For high tailwater elevations, the HEC-RAS program automatically reduces the amount of weir flow to account for submergence on the weir. Submergence is defined as the depth of water above the minimum weir elevation on the downstream side (cross section 2) divided by the height of the energy gradeline above the minimum weir elevation on the upstream side (cross section 3). The reduction of weir flow is accomplished by reducing the weir coefficient based on the amount of submergence. Submergence corrections are based on a trapezoidal weir shape or optionally an ogee.
spillway shape. The total weir flow is computed by subdividing the weir crest into segments; computing \( L, H \), a submergence correction, and a \( Q \) for each section; then summing the incremental discharges. The submergence correction for a trapezoidal weir shape is from *Hydraulics of Bridge Waterways* (Bradley, 1978). Figure 10.59 shows the relationship between the percentage of submergence and the flow reduction factor.

When the weir becomes highly submerged, the HEC-RAS program automatically switches to calculating the upstream water surface by the energy equation (standard step backwater) instead of using the pressure and weir flow equations. The criteria for when the program switches to energy based calculations is user controllable. A default maximum submergence is set to 0.95 (95%).

![Figure 10.59 Factor for Reducing Weir Flow for Submergence](image)

6. Combination Flow. Sometimes combinations of low flow or pressure flow occur with weir flow. In these cases, an iterative procedure is used to determine the amount of each type of flow. The HEC-RAS program continues to iterate until both the low flow method (or pressure flow) and the weir flow method have the same energy (within a specified tolerance) upstream of the bridge (cross section 3, Figure 10.50). The combination of low flow and weir flow can only be computed with the energy and Yarnell low flow methods.

The HEC-RAS program will begin checking for the possibility of pressure flow when the computed low flow energy grade line is above the maximum low chord elevation at the upstream side of the bridge. Once pressure flow is computed, the pressure flow answer is compared to the low flow answer and the higher \( Q \) value of the two is used.

The user has the option to direct the program to use the water surface, instead of energy, to trigger the pressure flow calculation.
G. Selecting a Bridge Modeling Approach. Several choices are available to the user when selecting methods for computing the water surface profile through a bridge. For low flow (water surface is below the maximum low chord of the bridge deck), the user can select any or all of the four available methods. For high flows, the user must choose between either the energy based method or the pressure and weir flow approach. The choice of methods should be considered carefully. The following discussion provides some basic guidelines on selecting the appropriate methods for various situations.

1. Low Flow Methods. For low flow conditions (water surface below the highest point on the low chord of the bridge opening), the energy and momentum methods are the most physically based, and in general are applicable to the widest range of bridges and flow situations. Both methods account for friction losses and changes in geometry through the bridge. The energy method accounts for additional losses due to flow transitions and turbulence through the use of contraction and expansion losses. The momentum method can account for additional losses due to pier drag. The FHWA WSPRO method originally was developed for bridge crossings that constrict wide flood plains with heavily vegetated overbank areas. The method is an energy-based solution with some empirical attributes (the expansion loss equation in the WSPRO method utilizes an empirical discharge coefficient). The Yarnell equation is an empirical formula. The following examples are some typical cases where the various low flow methods might be used:

- In cases where the bridge piers are a small obstruction to the flow, and friction losses are the predominate consideration, the energy-based method, the momentum method, and the WSPRO method should give the best answers.
- In cases where pier losses and friction losses are both predominant, the momentum method should be the most applicable. But any of the methods can be used.
- Whenever the flow passes through critical depth within the vicinity of the bridge, both the momentum and energy methods are capable of modeling this type of flow transition. The Yarnell and WSPRO methods are for subcritical flow only.
- For supercritical flow, both the energy and the momentum method can be used. The momentum based method may be better at locations that have a substantial amount of pier impact and drag losses. The Yarnell equation and the WSPRO method are applicable only to subcritical flow situations.
- For bridges in which the piers are the dominant contributor to energy losses and the change in water surface, either the momentum method or the Yarnell equation would be most applicable. However, the Yarnell equation is only applicable to Class A low flow.
- For long culverts under low flow conditions, the energy based standard step method is the most suitable approach. Several sections can be taken through the culvert to model changes in grade or shape or to model a very long culvert. This approach also has the benefit of providing detailed answers at several locations within the culvert, which is not possible with the culvert routines in HEC-RAS. However, if the culvert flows full, or if it is controlled by inlet conditions, the culvert routines would be the best approach. For a detailed discussion of the culvert routines within HEC-RAS, see Chapter 6 of the Hydraulic Reference Manual.

2. High Flow Methods. For high flows (flows that come into contact with the maximum low chord of the bridge deck), the energy-based method is applicable to the widest range of problems. The following examples are some typical cases where the various high flow methods might be used:

- When the bridge deck is a small obstruction to the flow, and the bridge opening is not acting like a pressurized orifice, the energy-based method should be used.
- When the bridge deck and road embankment are a large obstruction to the flow, and a backwater is created due to the constriction of the flow, the pressure and weir method should be used.
- When the bridge and/or road embankment is overtopped, and the water going over top of the bridge is not highly submerged by the downstream tailwater, the pressure and weir method should be used. The pressure and weir method will automatically switch to the energy method if the bridge becomes 95% submerged. The user can change the percent submergence at which the program switches from the pressure and weir method to the energy method. This is accomplished from the Deck/Roadway editor in the Bridge/Culvert Data editor.
- When the bridge is highly submerged, and flow over the road is not acting like weir flow, the energy-based method should be used.
10.7 SINGLE OPENING DESIGN

A. Single Opening Design Approach. The term "single opening" refers to a stream crossing requiring only one opening (of one or more spans) in the highway embankment.

The steps discussed in this section are described to provide the designer with a means of establishing an initial size of opening. The effect of the trial opening must then be analyzed using the designer's selection of methods outlined in Section 10.6. If the resulting backwater or the through bridge velocities are unacceptable, the designer must modify the opening until the estimated conditions are satisfactory for both the design and check flood conditions. The Department recommends the use of automated procedures for such analyses.

B. Recommended Procedure for Single Opening Design. The recommended procedure for establishing a single structure length and elevation of the low chord begins with estimating the design flood, obtaining accurate controlling cross sections, and determining the design and check flood water surface profiles (Chapter 7, Hydrology, and Chapter 8, Open Channels). Also useful, and sometimes necessary for complete documentation, is a compilation of past flood history, existing structures, and other characteristics of the highway crossing of the stream.

1. As a trial, assume an average through-bridge velocity (vt) that is less than the maximum allowable velocity but which is not lower than the unconstricted average velocity.

2. Apply the unconstricted design water surface elevation to the section and find the cross sectional area (At) subtended by this water surface that will satisfy the Continuity Equation for trial velocity and design discharge.

\[ A_t = \frac{Q}{v_t} \]

where: 
- \( A_t \) = submerged cross sectional area, \( \text{m}^2 \) (\( \text{ft}^2 \))
- \( Q \) = design discharge, \( \text{m}^3/\text{s} \) (\( \text{cfs} \))
- \( v_t \) = trial velocity, \( \text{m/s} \) (\( \text{ft/s} \))

3. By inspection of the section, estimate an average depth of water (Dt) in the cross section where the bridge is to be located.

4. Find the trial length (Lt) of the bridge using Equation 10.23.

\[ L_t = \frac{A_t}{D_t} \]

5. Position the abutments in the stream cross section (same cross section as in Step 3) so that they are approximately \( L_t \) apart and at locations which appear to maximize the through-bridge area.

6. Find the waterway area (Aw) below the design high-water within the structure limits.

7. Use the Continuity Equation to find the average through-bridge velocity (vb) for the actual waterway area (Aw).

\[ v_b = \frac{Q}{A_w} \]

8. If \( v_b \) is close to the target average velocity, the initial bridge length may be reasonable. (The allowable maximum velocity should be evaluated and established based on individual site characteristics.) This length usually must be adjusted slightly to fit standard span length requirements.
If $v_b$ is much lower or greater than the allowable maximum velocity, the length should be adjusted as necessary, repeating Steps 6 and 7. This routine should be repeated until the average through-bridge velocity is close to the target velocity. To minimize the cost of the structure, it usually is desirable to adjust the bridge length so that the design velocity is at or very near the maximum allowable velocity.

9. Establish a low chord (as discussed in Section 10.4.G. on Freeboard).

10. Estimate the backwater caused by the constriction of the bridge opening using the procedures outlined in Section 10.6. The bridge length may need to be adjusted to ensure that the backwater effects are not excessive.

11. Determine the maximum scour potential envelope as discussed in Section 10.9.

10.8 MULTIPLE OPENING DESIGN

A. Multiple Opening Design Approach. In situations where a bridge crosses a relatively wide flood plain with multiple discharge concentrations, it may be necessary to design multiple openings. A multiple opening configuration usually consists of a main channel bridge with relief openings. This type of crossing provides openings at or near the flow concentrations. The result is a reduction in along-embankment flow and backwater effects.

This type of problem can be modeled in two ways within HEC-RAS. The preferred method is to use the multiple opening capability in HEC-RAS, which is discussed in detail in the HEC-RAS Reference Manual. A second method is to model the two openings as divided flow. This method would require the user to define the flow path for each opening as a separate reach.

Refer to RE-45, Hydraulic Reference Manual for HEC-RAS (USACE, 1995b) for more information on analysis of multiple openings.

10.9 BRIDGE SCOUR

A. Scour Components. Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Potential scour can be a significant factor in the analysis of a stream crossing system. The design of a crossing system involves an acceptable balance between a waterway opening that will not create undue damage by backwater or suffer undue damage from scour and a crossing profile sufficiently high to provide the required traffic service.

For simplicity, scour is considered to consist of three components:

- Long-term aggradation and degradation.
- Contraction scour.
- Local scour.

Long-term aggradation and degradation is also referred to as natural scour as discussed in Section 10.5.B.5.

For a comprehensive evaluation of scour which includes defining the various types of scour and equations and computational methodologies used to quantify it, refer to HEC-18, Evaluating Scour at Bridges (FHWA, 2001). The remainder of this section provides an introduction to the concepts associated with scour.

B. Rates of Scour. The rate of scour in different materials and under different flow conditions depends on the following factors:

- Erosive power in the flow.
- Erosion resistance of the material.
- A balance between sediment transported into and out of a section.
In erosion resistant materials, equilibrium may not be reached in any one flood event but may develop over a long series of events.

Generally, the methods currently available do not specifically accommodate cohesive bed materials, nor is time dependency considered. Therefore, the results of any scour calculations should be considered only as an indication of the maximum potential scour. Judgment based on experience must be exercised to decide whether or not calculated depths are likely for the given site conditions and life expectancy of the bridge.

C. Requirements for Scour Analysis. Design projects involving new, rehabilitated, widened, and existing bridges over waterways should include estimates of the potential scour envelope using velocities and flow depths resulting from $Q_{100}$, $Q_{500}$, flood of record, and $Q$-overtopping. In some cases, other events could produce worse scour. Such events with return periods not exceeding 500 years should be considered. In all cases, subsequent foundation design is based on the worst case scour. The designer should refer to HEC-18, *Evaluating Scour at Bridges* (FHWA, 2001) for a detailed discussion of analytical procedures.

D. Aggradation and Degradation. The depth and area of general scour at a waterway opening may be affected by any or all of the following factors:

- Slope, natural alignment, and shifting of the channel.
- Type and amount of bed material in transport.
- Nature and occurrence of flood events.
- Accumulations of debris.
- Constriction or realignment of flow due to the stream crossing.
- Layout and geometry of hydraulic structure works.
- Geometry and orientation of piers.
- Classification, stratification, and consolidation of bed and sub-bed materials.
- Placement or loss of riprap and other protective materials.
- Natural or man-made changes in flow or sediment regimes.
- Failures such as collapse of a nearby structure.

It is important to consider the potential for long-term aggradation and degradation. Generally, projections based on evaluation of the history of the site, or ones similar to the site, may suffice. Sometimes, estimation of long-term aggradation and degradation may require a qualitative determination based on general geomorphic and river mechanics relationships, an engineering geomorphic analysis based on qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios or future conditions, and mathematical models such as BRI-STARS and the USACE HEC-6 to make predictions of quantitative changes in streambed elevations due to changes in the stream and watershed.

E. Contraction Scour. Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or by a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the contraction. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases, and the velocity and shear stress decrease until relative equilibrium is reached, i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach.

Contraction scour is typically cyclic. That is, the bed scours during the rising stage of the runoff event, and fills on the falling stage. The contraction of flow due to a bridge can be caused either by a natural decrease in flow area of the stream channel, by abutments projecting into the channel, or by the piers blocking a large portion of the flow area. Contraction also can be caused by the approaches to a bridge cutting off the flood plain flow. This can cause clear-water scour on the setback portion of a bridge section (the portion of the bridge located in the floodplain) or within relief bridges because the flood plain flow normally does not transport significant concentrations of bed material sediments.

Depending on the stream flow, contraction scour can be either live-bed or clear-water. Live-bed scour occurs when the bed material upstream of the constriction is in motion. The scour that results at the constriction is reflective of an equilibrium condition between the sediment transported into the section and that transported away from the
section. Under live-bed conditions, scour holes created during the rising stages of a flood often are refilled during the recession stages.

Clear-water scour occurs when the bed material is not in motion outside the contracted section. The sediment transported into the contracted section is essentially zero. Clear-water scour occurs when the shear stress induced by the water flow exceeds the critical shear stress of the bed material. Generally, when clear-water scour occurs, there is no refilling during the recession of the flood due to the lack of sediment supply. During the initial stages of a flood, clearwater scour could occur then be followed by live-bed scour at higher flood stages.

Typical clear-water scour situations include the following:

- Streams with coarse bed material.
- Flat gradient streams during low flow.
- Local deposits of bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation).
- Armored stream beds in which tractive forces are large enough to penetrate the armor only at the piers and abutments.
- Vegetated channels in which the tractive forces are large enough to penetrate the cover only at piers and abutments.

F. Pier Scour. Either live-bed or clear-water scour may occur at pier locations. The upstream part of a local scour hole tends to have the shape of a truncated cone with the cone angle approximating the angle of repose of the sediment in water. Downstream slopes are flatter where the flow mixes with other flow, and a bar is formed downstream of the hole. The lateral extent of the scour hole can be determined from the wetted angle of repose of the material and the depth of scour.

G. Abutment Scour. Several abutment scour equations currently exist and are presented in HEC-18, *Evaluating Scour at Bridges, Fourth Edition* (FHWA, 2001b). However, none of the equations presented to date gives consistently acceptable results. Generally, they give conservative estimates even for low Froude numbers. Abutments should be protected to reduce the potential for scour failure by the use of properly designed mitigative measures according to the guidance in Publication 15M, Design Manual, Part 4, *Structures*.

H. Total Scour Envelope. In reality, a total scour envelope at any given section is the result of a complex interaction of flow, sediment transport, bed material, and time. Currently, the procedures available assume that components of scour (long-term degradation, contraction, and local scour) act independently of each other and are ultimate depths for non-cohesive bed materials. The total scour envelope, then, is considered to be the summation of the individual components at the appropriate locations. All components of scour are considered to be equal to or greater than zero. Any negative scour depths are set to zero. Without better methods, the assumption is that the natural degradation and contraction scour depths occur evenly across the portion of the cross section for which they were estimated. Where local scour is considered to occur (at piers and abutments), the total scour is assumed to be the sum of natural degradation, contraction scour, and local scour.


J. Other Scour Considerations.

1. Borrow. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause scour at the bridge. Scour occurs upstream of the borrow because of the increased gradient of the stream bed. The bed load of the stream will be deposited in the borrow area and scour occurs downstream as the stream regains its bed load.

If there is any concern about the effects of borrow from a stream, sediment transport models such as BRISTARS or HEC-6 should be used.

2. In-Stream Mining and Dredging. In-stream mining for aggregates and dredging for navigation and flood control can be extremely damaging in cases where so much material is removed from the stream bed that all of the incoming sediment supply is trapped and degradation of long reaches occurs.
At some locations where dredging may be necessary, measures to stabilize the stream bed elevation and the stream bank may be required. Otherwise, the pier and abutment foundations must be set below the expected future elevation due to dredging of the stream bed.

3. Armoring. Armoring occurs because a stream or river is unable, during a particular flood, to move the more coarse material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially but later become checked by armoring before the full scour potential is reached for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor on the stream bed or in the scour holes and thus limit further scour for a particular discharge. This armoring effect can decrease scour hole depths which were predicted based on formulas developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs than the flood used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring also may cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges since they encourage difficult to assess plan-form changes. Bank widening also spreads the approach flow distribution, which in turn results in a more severe bridge opening contraction.

4. Scour Resistant Materials. Caution is necessary in determining the scour resistance of bed materials and the underlying strata. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour may not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, this material is replaced with more easily eroded material. Thus, at some later date another flood may reach the predicted scour depth. Serious scour has been observed to occur in materials commonly perceived to be scour resistant such as consolidated soils and glacial till, as well as so-called bed rock streams and streams with gravel and boulder beds.

5. Scour Analysis Methods. Before the various scour forecasting methods for contraction and local scour can be applied, it first is necessary to (1) obtain the fixed bed channel hydraulics, (2) estimate the profile and plan form scour or aggradation, (3) adjust the fixed bed hydraulics to reflect these changes and (4) compute the bridge hydraulics. Two methods are provided in this chapter for combining the contraction and local scour components to obtain total scour. The first method, identified as Method 1, has application when armoring is not a concern or insufficient information is available to permit its evaluation, or where more precise scour estimates are not deemed necessary. The second method, Method 2, can be used when stream bed armoring is of concern, more precise contraction scour estimates are deemed necessary, or deposition is expected and is a primary concern.

(a) Method 1. This is considered to be a conservative practice since it assumes that the scour components develop independently. The potential local scour to be calculated using this method would be added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. The general approach with this method is as follows:

- Estimate the natural channel's hydraulics for a fixed bed condition based on existing conditions.
- Assess the expected profile and plan form changes.
- Adjust the fixed bed hydraulics to reflect any expected profile or plan form changes.
- Estimate contraction scour using the empirical contraction formula and the adjusted fixed bed hydraulics assuming no bed armoring. If the reach is expanding, contraction scour is not expected.
- Estimate local scour using the adjusted fixed bed channel and bridge hydraulics assuming no bed armoring.
- Add the local scour to the contraction scour to obtain the total scour.

(b) Method 2. This analysis method is based on the premise that the contraction and local scour components do not develop independently. As such, the local scour estimated with this method is determined based on the expected changes in the hydraulic variables and parameters due to contraction.
scour or deposition; i.e., through what may prove to be an iterative process, the contraction scour and channel hydraulics are brought into balance before these hydraulics are used to compute local scour. Additionally, with this method the effects of any armoring may be considered. The general approach for this method is as follows:

- Estimate the natural channel's hydraulics for a fixed bed condition based on existing site conditions.
- Estimate the expected profile and plan form changes based on the procedures in the Model Drainage Manual and any historic data.
- Adjust the natural channel's hydraulics based on the expected profile and plan form changes.
- Select a trial bridge opening and compute the bridge hydraulics.
- Estimate contraction scour.
- Revise the natural channel's geometry to reflect these contraction scour or deposition changes and then again revise the channel's hydraulics (repeat this iteration until there is no significant change in either the revised channel hydraulics or bed elevation changes -- a significant change would be a 5% or greater variation in velocity, flow depth or bed elevation).
- Using the foregoing revised bridge and channel hydraulic variables and parameters obtained considering the contraction scour the local scour.
- Extend the local scour assessment below the predicted contractions scour depths in order to obtain the total scour.

10.10 DECK DRAINAGE

Effective bridge deck drainage is important for several reasons. These include the susceptibility of the deck structural and reinforcing steel to corrosion from deicing salts, ice forming on bridge decks while other roadway surfaces are still ice-free, and the possibility of hydroplaning on decks with little surface texture. Bridge decks often are less effectively drained than approach roadways because of a lower cross slope; uniform cross slopes for traffic lanes and shoulders; parapets or curbs which contain the water and debris within the roadway section; grates, scuppers and curb openings which usually are not depressed and are smaller than on roadway sections; and, where used for downspouts, clogging in sharp bends of small diameter pipes.

Deck drainage can be improved by providing sufficient gradient to cause the water to flow to inlets or off the ends of the bridge, avoiding zero gradients and sag-vertical curves on bridges, intercepting all flow from curbed roadways before it reaches the bridge. Currently, there is a trend toward the use of watertight joints and carrying all deck drainage to the bridge ends for disposal because deck drains may be difficult to maintain.

A. Deck Inlets. Inlets used on bridge decks include grates, scuppers, curb inlets and slotted curbs. Inlets should be spaced to avoid spread on the roadway which may cause interference with traffic or the creation of traffic hazards. Collection systems and downspouts should be avoided, where possible. Collection systems should be designed with cleanouts at all bends, sufficient gradient to minimize problems with debris, and runs as short as practical. Collection systems with excessive gradients often have clogging problems because of insufficient flow depths to carry debris introduced through the inlets.
Deck drains over traveled ways should be spaced so that water does not drain directly onto the roadway or railroad below. Where downspouts are used, splash basins should be considered to minimize erosion. Drainage should not be allowed to discharge against any part of the structure, Chapter 13, *Storm Drainage Systems* and *Bridge Drainage Systems* (Transportation Research Board, 1979a).

**B. Bridge End Drains.** Because of the vulnerability of approach roadway shoulders and foreslopes to erosion from concentrated flow, sufficient inlet capacity should be provided off the bridge ends to intercept flow from the bridge. A closed conduit often is preferable to an open chute down the foreslope because it controls the water in a more positive manner, is more aesthetically pleasing, and is less susceptible to damage by maintenance equipment.

When bridge end drains are not provided with the bridge construction, temporary provisions for protecting the approach fill from erosion should be utilized until permanent measures are installed and functional.

**10.11 HYDRAULIC-RELATED CONSTRUCTION CONSIDERATIONS**

Numerous considerations that are within the purview of the construction engineer and/or the contractor can affect the integrity of the hydraulic design and the highway-stream crossing system. Also, features of the design may be logical insofar as the engineer is concerned, but ill-conceived from the viewpoint of the construction engineer. Lines of communication between the designer and construction personnel should be established to ensure that designs are not unnecessarily complex or difficult to construct and that construction methods and measures do not invalidate design assumptions or create conditions which will adversely affect the stream crossing.

The responsibility for construction-related hydraulic considerations of stream crossings ordinarily rests with the contractor, but in some cases the highway agency may include construction-related details in the plans in order to mitigate potential environmental effects or reduce the risk of failure during construction. Whether the highway agency or the contractor assumes the risk and responsibility, hydraulic considerations during construction usually differ from the design considerations for the completed facility.

**A. Verification of Plans.** Plans should be checked to verify that site conditions at the stream crossing have not changed from those that existed at the time design plans were completed. Meander migration, bank caving, aggradation, headcutting or other natural or construction-induced changes in the channel may have occurred which would require that the designer reconsider decisions made on the basis of conditions which were different from those which existed at the beginning of construction. The changed conditions may require river control works,
revisions to pier locations and orientation, rearrangement of spans, or other modifications of the design to accommodate the changes that have occurred.

Dependent upon the time that has elapsed between completion of the design plans and the beginning of construction, changes in land use could significantly affect the validity of design considerations. Commercial mining of materials for construction is a rather common practice that can change flow velocities, volume and character of bed load, and flow direction and distribution at the crossing site. Land clearing for agricultural purposes may create a need to reconsider the location and size of waterway openings and the need for spur dikes. Land development near the site could change damage risk considerations for the crossing.

Hydraulic and structural designers should be consulted regarding the need to modify the design at any stream crossing which has changed significantly from the conditions which existed during design.

B. Plan Changes. Plan changes that become necessary during construction that affect pier or abutment locations, configuration or orientation, pile tip or foundation elevation should be approved by the engineer prior to proceeding with structural redesign and construction. Scour, debris and backwater considerations for the changed conditions may indicate that the proposed plan change is not advisable or that an alternative to the proposed change would serve the construction need without adversely affecting the hydraulics.

C. Borrow Areas. Borrow areas should be located so that they will not contribute to the hazards of the stream crossing. Stream bed and bank borrow can induce changes in the stream that may cause active meandering, a new channel to be formed, or result in deposition of the bed load which will cause clear water scour in the waterway. Borrow areas established along the embankment can cause concentrated flow along the embankment and contribute to serious scour at the bridge abutment.

Borrow area drainage and dewatering should be accomplished in a manner that will not contribute to sedimentation in the stream. It may be desirable to provide settling basins to remove contaminants from such drainage prior to release into the stream.

Full details on considerations in borrow area location are discussed in previous sections of this manual.

D. Detours, Contractor Crossings and Work Areas. Stream crossings for detours are built to much lesser standards than the crossing designed for the highway. Temporary stream crossing structures are usually substantially smaller than the permanent crossing structure because of the lower probability of occurrence of an extreme flood event during the relatively short construction phase. This is good practice from both hydraulics engineering and economic points of view. During a one-year construction period, the odds are 4-to-1 against a flood as large as a 5-year flood, and there is an even chance that the mean annual event will not be exceeded. The odds that a 10-year flood will occur during a one-year construction period are 9-to-1 against and 27-to-1 against occurrence in a three-year construction period. Similarly, the probability of exceedance during other construction periods can be read from the curve or computed with the equation:

\[ Risk = R = 1 - \left(1 - \frac{1}{T_R}ight)^n \]

where \( R \) is equal to the risk, \( n \) equals the number of years under construction, and \( T_R \) represents the return period.

It follows that the criteria used for the hydraulic design of detour stream crossings should be based on risk factors which should be evaluated considering the probability of flood exceedance during the anticipated service life of the detour (the construction period for the highway crossing), the risk to life and property, and traffic service requirements.

The smaller temporary structure will create excessive backwater and scour even during minor floods unless the gradeline is kept low enough to be overtopped before upstream damage occurs. The structure should be anchored to avoid loss during minor floods.
As in the case of the design for highway-stream crossings, detour designs should accommodate floods larger than the event for which they are designed in order to avoid undue liability for damages from excessive backwater and to reduce the probability of losing the detour stream crossing structure during a larger flood. In most instances, the conveyance of floods larger than the detour design flood is provided for by a low roadway profile which allows overflow without creating excessive velocities or backwater.

PennDOT prefers a hydraulic design of detour stream crossings by the highway agency because of environmental considerations and risks associated with contractor designs. Designs furnished to the contractor vary from a required waterway opening to a complete design of the temporary stream crossing. Considering that both the approach roadway profiles and the waterway opening are integral parts of a stream crossing system, detour designs should not be limited to specification of a required waterway area.

Temporary stream crossings necessary for the construction of highways usually are the responsibility of the contractor. It may be desirable in some instances, however, for the highway agency to design such crossings in order to minimize or mitigate the adverse effects on the stream environment, to facilitate securing permits, or to reduce the risk assumed by the contractor and thereby reduce construction costs.

E. Environmental and Ecological Aspects. Minimum disturbance of the banks and bed of a stream during the construction period will reduce erosion damage to the banks, sedimentation, and harm to fish and wildlife. Embankments in or along streams should be constructed of erosion-resistant material and/or protected against erosion to avoid adverse sediment concentrations which contribute to the turbidity of the stream.

Consideration should be given to precluding instream operations that would cause turbidity during the spawning season of certain types of fish. This information is available from the Pennsylvania Fish and Boat Commission. Detours and construction roads are other sources of turbidity and either should be constructed at a time that fishery activities will not be disturbed, or provisions should be made to control any harmful effects of erosion. Silts and clays will generally flush out of the substrate over a period of time, but sands tend to become embedded. Gravel and rock similar to the gradations found in the existing substrate will do the least damage to the aquatic habitat.

Pumping of cofferdams and other dewatering operations may have a discharge of unacceptable quality to the receiving stream. Mitigation measures such as settling basins may be necessary if the ecosystem of the stream would be upset by the temporary degradation of water quality.

F. Hydrologic Information. The hydrology for the construction site is the same as for the design of a bridge. However, temporary structures will have a different design frequency storm than the bridge and it will depend upon the how long the temporary structure is in place. The hydrology for the construction site is the same as for the design of a bridge; however, the design storm will depend upon how long the temporary structure is in place. The design storm for the temporary structure will usually be different from the one used for the bridge.

G. Cofferdams, Caissons, Barges, and Falsework. Cofferdams, causeways, falsework and occasionally contractor's equipment, such as barges, constrict the stream channel more than the completed substructure and consequently have greater potential for causing scour and bank caving, and for collecting debris. Scheduling of work to avoid flood seasons is especially important if these types of operations will be involved.

H. Feedback. Most highway designers do not have an opportunity to participate in the construction of the works that they have created. For this reason, designs that could be improved upon for construction purposes tend to be perpetuated simply because the designer is not informed of the deficiencies.

Construction engineering personnel are encouraged to invite designers to visit construction sites to discuss problems with designs and possible improvements in future designs. Upon completion of a project, a design critique conducted jointly by designers and field personnel can be a very useful learning experience for both.

10.12 HYDRAULIC-RELATED MAINTENANCE CONSIDERATIONS

A sizeable proportion of highway maintenance resources are expended on protecting and preserving the capital investment in stream crossings. Many expenditures are made on an emergency basis, while many others are made to maintain the hydraulic and structural integrity of the crossing and to guard against future damage by flood flow. As
is true of all aspects of highway and bridge maintenance, it is important that when maintenance work is undertaken, it is a high-priority need and cost-effective.

Engineers and other highway personnel with expertise in specialized areas can provide important expert assistance in assessing priority needs and in recommending cost-effective repairs and protective measures. Similarly, maintenance personnel can provide important information to engineers which will enable the engineers to provide solutions for problems at existing crossings and possibly avoid similar problems in future designs.

Discussion in the following sections is intended to advocate the merits of utilizing the services of engineers in the maintenance of highway-stream crossing systems. Streams are dynamic systems, and, in general, it is necessary to provide countermeasures at each problem site commensurate with the hazard and the resources available. Therefore, it is not the intention here to provide a detailed discussion of conditions which warrant corrective measures, or to prescribe the measures which should be undertaken.

A. Maintenance Inspections. The National Bridge Safety Inspection Program has been expanded to require detailed consideration of scour conditions that could threaten the structural stability of the bridge. The FHWA recommends that a team consisting of structural, hydraulic and geotechnical personnel be employed in the evaluation of bridge scour. In addition to regularly scheduled maintenance inspections, stream crossings may be inspected following flood events to discover conditions which may threaten the integrity of the crossing. Items that should be noted and commented on include bend migration, aggradation, degradation, unusual scour depths, bank erosion, approach fill erosion, bank protection and river training device damage or destruction, and changes in channel alignment and flow conditions. Conditions which appear to threaten the bridge or approaches should be referred to hydraulic and structural engineers for recommendations on countermeasures to be undertaken. PennDOT's inspection manual, Publication 238, Bridge Safety Inspection Manual provides detailed discussion of the requirements for maintenance inspection teams and processes.

B. Flood Damages. Although maintenance personnel often are occupied with emergency repair during times of floods, data which can be collected only during and immediately following a flood, such as highwater elevations and scour depths, should be given attention for scour critical bridges. These data are important to any decision regarding the necessity for providing remedial or corrective measures and the measures that should be used.

Some of the serious problems that develop during floods have been anticipated by the designers. For example, scour in alluvial streams is a well-known phenomenon, and although predictive techniques are not precise, bridges are designed to withstand the anticipated scour; therefore, it is important to determine the extent and depth of scour that develops during a flood, but corrective measures are not necessary unless the expectations of the designer were exceeded. Similarly, the "as-built" channel at a bridge may not need to be restored if piers were designed in anticipation of degradation or bend migration through the bridge opening.

Stream bank protection and river training measures are generally "temporary" since it is not cost-effective to provide protection against the worst possible contingency. Most such measures are intended to provide protection against most of the floods that occur but eventually will be damaged or destroyed. Thus, the designer is "buying time" when the measures are specified with the expectation that other measures will be necessary after the stream defeats the original installation. At that time, it is probable that restoration of the original installation would be unlikely to stabilize the banks or train the stream in its changed condition, and a decision must be made on the need for new or additional measures.

The above discussion is to illustrate that all "damage" need not be repaired, and cost-effective repair or reconstruction may not restore measures as originally constructed. Maintenance files should contain data on "as-built" construction to aid in making decisions regarding the need for corrective measures. It may be expedient to consult structural and hydraulic engineers to review the design and advise on the necessity for emergency measures. Some protective measures cannot be provided or existing measures cannot or need not be repaired during the flood in which damage occurs. After the flood subsides, there is an opportunity to evaluate the changed conditions and the damage caused by the flood and to make new decisions regarding protective measures to be employed. The expertise of hydraulic and structural engineers should be utilized to ensure that cost-effective measures are provided.

1. Remedial Construction and Repair. Additional protective measures often are required at stream crossings to protect the crossing from destruction by the stream. Bend migration may be endangering the approach embankment and/or the structure, or an unanticipated headcut or scour hole may have exposed pier footings or
pilings to an unacceptable depth. These conditions should be corrected as soon as practical, and hydraulic and structural engineers should be consulted regarding the remedial measures or countermeasures to be employed. It should be kept in mind that the security of the crossing rather than that of any existing protective measures is the objective of the maintenance effort. Relatively inexpensive, expendable measures may be more cost-effective than more expensive, relatively permanent measures.

2. Recurring Damage. At some stream crossings, similar damage occurs during each flood event, requiring relatively frequent, recurring maintenance. Some of the problems can be eliminated by remedial construction, as by the construction of spur dikes, finger dikes or downstream dikes to avoid damage to approach embankments. The logical and feasible solution to other recurring damage problems may be to discontinue repairing the damage if the crossing and adjacent property are not endangered. An example of this type of problem might be a scour hole that develops at midspan, or downstream, of a structure on the floodplain.

Many highway-stream crossing systems were designed with relatively low profiles where the roadway encroaches on the floodplain. Overflow over the roadway may cause traffic interruption and embankment damage even though the bridge over the stream is above highwater. The seemingly obvious solution is to raise the profile of the roadway, but this may have implications that are not immediately apparent. A higher profile in the floodplain will cause higher backwater, force larger floods through the bridge waterway, and change flow distribution downstream of the highway facility. These changes may significantly increase the risk of flood damage to property upstream and downstream of the highway, and severe scour may occur at the bridge because larger flows are forced through the opening. Also, the probability that the bridge superstructure will become submerged will be greater and the hazard of failure from drag, debris, ice or buoyant forces will be increased.

Recurring maintenance requirements at some stream crossings are such that the only remedy is reconstruction. This often occurs with aggrading and degrading streams where continual maintenance and countermeasures are expensive and often ineffective. The need to reconstruct may also arise from changes in the watershed which have changed the hydrology of the stream. Maintenance personnel should take steps to ensure that design personnel are fully cognizant of the history of the crossing and the reasons for its inadequacy.

10.13 APPURTENANCES

A. Bridge Rail. The type of bridge rail used can be an important hydraulic consideration. This is particularly true in instances where overtopping of the bridge is a factor.

A solid bridge rail used where the bridge superstructure overtops will constitute a significant impediment to flood flow.

A more desirable type of rail for accommodation of flood flow is a style which will offer the floodwater an opening. An open slender type of bridge railing will have a lower backwater and smaller lateral forces than a more impervious type. However, parapets must always be designed for traffic loads appropriate to the traffic on the anticipated bridge.

10.14 DESIGN DOCUMENTATION

Design data should be assembled in an orderly fashion and retained for future reference. The amount and detail of documentation for each highway-stream crossing system should be commensurate with the risk and the importance of the crossing. For example, a small stream in a rural area would not ordinarily require the same degree of documentation as a small stream in a developed area.

Design data and documentation are important in the post-construction period for the following reasons:

- The performance of structures over a period of time, as compared with information developed for the design, is very helpful in evaluating design policies and procedures and the validity of design assumptions.
In the event of failure, contributing causes can be identified, compared with design assumptions and computations, and considered in the design of replacement structure(s).

Documentation of data for existing structures is a valuable source of information when structures are replaced, repaired or rehabilitated, and for the design of other structures in the vicinity.

Information collected and analyzed for purposes of highway design can be of value to others considering plans for the vicinity.

Records of data, analyses, and decisions are essential for responding to subsequent complaints and litigation.

Project files are the most permanent of all highway agency records and are a convenient and appropriate place for recording the results of analyses and decisions. Chapter 4, *Documentation and Document Retention*, should be referenced for required documentation for bridge projects.

### 10.15 CHAPTER 10 NOMENCLATURE

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Net area of the bridge opening in Orifice Equation</td>
<td>m² or ft²</td>
</tr>
<tr>
<td>A</td>
<td>Cross sectional area of flow</td>
<td>m² or ft²</td>
</tr>
<tr>
<td>B</td>
<td>Flow weighted distance in FHWA WSPRO Method Equations</td>
<td>m or ft</td>
</tr>
<tr>
<td>C</td>
<td>Coefficient of discharge for fully submerged pressure flow in Orifice Equation</td>
<td>dimensionless</td>
</tr>
<tr>
<td>C_d</td>
<td>Coefficient of discharge for pressure flow in Pressure-Weir Flow Methods</td>
<td>dimensionless</td>
</tr>
<tr>
<td>F</td>
<td>Force of impact imparted by debris in Debris Force Equation</td>
<td>N or lb</td>
</tr>
<tr>
<td>F_d</td>
<td>Drag force per unit of bridge length in Drag Force Equation</td>
<td>N/m or lb/ft</td>
</tr>
<tr>
<td>g</td>
<td>Gravitational acceleration constant</td>
<td>m/s² or ft/s²</td>
</tr>
<tr>
<td>H</td>
<td>The difference between the energy grade elevation upstream and the water surface elevation downstream in Orifice Equation</td>
<td>m or ft</td>
</tr>
<tr>
<td>H</td>
<td>Depth of submergence of superstructure in Drag Force Equation</td>
<td>m or ft</td>
</tr>
<tr>
<td>H_3-2</td>
<td>Drop in water surface elevation from section 3 to 2 in Yarnell Equation</td>
<td>m or ft</td>
</tr>
<tr>
<td>h, h_1, h_2, h_3, h_4</td>
<td>Water surface elevation, water surface elevation at respective cross sections in FHWA WSPRO Method Equations</td>
<td>m or ft</td>
</tr>
<tr>
<td>h_L, h_L(4-1)</td>
<td>Energy losses, energy losses from cross section 4 to 1 in FHWA WSPRO Method Equations</td>
<td>m or ft</td>
</tr>
<tr>
<td>h_f</td>
<td>Energy losses due to friction</td>
<td>m or ft</td>
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<tr>
<td>h_(1-2)</td>
<td>Energy losses due to friction from section 1 to section 2 in Yarnell Method Equations</td>
<td>m or ft</td>
</tr>
<tr>
<td>h_e</td>
<td>Energy losses due to expansion</td>
<td>m or ft</td>
</tr>
<tr>
<td>K</td>
<td>Yarnell's pier shape coefficient</td>
<td>dimensionless</td>
</tr>
<tr>
<td>K</td>
<td>Total loss coefficient in modified form of Orifice Equation</td>
<td>dimensionless</td>
</tr>
<tr>
<td>K, K_1, K_2</td>
<td>Conveyance, conveyance at respective cross sections in FHWA WSPRO Method Equations</td>
<td>m³/s or cfs</td>
</tr>
<tr>
<td>L</td>
<td>Effective length of the weir in Weir Flow Equation</td>
<td>m or ft</td>
</tr>
<tr>
<td>L_ave</td>
<td>Effective flow length in approach reach in FHWA WSPRO Method Equations</td>
<td>m or ft</td>
</tr>
<tr>
<td>M</td>
<td>Mass of debris in Debris Force Equation</td>
<td>kg or slugs</td>
</tr>
<tr>
<td>P_1, P_2</td>
<td>Sinuosity in Channel Modification Equations, (ratio of stream length/valley length or valley slope/channel slope)</td>
<td>m/m or ft/ft</td>
</tr>
<tr>
<td>Q</td>
<td>Total discharge, design discharge</td>
<td>m³/s or cfs</td>
</tr>
<tr>
<td>S</td>
<td>Stopping distance in Debris Force Equation</td>
<td>m or ft</td>
</tr>
<tr>
<td>S_1, S_2</td>
<td>Channel slope in Channel Modification Equations</td>
<td>m/m or ft/ft</td>
</tr>
<tr>
<td>V, V_1, V_2, V_3, V_4</td>
<td>Velocity, velocity at respective cross sections in Yarnell Equation and FHWA WSPRO Method Equations</td>
<td>m/s or ft/s</td>
</tr>
<tr>
<td>v</td>
<td>Velocity of flow</td>
<td>m/s or ft/s</td>
</tr>
</tbody>
</table>
10.16 REFERENCES


Chapter 10 - Bridge Hydraulics


Federal Highway Administration (1990a). *Bridge Waterways Analysis Model* (WSPRO), FHWA-IP-89-027.


