10. Bridges

10.1. Introduction

10.1.1 Definition

Bridges are defined as:

- structures that transport vehicular traffic over waterways or other obstructions,
- part of a stream crossing system that includes the approach roadway over the flood plain, relief openings, and the bridge structure, and
- legally, structures with a centerline span of 20 feet or more. However, structures designed hydraulically as bridges as described above are treated in this chapter, regardless of length.

10.1.2 Analysis/Designs

Proper hydraulic analysis and design is as vital as the structural design.

Stream crossing systems should be designed for:

- minimum cost subject to criteria,
- desired level of hydraulic performance up to an acceptable risk level,
- mitigation of impacts on stream environment, and
- accomplishment of social, economic and environmental goals.

10.1.3 Purpose Of Chapter

- To provide guidance in the hydraulic design of a stream crossing system through:
    - o appropriate policy and design criteria, and
    - o technical aspects of hydraulic design.
- To present non-hydraulic factors that influence design including:
    - o environmental concerns,
    - o emergency access, traffic service, and
    - o consequence of catastrophic loss.
- To present a design procedure which emphasizes hydraulic analysis using the computer programs WSPRO and HEC-2.
- To present a brief section on design philosophy. A more in-depth discussion is presented in the AASHTO Highway Drainage Guidelines, Chapter VII(1).

10.2. Policy

10.2.1 General Policy

Policy is a set of goals and/or a plan of action. Federal policies and state policies that broadly apply to drainage design are presented in the Policy Chapter of this manual. Policies that are unique to bridge crossings are presented in this section.

The hydraulic analysis should consider various stream crossing system designs to determine the most cost effective proposal consistent with design constraints.

Policy provides guidelines subject to change as approved by the Department.

10.2.2 DOT&PF Policy

These policies identify specific areas for which quantifiable criteria can be developed.

- The final design selection should consider the maximum backwater allowed by the National Flood Insurance Program in regulatory floodways. However, it is the general Department policy to design for the maximum backwater increase which is economically feasible. Consideration must be given to the effects, economic or otherwise, of a backwater increase on adjacent private property.
- The final design should not significantly alter the flow distribution in the flood plain.
- The "crest-vertical curve profile" should be considered as the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge.

- A specified clearance, generally three feet, should be established to allow for passage debris. Clearance for the passage of ice or for icing conditions should be evaluated on a case-by-case basis, but is generally three feet or greater. For navigation channels, a vertical clearance conforming to Federal requirements should be established based on normally expected flows during the navigation season.
- Degradation or aggradation of the river as well as contraction and local scour shall be estimated, and appropriate positioning of the foundation, below the total scour depth if practicable, shall be included as part of the final design.

10.3. Design Criteria

10.3.1 General Criteria

Design criteria are the tangible means for placing accepted policies into action and become the basis for the selection of the final design configuration of the stream-crossing system. Criteria are subject to change when conditions so dictate as approved by State Hydraulics Engineer.

Following are the AASHTO general criteria related to the hydraulic analyses for the location and design of bridges as stated in the Highway Drainage Guidelines.

- Backwater will not significantly increase flood damage to property upstream of the crossing.
- Velocities through the structure(s) will not damage either the highway facility or increase damages to adjacent property.
- Maintain the existing flow distribution 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.
- Acceptable risks of damage or viable measures to counter the vagaries of alluvial streams.
- Minimal disruption of ecosystems and values unique to the flood plain and stream.
- Provide a level of traffic service compatible with that commonly expected for the class of highway and compatible with projected traffic volumes.
- Design choices should support costs for construction, maintenance and operation, including probable repair and reconstruction and potential liability, that are affordable.

10.3.2 Department Criteria

These criteria augment the general criteria. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using the water surface profile programs such as WSPRO or HEC-2.

<u>Travelway</u>

Inundation of the travelway dictates the level of traffic services provided by the facility. The travelway overtopping flood level identifies the limit of serviceability. Desired minimum levels of protection from travelway inundation for functional classifications of roadways are presented in Chapter 7.

<u>Risk Evaluation</u>

The selection of hydraulic design criteria for determining the waterway opening, road grade, scour potential, riprap and other features shall consider the potential impacts to:

- interruptions to traffic,
- adjacent property,
- the environment, and
- the infrastructure of the highway.

The consideration of the potential impacts constitutes an assessment of risk for the specific site. The least total expected cost (LTEC) alternative should be developed in accordance with FHWA HEC-17(3) where a need for this type of analysis is indicated by the risk assessment. This analysis provides a comparison between other alternatives developed in response to considerations such as environmental, regulatory, and political. (See Section 10.6.7)

<u>Design Floods</u>

Design floods for such things as the evaluation of backwater, clearance and overtopping shall be established predicated on risk based assessment of local site conditions. They shall reflect consideration of traffic service, environmental impact, property damage, hazard to human life, and flood plain management criteria. Travelway inundation from Section 10.3.2 which represents a frequency based design, shall be used to establish the minimum design flood.

Backwater/Increases Over Existing Conditions

Conform to FEMA regulations for sites covered by the National Flood Insurance Program (NFIP). Minimize backwater of the 1% exceedence probability flood for sites not covered by NFIP.

Clearance

Where practicable a minimum clearance of 3 ft shall be provided between the design approach water surface elevation and the low chord of the bridge for the final design alternative to allow for passage of ice and debris. Where this is not practicable, the clearance should be established by the designer based on the type of stream and level of protection desired as approved by the Department.

Flow Distribution

The conveyance of the proposed stream-crossing location shall be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility shall not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment shall be investigated if there is more than a 10% redistribution of flow.

Scour

Design for bridge foundation scour considering the magnitude of flood, through the 1% event, that generates the maximum scour depth. The design shall use a geotechnical design practice safety factor of from 2 to 3. The resulting design should then be checked using a super flood and a geotechnical design practice safety factor of 1.0 (See Section 10.6.8). The superflood is defined as the 0.2% event, 1.7 times the magnitude of the 1% event, or the overtopping flood, whichever is the least.

Abutment and embankment scour protection shall have a top elevation at least 1 ft above the design approach water surface elevation where practicable.

10.4. Design Procedure

10.4.1 Survey Accuracy (Computation Method)

The design for a stream crossing system requires a comprehensive engineering approach that includes formulation of alternatives, data collection, selection of the most cost effective alternative according to established criteria, and documentation of the final design.

Water surface profiles are computed for a variety of technical uses including:

- flood insurance studies,
- flood hazard mitigation investigations,
- drainage crossing analysis, and
- longitudinal encroachments.

The completed profile can affect the highway bridge design and is the mechanism for determining the effect of a bridge opening on upstream water levels.

Errors associated with computing water surface profiles with the step backwater profile method can be classified as:

- data estimation errors resulting from incomplete or inaccurate data collection and inaccurate data estimation,
- errors in accuracy of energy loss calculations depending on the validity of the energy loss equation employed and the accuracy of the energy loss coefficients (Manning's n-value is the coefficient measuring boundary friction),
- inadequate length of stream reach investigated, and
- significant computational errors resulting from using cross-sectional spacings which are incorrectly considered to be adequate.

The errors are due to inaccurate integration of the energy loss-distance relationship that is the basis for profile computations. This error may be reduced by adding interpolated sections (more calculation steps) between surveyed sections.

10.4.2 Design Procedure Outline

The following design procedure outline shall be used. Although the scope of the project and individual site characteristics make each design an unique one, this procedure shall be applied unless indicated otherwise by the Department.

I. Data Collection

A. Survey

1. Topography

2. Geology

3. Highwater marks

4. History of debris accumulation, ice, and scour

5. Review of hydraulic performance of existing structures

6. Maps, aerial photographs

7. Rainfall and stream gage records

8. Field reconnaissance

B. Studies by other agencies

1. Federal Flood Insurance Studies

2. Federal Flood Plain Studies by the ACOE, SCS, etc.

3. State and Local Flood Plain Studies

4. Hydraulic performance of existing bridges

C. Influences on hydraulic performance of site

1. Other streams, reservoirs, water intakes

2. Structures upstream or downstream

3. Natural features of stream and flood plain

4. Channel modifications upstream or downstream

5. Flood plain encroachments

6. Sediment types and bed forms

D. Environmental impact

1. Existing bed or bank instability

2. Flood plain land use and flow distribution

3. Environmentally sensitive areas (fisheries, wetlands, etc.)

E. Site-specific Design Criteria

1. Preliminary risk assessment

2. Application of agency criteria

II. Hydrologic Analysis

A. Watershed morphology

l. Drainage area

2. Watershed and stream slope

3. Channel geometry

B. Hydrologic computations

l. Discharge for historical flood that complements the high water marks used for calibration

2. Discharges for specified frequencies

III. Hydraulic Analysis

A. Computer model calibration and verification

B. Hydraulic performance for existing conditions

C. Hydraulic performance of proposed designs

IV. Selection of Final Design

A. Risk assessment/Least-cost alternative (LTEC)

B. Measure of compliance with established hydraulic criteria

C. Consideration of environmental and social criteria

D. Design details such as riprap, scour abatement, river training

V. Documentation

A. Complete project records, permit applications, etc.

B. Complete correspondence and reports

Checklist and Risk Assessment forms are presented in Appendix A.

10.4.3 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 WSPRO or HEC-2 unless indicated otherwise by the Department

Alternative methods of analysis of bridge hydraulics are discussed in this section but emphasis is placed on the use of WSPRO.

It is impracticable to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations. However, an example of the basic manual calculations is included in Appendix B as an explanation of the various aspects of bridge hydraulics.

The hydraulic variables and flow types are defined in Figures 10-1 and 10-2 on the next two pages.

- Backwater (h1) is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross-section (Section 1). It is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also 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 10-2.

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

- Type 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 the Type IIB it 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.

10.4.4 Methodologies

Momentum

The Corps of Engineers HEC-2 model uses a variation of the momentum method in the special bridge routine when there are bridge piers. The momentum equation between cross-sections 1 and 3 (Figure 10-2) is used to detect Type II flow and solve for the upstream depth in this case with critical depth in the bridge contraction.

This model has been used for the majority of the flood insurance studies performed under the NFIP. However, it is recognized that the bridge analysis routines in "Hydraulics of Bridge Waterways", FHWA:HDS-1 and WSPRO may yield a better definition of actual hydraulic performance at a bridge site. However it is also recognized that HEC-2 may be more computationally stable for difficult flow models.

Energy (HDS-1)

The method developed by FHWA described in HDS-1 and Appendix B is an energy approach with the energy equation written between cross sections 1 and 4 as shown in Figure 10-1 for Type I flow. The backwater is defined in this case as the increase in the approach water surface elevation relative to the normal water surface elevation without the bridge.

This model utilizes a single typical cross section to represent the stream reach from points 1 to 4 on Figure 10-1. It also requires the use of a single energy gradient. This method is no longer recommended for final design analysis of bridges due to its inherent limitations but it may be useful for preliminary analysis and training. Studies performed by the Corps of Engineers for the FHWA show the need to utilize a multiple cross section method of analysis in order to achieve reasonable stage-discharge relationships at a bridge.

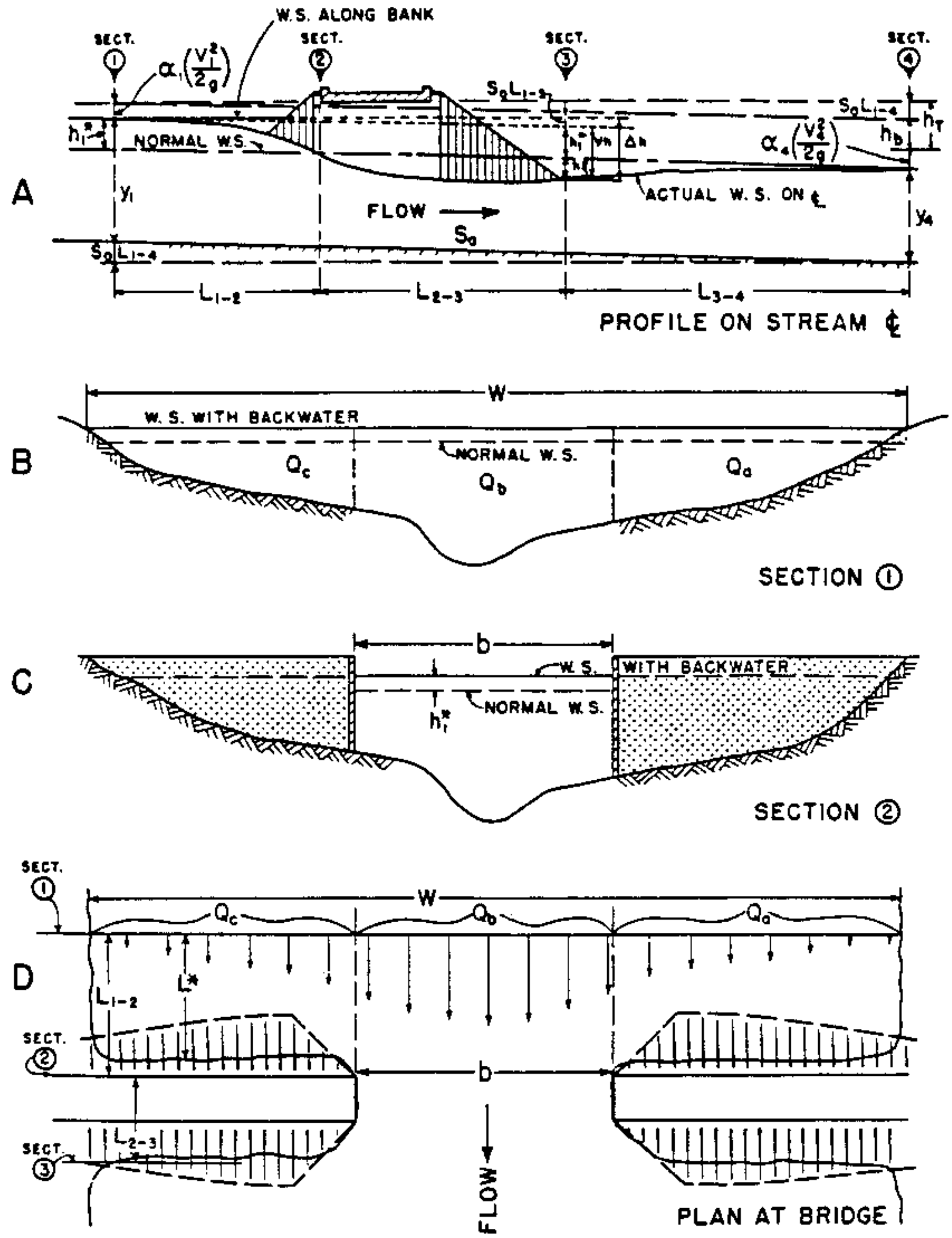

Figure 10-1
Bridge Hydraulics Definition Sketch

Source: HDS-1

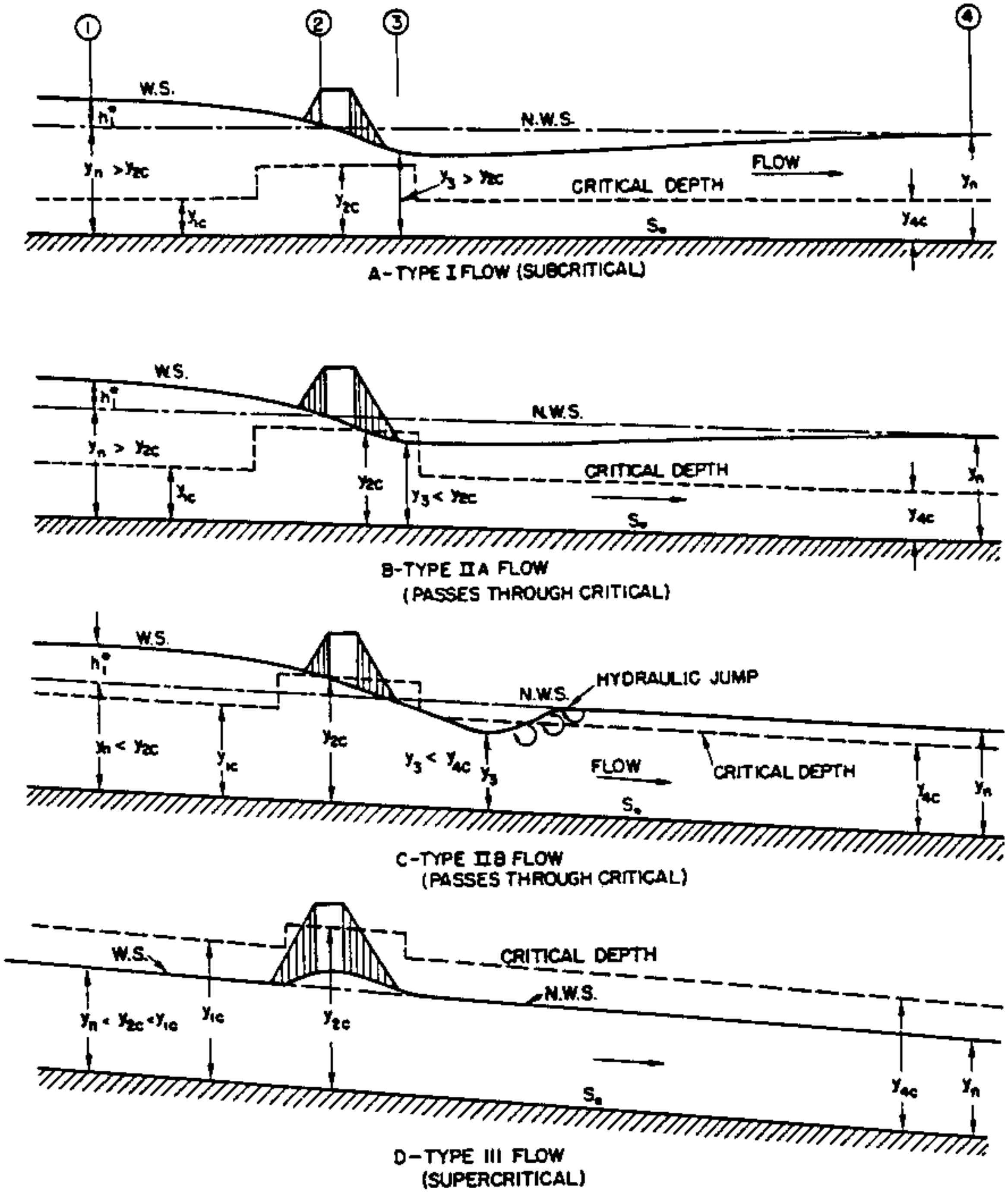

Figure 10-2
Bridge Flow Types

Source: HDS-1

Energy (WSPRO)

WSPRO combines step-backwater analysis with bridge backwater calculations. This method allows for pressure flow through the bridge, embankment overtopping, and flow through multiple openings and culverts. The bridge hydraulics still rely on the energy principle, but there is an improved technique for determining approach flow lengths and the introduction of an expansion loss coefficient. The flow-length improvement was found necessary when approach flows occur on very wide heavily-vegetated flood plains. The program also greatly facilitates the hydraulic analysis required to determine the least-cost alternative.

The use of WSPRO is recommended for both preliminary and final analyses of bridge hydraulics. Even if only a single surveyed cross section is available the input-data propagation features of WSPRO make it easy to apply with more comprehensive output available than with HDS-1.

Other Models

The USGS computer model E431 and the U.S. Soil Conservation Service computer model WSP-2 are recognized methods for computing water surface profiles.

Two-Dimensional Modeling

The water surface profile and velocities in a section of river are often predicted using a computer model. In practice, most analysis is performed using one-dimensional methods such as the standard step method found in WSPRO or HEC-2. 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 more complex and require more time to set up and calibrate. They require somewhat more field data than a one-dimensional model and, depending on complexity, may require a little more computer time.

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. The USGS has developed a two-dimensional flow finite element computer program for the FHWA that is designated FESWMS. This program 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, spur dikes, flood plain encroachments, multiple channels, flow around islands and flow in estuaries. A similar program named FastTabs, which uses two dimensional finite element analysis code from the Corps of Engineers and a graphical interface developed at Brigham Young University, provides much the same analysis tools while providing an improved input and output interface.

Physical Modeling

Complex hydrodynamic situations defy accurate or practicable mathematical modeling. Physical models should be considered when:

- hydraulic performance data is needed that cannot be reliably obtained from mathematical modeling,
- risk of failure or excessive over-design is unacceptable, and
- research is needed.

The constraints on physical modeling are:

- size(scale),
- cost, and
- time.

10.4.5 WSPRO 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 bridge backwater. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge without spur dikes are shown in Figure 10-3. 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 spur dikes and a roadway profile are specified.

Pressure flow through a 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 the following table:

Flow Classification According to Submergence Conditions (WSPRO User Instructors Manual - 1987)

Flow Through Bridge	Flow Through Bridge Opening
Opening Only	and Over Road Grade
Class 1 - Free surface flow	Class 4 - Free surface flow
Class 2 - Orifice flow	Class 5 - Orifice flow
Class 3 - Submerged orifice flow	Class 6 - Submerged orifice flow

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.

A user's instruction manual for WSPRO should serve as a source for more detailed information on using the computer program.

10.4.6 HEC-2 Modeling

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 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 bridge backwater. 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 10-4.

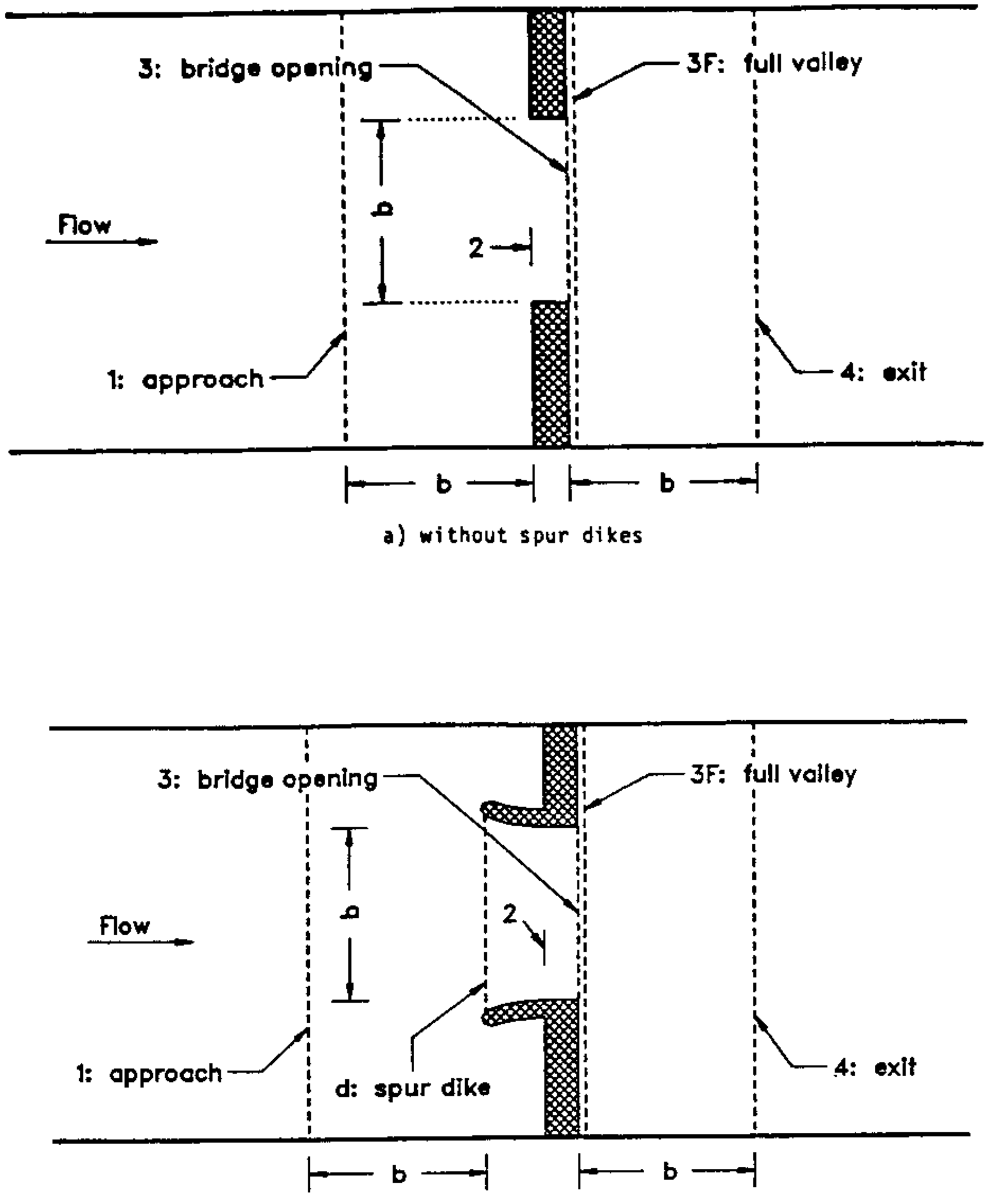

Figure 10-3
Cross-Section Locations For Stream Crossing With A Single Waterway Opening

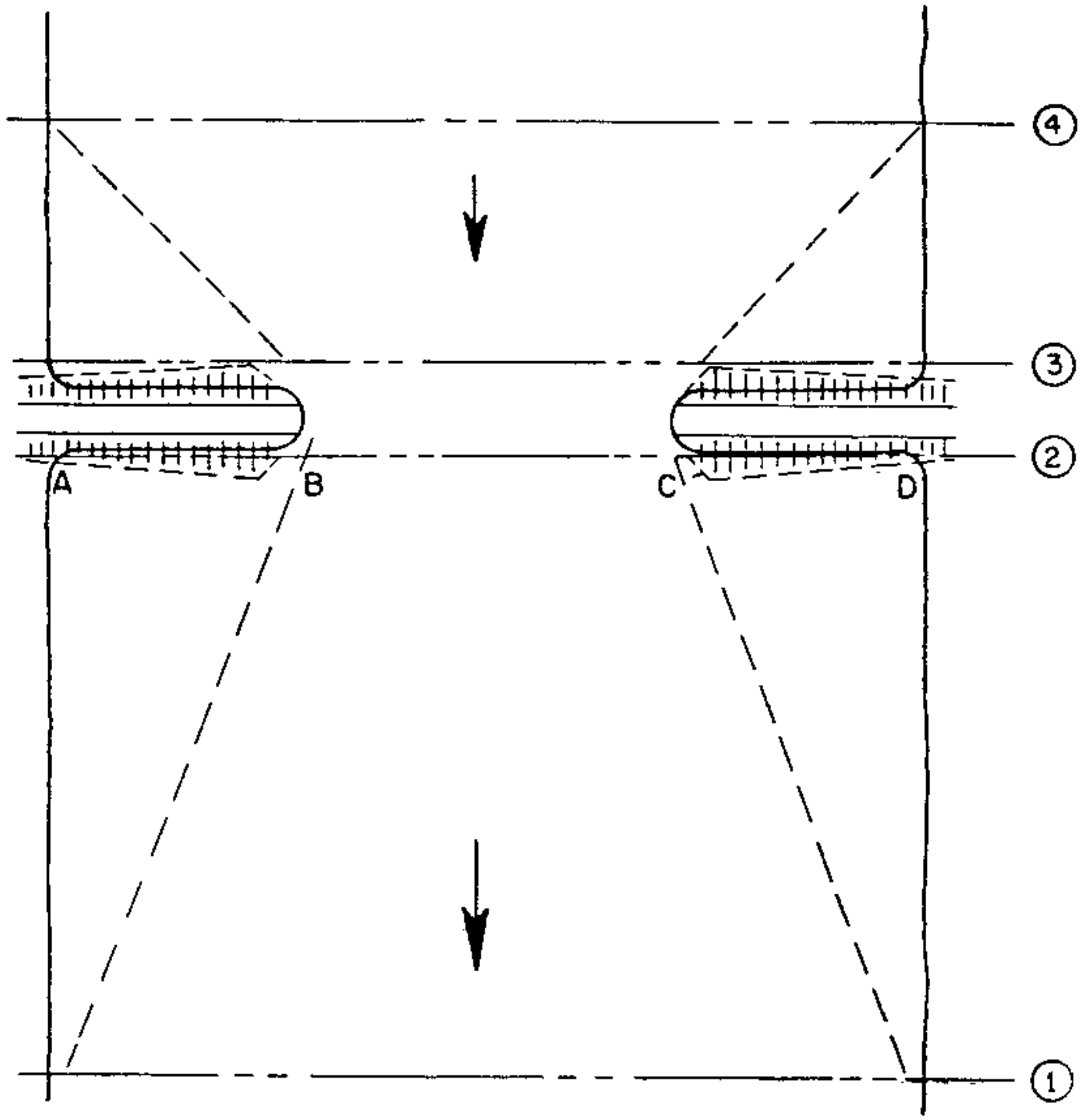

Figure 10-4
Cross-Section Locations In The Vicinity Of Bridges

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, even though data was prepared for the special bridge method, for bridges without piers and under low flow control.

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 are 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 flood plain 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. Normally the computations are based on the assumption that all area below the water surface elevation is effective in passing the discharge. However, if the water surface elevation at a particular cross section is less than the top of levee elevations, and if the water cannot enter or leave the overbanks upstream of that cross section,then the flow areas in these overbanks should not be used in the computations. Variable IEARA on the X3 card and the bank stations coded in fields three and four on the X1 card are used for this condition. By setting IEARA equal to ten the program will consider only flow confined by the levees, unless the water surface elevation is above the top of one or both of the levees, in which case flow area or areas outside the levee(s) will be included. If this option is employed and the water surface elevation is close to the top of a levee, it may not be possible to balance the assumed and computed water surface elevations due to the changing assumptions of flow area when just above and below the levee top. When this condition occurs, a note will be printed that states that the assumed and computed water surface elevations for the cross section cannot be balanced. A water surface elevation equal to the elevation which came closest to balancing will be adopted. It is then up to the program user to determine the appropriateness of the assumed water surface elevation and start the computation over again at that cross section if required.

It is important for the user to study carefully the flow pattern of the river where levees exist. If, for example, a levee were open at both ends and flow passed behind the levee without overtopping it, IEARA equals zero or blank should be used. Also, assumptions regarding effective flow areas may change with changes in flow magnitude. Where cross section elevations outside the levee are considerably lower than the channel bottom, it may be necessary to set IEARA equal to ten to confine the flow to the channel.

A user's instruction manual for HEC-2 is available and should serve as a source for more detailed information on using this computer model.

10.4.7 General Modeling

The above discussions illustrate the complexities involved in modeling open channel flow even with relatively simple one dimensional approximations. There are other issues as well of which the designer need be aware if he is to use these tools effectively and accurately, such as optimizing the placement of cross sections, the use of interpolated cross sections, identification of subtle convergence errors such as slope swapping, treatment of supercritical flow, the use of parameters which can be "tweaked", and others. It cannot be stressed too strongly that the use of these tools without proper training can lead to inaccurate or erroneous results.

10.5. Bridge Scour and Aggradation

10.5.1 Introduction

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour and also loss of hydraulic area due to deposition (aggradation). Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis. The hydraulic engineer must endeavor to always be aware of and use the most current scour forecasting technology.

The FHWA issued a Technical Advisory (TA 5140.20) on bridge scour in September 1988. This document "Interim Procedures for Evaluating Scour at Bridges" was an attachment to the Technical Advisory. The interim procedures were replaced by HEC-18 issued in 1991 and revised in 1993. Users of this manual should consult HEC-18 for a more thorough treatise on scour and scour prediction methodology. A companion FHWA document to HEC-18 is HEC-20, "Stream Stability at Highway Structures".

The inherent complexities of stream stability, furthered complicated by highway stream crossings, requires a multilevel solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative analysis using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations. This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in HEC-20.

Less hazardous perhaps are problems associated with aggradation. Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the traveling public due to more frequent overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these consequences. Also, aggradation in a stream reach may serve to moderate potential scour depths. Aggradation is sometimes referred to as negative scour.

10.5.2 Scour Types

Present technology dictates that bridge scour be evaluated as interrelated components:

- long term profile changes (aggradation/degradation),
- plan form change (lateral channel movement),
- contraction scour/deposition, and
- local scour.

Long Term Profile Changes

Long term profile changes can result from stream bed profile changes that occur from aggradation and/or degradation.

- Aggradation is the deposition of bedload due to a decrease in the energy gradient.
- Degradation is the scouring of bed material due to increased stream sediment transport capacity which results from an increase in the energy gradient.

Forms of degradation and aggradation shall be considered as imposing a permanent future change for the stream bed elevation at a bridge site whenever they can be identified.

Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

Contraction

Channel contraction scour results from a constriction of the channel which may, in part, be caused by bridge piers in the waterway. Highways, bridges, and natural channel contractions are the most commonly encountered cause of constriction scour. Two practices are provided in this manual for estimating contraction scour.

1. Sediment routing practice - This practice should be considered should either bed armoring or aggradation from an expanding reach be expected to cause an unacceptable hazard.
2. Empirical practice - This practice is adapted from laboratory investigations of bridge contractions in non-armoring soils and, as such, must be used considering this qualification. This practice does not consider bed armoring and its application for aggradation may be technically weak.

The same empirical practice algorithms used in this manual to evaluate a naturally contracting reach may also be used to evaluate deposition in an expanding reach provided armoring is not expected to occur. With deposition the practice of applying the empirical equations "in reverse" is required; i.e., the narrower cross section is upstream which results in the need to manipulate the use of the empirical "contraction scour" equation. This need to manipulate the intended use of an equation does not occur with the sediment routing practice which is why it may be more reliable in an expanding reach.

Local Scour

Exacerbating the potential scour hazard at a bridge site are any abutments or piers located within the flood flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry. However, the importance of these geometric variables will vary. As an example, increasing the pier or cofferdam width either through design or debris accumulation will increase the amount of local scour, but only up to a point in subcritical flow streams. After reaching this point, pier scour should not be expected to measurably increase with increased stream velocity or depth. This threshold has not been defined in the more rare, supercritical flowing streams.

10.5.3 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 arrested by armoring before the full scour potential is reached again 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 formulae developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs than used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring may also cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further, 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.

10.5.4 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 scoured 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.

10.5.5 Scour Analysis Methods

Before the various scour forecasting methods for contraction and local scour can be applied it is first necessary to (l) 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 manual for combining the contraction and local scour components to obtain total scour. The first method identified as Method 1 shall 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. The second method, Method 2, shall have application where armoring is not a concern or insufficient information is available to permit its evaluation, or where more precise scour estimates are not deemed necessary.

Method 1

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 also 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 this 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 or deposition.
- Once again 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 or deposition, calculate the local scour.
- Extend the local scour assessment below the predicted contractions scour depths in order to obtain the total scour.

Method 2

This is considered to be a conservative practice as it assumes that the scour components develop independently. Thus, as indicated with Method 1 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, estimate the deposition by "reversing" the empirical equation application and considering deposition as "negative" scour.
- Estimate local scour using the adjusted fixed bed channel and bridge hydraulics assuming no bed armoring.
- Add the local scour to the contraction scour or aggradation ("negative" scour) to obtain the total scour.

10.5.6 Scour Assessment Procedure

Bridge scour assessment shall normally be accomplished by collecting the data and applying the general procedure outlined in this section.

Site Data

<u>Bed Material</u>

Obtain bed material samples for all channel cross sections when armoring is to be evaluated. If armoring is not being evaluated, this information need only be obtained at the site. From these samples try to identify historical scour and associate it with a discharge. Also, determine the bed material size distribution in the bridge reach and from this distribution determine d16, d50, d84, and d90.

<u>Geometry</u>

Obtain existing stream and flood plain cross sections, stream profile, site plan and the stream's present, and where possible, historic geomorphic plan form. Also, locate the bridge site with respect to such things as other bridges in the area, tributaries to the stream or close to the site, bed rock controls, manmade controls (dams, old check structures, river training works, etc.), and downstream confluence with other streams. Locate (distance and height) any "headcuts" due to natural causes or such things as gravel mining operations. Upstream gravel mining operations may absorb the bed material discharge resulting in the more adverse clear water scour case discussed later. Any data related to plan form changes such as meander migration and the rate at which they may be occurring are useful.

<u>Historic Scour</u>

Any scour data on other bridges or similar facilities along the stream.

<u>Hydrology</u>

Identify the character of the stream hydrology; i.e., perennial, ephemeral, intermittent as well as whether it is "flashy" or subject to broad hydrograph peaks resulting from gradual flow increases such as occur with general thunderstorms or snowmelt.

Geomorphology

Classify the geomorphology of the site; i.e., such things as whether it is a flood plain stream, crosses a delta, or crosses an alluvial fan; youthful, mature or old age.

General

Step 1

Decide which analysis method is applicable. Method 2 shall be used as the general method to quickly evaluate existing bridges to identify significant potential scour hazards or, where armoring or an expanding reach are obviously not of concern, on a proposed bridge. Method 1 shall be used to evaluate bridges where armoring or an expanding reach are of concern as well as where Method 2 indicates a significant potential scour hazard may exist.

Step 2

Determine the magnitude of the base flood and "super flood" as well as the magnitude of the incipient overtopping flood, or relief opening flood. Accomplish steps 3 through 12 using the discharge that places the greatest stress on the bed material in the bridge opening.

Step 3

Determine the bed material size which will resist movement and cause armoring to occur.

Step 4

Develop a water surface profile through the site's reach for fixed bed conditions using WSPRO or HEC-2.

Step 5

Assess the bridge crossing reach of the stream for profile bed scour changes to be expected from degradation or aggradation. Again, take into account past, present and future conditions of the stream and watershed in order to forecast what the elevation of the bed might be in the future. Certain plan form changes such as migrating meanders causing channel cutoffs would be important in assessing future streambed profile elevations. The possibility of downstream mining operations inducing "headcuts" shall be considered. The quickest way to assess streambed elevation changes due to "headcuts" (degradation) is by obtaining a vertical measurement of a downstream "headcut(s)" and projecting that measurement(s) to the bridge site using the existing stream profile assuming the stream is in regime; if it is not, then it may be necessary to estimate the regime slope. A more time consuming way to assess elevation changes would be to use some form of sediment routing practice in conjunction with a synthetic flood history.

Step 6

Assess the bridge crossing reach of the stream for plan form scour changes. Attempt to forecast whether an encroaching meander will cause future problems within the expected service life of the road or bridge. Take into account past, present and expected future conditions of the stream and watershed in order to forecast how such meanders might influence the approach flow direction in the future. The sediment routing practice discussed later for computing channel contraction scour or aggradation may prove useful in making such assessments -- particularly if coupled to a synthetic flood history. This forensic analysis on a site's past geomorphological history to forecast the future may prove useful. Otherwise this assessment have to be largely subjective in nature.

Step 7

Based on the expected profile and plan form scour changes, adjust the fixed bed hydraulic variables and parameters.

Step 8

Assess the magnitude of channel or bridge contraction scour using Method 1 or Method 2 based on the fixed bed hydraulics of Step 7.

Step 9

Assess the magnitude of local scour at abutments and piers using Method 1 or Method 2.

Step 10

Plot the scour and aggradation depths from foregoing steps on a cross section of the stream channel and flood plain at the bridge site. Using judgment, enlarge any overlapping scour holes (discussed later). Treat any aggradation as a negative scour.

Step 11

Evaluate the findings of Step 10. If the scour is unacceptable, consider the use of scour countermeasures or revise the trial bridge opening and repeat the foregoing steps.

Step 12

Once an acceptable scour threshold is determined, the geotechnical engineer can make a preliminary foundation design for the bridge based on the scour information obtained from the foregoing procedure and using commonly accepted safety factors. The structural engineer should evaluate the lateral stability of the bridge based on the foregoing scour.

Step 13

Repeat the foregoing assessment procedures using the greatest bridge opening flood discharge associated with the selected "super flood". These findings are again for the geotechnical engineer to use in evaluating the foundation design obtained in Step 12. A foundation design safety factor of 1.0 is commonly used to ensure that the bridge is marginally stable for a flood associated with the "super flood".

10.6. Philosophy

10.6.1 Introduction

Any stream is a dynamic natural system which, as a result of the encroachment caused by elements of a stream-crossing system, will respond in a way that may well challenge even an experienced hydraulic engineer. The complexities of the stream response to encroachment demand that: (l) hydraulic engineers must be involved from the outset in the choice of alternative stream crossing locations, and (2) at least some of the members of the engineering design team must have extensive experience in the hydraulic design of stream-crossing systems. Hydraulic engineers should also be involved in the solution of stream stability problems at existing structures.

This section discusses qualitatively some of the design issues which contribute to the overall complexity of spanning a stream with a stream-crossing system. A much more thorough discussion of design philosophy and design considerations is found in the AASHTO Highway Drainage Guidelines, "Hydraulic Analyses for the Location and Design of Bridges".

10.6.2 Location Of Stream Crossing

Although many factors, including nontechnical ones, enter into the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include flood plain width and roughness, flow distribution and direction, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location also affect environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. Finally, the hydraulics of a particular site determine whether or not certain national objectives such as wise use of flood plains, reduction of flooding losses, and preservative of wetlands can be met.

10.6.3 Coordination, Permits, Approvals

The interests of other government agencies must be considered in the evaluation of a proposed stream-crossing system, and cooperation and coordination with these agencies, especially water resources planning agencies, must be undertaken. Coordination with the Federal Emergency Management Agency (FEMA) is required when a:

- proposed crossing encroaches on a regulatory floodway and would require an amendment to the floodway map,
- proposed crossing encroaches on a flood plain where a detailed study has been performed but no floodway has been designated and the maximum one foot increase in the base flood would be exceeded,
- community is expected to enter into the regular program within a reasonable period and detailed flood plain studies are underway, and
- community is participating in the emergency program and the base flood elevation in the vicinity of insurable buildings is increased by more than one foot.

Whenever practicable, the stream-crossing system shall avoid encroachment on a regulatory floodway within a flood plain. When this is not feasible, modification of the floodway itself shall be considered. If neither of these alternatives is feasible, FEMA regulations for "floodway encroachment where demonstrably appropriate" shall be met.

Designers of stream-crossing systems must be cognizant of relevant local, State, and Federal laws and permit requirements. Federal permits are required for construction of bridges over navigable waters and are issued by the U.S. Coast Guard. Permits for other construction activities in navigable waters are under the jurisdiction of the U.S. Army Corps of Engineers. Applications for Federal permits may require environmental impact assessments under the National Environmental Policy Act of l969.

10.6.4 Environmental Considerations

Environmental criteria which must be met in the design of stream-crossing systems include the preservation of wetlands and protection of aquatic habitat. Such considerations often require the expertise of a biologist. Water quality considerations shall also be included in the design process insofar as the stream-crossing system affects the water quality relative to beneficial uses. As a practical matter with bridges, the hydraulic design criteria related to scour, degradation, aggradation, flow velocities, and lateral distribution of flow, for example, are important criteria for evaluation of environmental impacts as well as the safety of the stream-crossing structures.

10.6.5 Stream Morphology

The form and shape of the stream path created by its erosion and deposition characteristics comprise its morphology. A stream can be braided, straight, or meandering, or it can be in the process of changing from one form to another as a result of natural or manmade influences. A historical study of the stream morphology at a proposed stream-crossing site is extremely important. This study should also include an assessment of any long-term trends in aggradation or degradation. Braided streams and alluvial fans shall especially be avoided for stream-crossing sites whenever possible.

10.6.6 Surveys

The purpose of surveys is to gather all necessary site information. This shall include such information as topography and other physical features, land use and culture, flood data, basin characteristics, precipitation data, historical high-water marks, existing structures, channel characteristics, and environmental data. A site plan shall be developed on which much of the survey data can be shown.

10.6.7 Risk Evaluation

The evaluation of the consequence of risk associated with the probability of flooding attributed to a stream-crossing system is a tool by which site specific design criteria can be developed. This evaluation considers capital cost, traffic service, environmental and property impacts, and hazards to human life.

The evaluation of risk is a two stage process. The initial step, identified as risk assessment, is more qualitative than a risk analysis and serves to identify threshold values that must be met by the hydraulic design. A form to be used for documenting this assessment is presented in Appendix A.

In many cases where the risks are low and/or threshold design values can be met, it is unnecessary to pursue a detailed economic analysis. In those cases where the risk are high and/or threshold values cannot be met, a Least Total Expected Cost (LTEC) analysis should be considered.

The results of a least-cost analysis can be presented in a graph of total cost as a function of the overtopping discharge. The total cost consists of a combination of capital costs and flood damages (or risk costs). Risk costs decrease with increases in the overtopping discharge while capital costs simultaneously increase. The overtopping discharge for each alternative is determined from a hydraulic analysis of a specific combination of embankment height and bridge-opening length. The resulting least-cost alternative provides a tradeoff comparison. If, for example, environmental criteria result in an alternative that is different from the least-cost alternative, the economic tradeoff cost of that alternative can be given as the difference between its cost and the minimum cost provided by a LTEC analysis.

The alternatives considered in the least-cost analysis do not require the specification of a particular design flood. This information is part of the output of the least-cost analysis. In other words, the least-cost alternative has a specific risk of overtopping that is unknown before the least-cost alternative has been determined. Therefore, design flood frequencies are used only to establish the initial alternative. Thereafter, specific flood-frequency criteria such as the 50-year flood requirement for interstate highways and the 100-year flood plain requirements for flood insurance should be considered only as constraints on the final design selection. Deviation from the least-cost alternative may be necessary to satisfy these constraints and the trade-off cost for doing so can be obtained from the least-cost analysis.

Risk based analysis does not recognize some of the intangible factors that influence a design. The minimum design that results from this type of analysis may be too low to satisfy the site condition.

10.6.8 Scour

The extreme hazard posed by bridges subject to bridge scour failures dictates a different philosophy in selecting suitable flood magnitudes to use in the scour analysis. With bridge flood hazards other than scour, such as those caused by roadway overtopping or property damage from inundation, a prudent and reasonable practice is to first select a design flood to determine a trial bridge opening geometry. This geometry is either subjectively or objectively selected based on the initial cost of the bridge along with the potential future costs for flood hazards. Following the selection of this trial bridge geometry, the base flood (100-year) is used to evaluate this selected opening. This two step evaluation process is used to ensure the selected bridge opening based on the design flood contains no unexpected increase in any existing flood hazards other than those from scour or aggradation. With bridge scour, not only is it required to consider bridge scour or aggradation from the base flood, but also an even larger flood termed herein as the "super flood", defined as the 500-year flood or the overtopping flood, whichever is least.

Scour prediction technology is steadily developing, but lacks at this time, the reliability associated with other facets of hydraulic engineering. Several formulae for predicting scour depths are currently available and others will certainly be developed in the future. The designer should strive to be acquainted with the "state of practice" at the time of a given analysis and is encouraged to be conservative in the resulting scour predictions.

First discussion is warranted as to what constitutes the greatest discharge passing through the bridge opening during a particular flood. Even where there are relief structures on the flood plain or overtopping occurs, some flood other than the base flood or "super flood" may cause the worse case bridge opening scour. This situation occurs where the bridge opening will pass the greatest discharge just prior to incurring a discharge relief from overtopping or a flood plain relief opening. Conversely care must be exercised in that a discharge relief at the bridge due to overtopping or relief openings may not result in reduction in the bridge opening discharge. Should a reduction occur, the incipient overtopping flood or the overtopping flood corresponding to the base flood or "super flood" would be used to evaluate the bridge scour.

With potential bridge scour hazards a different flood selection and analysis philosophy is considered reasonable and prudent. The foregoing trial bridge opening which was selected by considering initial costs and future flood hazard costs shall be evaluated for two possible scour conditions with the worse case dictating the foundation design -- and possibly a change in the selected trial bridge opening.

First, evaluate the proposed bridge and road geometry for scour using the base flood, incipient overtopping flood, overtopping flood corresponding to the base flood, or the relief opening flood whichever provides the greatest flood discharge through the bridge opening. Once the expected scour geometry has been assessed, the geotechnical engineer would design the foundation. This foundation

Design would use the conventional foundation safety factors and eliminate consideration of any stream bed and bank material displaced by scour for foundation support.

Second, impose a "super flood" on the proposed bridge and road geometry. This event shall be greater than the base flood and shall be used to evaluate the proposed bridge opening to ensure that the resulting potential scour will produce no unexpected scour hazards. Similar to the base flood to evaluate the selected bridge opening, use either the "super flood", or the overtopping flood, whichever imposes the greatest flood discharge on the selected bridge opening. The foundation design based on the base flood would then be reviewed by the geotechnical engineer using a safety factor 1.0 and again, taking into account any stream bed and bank material displaced by scour from the "super flood".

10.6.9 Preventive/Protection Measures

Based on an assessment of potential scour provided by the Hydraulic Engineer, the structural designers can incorporate design features that will prevent or mitigate scour damage at piers. In general, circular piers or elongated piers with circular noses and an alignment parallel to the flow direction are a possible alternative. Spread footings should be used only where the stream bed is extremely stable below the footing and where the spread footing is founded at a depth below the maximum scour computed in Section 10.6.8. Drilled shafts or drilled piers are possible where pilings cannot be driven. Protection against general stream bed degradation can be provided by drop structures or grade-control structures in, or downstream of the bridge opening.

Rock riprap is often used, where stone of sufficient size is available, to armor abutment fill slopes and the area around the base of piers. Riprap design information is presented in Appendix B of this chapter and in Chapter 17.

Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment slope should be avoided. Embankment overtopping may be incorporated into the design but should be located well away from the bridge abutments and superstructure.

Spur dikes are recommended to align the approach flow with the bridge opening and to prevent scour around the abutments. They are usually elliptical shaped with a major to minor axis ratio of 2.5 to 1. Some states have found that a length of approximately 150 ft provides a satisfactory standard design. Their length can be determined according to HDS-1 (2). Spur dikes, embankments, and abutments shall be protected by rock riprap with a filter blanket or other revetments approved by the Department.

10.6.10 Deck Drainage

Improperly drained bridge decks can cause numerous problems including corrosion, icing, and hydroplaning. Whenever possible, bridge decks should be watertight and all deck drainage should be carried to the ends of the bridge. Drains at the end of the bridge should have sufficient inlet capacity to carry all bridge drainage.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of the interceptors shall conform to the procedures in HEC-12.

10.6.11 Construction/Maintenance

Construction plans, including shop drawings and temporary structure details, should be reviewed by the Department's Hydraulic Engineer. Temporary structures and crossings used during construction should be designed for a specified risk of failure due to flooding during the construction period. The impacts on normal water levels, fish passage, and normal flow distribution must be considered. The Department's Hydraulic Engineer should attend the Pre-construction meeting to discuss any hydraulic concerns with the Contractor and the Department's Construction personnel.

All borrow areas existing within the flood plain shall be chosen so as to minimize the potential for scour and adverse environmental effects within the limits of the bridge and its approaches on the flood plain.

The stream-crossing design shall incorporate measures which reduce maintenance costs whenever possible. These measures include spur dikes, retards, guide dikes, jetties, riprap protection of abutments and embankments, embankment overflow at lower elevations than the bridge deck, and alignment of piers with the flow.

10.6.12 Waterway Enlargement

There are situations where roadway and structural constraints dictate the vertical positioning of a bridge and result in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases.

It is possible to increase the effective area by excavating a flood channel through the reach affecting the hydraulic performance of the bridge. There are, however, several factors that must be accommodated when this action is taken.

- The flow line of the flood channel should be set above the stage elevation of the dominant discharge: (See AASHTO Highway Drainage Guidelines)
- The flood channel must extend far enough up and downstream of the bridge to establish the desired flow regime through the affected reach.
- The flood channel must be stabilized to prevent erosion and scour.

10.6.13 Auxiliary Openings

The need for auxiliary waterway openings, or relief openings as they are commonly termed, arises on streams with wide flood plains. The purpose of openings on the flood plain is to pass a portion of the flood flow in the flood plain 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 has predictable capacity during flood events.

Basic objectives in choosing the location of auxiliary openings include:

- maintenance of flow distribution and flow patterns,
- accommodation of relatively large flow concentrations on the flood plain,
- avoidance of flood plain flow along the roadway embankment for long distances, and
- crossing of significant tributary channels.

The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow. The development of 2-

D models is a major step toward more adequate analysis of complex stream-crossing systems.

The most complex factor in designing auxiliary openings is determining the division of flow between the two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. The design of auxiliary openings should usually be generous to guard against that possibility.

10.7. References

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Kindsvater, C.E., "Discharge Characteristics of Embankment-Shaped Weirs, "U.S. Geological Survey, WSP 1607-A, 1964.

Matthai, H.F., "Measurement of Peak Discharge at Width Contractions by Indirect Methods, "U.S. Geological Survey, Techniques of Water Resources Investigations, Book 3, Ch. A4, 1967.

Schneider, V.R., Board, J.W., Colson, B.E., Lee, F.N., and Druffel, L., "Computation of Backwater and Discharge at Width Constriction of Heavily Vegetated Flood Plains, "U.S. Geological Survey, WRI 76-129, 1977.

Federal Highway Administration, "Drainage at Highway Pavements", HEC-12, 1984.

Appendix A: Forms

Design Procedure Form

Engineer ____________________________

City/Borough ________________________

Project ____

Description _______________________________________

MAPS:

____

USGS Quad Scale Date
Other Maps
Local Zoning Maps
Flood Hazard Delineation (Quad.)
Flood Plain Delineation (HUD)
Flood Insurance Firm & FHBM
Local Land Use
Soils Maps
Geologic Maps
Aerial Photos Scale Date

____

STUDIES BY INTERNAL SOURCES:

____

Hydraulics Sect. Records
Regional Drainage Records
Maintenance Records
Flood Record (High Water, Newspaper)

____

STUDIES BY EXTERNAL AGENCIES:

____

FEMA Flood Insurance Studies
USACE Flood Plain Inform. Report
SCS Watershed Studies PFP-HYDRO
Local Watershed Management
USGS Gages & Studies
Interim Flood Plain Studies
Water Resources Data
Regional Planning Data
Forestry Service
Utility Company Plans

____

TECHNICAL RESOURCES:

____

ADOT&PF Drainage Manual
Technical Library

____

____

DISCHARGE CALCULATIONS:

____

Drainage Areas
Formula
HEC-1
SCS
Gaging Data - Regional Analysis
Regression Equations
Area-Discharge Curves
Log-Pearson Type III Gage Rating

____

HIGH WATER ELEVATIONS:

____

ADOT&PF Survey
External Sources
Personal Reconnaissance

____

____

FLOOD HISTORY:

____

____

____

CALIBRATION OF HIGH WATER DATA:

____

Discharge and Frequency of H.W. el.
Influences Responsible for H.W. el.

____

____

____

Analyze Hydraulic Performance of Facility for Min. Flow through 100 Yr.

Analyze Hydraulic Performance of Proposed Facility for Min. Flow through 100 Yr.

Reconaissance Reports

____

____

Design Procedure Form (continued)

DATA REPORTS:

ADOT&PF Data
Other Agency Data

ENVIRONMENTAL REPORTS:

Agency
Surface Water Envir. Study
Surface Water Envir. Revisions
Recon. Reports

Location Report
Location Revisions Report
Drainage Survey Inspec. Report
Drainage Sur. Insp. Report Revisions
Hydraulic Design Report
Hydra. Design Report Revisions
Construction Report
Construction Report Revisions
Hydraulic Operation Report
Hydra. Oper. Report Revisions

TECHNICAL AIDS:

ADOT&PF Drainage Manual
ADOT&PF & FHWA Directives
Technical Library

DESIGN APPURTENANCES:

Dissipators
Riprap
Erosion & Sediment Control
Fish & Wildlife Protection
External Sources
Personal Reconnaissance
Maintenance Records

COMPUTER PROGRAMS:

HY8, CDS
Direct Step Water Surface Profile
USACE HEC-2 Water Surface Profile
FHWA Bridge Backwater
Log-Pearson Type III Analysis
WSPRO
PFP-HYDRA
FESWMS
FastTabs

Compiled by: ______________________ Scheme No. ____________

Date ______________________________

Preliminary Risk Assessment Checklist

(Predicated on Engineering Judgement based on Survey and Plans)

Check Off

1. Potential risk to human life due to flood pool upstream and/or "Dam Break - Flood Wave" downstream ____________

2. Damage to adjacent property by changes in hydraulic characteristics ____________

3. Damage to highway facility _______________________________ ____________

   ____

   ____

   ____

4. Traffic Service

   ADT ___________ Detours Available _________ ____________

   Describe detour (i.e. Rte...to Rte... to Rte..., Length...miles)____

   ____

   ____

   ____

5. Flood Plain Management Criteria ____________
   Specify: ____

   ____

   ____

   ____

Preliminary Risk Assessment Checklist (continued)

6. Flood Plain Impacts ______________________________________ ____________

____

____

____

7. Other Pertinent Factors ___________________________________ ____________

____

____

____

ADOT&PF Risk Assessment For Final Design

LOCATION

City/Borough ______________________ Civil Twp. _____ Sec. _____ Twp. _____ Range _____
Over (River, Cr., Dr. Ditch) ______________________________ Road No. _________________
Project No. _________________________ Design Number ___________ FHWA No. ________
Assessment Prepared by ________________________________ Date ____________________

1. HYDROLOGIC EVALUATION

A. Nearest Gaging Station on this stream __________ (None _______) ___________
____

B. Are flood studies available on this stream: _______________________________

C. Flood Data:

Q10 ______ cfs Est. Bkwtr. _____ ft. Q25 cfs Est. Bkwtr. _____ ft.

Q50 ______ cfs Est. Bkwtr. _____ ft. Ql00 cfs Est. Bkwtr. _____ ft.

Q500 _____ cfs or Overtopping ________________cfs Est. Bkwtr. _____ ft.

Drainage Area _______________ Method Used to compute Q ____________________

D. Does the crossing require outside agency approval? Yes _______ No _______
List Agencies: ____

2. PROPERTY RELATED EVALUATIONS

A. Damage potential: Low _________ Moderate _________ High _______________
List buildings in flood plain (attachments as necc.)___________________ Location ______________
Floor Elevation ________________________
Upstream Land Use ____
Anticipate Any Change? ____

B. Any flood zoning? (FIA Studies, etc.) Yes __________ No ____________
Type of Study ____
Base flood elevation __________________________ (100 year)
Regulatory floodway width ________________________ (As noted in FIA studies)
Comments: ____
____

ADOT&PF Risk Assessment For Final Design (continued)

3. ENVIRONMENTAL CONSIDERATIONS

A. List commitments in Environmental Documents which affect Hydraulic Design (None _____)

____

____

____

____

4. HIGHWAY AND BRIDGE (CULVERT) RELATED EVALUATIONS

A. Note any outside features which might affect Stage, Discharge or Frequency.
Levees ____ Aggradation/Degradation ____ Reservoirs _____ Diversions ____
Explanation ____

____

B. Roadway Overflow Section (None _____) Length ___________ Elev. __________
Embankment: Soil Type ___________________ Type Slope Cover ___________________________
Comments: ____

5. MISCELLANEOUS COMMENTS

A. Is there unusual scour potential? Yes ____ No ____ Protection Needed? ____

B. Are banks stable? _________________________________Protection Needed? ______

C. Are spur dikes needed? Yes ______ No ______

D. Does stream carry appreciable amount of ice? _____ Elev. of high ice ______

E. Does stream carry appreciable amount of large driftwood? __________________

F. Comments: ____

____

____

6. TRAFFIC RELATED EVALUATIONS

A. Present Year __________ Traffic Area ________ VPD % Trucks __________

B. Design Year ___________ Traffic Area ________ VPD % Trucks ___________

C. Emergency Route ________ School Bus Route ___________ Mail Route __________

D. Detour Available? ________ Length of Detour _______________________ Miles

Comments: ____

____

7. PRESENT FACILITY

A. Low Roadway Elevation: ____

B. Bridge Hydraulic Capacity at point of overtopping ____________________ cfs
____ Frequency (if less than Q500)

C. Is flash flooding likely? Yes ________ No _______

Comments: ____
____

8. ALTERNATIVES

A. Recommended Design ____
Low Superstructure (Bridge) ____________ Top Opening (culvert) ____________
Low Roadway Grade ____

B. Were other hydraulic alternates considered? Yes ______ No _____
Discussion: ____
____
____
____
____

C. Is this assessment commensurate with the risks identified (Yes ___ No ___)
or is further analysis needed? (Yes ____ No ____)

Appendix B: Miscellaneous

Backwater Calculations

Introduction

This Appendix addressed the manual calculation of bridge backwater as presented in FHWA HDS-1. It also addresses the design of riprap at bridge abutments and piers as presented in the FHWA HEC-11 (also see Chapter 17).

The information presented in this Appendix covers the necessary calculations. The user should refer to the referenced publication for a more complete coverage of the subject: Hydraulics Of Bridge Waterways

Backwater

The expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, section 1, and a point downstream from the bridge at which normal stage has been reestablished, section 4 (Figure D.1). The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between sections 1 and 4, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_1^* = K^*\alpha_2\, Vn^2/2g +$$

$$(\alpha_1\, [(A_{n2}/A_4)^2 - (A_{n2}/A_1)^2])\ V_{n2}^2/2g$$

h_1^* = total backwater, ft

K^* = total backwater coefficient

α_1 & α_2 = as defined below

A_{n2} = gross water area in constriction measured below normal stage, ft^2

V_{n2} = average velocity in constriction* or Q/A_{n2}, ft/s

A_4 = water area at section 4 where normal stage is reestablished, ft^2

A_1 = total water area at section 1, including that produced by the backwater, ft^2

To compute backwater, it is necessary to obtain the approximate value of h_1^* by using the first part of the expression:

$$h_1^* = [K^*\alpha_2\, (V_{n2}^2)]/2g$$

The value of A1 in the second part of expression, which depends on h_1^*, can then be determined and the second term of the expression evaluated:

$$\alpha_1\, [(A_{n2}/A_4)^2 - (A_{n2}/A_1)^2]\ V_{n2}^2/2g$$

This part of the expression represents the difference in kinetic energy between sections four and one, expressed in terms of the velocity head, $V_{n2}^2/2g$.

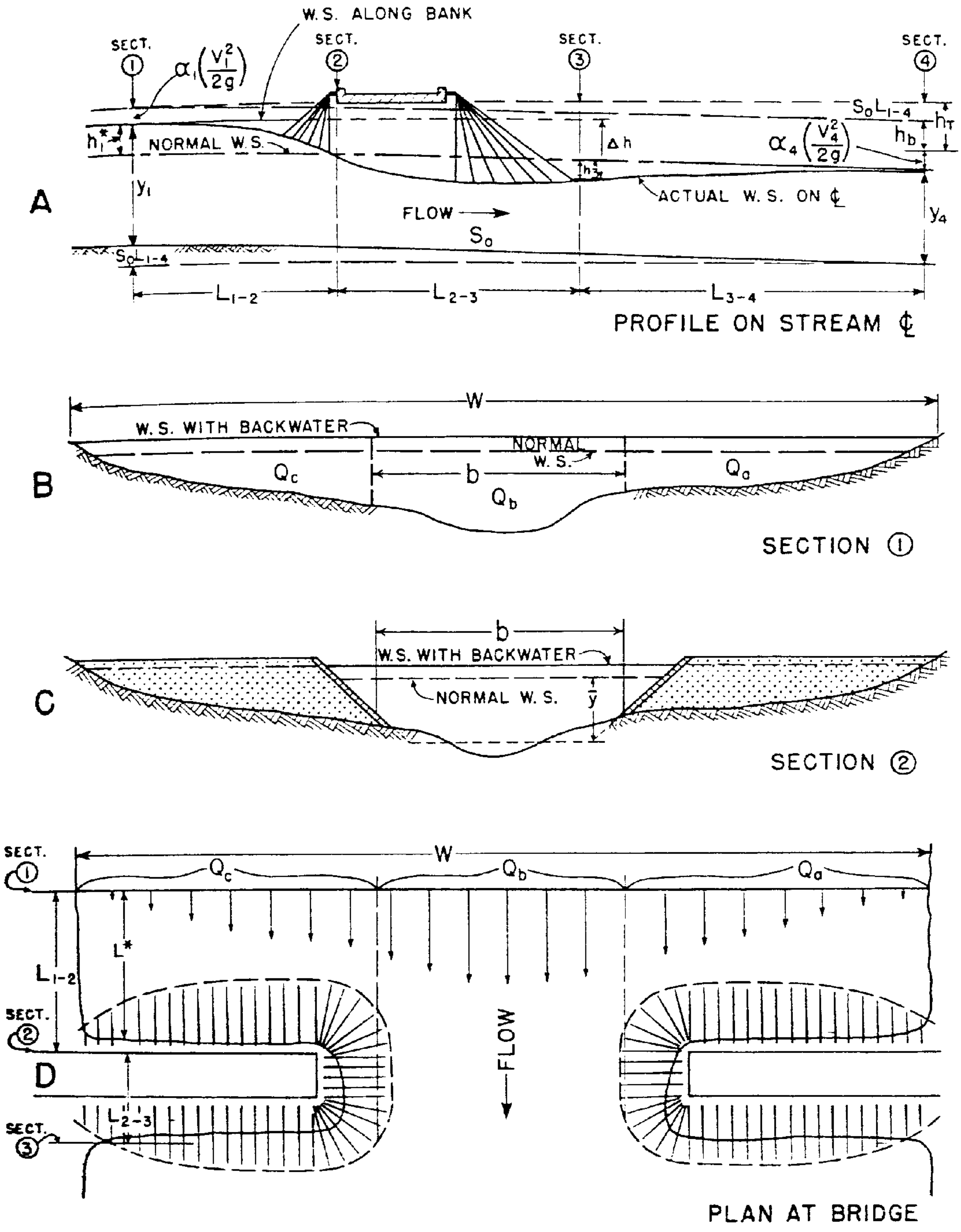


Figure D.1
Normal Crossing: Spillthrough Abutments

Bridge Opening Ratio

$$M = Qb/(Q_a + Q_b + Q_c)$$

Kinetic Energy Coefficient

$$\alpha_1 = (qv^2)/QV_1^2$$

Where:

v = average velocity in a subsection

q = discharge in same subsection

Q = total discharge in river

V_1 = average velocity in river at section 1 or Q/A_1

Width of Constriction

$b = A_{n2}/y$ (Figure D.1)

Backwater Coefficient

$$K^* = K_b + \Delta K_p + \Delta K_s + \Delta K_e$$

Where:

K_b is the base constriction coefficient

ΔK_p is the pier coefficient

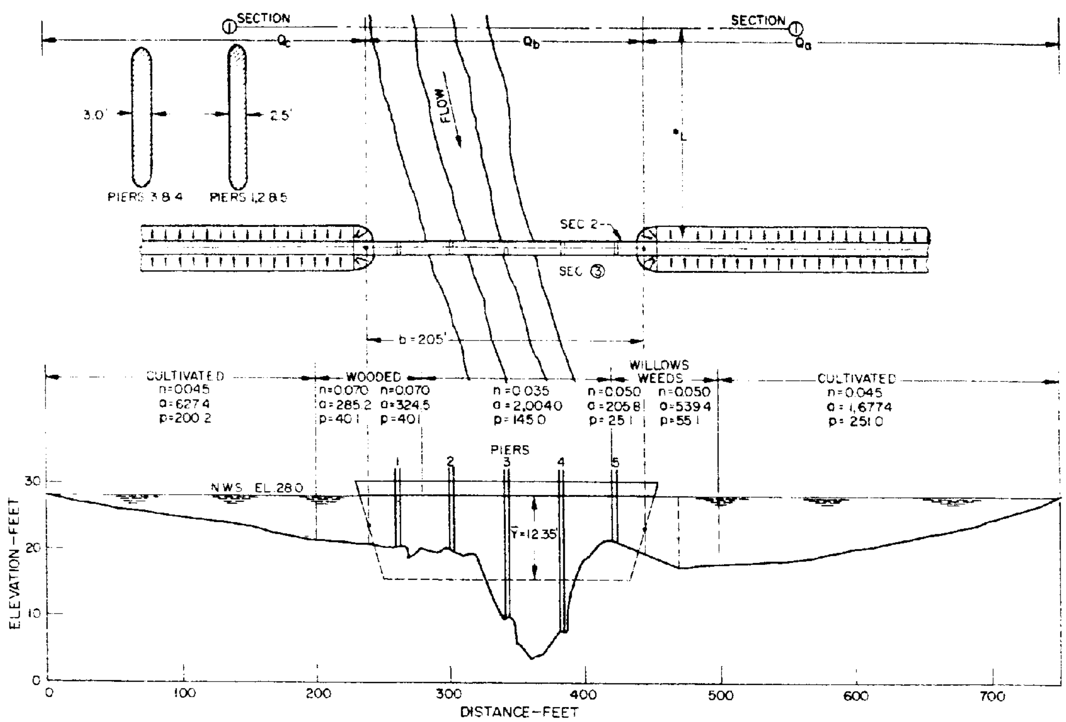
ΔK_s is the skew coefficient

ΔK_e is the eccentricity coefficient

Individual coefficient values are obtained from figures in HDS-1.

<u>Example D.1</u>

The channel crossing is shown in Figure D.2 with the following information: Cross section of river at bridge site showing areas, wetted perimeters, and values of Manning n; normal water surface for design = El 28.0 ft at bridge; average slope of river in vicinity of bridge S_o = 2.6 ft/mi or 0.00049 ft/ft; cross section under bridge showing area below normal water surface and width of roadway = 40 ft. Find the Bridge Backwater caused by this roadway crossing



The stream is essentially straight, the cross section relatively constant in the vicinity of the bridge, and the crossing is normal to the general direction of flow.

Figure D.2 Channel Crossing

Solution

Under the conditions stated, it is permissible to assume that the cross sectional area of the stream at section 1 is the same as that at the bridge. The approach section is then divided into subsections at abrupt changes in depth or channel roughness as shown in Figure D.2. The conveyance of each subsection is computed as shown in columns 1 through 8 of Table D.l. The summation of the individual values in column 8 represents the overall conveyance of the stream at section 1 or K_1 = 879,489. Note that the water interface between subsections is not included in the wetted perimeter. Table D.1 is set up in short form to better demonstrate the method. The actual computation would involve many subsections corresponding to breaks in grade or changes in channel roughness.

Table D.1 Calculation Summary

	Sub-Section	n	$\frac{n}{1.49}$	a	p	$r=\frac{a}{p}$	$r^{2/3}$	$k=\frac{1.49}{n}ar^{2/3}$	$q=Q\frac{k}{K_1}$	$v=\frac{q}{a}$	qv^2
	(1)	(2)	(3)	sq.ft. (4)	ft. (5)	ft. (6)	(7)	(8)	cfs (9)	fps (10)	(11)
Q_a	0-200 200-240	.045 .070	33.0 21.2	627.4 285.2	200.2 40.1	3.134 7.112	2.142 3.698	44,349 22,359	983.3 495.7	1.57 1.74	2,424 1,501
Q_b	240-280 280-420 420-445	.070 .035 .050	21.2 42.5 29.7	324.5 2,004.0 205.8	40.1 145.0 25.1	8.092 13.821 8.199	4.031 5.759 4.066	27,732 490,492 24,852	614.8 10,875.2 551.0	1.89 5.43 2.68	2,196 320,654 3,958
Q_c	445-500 500-750	.050 .045	29.7 33.0	539.4 1667.4	55.1 251.0	9.789 6.683	4.576 3.548	73,309 196,396	1,625.4 4,354.6	3.01 2.60	14,726 29,436
				A_n=2,534 sq.ft				K_1=879,489	Q=19,500 cfs		Σqv^2= 374,895
				A_{n2}=2,534 sq.ft.					Q_b=12,040 cfs		

S_o=0.00049 ft/ft

Since the slope of the stream is known (2.6 ft/mi) and the cross sectional area is essentially constant throughout the reach under consideration, it is permissible to solve for the discharge by what is known as the slope-area method or;

$$Q = K_1 S_o^{1/2} = 879{,}489(0.00049)^{1/2} = 19{,}500 \text{ cfs}$$

To compute the kinetic energy coefficient, it is first necessary to complete columns 9, 10, 11 of Table D1; then:

$$\alpha_1 = \frac{qv^2}{QV_1^2} = \frac{374{,}895}{19{,}500(19{,}500/5{,}664)^2} = 1.62$$

The sum of the individual discharges in column 9 must equal 19,500 cfs. The factor M is the ratio of that portion of the discharge approaching the bridge in width b, to the total discharge of the river:

$$M = Q \qquad M = Q_b/Q = 12{,}040{,}19500 = 0.62$$

Entering Figure 5 in HDS-1 with α_1 = 1.62 and M = 0.62, the value of α_2 is estimated as 1.40.

Entering Figure 6 in HDS-1 with M = 0.062, the base curve coefficient is K_b = 0.72 for bridge waterway of 205 ft.

As the bridge is supported by five solid piers, the incremental coefficient (ΔK_p) for this effect is determined. Referring to Figure D.2 and Table D1: the gross water area under the bridge for normal stage, A_{n2}, is 2,534 sq ft and the area obstructed by the piers, A_p, is 180 sq ft; so:

$$J = A_p/A_{n2} = 180/2{,}534 = 0.071$$

Entering Figure 7A in HDS-1 with J = 0.071 for solid piers, the reading from the ordinate is ΔK =

0.13. This value is for M = 1.0. Now enter Figure 7B in HDS-1 and obtain the correction factor σ , for M = 0.62 which is 0.84. The incremental backwater coefficient for the five piers, $\Delta K_p = \Delta K\sigma = 0.13 \times 0.84 = 0.11$

The overall backwater coefficient:

$K^* = K_b + \Delta K_p = 0.72 + 0.11 = 0.83$,

$$V_{n2} = \frac{Q}{A_{n2}} = \frac{19{,}500}{2{,}534} = 7.70 \text{ ft/s}$$

and

$V_{n2}^2/2g = 0.92$ ft

The approximate backwater will be:

$K^*\alpha\ 2V_{n2}^2/2g = 0.83 \times 1.40 \times 0.92 = 1.07$ ft

Substituting values in the second half of expression for difference in kinetic energy between sections 4 and 1 where A_{n1} = 5664 sq ft = A_4.

A_1 = 6384 sq ft, and $A_{n2} = 2534\ \text{ft}^2$

$\alpha_1\ [(A_{n2}/A_4)^2 - (A_{n2}/A_1)^2]\ V_{n2}/2g$

$1.62[(2{,}534/5664)^2 - (2{,}534/6384)^2]\ (.92) =$

$(1.62)(.042)(.92)=.06$ ft

Then total backwater produced by the bridge is

$h_1^* = 1.07 + 0.06 = 1.13$ ft

Riprap At Piers And Abutments

Abutments

The equation for determining the required size of riprap stone at abutments is:

$$D_{50} = 0.001\ V_a^3 / (d_{avg}^{0.5}\ K_1^{1.5})\ (Csg)\ (Csf)$$

Where:

D_{50} = the median riprap particle size;

C = correction factor (described below);

V_a = the average velocity adjacent to abutment (ft/s);

d_{avg} = the average flow depth in the main flow channel (ft); and

K_1 is defined as:

$$K_1 = 1-(\sin^2\theta/\sin^2\varphi)^{0.5}$$

Where

θ = the bank angle with the horizontal ; and

φ = the riprap material's angle of repose.

$$C = C_{sg}C_{cf}$$

$$C_{sg} = 2.12 / (S_s - 1)^{1.5}$$

Where:

S_s = the specific gravity of the rock riprap.

$$C_{sf} = (SF / 1.2)^{1.5}$$

Where:

SF = the stability factor to be applied.

When applying the equation for riprap design at abutments a velocity in the vicinity of the abutment should be used instead of the average section velocity. The velocity in the vicinity of bridge abutments is a function of both the abutment type (vertical, wingwalled, or spillthrough), and the amount of constriction caused by the bridge. However, information documenting velocities in the vicinity of bridge abutments is currently unavailable. Until such information becomes available, it is recommended that the equation be used with a stability factor of 1.6 to 2.0 for turbulently mixing flow at bridge abutments.

Note that the average velocity and depth used in the equation for riprap design at bridge constrictions for abutment protection is the average velocity and depth in the constricted cross section at the bridge. Flow profiles at bridge sections are nonuniform. The recommended procedure for computing the average depth and velocity at bridge constrictions is:

- Model the reach in the vicinity of the crossing using WSPRO, HEC-2, or some other model with bridge loss routines.
- Compute the average depth and velocity in the constriction as the average of the depth and velocity for modeled cross sections at the entrance to, and exit from the bridge constriction.

As outlined above, the average section flow depth and velocity used in the equation are main channel values. The main channel is typically defined as the area between the channel banks. However, when the bridge abutments are located on the floodplain a sufficient distance from the natural channel banks so as not to be influenced by main channel flows, the average depth and velocity on the floodplain within the constricted section should be used in the riprap design relationship. Most standard computerized bridge backwater routines provide the necessary depths and velocities as a part of their standard output. If hand normal depth computations are being used, the computations must consider conveyance weighted effects of both floodplain, and main channel flows.

When there is no overbank flow and the bridge spillthrough abutment on the channel bank matches the slope of the main channel banks upstream and downstream, use the design procedure without modifications.

Piers

The FHWA is currently evaluating various equations for selection of riprap at bridge piers. Present research indicates that velocities in the vicinity of the base of a pier can be related to the velocity in the channel upstream of the pier. For this reason, the interim procedure presented below is recommended for designing riprap at piers:

Determine the D_{50} size of the riprap using the rearranged Ishbash equation to solve the stone diameter (in feet), for fresh water:

$$D_{50} = [1/2(1.384V_s^2)]/[(s-1)2g]$$

Where:

D_{50} = average stone diameter (ft)

V_s = velocity against stone (ft/s)

s = specific gravity of riprap material (1b/ft^2)

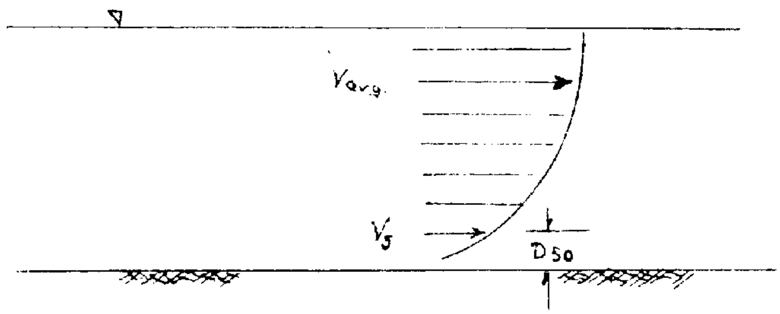
g = 32.2 ft/s^2

Parola determined that the velocity acting against the stone around a pier could be obtained by multiplying the average (in the vertical) approach velocity by a factor that ranges from 1.50 for a circular pier to 1.70 for a rectangular pier.

Replace V_s by 1.50 V_a for circular piers by 1.70 V_a for rectangular piers.

One parameter in the equation is the velocity against the stone. This velocity should be measured adjacent to the bed or riprap material. As shown in the figure below, the velocity value that would normally be obtained from computer models is representative of the average velocity. The shear velocity adjacent to the bed is usually of a lesser magnitude than the average velocity.

The Federal Highway Administration has furnished the following formula by which the average velocity may be converted into the shear velocity. The D_{50} term is really a depth measurement. It is indicating the depth or height above the stream bed at which the shear velocity will be computed. One assumes a size riprap that would be required and thereby determines a D_{50}. Applying this formula, one finds the shear velocity which is then applied to the riprap equation. Working through the riprap equation a final answer is derived for the required stone size. This required D_{50} is then compared to the assumed D_{50} that was used in determining the shear velocity. If the computed D_{50} is approximately equal to the assumed D_{50} then the calculation may be considered valid. If the D_{50}s are not equal, a new assumption should be made and the process repeated.



Formula:

$V_{Shear} = V_{avg}[\log 30.7/\{\log(10.93y/D_{50} + 1)\}]$

Where:

V avg = V_{n2} = Vel. at upstream face of pier

y = depth of flow (ft.) associated with V avg

D_{50} = Assumed riprap MDS(ft.)

Example: y = 8′

SF = 1.5

Vavg. = 10 ft/s

Vavg x 1.5 = 15 ft/s

Assume $D_{50} = 1'$

$V_{Shear} = V_{avg}[\log 30.7/\{\log(10.93y/D_{50} + 1)\}]$

$V_{Shear} = 15[\log 30.7/\{\log(10.93x8/1 + 1)\}] = 11.5$ ft/s

$D_{50} = [.5(1.384x11.5^2)]/[1.65x64.4] = 0.9$ ft

Conclusion: Assumed D_{50} of 1.0 ft approximately equals computed D_{50} of 0.9 ft. Therefore, the solution is satisfactory. Use D_{50} equals approximately 1.0 ft.

Provide a mat width that extends horizontally at least two times the pier width measured from the pier face.

Place the mat below the streambed a depth equivalent to the contraction scour. The thickness should be three stone diameters or more.