OFFICE OF STRUCTURES
MANUAL ON HYDROLOGIC AND HYDRAULIC DESIGN

CHAPTER 9 CHANNELS

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9.1 Introduction

The primary purpose of Chapter 9 is to provide information to the user regarding the policies and practices of the Maryland State Highway Administration with regard to stream crossings. In addition to summarizing this information, it may also serve as a means of locating related policies and practices in other chapters of the Manual. The basic equations for computing open channel flow have been moved to Chapter 9 Appendix A

9.1.1 Definitions

Open channels are natural or man-made conveyances for water in which:

- the water surface is exposed to the atmosphere, and the gravity force component in the direction of motion is the driving force.

There are various types of open channels encountered by the designer of transportation facilities:

- •natural channels - streams, estuaries
- •roadside channels or ditches,
- •irrigation channels, and
- •drainage ditches.

Only natural channels will be addressed in this chapter. The principles of open channel flow hydraulics are applicable to all drainage facilities including culverts.

Stream channels are:

- usually natural channels with their size and shape determined by natural forces,
- usually compound in cross section with a main channel for conveying low flows and a flood plain to transport flood flows, and
- usually shaped geomorphologically by the long term history of sediment load and water discharge which they experience.

9.1.2 Significance

Hydraulic analysis of highway stream crossings and/or longitudinal flood plain encroachments is necessary for the design of transportation drainage systems in order to determine:

- the potential for flooding of the highway or adjacent lands,

- the potential for damage to a structure or its highway approaches as a result of scour, erosion, or hydraulic forces,

- the potential for changes in the stability of the river system as a result of long term, on-going processes of degradation, aggradation or lateral movement of the channel bed and banks,

9.1 Introduction (continued)

- appropriate location and design considerations to minimize any adverse effect of the above noted conditions on the highway or on adjacent lands.

9.1.3 Location and Design Alternatives

Hydraulic analysis associated with natural channels is a process which selects and evaluates alternatives according to established policy and criteria. These standards established by the Maryland State Highway Administration serve as a framework for the design, construction and maintenance of highway facilities that meet their intended purpose of safety of the traveling public and structural integrity of the facility while serving to maintain and enhance the natural values of the stream and its flood plain.

9.2 Policy

Highway designs affecting natural channels and their flood plains are to be developed in a manner that complies with applicable Federal and State laws and regulations as per the guidance in Chapter 2, Legal, including:

- regulations of the Department of Natural Resources and the Maryland Department of the Environment pertaining to construction on Non-tidal waters and flood plains (See the selected excerpts from the Annotated Code of Maryland –COMAR in Appendix A of Chapter 2 Legal)
- wetlands and flood plain management regulations promulgated by the Federal Highway Administration (FHWA), the Corps of Engineers, the Federal Emergency Management Agency (FEMA) and others.

The Office of Structures will serve as the lead office for all highway crossings or encroachments on flood plains with a drainage area equal to or greater than one square mile. The Office of Highway Development will serve as the lead office for crossings or encroachments on flood plains with drainage areas of less than one square mile, except that the Office of Structures will handle all crossings where the existing structure is classified as a bridge and will also be involved in studies and designs for replacement of certain small drainage structures - See Chapter 1, Introduction and Chapter 13, Culverts

Appropriate coordination within SHA Offices and at Interagency Reviews is initiated and maintained throughout project development in accordance with the discussion presented in Chapter 3, Policies and Procedures, and Chapter 5, Project Development. The objectives of this coordination include:

- identification of matters pertaining to wetlands, fish passage, stream stability and enhancement, and other environmental concerns,

- early identification of alternative locations and designs that meet SHA criteria for highway construction within flood plains and wetlands, and

9.2 Policy

- early resolution of concerns of Federal and State agencies about the highway location and design, prior to submission of plans for TS&L approval.

Hydrologic and hydraulic studies are to be carried out by the Office of Structures to identify changes expected to occur as a result of proposed highway projects, including changes in water surface elevations, flow and velocity distributions, shear stresses and other hydraulic characteristics. Such changes need to be evaluated with regard to the safety and stability of the highway, and the Federal and State laws and regulations regarding wetlands and flood plain management. In accordance with the provisions of Section 9.3.2 of this chapter, the effects of proposed highway construction on the flooding of adjacent lands should also be evaluated for the magnitude of flood flows expected to occur as a result of ultimate development in the watershed. Particular attention needs to be given to the following considerations during the development of hydrologic and hydraulic studies:

- potential scour and erosion at the highway structure,
- stability of the stream channel,
- incremental flooding of adjacent lands as a result of highway construction,
- utilization of appropriate temporary and permanent pollution/sediment controls, procedures, and devices to minimize any adverse effects of highway construction on the natural channel and its flood plain.
- construction of temporary roads and stream crossings to serve the contractor's operation and to maintain traffic during construction.
- passage of fish and wildlife.
- avoidance of habitat of endangered plants and animals
- means for diverting stream flow during construction.

9. 3 Design Criteria

9.3.1 General

The design criteria in this section establish the standards by which SHA policies are to be carried out. They form the basis for consideration of the hydraulic aspects of the location and design of highway projects in flood plains. Where the design criteria in this chapter are inter-related with design criteria in other chapters, appropriate cross-references are provided. Appendix B to Chapter 2 contains pertinent excerpts and references of the Annotated Code of Maryland (COMAR).

9.3.2 Stream Channels

Non-tidal waterways are regulated by the Non-tidal Wetlands and Waterway Division of the Water Management Administration. A Non-tidal Wetlands and Waterway Permit will normally be required from this agency to approve proposed work affecting channels and flood plains. Hydrologic and hydraulic reports are to be submitted in support of the permit application, and water surface profiles need to be computed to evaluate the effect of proposed highway construction on the flood plain. The following criteria apply to development of technical reports prepared in support of permit applications:

1. Under certain conditions for in-kind replacement of existing bridges, development of water surface elevations may not be required (See Chapter 10).

2. When highway crossings or encroachments are proposed on flood plains that are under the jurisdiction of the Federal Emergency Management Agency (FEMA), appropriate studies and coordination shall be carried out to ensure consistency with the FEMA flood plain regulations (See Chapters 2 and 5). These studies are to be performed concurrently with the hydrologic, geomorphic and hydraulic studies necessary to meet State requirements affecting construction in flood plains under the jurisdiction of the Maryland Department of the Environment (MDE).

3. Approved hydraulic water surface profile models include HEC-RAS, Version 4.1, the standard model (Reference 42) and HEC-2, (Reference 40) for special studies involving FEMA flood plain management studies. The HY-8 Culvert Programs, References 19, may also be appropriate for some sites, when used in concert with HEC-RAS. Use of any other model will require prior SHA approval (See Chapter 10).

4. Flood discharges for the watershed should normally be computed on the basis of pre-construction (presently existing) conditions in the flood plain at the project site and land use as described in Item 6 below. Because of the large size of the watersheds of channels requiring bridge structures, and the very minor effects that most SHA structures have on flood flows, the difference in flood discharge estimates for pre-construction and post-construction conditions in the flood plain will generally not be significant, and can be considered as being the same when approved by the Office of Structures. However, if there are site conditions that would make a significant difference between pre-construction and post-construction hydrology, such as large changes in flood plain storage, the Engineer will be expected to compute and evaluate both the pre-construction and post-construction flood flow discharges. (See Chapter 8 Hydrology)

5. There are a number of considerations involved in the design of a highway crossing of a flood plain:

5.1 Considerations involved in satisfying the design specifications and standards of the Office of Structures. The procedures to be used to select the design discharge are presented below. Further discussion concerning the evaluation of the effect of flood waters on the safety of the traveling public and the stability of the structure are presented in Chapters 3, 8 and 10.

Structures and their approach roadways shall, as a minimum, be designed for the passage of the design year flood (based on ultimate development in the watershed) in accordance with the information in Table 1. The water surface elevation along the approach roadways for the design year flood (which should be coincident with the energy line of flow at the crossing for 1-D models) should not exceed the elevation of the bridge deck or the edge of the traffic lane. Designs for a higher recurrence interval flood may be used where justified to reduce the flood hazard to traffic or to adjacent properties. Where appropriate, consideration should be given to providing freeboard to facilitate passage of debris. Water surface profiles shall be developed for each structure (1) for the design year flood, (2) for evaluation of scour as described in Chapter 11 and (3) for the 2, 10 and 100 year floods, based on ultimate development in the watershed as described in Chapters 8, 9 and 10. *A design exception will be necessary in order to design for a flood with a lower recurrence interval than those listed in Table 9-1 below:*

Table 9-1 Recurrence Interval for Design Flood

Highway Classification (See Highway Location Manual)	Recurrence Interval for Design Flood (years)
Interstate, other Freeways and Expressways, and Rural, Urban and Other Principal Arterials	100
Intermediate and Minor Arterials	50
Major and Minor Collectors	25
Local Streets	10

Table 1 Notes

- Interstate, Freeway, Expressway and Arterial ramps and frontage roads should be assigned a design flood recurrence interval consistent with the crossroad being serviced by the ramps and frontage roads; however, the hydraulic design of ramp structures must not interfere with or compromise the designs of the structures carrying the higher class traffic lanes.
- *Any on-system structure that will be overtopped by flood waters having a recurrence interval smaller than the 25 year flood shall be posted for flooding.*
- In addition to the design flood, floods with the following recurrence intervals need to be evaluated during the design process:
  - bankfull stage for geomorphology studies (Chapter 14)
  - 2, 10 and 100 year floods (Chapter 10)
  - Overtopping ,100- year and 500-year floods for scour evaluation (Chapter 11)

5.2 Considerations involved in satisfying the Federal and State laws and regulations affecting flood plain management, including anticipated changes to flood elevations on adjacent lands and environmental impacts caused by the highway. These considerations are discussed and referenced below.

6. For purposes of evaluating the effect of the proposed highway project on flooding of adjacent lands and the hydraulic adequacy of a structure, flood discharge estimates shall be developed, as a minimum, for the 2, 10 and 100-year floods, based on ultimate development of the associated watershed as depicted on current zoning maps (See Chapter 8 and Chapter 2).

7. The Engineer may use estimates of flood discharges, based on existing development of the watershed, to evaluate other aspects of locating highways in flood plains including:

   - concerns about fish passage and stream stability,
   - locations where temporary crossings or stream diversions are to be installed during construction, and
   - investigations of flooding complaints.

8. Two water surface profiles shall be developed for each flood discharge estimate selected by the Engineer using the criteria in Items 6 and 7 above:

   - A water surface profile based on pre-construction (existing) conditions in the flood plain, and
   - A water surface profile based on post-construction conditions in the flood plain.

Both water surface profiles shall be continued upstream of the project to a point where the difference in elevations is 0.1 foot or less. The differences in elevations shall be tabulated and the maximum changes in elevations (typically for the 100 year flood) shall be evaluated using the criteria in Table 9-1:

9. Changes in the flow distribution and velocity of flow in the channel and on the flood plain should be minimized, to the extent practicable. Particular attention should be given to this consideration when there are improved properties within the flood plain limits that are affected by the project.

Table 9-2. Actions Required to Mitigate Anticipated Increases in Flood Water Elevations on Adjacent Lands Due to Highway Construction

Rise in water Surface beyond SHA R-O-W	Undeveloped Flood Plain	Developed Flood Plain (See Note 1)
No Increase (See Note 2)	No Action Required	No Action Required
Greater than 0.10 foot but not greater than 1.0 foot	Notify Property Owners; Determine need for mitigation on a case-by-case basis.	Avoid increases in flood water elevations where practicable; Where necessary, notify property owners and mitigate flood hazard through purchase of property, flood easements or other appropriate means.
Greater than 1.0 feet (See Note 3)	Avoid such increases where practicable; Where necessary, notify property owner and mitigate flood hazard through purchase of property, flood easements or other appropriate means.	Avoid increases in flood water elevations where practicable; Where necessary, notify property owners and mitigate flood hazard through purchase of property, flood easements or other appropriate means.

Note 1: Development on flood plains includes most buildings except for minor structures such as utility sheds. Also excluded in the definition of development are facilities such as walkways, tennis courts, picnic tables and parking lots. This criterion also applies to land presently zoned for development.

Note 2: A numerical increase of 0.10 foot or less is considered to be no increase in the flood elevation. This definition allows for minor fluctuations by computational models.

Note 3: Normal design practice is to limit rises to 1.0 foot or less as indicated above. Rises greater than 1.0 foot should be limited to special cases where it is not practical to meet the 1.0 foot limit.

9.3.3 Tidal Waters

The SHA is required to obtain either a State Tidal Wetlands License or a Tidal Wetlands Permit from the Maryland Board of Public Works for highway projects located in tidal areas. The basis for the granting of this license/permit is concurrence with State regulations governing work in wetland areas and with other environmental concerns. These concerns are addressed during the project development process through the SHA project development procedures (See Chapter 5).

Hydrologic and hydraulic studies are not required to be included with the application for the tidal wetlands license/permit because of the following considerations:

- Water surface elevations in tidal areas are controlled mainly by the tides, and the presence of highway structures on tidal waterways will normally have an insignificant effect on storm tide elevations.

- It is the practice of the SHA to minimize placement of fill in tidal waters and in adjacent wetlands.

One possible exception to the above general rule is the case of a structure located in a tidal area, but where water surface elevations are controlled by flood flows from the upland watershed rather than by the tides. An example of this case is the Woodrow Wilson Bridge over the Potomac River. The recommended procedure for this particular situation is to treat the design as a non-tidal crossing and submit it to the Non-tidal Wetlands and Waterway Division for their review and determination as to the type of permit that is required. Under certain circumstances, when both types of wetlands are present, it may be necessary to obtain both a tidal and a non-tidal permit for a highway crossing of this type of site.

Hydrologic and hydraulic studies of structures in tidal areas are necessary for purposes of evaluating the stability of the structures and for determining the magnitude and recurrence interval of the over-topping flood, where appropriate. Discussion of these studies is presented in Chapter 10.

9.4 Open Channel Flow Equations

See Chapter 9, Appendix A

9.5 Hydraulic Analysis

9.5.1 General

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of highway stream crossings.

The model used in hydraulic analysis by the Office of Structures is the Corps of Engineers HEC-RAS model, Version 4.1. (In some special cases, an existing HEC-2 model may be utilized by converting it to a HEC-RAS model.) HEC-RAS is based on the step-backwater method and is used to compute the complete water surface profile in a stream reach, to evaluate the unrestricted water surface elevations for bridge hydraulic design, or to analyze other gradually-varied flow problems in streams. (See Chapter 10)

The culvert program HY-8 developed by the FHWA (Reference 19), can be used to effectively model flow through large culverts provided that the design conditions, such as tailwater, are properly determined. (See Chapter 10).

9.5.2 Cross-sections

Cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation which locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible, but in wide flood plains or bends it may be necessary to use a section along intersecting straight lines, i.e. a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors. (See Chapter 4).

9.5.2.1 Beginning Water Surface Elevation

The Engineer is expected to use judgment in the selection of the downstream starting water surface elevation. Office and field studies should be carried out to investigate the location of any control sections on the river. Particular attention needs to be focused on locations at river confluences or on regulated rivers. A study of the flood frequencies, hydrographs, and stage-discharge relations of confluencing rivers can serve to provide useful information in this regard. Similarly, agencies responsible for regulating flow in rivers should be able to provide information that will be helpful in estimating river stages for floods of various return periods.

The essential issue is that the starting water surface elevation should be selected in consideration of the purpose of the hydraulic study. If the concern is to evaluate anticipated worst-case scour conditions, it is prudent to select a conservatively low starting water surface elevation and reasonable “n” values for winter conditions. Conversely, if flooding of developed properties is a concern, a conservatively high starting water surface elevation and “n” values for summer conditions should be considered. (See also Chapter 3, Appendix A and Chapter 10)

9.5 Hydraulic Analysis

If there is a control point downstream of the structure at which the water flows through critical depth, then this point should be selected as the starting point for computing the water surface profile. In most cases, however, there are no significant downstream controls, and the beginning water surface elevation must be estimated so as to represent normal depth for the given flow conditions. This estimation procedure is essentially one of trial and error. It has been the experience of engineers in the Office of Structures that the best approach to this process is to input the friction slope at the downstream section, and then correct this slope based on an evaluation of the results of the first one or two runs of the water surface profile. See section 9.5.4 for a method of estimating the downstream station for the beginning of the HEC-RAS program.

Normally, an average value of the water surface slope for several sections is used to determine the initial value of the friction slope. (See also Chapter 10)

9.5.2.2 Manning's "n" Value Selection
See Chapter 3 Appendix A

9.5.2.3 Calibration

The model used to develop water surface profiles should be calibrated, when reliable information is available. The following parameters, in order of preference, should be used for calibrations: Manning's n, slope, discharge, and cross-section.

Reliable high water marks are often difficult to locate, either in the field or in the files of the SHA or other governmental agencies; nevertheless, the Engineer needs to check to see if such information is available from observations during field investigations, or from discussions with local residents, personnel within the SHA and other agencies. If no reliable high water marks are available, use of maximum and minimum n values as discussed above will help to establish reasonable limits with regard to flood water elevations and the resulting hydraulic characteristics of the flood flows.

9.5.2.4 Switchback Phenomenon

If the cross-section is improperly subdivided, the mathematics of the Manning's Equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area which causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross-sections should be used in order to avoid the switchback.
This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any

cross-section being used in a step backwater program. **For this reason, the cross-section should always be subdivided with respect to both vegetation and geometric changes.** Note that the actual n value itself may be the same in adjacent subsections.

9.5.3 Step-backwater Models

The computation of water surface profiles by HEC-RAS is based on the standard step method in which the stream reach of interest is divided into a number of sub reaches by cross sections spaced such that the flow is gradually-varied in each sub reach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section. (See Chapter 9 Appendix A)

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles

9.5.4 Profile Computation

The Corps of Engineers (Reference 39) developed equations for determining the upstream and downstream reach lengths in a hydraulic study as follows:

$$\mathbf{Ldn = 8{,}000\ (HD^{0.8}/S)} \qquad \mathbf{(9.12)}$$

$$\mathbf{Lu = 10{,}000\ [(HD^{0.6})(HL^{0.5})]/S} \qquad \mathbf{(9.13)}$$

Where:

Ldn = downstream study length (along main channel), ft (for normal depth starting conditions)

Lu = estimated upstream study length (along main channel), ft (required for convergence of the modified profile to within 0.1 feet of the base profile)

HD = average hydraulic depth (1- percent chance event flow area divided by the top width), ft

S = average reach slope, ft/mile

HL = head loss ranging between 0.5 and 5.0 feet at the channel crossing structure for the 100-year flood, ft

Reference 39 is a valuable source of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open-channels. This reference contains more specific guidance on cross-section determination, location, and spacing and stream reach determination. This Reference investigates the accuracy and reliability of water surface profiles related to "n" value determination and the survey or mapping technology used to determine the cross-section coordinate geometry.

9.5 Hydraulic Analysis (continued)

9.5.5 Water and Sediment Routing

Sediment transport is a complex subject, and is not generally necessary for the studies conducted by the Office of Structures in connection with the hydraulic design of bridges. The most practical reference pertaining to sediment transport is the FHWA Manual Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments dated December 2001. FHWA presents a workshop (SHA Number ENBH (T) 202) to give practicing engineers experience in working with sediment transport models.

Currently, the procedure in Chapter 14, Stream Morphology, is used to evaluate the effect of sediment transport and its effect on hydraulic design and scour considerations. If the need should arise for more complex sediment transport studies, OOS would consider obtaining the services of a consultant with special expertise in this field of study.

9.5.6 Two-dimensional models

Two dimensional flow models have been under development for some time, and are of great interest to Bridge and Hydraulic Engineers because they have the capability of calculating flow velocities as vectors on a two-dimensional horizontal plane and depicting both the magnitude and the direction of the velocity vector. The FESWMS-2DH model developed by the FHWA (Finite Element Surface Water Modeling System: Two Dimensional Flow in a Horizontal Plane, Reference 21) has been used successfully on a number of projects where conditions cannot be accurately represented by one dimensional models such as HEC-RAS. Examples of such conditions include wide flood plains, bends in rivers, and confluences with other rivers.

SHA has experimented with the FESWMS model at a few locations. Such 2-D studies cost a great deal more and take more time than a HEC-RAS study. In once of the cases studies, it was an open question as to whether the results were significantly better than the results from the HEC-RAS study.

Since FESWMS is a relatively complex model requiring considerable expertise by the user in order to understand and properly apply the model, its use will be subject to approval of the SHA on a project by project basis.

9.6 Design Procedure

9.6.1 General

The design procedure for all types of channels has some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

9.6.2 Stream Channels

The analysis of a stream channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain (See Section 9.7).

A step-by-step procedure and check list is provided in Chapter 3 on the development of a hydraulic study. Appropriate modifications to this general procedure will need to be made on a project by project basis to reflect field conditions.

9.7 Stream Morphology

9.7.1 Introduction

The form assumed by a natural stream, which includes its cross-sectional shape as well as its planform, is a function of many variables for which cause-and-effect relationships are difficult to establish. The stream may be graded or in equilibrium with respect to long time periods, which means that on the average it discharges the same amount of sediment that it receives although there may be short-term adjustments in its bedforms in response to flood flows. On the other hand, the stream reach of interest may be aggrading or degrading as a result of deposition or scour in the reach, respectively. The planform of the stream may be straight, braided, or meandering. These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form, and bank erosion with time.

A qualitative assessment of the river response to proposed highway facilities is possible through a thorough knowledge of river mechanics and accumulation of engineering experience. Engineers are expected to conduct a preliminary stream morphology report (See Chapter 14) during field reviews of the project site, and to discuss this report with the Office of Structures. Depending on the findings of the preliminary report, a detailed stream morphology report may be needed to discuss environmental concerns, such as fish passage, and to address the potential for lateral stream movement and long term degradation or aggradation. Specific guidance for the conduct of such studies is presented in Chapter 14.

The Rosgen procedure for classifying streams has been used by the SHA since it represents a practical approach for field classification and provides guidance with regard to the likely effect that construction of highways stream crossings may have on stream stability. Knowledge of and experience with the Rosgen classification procedures is helpful in applying the analytical and design procedures under development by the SHA for structure crossings of streams. Chapter 14, Stream Morphology, provides detailed guidance in regard to the design procedures recommended by the SHA to evaluate the effects of highway construction on stream morphology. Questions regarding the evaluation of stream morphology should be referred to the Office of Structures, References 35, 43 and 44 provide detailed descriptions and explanations regarding the application of the Rosgen classification system.

9.7.2 Countermeasures

A countermeasure is defined as a measure incorporated into a highway crossing of a stream to control, inhibit, change, delay, or minimize stream and bridge stability problems. They may be installed at the time of highway construction or retro-fitted to resolve stability problems at existing crossings. Retro-fitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop.

The selection of an appropriate countermeasure for a specific stream stability problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs. The recommended approach in selecting a specific countermeasure for an OOS project is as follows:

- Determine the scope of the potential stream stability problem, and alternatives available to address the problem.
- Meet with OOS engineers to determine the most appropriate option or options to address the problem.

Below is a brief discussion of possible countermeasures for some common river stability problems. The reader is encouraged to consult the information presented in Chapter 11 as well as the references listed at the end of this chapter for detailed information on the design and construction of the countermeasures. Particular attention should be given to the FHWA Hydraulic Engineering Circular 23, Bridge Scour and Stream Instability Countermeasures. SHA regularly uses rock structures such as cross vanes and other river controls to serve in stabilizing streams where there are problems with channel stability.

9.7.2.1 Channel Movement

The best countermeasure against channel movement is a crossing location on a relatively stable location such as a straight reach of stream between bends. However, in most cases the approach used by the Office of Structures is to estimate the extent of channel movement expected to occur over the design life of the bridge. This has been designated the Lateral Channel Movement Zone or LCMZ. Each foundation element within this LCMZ is then designed for the potential that the channel may move to the particular pier or abutment (See Chapters 11 and 14).

Other counter measures include the protection of an existing bank line, the establishment of a new flow line or alignment, and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, bio-engineering techniques and channel relocations. Measures may be used individually or a combination of two or more measures may be used to stabilize lateral channel movement. (References 23 and 29).

9.7.2.2 Channel Braiding

Countermeasures used at braided streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels.

The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Spur dikes at bridge ends used in combination with revetment on highway fill slopes, riprap on highway fill slopes only, and spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

9.7 Stream Morphology

9.7.2.3 Degradation

Degradation in streams can cause the loss of bridge piers in stream channels, and piers and abutments in caving banks. A check dam, which is a low dam or weir constructed across a channel, can be a successful technique for halting degradation on small to medium sized streams. Use of a check dam will require the evaluation of its effect on fish passage, and construction of fish ladders or other devices, where necessary, to maintain the accessibility of the area above the dam to the fish. For most OOS bridge projects, however, the approach is to locate the bridge foundations below the elevation of the estimated degradation. (See Chapter 11 and Chapter 14)

Longitudinal stone dikes placed at the toe of channel banks can be effective counter measures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful counter measure against degradation problems (Reference 29).

9.7.2.4 Aggradation

Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or combinations of these.

Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation.

Another technique which shows promise is the submerged vane technique developed by the University of Iowa. The studies suggest that the submerged vane structure may be an effective, economic, low-maintenance, and environmentally acceptable sediment-control structure with a wide range of applications (References 29, 33 and 34). This device has not been tested in Maryland.

9.8 Fish Passage

The Office of Structures is currently revising its guidance on designing structures to accommodate fish passage. There are a number of studies now underway, and the information obtained from these studies will be used to reformulate the OOS guidelines and approaches to providing for fish passage at highway structures. (See Chapter 13)

9.9 References

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CHAPTER 9 CHANNELS

APPENDIX A

Hydraulic Design Equations for Open Channel Flow

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CHAPTER 9 APPENDIX A

Hydraulic Design Equations for Open Channel Flow

Introduction

The Equations presented in this Appendix are, for the most part, the basic open channel flow equations covered in standard text books. During the re-edit of Chapter 9 Channels, a determination was made to retain these equations for the use of the Manual user, but to relocate them to this Appendix. Please contact the Office of Structures, H&H Structures Division if you have comments or questions relating to the use of these equations.

9.4 Open Channel Flow

9.4.1 General

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (Reference 23). The basic principles of fluid mechanics -- continuity, momentum, and energy -- can be applied to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principal problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels.

9.4.2 Definitions

Specific Energy

Specific energy E is defined as the energy level relative to the channel bottom. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head:

$$\mathbf{E = y + \alpha (V^2/2g)} \qquad \textbf{(9.1)}$$

where:

y = depth, ft

α = kinetic energy correction coefficient

V = mean velocity, ft/s

g = gravitational acceleration, 32.2 ft/s^2

The kinetic energy correction coefficient is taken to have a value of one for turbulent flow in prismatic channels but may be significantly different than one in natural channels.

Kinetic Energy Correction Coefficient

As the velocity distribution in a river varies from a maximum at the design portion of the channel to essentially zero along the banks, the average velocity head, computed as $(Q/A)^2/2g$ for the stream at a section, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, α, defined as:

$$\boldsymbol{\alpha = [\Sigma(qv^2)/(QV^2]} \qquad \textbf{(9.2)}$$

Where:

v = average velocity in subsection, ft/s

q = discharge in same subsection, cfs

Q = total discharge in river, cfs

V = average velocity in river at section or Q/A, ft/s

9.4.2 Definitions (continued)

Total Energy Head

The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The locus of the energy head from one cross section to the next defines the energy grade line. See Figure 9-1 for a plot of the specific energy diagram.

Steady and Unsteady Flow

A steady flow is one in which the discharge passing a given cross-section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

Uniform Flow and Non-uniform Flow

A non-uniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the direction of flow direction; however, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

Gradually-varied and Rapidly-Varied

A non-uniform flow in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected, is referred to as a gradually-varied flow; otherwise, it is considered to be rapidly-varied.

Froude Number

The Froude number is an important dimensionless parameter in open channel flow. It represents the ratio of inertia forces to gravity forces and is defined by:

$$\mathbf{F = V/(gd)^{.5}} \qquad \mathbf{(9.3)}$$

Where:

V = mean velocity = Q/A, ft/s

g = acceleration of gravity, ft/s^2

d = hydraulic depth = A/T, ft

A = cross-sectional area of flow, ft^2

T = channel top width at the water surface, ft

This expression for Froude number applies to any single section channel of non-rectangular shape.

9.4.2Definitions (continued)

Critical Flow

The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge when the specific energy is held constant. These relationships are illustrated in Figure 9-1.

Subcritical Flow

Depths greater than critical occur in subcritical flow and the Froude number is less than one. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.

Supercritical Flow

Depths less than critical depth occur in supercritical flow and the Froude number is greater than one. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

9.4 Open Channel Flow (continued)

Hydraulic Jump

A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures.

9.4.3 Flow Classification

The classification of open channel flow can be summarized as follows.

Steady Flow

1. Uniform Flow

2. Non-uniform Flow

a. Gradually Varied Flow

b. Rapidly Varied Flow

Unsteady Flow

1. Unsteady Uniform Flow (rare)

2. Unsteady Non-uniform Flow

a. Gradually Varied Unsteady Flow

b. Rapidly Varied Unsteady Flow

The steady uniform flow case and the steady non-uniform flow case are the most fundamental types of flow used in highway engineering hydraulic models like HEC-2 and WSPRO for purposes of developing water surface profiles. However, the gradually varied unsteady flow case is used for purposes of evaluating tidal flow.

9.4 Open Channel Flow (continued)

9.4.4 Equations

The following equations are those most commonly used to analyze open channel flow. The use of these equations in analyzing open channel hydraulics is discussed in Section 9.5.

Manning's Equation

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in steady, uniform flow and is computed from Manning's equation:

$$\mathbf{Q = (1.49/n)AR^{2/3}S^{1/2}} \qquad \textbf{(9.4)}$$

Where:

Q = discharge, cfs

n = Manning's roughness coefficient

A = cross-sectional area of flow, ft^2

R = hydraulic radius = A/P, ft

P = wetted perimeter, ft

S = channel slope, ft/ft

9.4.4 Equations (continued)

The selection of Manning's n is generally based on observation; however, considerable experience and judgment is necessary to arrive at an appropriate n value for a given site. A study by the Corps of Engineers (Reference 39) has shown that errors in the selection of Manning n values will normally result in greater errors in the elevation of flood water surface profiles than errors in any of the other variable used in the preparation of hydraulic studies of streams and their flood plains. **Accordingly, the Engineer is expected to carefully apply the procedures developed by the U.S. Geological Survey as presented in Reference 22 to determine n values for channels and their flood plains. (See also Chapter 3, Appendix A)**

Reference 5, also prepared by the U.S. Geological Survey, and is recommended for use as a guide for checking whether the values obtained from the procedures in Reference 22 are reasonable.

The most accurate estimates of Manning's n values can be obtained at locations where high water marks are available and can be correlated with a known discharge.

9.4 Open Channel Flow (continued)

The following guidance is offered with regard to the selection of appropriate Manning's n values:

- For major flood flows, the bed of sand channels can be expected to assume the plain bed form of the upper regime.
- Particular care should be exercised in the estimation of n values for wooded flood plains. There is an apparent tendency for Engineers to underestimate flood plain n values, and consequently to overestimate the amount of overbank flow. Errors of this type result in unrealistic values of live bed contraction scour, as explained in Chapter 10.

Additional discussion of Manning's n values is contained in Section 9.5.2.1.

If the normal depth computed from Manning's equation is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

9.4.4 Equations

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance K:

$$\mathbf{K = (1.49/n)AR^{2/3}} \qquad \textbf{(9.5)}$$

and then Manning's Equation can be written as:

$$\mathbf{Q = KS^{1/2}} \qquad \textbf{(9.6)}$$

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the flow distribution through the opening in a proposed stream crossing. Appendix 9C presents an illustration of how conveyance can be used to determine the distribution of flood flow within the channel and on the flood plain.

9.4 Open Channel Flow (continued)

Continuity Equation

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form:

$$\mathbf{Q = A_1V_1 = A_2V_2} \qquad \mathbf{(9.7)}$$

Where:

Q = discharge, cfs

A = flow cross-sectional area, ft^2

V = mean cross-sectional velocity, ft/s (which is perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path. The continuity equation can be used together with Manning's Equation to obtain the steady uniform flow velocity as:

$$\mathbf{V = Q/A = (1.49/n)R^{2/3}S^{1/2}} \qquad \mathbf{(9.8)}$$

Energy Equation

The energy equation expresses conservation of energy in open channel flow expressed as energy per unit weight of fluid which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities which give the total energy head at any cross section when added. Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$\mathbf{h_1 + \alpha_1(V_1^2/2g) = h_2 + \alpha_2(V^2{}_2/2g) + h_L} \qquad \mathbf{(9.9)}$$

Where: h_1 and h_2 are the upstream and downstream stages, respectively, ft

α = kinetic energy correction coefficient

V = mean velocity, ft/s

h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, ft

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y, i.e. h=z+y. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

9.5.3 Step-backwater Models

The computation of water surface profiles by HEC-RAS is based on the standard step method in which the stream reach of interest is divided into a number of sub reaches by cross sections spaced such that the flow is gradually-varied in each sub reach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section.

The method requires definition of the geometry and roughness of each cross section as discussed in Section 9.5.1. Manning's n values can vary both horizontally across the section as well as vertically. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line can all be specified.

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the Corps of Engineers HEC-RAS be used.

The HEC-RAS Version 4.0 and HEC-2 programs developed by the Corps of Engineers are widely used for calculating water surface profiles for steady gradually varied flow in natural and man-made channels. SHA recommends HEC-RAS for most locations except where previous flood studies for FEMA or other flood control agencies have been made. In this case, it may be easier to use the existing HEC-2 models in order to compare existing and proposed water surface profiles. Both subcritical and supercritical flow profiles can be calculated by these programs. The effects of bridges, culverts, weirs, and structures in the floodplain may be also considered in the computations. These programs are also designed for application in flood plain management and flood insurance studies.

9.5.3 Step-Backwater Models (continued)

To amplify on the methodology, the energy equation is repeated from Section 9.4.4:

$$\mathbf{h_1 + \alpha_1(V_1^2/2g) = h_2 + \alpha_2(V_2^2/2g) + h_L} \qquad \mathbf{(9.10)}$$

Where:

h_1 and h_2 are the upstream and downstream stages, respectively, ft

α = kinetic energy correction coefficient

V = mean velocity, ft/s

h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, ft

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y, i.e., h = z+y. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope, and cross-sectional properties.

The total head loss is calculated from:

$$\mathbf{h_L = K_m[(\alpha_1 V_1^2/2g) - (\alpha_2 V_2^2/2g)] + S_e L} \qquad \mathbf{(9.11)}$$

Where:

K_m = the minor loss coefficient

S_e = the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique (Shearman, 1990 and HEC-2), ft/ft

These equations are solved numerically in a step-by-step procedure called the Standard Step Method from one cross section to the next.

The default values of the minor loss coefficient K_m are zero and 0.1 for contractions and 0.5 and 0.3 for expansions in WSPRO and HEC-2, respectively. For HEC-RAS, the default coefficients for typical bridge sections are 0.3 for contractions and 0.5 for expansions.

9.5.4 Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross-section interval should be used, or the range of starting water-surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis.

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles