OFFICE OF STRUCTURES
MANUAL ON HYDROLOGIC AND HYDRAULIC DESIGN

CHAPTER 10 BRIDGES

OFFICE OF
STRUCTURES

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10.1 Introduction

10.1.1 Definition

Bridge structures are defined as:

- Structures that transport vehicular traffic over waterways or other obstructions,

- Any highway structure over a waterway placed on footings

- Part of a stream crossing system that includes the approach roadway over the flood plain, relief openings, and the structure itself,

- Legally, all structures with a centerline span of 20 feet (6.1 m) or more. This chapter addresses structures designed hydraulically as bridges, regardless of their length. The design of culverts that meet the definition of a bridge structure is addressed in Chapter 13 and (for bottomless culverts) Chapter 11, Appendix C

As a general rule, the Office of Structures is responsible for structures with a drainage area of one square mile or greater while the Office of Highway Development handles structures with smaller drainage areas. However, the Office of Structures will normally handle cast-in-place box culverts, structures placed on footings and certain small culverts as described in Chapter 13, Culverts

101.2 Purpose of Chapter

- To establish policies and procedures for the location and hydraulic design of bridge structures over waterways and to promote early, active and continuing involvement and coordination in the project development process by structural, geotechnical and hydraulic engineers.

- To emphasize the need for full consideration of public safety and traffic service, and in particular the consequences of catastrophic loss through bridge collapse or failure.

- To emphasize the importance of stream geomorphology and environmental considerations in the selection, location and design of bridge structures.

- To present a design approach which emphasizes a comprehensive investigation of field conditions, an appropriate level of hydrologic, hydraulic and geomorphologic analyses, and thorough evaluation and verification of study results.

Chapters 8 (Hydrology), 9 (Channels), 10 (Hydraulic Design of Bridges), Chapter 11 (Evaluating Scour at Bridges), Chapter 13, Culverts and Chapter 14, Stream Geomorphology are closely related and need to be considered as a unit for purposes of defining the process of locating and designing bridges in flood plains. Cross-references are provided to appropriate policies or guidelines in these other chapters to avoid redundant text in this chapter.

10.2 Policy

10.2.1 General

Policy is a set of goals and/or a plan of action. Federal and State policies that broadly apply to the hydraulic design of structures are presented throughout the various chapters of this manual. Policies that apply to the hydraulic design of bridges are presented in this section.

The policies and procedures described in this chapter establish the design process representative of the present "normal engineering practice" or "State of Practice" for the Office of Structures. They outline the approach to be followed by a "reasonably competent and prudent designer" in evaluating, selecting, and approving a final design.

The mission of the Office of Structures is to design safe, economical and aesthetic highway structures for the traveling public through a dedicated work force committed to teamwork, communication, professionalism and customer service.

10.2.2 Type, Size and Location of Structures (TS&L)

The detail of location and design studies should be commensurate with the risk associated with the structure, its approach roads and with other economic, engineering, social or environmental concerns. Consideration of safety, traffic service, waterway adequacy, structural stability, environmental compatibility and cost-effectiveness are to be addressed throughout the project development stage through:

- hydrologic, geomorphologic, hydraulic, scour and structural engineering studies,
- use of a team effort of engineers with expertise in the study areas described above.
- development of alternative designs, and selection and approval of the final design in accordance with the project development procedures in Chapters 3 Policy and 5 Project Development.

Hydrologic studies are to be carried out in accordance with the procedures in Chapter 8.

Geomorphology studies are to be conducted using the procedures set forth in Chapter 14, Stream Morphology.

Hydraulic studies are discussed in this chapter as well as in Chapters 3, 9 and 13.

The primary responsibility of the Engineer is to provide for the public safety. Structures are to be designed to accommodate the design flood and to remain stable in resisting damage from scour and hydraulic forces for extreme flood events in accordance with the procedures in Chapter 11. Bridge deck drainage systems are to be designed to limit the spread of water onto the traveled way in accordance with the policy and guidance in Chapter 12, Bridge Decks.

10.2 Policy

An important aspect of susceptibility to flood damage is channel stability. Structures over streams should be designed, to the extent practicable, to enable the stream to transport its water and sediment discharges over long periods of time without significantly changing its plan form, profile or cross-sectional characteristics. Stream restorations and enhancements should be considered to help to stabilize degraded streams or to maintain existing stable streams. Procedures for accomplishing this goal are set forth in Chapter 14, Stream Geomorphology, Chapter 3, Policy, Chapter 13, and Chapter 13, Culverts.

The H&H Team Leader needs to form a team with the requisite experience, knowledge and skills discussed in this section, and to utilize these individuals throughout the project development process as described in Section 10.4.1. This project development process is described at greater length in Chapters 3 and 5:

10.2.2.1. Location of Structure

The location of structures should be supported by analyses of alternatives with consideration given to safety, engineering, economic, social and environmental concerns, as well as costs of construction, operation, maintenance and inspection associated with the structures and with the relative importance of the above noted concerns.

The Office of Structures reviews and approves the proposed horizontal and vertical alignments of bridge structures and approach roads in flood plains prior to development of the TS&L. This preliminary attention to the location of acceptable crossing and encroachment locations serves to promote the selection of safe and cost effective alternatives consistent with other design objectives and constraints. The selection of the alignment and grade of the structure and its approach roads needs to consider initial capital costs of construction and flood hazards including:

- The hydrologic and hydraulic characteristics of the waterway and its flood plain including channel stability, flood history and, in coastal areas, tidal ranges and periods,

- The effect of the proposed structure on flood flow patterns, stream geomorphology and stream stability, and the resulting scour potential at bridge foundations,

- The effect of the proposed structure (particularly culverts) on fish and wildlife passage.

- Avoidance of locations which create or augment hazardous hydraulic flow conditions that may endanger the stability of the structure or adversely affect adjacent development. Other concerns include delays and detours to traffic and disruption of the commerce and transportation systems of the region as a result of closure of a structure and its approach roads due to overtopping by floods or damage due to scour,

- Availability of routes for emergency evacuation,

- Flood hazards to adjacent properties,

10.2 Policy

- Consideration of environmental impacts and benefits of the proposed project as set forth in Chapter 3, Policy.

The evaluation of these factors is considered to constitute an assessment of risk for the specific site, and should be summarized in the hydraulic design report. A similar assessment should be made for temporary structures built by the contractor for use in the construction of the highway project.

For locations involving severe flood hazards, a more detailed risk analysis (See Reference 3) may provide a means of evaluating these flood hazards with respect to other environmental, regulatory or political considerations. A detailed risk analysis is seldom necessary for highway structures in Maryland since the application of State flood plain regulations has served to minimize the potential for severe flood hazards.

10.2.2.2 Structure Type and Size

The hydraulic analyses prepared during project development shall consider various stream crossing locations and alternatives; and structure designs to determine a cost effective alternative consistent with other design objectives and constraints:

- Structures and their approach roadways shall be designed for the passage of a design year flood in accordance with the criteria in Section 10.3.1 so that flood waters do not encroach upon the edge of the traffic lane of the approach roadways or the bridge deck. A written request must be submitted for the approval of the Deputy Chief Engineer, Office of Structures to obtain a design exception for selection of a design flood with a lesser recurrence interval.

- The structure shall be designed to be stable for anticipated worst case conditions of scour in accordance with the provisions of Chapter 11. In some cases, the Engineer may determine that an initial scour assessment at the TS&L stage will serve adequately to determine the acceptability of the proposed preliminary design. For this situation, the final scour report can be developed at a later stage of project development in accordance with the provisions of Chapter 5.

- The size of the waterway opening provided for structures over waterways shall be adequate to meet the requirements of Federal and State regulations for flood plain management (See Chapter 9), and for resistance to scour (See Chapter 11).

- Where diversion of flow to another watershed is expected to occur as a result of increased backwater or modification of existing flood flow patterns due to the highway construction, an evaluation of the flow diversion needs to be carried out. This study should serve to ensure compliance with regard to any legal requirements pertaining to flood hazards in the other watershed.

10.2 Policy

10.2.3 Flood Plain Management

Bridges and their approaches on flood plains shall be located and designed with regard to the goals and objectives of flood plain management including:

- prevention of uneconomic, hazardous or incompatible use and development of flood plains,

- avoidance of significant transverse and longitudinal encroachments, where practicable,
- minimization of adverse highway impacts and mitigation of unavoidable impacts, where practicable,

- consistency with the intent of the standards and criteria of the National Flood Insurance Program, where applicable. Coordination with FEMA (Federal Emergency Management Agency) shall be carried out through the local community as described in Chapter 5, Appendix B; consistency with the intent of State Regulations as set forth in COMAR 08.05.03, Waterway Construction (See Chapter 9).

- The predicted values of the 2, 10 and 100-year flood, <u>based on ultimate development in the watershed</u>, serve as the present engineering standard for evaluating and regulating flood plain uses under the flood plain regulations of the State of Maryland (See Chapter 8, Hydrology). Water surface profiles for these flood discharges and the design discharge (Section 10.3.1) shall be developed using the policies and procedures described in this chapter and in Chapter 3, Policy and Chapter 9, Channels.

- The predicted value of the 100-year flood, based on existing development in the watershed, serves as the present engineering standard for evaluating and regulating flood plains under the National Flood Insurance Program managed by FEMA.

- The final design shall be consistent with State Regulations (COMAR 08.05.03 Waterway Construction) and the National Flood Insurance Program regarding permissible increases in flood water elevations, unless exceedence of such limits can be justified by special hydraulic conditions (See Chapter 9, Stream Channels).

- Hydrologic and Hydraulic Reports are to be prepared and submitted to the Maryland Department of the Environment (MDE) and the Federal Emergency Management Agency (FEMA), where appropriate, for the necessary reviews and approvals. These reports shall demonstrate consistency with the Federal and State flood plain regulations. Chapter 3 describes the detailed information to be considered in preparing Hydrologic and Hydraulic Reports for the MDE. Chapter 5 describes the procedures to be used and the information to be included in reports submitted to FEMA. It is productive to submit hydraulic reports to MDE and FEMA for concurrent review so that the concerns of both agencies can be addressed in an efficient manner.

10.2.4 Tidal Bridges: Special procedures are required in the design tidal bridges as described in Section 10.4.5

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 and when approved in writing by the Deputy Chief Engineer, Office of Structures. The following general criteria of the State Highway Administration apply to the hydraulic analyses for the location and design of bridges.

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 2. 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 and 9. *A design exception will be necessary in order to design for a flood with a lower recurrence interval than those listed in Table 1 below:*

Table 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.

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- *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 shall be evaluated during the design process:
  - bankfull stage for geomorphology studies
  - 2, 10 and 100 year floods as explained in Section 10.2 Flood Plains,
  - Overtopping ,100- year and 500-year floods for scour evaluation

10.3.2 Guard Rail and Median Barriers

Open guard rail and median barrier sections should be considered for approach roads to structures within the limits of the 100-year flood plain. If roadway or bridge designers intend to specify use of solid barriers within flood plain limits, this condition should be determined at an early stage in the project development process so that the effect of the barrier rails can be taken into account in the hydraulic design and the TS&L plans.

10.3.3 Fish and Wildlife

Full consideration is to be given to providing reasonable conditions for the passage of fish and wildlife, and for providing opportunities to enhance habitat for local species. Such considerations routinely include evaluation of channel alternatives and opportunities for stream restoration and other enhancements.

10.3.4 Structure Type

The following items should be considered in the selection of the structure type for a stream crossing:

1) Typical structure types are listed below. See Chapter 13 for a discussion of the relative advantages of culverts vs. bridges:
   - pipe culverts
   - steel pipe arch culverts
   - box culverts or rigid frames
   - bridges and bottomless arch structures

2) Use of continuous spans, where feasible, instead of simple spans to provide for a greater measure of redundancy and safety (See Chapter 11, Evaluating Scour at Bridges).

3) Use of stub abutments with spillthrough slopes, where practicable, in lieu of vertical wall abutments to provide an open design less susceptible to damage from scour (See Chapter 11).

4) Use of streamlined shapes for the superstructure (especially where overtopping by flood waters is to be expected) as well as substructure elements to minimize the extent of

horizontal hydraulic forces acting on the structure and to facilitate passage of ice and debris.

5) Consider clearances of the superstructure (freeboard) above design high water for passage of ice and debris based on the structure type and the characteristics of the stream being crossed. For navigation channels, horizontal and vertical clearances conforming to Federal and State requirements are to be provided.

6) Location and design of piers and abutments in accordance with the guidance in Chapter 11. to minimize the scour potential and to maintain the existing flow distribution in the channel and on the flood plain.

10.3.5 Structure Size and Location

The size of the waterway opening provided for structures over waterways shall be adequate to meet the requirements of Federal and State regulations) for flood plain management (Chapter 9). In addition, the following standards should be considered in the selection of the size of the structure, consistent with other structural, geometrical, and cost considerations and limitations:

1) Set the bridge deck elevation as high above the stream bed as is practical, to facilitate passage of ice and debris.

2) Set the bridge abutments well back (ten feet or more) from the channel banks to minimize problems with lateral migration of the channel and with passage of ice, debris and wildlife.

3) Eliminate or limit the number of piers in the main channel; where feasible, avoid placement of a pier at the channel thalweg.

4) The design of the waterway area and the location of substructure elements should be carried out to accomplish the following objectives for the design year flood:

   - Backwater should not increase flood damage to developed properties upstream of the crossing (See Chapter 9).

   - Velocities through the structure(s) should damage neither the highway facility nor adjacent property.

   - The existing flow patterns should be maintained to the extent practicable.

   - Changes to the flow depth and velocity in the channel upstream of the structure should not be modified to the extent that they create a problem with sediment deposition.

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- Ecosystems and values unique to the flood plain and stream should be preserved and enhanced, where practical to do so.

- Pier spacing and orientation, and abutment type and location should be selected so as to minimize (1) obstructions to the flow; (2) the disruption of existing flood flow patterns (Chapters 9 and 14); and (3) the collection of debris. Please refer to Appendix C of Chapter 10 for guidance in sizing bridge support elements to facilitate the passage of debris.

5. The "crest-vertical curve profile" should be considered as the preferred highway crossing profile when designing for embankment overtopping since this design serves to provide for relief from the hydraulic forces acting at the bridge.

10.3.6 In-Kind Replacements

In some cases, it may be advantageous to replace an existing structure using the "in-kind" replacement procedure developed by the Maryland Department of the Environment (MDE). The types of "in-kind" replacements and the procedures to follow for this design approach are described in detail in Chapter 5. One advantage of an in-kind replacement is the acceptance by MDE of simplified hydraulic studies for purposes of granting the necessary permits. However, this approach should not be used if the Engineer determines that:

- The existing waterway area is inadequate,

- The existing structure is vulnerable to damage from scour,

- The roadway profile is too low to provide for safe and adequate traffic service, or

- A modified design would result in a safer, more cost-effective structure.

Where one or more of the above conditions are found to exist, the recommended design approach is to conduct a full hydrologic and hydraulic study and to design the structure in accordance with the policies and procedures described in this manual.

10.4 Design Procedures

10.4.1 Overview

The design for a stream crossing system requires a comprehensive engineering approach as presented in Table 10- 2 below.

10.4.1.1 Typical Steps Involved in the Design of a Bridge

Table 10-2 below sets forth the steps involved in the design process for a typical bridge design project (See also Chapter 3, Policy).

Table 10 – 2 Design Process

1. Establish Design Objectives and Priorities,
2. Hydrologic Analysis
3. Existing Condition Hydraulics
4. Geomorphology and Environmental Studies (including stream stability and consideration of channel restoration and enhancement opportunities),
5. Conceptual Design for Channel Stability
6. Assessment of Structure/ Stream Channel Alternatives
7. Proposed Condition Hydraulics
   - Pre-TS&L
   - Semi-final channel stability design
   - Scour Evaluations
   - FEMA studies when required
8. Design Plans
   - Includes temporary measures during construction to maintain channel flow.

9) Documentation of the final design, including the filing of the H&H Sheet with project plans.

The study effort and scope for each of these steps can be expected to vary to a considerable degree from project to project; however, these nine steps are common elements of the design of most bridges. (See also Chapter 3 Policy). These elements are discussed below:

1. Establish Design Objectives and Priorities. It is not possible to achieve all design objectives to the same degree because some objectives may dictate significantly different designs than others. It is important to meet and reach agreement with the regulatory and review agencies on priorities at an early stage of project development. Also, at this stage of project development, the engineers conducting the various

10.4 Design Procedures

2. H&H studies should meet with the SHA staff in order to review the design process and to establish the scope of the work required to complete each step in the design process.

3. Hydrologic Analysis (Chapter 8). Determine and evaluate/verify the bankfull flow and various flood flows ranging from the flow selected for assessing fish passage to the 2-year flood and the 500-year flood.

4. Existing condition hydraulics. Collection and analyze information needed to prepare water surface profiles of bankfull flow and of various flood flows for the existing conditions at a bridge site. The flood flows selected for evaluation typically include the 2, 10 and 100-year discharges as well as the design flood. Water surface profiles developed for these flood flows establishes the baseline conditions for evaluating the effects of the proposed bridge at the site. In most cases, a one dimensional hydraulic analysis will be adequate using the HEC-RAS program of the Corps of Engineers. In some instances, special studies may be necessary to evaluate the site conditions. An example would be using a two-dimensional analytical program such as the FHWA program FESWMS-2dh for a site with complex hydraulic features. Important considerations in the existing system hydraulics include:

- Existing flood insurance studies and flood hazard mitigation investigations (Chapter 9)

- Existing development on the flood plain

- Evaluation of the waterway area and highway profile of the existing bridge for purposes of providing for traffic service and safety. (Performance of the existing structure and roadway in accommodating flood flows).

5. Geomorphology Study (Chapter 14). Evaluate the stability of the existing channel and its flood plain using the guidance in Chapter 14, Stream Morphology.

6. Conceptual Design for Channel Stability. This step is a follow-up to the geomorphology study. Its purpose is to develop studies to the point that the location of the stream channel and the installation of any channel controls necessary to stabilize the channel are clearly defined. It also includes consideration of opportunities for channel restoration and enhancement. This work needs to be completed in order to evaluate the performance of alternative channel and structure designs. It is at this stage that the extent of work in the channel is determined. If work beyond the presently proposed highway right-of-way is found to be necessary, such work must be approved and the ROW personnel notified of the need for additional acquisitions or easements.

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7. Assessment of Structure and Stream Channel Alternatives. Assess the compatibility with the stream geomorphology, evaluate project benefits and impacts and evaluate and compare water surface profiles of the proposed structure with those of the existing structure.

8. Proposed Condition Hydraulics (Pre-TS&L Studies). Finalize the stream channel design to the extent practicable and select a structure that is consistent with the stream morphology and the hydraulics of the site. After the most appropriate structure is selected, finalize water surface profiles to document compliance with applicable State (MDE) and Federal (FEMA) regulations. Conduct scour studies to assure that the structure remains stable for the worst-case scour conditions as determined from the guidance in Chapter 11.

9. Design Plans. This process of completing the plans extends from the TS&L to PS&E. The plans for the channel are completed and channel control structures for vertical and horizontal controls are detailed on the plans. Design plans for the structure, including any scour countermeasures, are completed. Erosion control features are evaluated and included on the plans along with detailed information for any temporary stream diversions during construction.

10. Documentation. The Structures H&H Sheet is completed and filed with the PS&E plans. All computations and reports are submitted to the Structures H&H Unit in hard copy and on computer CD disks

10.4.1.1 Evaluation of Errors

Water surface profiles (typically computed by the HEC-RAS model) have a variety of technical uses including:

- Design of the bridge waterway area and bridge and roadway profiles (Chapter 10),
- Evaluation of the substructure design for resistance to scour damage (Chapter 11).

The water surface profiles serve to depict the effect of a bridge opening on water surface levels upstream and downstream of the bridge.

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, estimation or correlation,

- Errors due to use of a one-dimensional model for a bridge site with complex hydraulic flow characteristics where the concept of one-dimensional flow is invalid,

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- Errors involving selection or estimation of input parameters such as Manning's n values, energy loss coefficients, starting water surface elevations, limits of ineffective flow areas, etc.

- Errors resulting from selection of an inadequate length of stream downstream or upstream from the structure being analyzed,

- Significant computational errors resulting from using improper locations or spacing of cross-sections.

- Errors 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.

These errors can be minimized or eliminated by careful evaluation of the procedures used during the collection of field and office data and during the preparation of the input for the computer model. The Assistant Division Chief of the Plats and Survey Division is available to provide assistance regarding the use of survey data. Attention should be given to addressing the error statements in the computer output of the HEC-RAS model

New surveys for the purpose of developing or determining water-surface profiles are to be referenced to mean sea level using the North American Vertical Datum (NAVD) of 1988. Prior to the use of this datum, surveys were referenced to mean sea level using the National Geodetic Vertical Datum (NGVD) of 1929. The new datum was established primarily to account for distortions in the 1929 survey network. The difference between these two datums varies depending on the geographical location. The NAVD datum may be either higher or lower than the NGVD datum. In Maryland, the NAVD datum is typically about 0.7 foot lower than the NGVD datum. If the NAVD datum is 0.7 foot lower than the NGVD datum for a specified point, it means that the elevation of the point will be 0.7 foot higher when it is converted from the NGVD datum to the NAVD datum.

In tidal waters, depth soundings are normally referenced to mean low water or mean low tide. The relationship between mean low water and mean tide level can be found from NOAA tide tables. To minimize the chance of errors due to erroneous correlation of different datums, the engineer is encouraged to prepare a Chart of Datums prior to commencement of work (See Chapter 10 Appendix A)

10.4.2 Design Procedure Outline

When using information from an SHA survey book, the book should be reviewed to establish the datum used in the survey.

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The developed water surface profiles for a proposed design need to be evaluated for conformance to the State requirements addressed in this Chapter and in Chapter 9. Factors to consider in this evaluation include:

- Changes in flood water elevations upstream and downstream attributable to the highway project,

- Changes in flow distribution and velocities due to the bridge and its approaches, and

- The flood hazards to highways users and abutting property owners. Engineering judgment is necessary to evaluate flood hazards affecting the safety of road users or abutting property owners

10.4.3 Hydraulic Performance of Bridges

The following Figures 1 through 4 have been excerpted from the FHWA Manual Hydraulics of Bridge Waterways, HDS 1 dated 1978

- Figure 1 depicts converging and diverging flow lines which may occur at a bridge for a typical normal bridge crossing which creates a contraction in the flow. Figure 1 illustrates the location of the approach section 1, the bridge section 2, the downstream bridge section 3 and section 4, the downstream limit of the stream reach affected by the bridge constriction.
- Figure 2 is of interest in the schematic representation of how bridge backwater is measured at the approach section as the rise in the water surface elevation due to the bridge constriction. It depicts a bridge with wingwall abutments
- Figure 3 is similar to Figure 2 for a bridge with spillthrough slopes at the abutments.
- Figure 4 depicts the types of flow encountered at a bridge constriction.

Backwater 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 and abutments. Backwater can also be the result of a "choking condition" in which the channel width under the structure is constricted to the point where critical depth occurs in the contracted opening. Backwater is the result of an increase in depth and specific energy upstream of the contraction in order to develop the additional energy required to push the flow through the bridge. This condition is illustrated in Figure 4 for different flow types.

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

- 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

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- Type IIA the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In Type IIB it is higher than the normal water surface elevation and a weak hydraulic jump occurs immediately downstream of the bridge contraction. This flow type is not desirable. Where practical, the bridge waterway area should be designed to eliminate these flow types

- Type III flow is supercritical approach flow which remains supercritical through the bridge contraction. Such a flow condition is subject to only minor increases in backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the bridge. Supercritical flow is seldom encountered in Maryland streams. If this condition is found to exist, the bridge should be designed so that there is no contraction of the flow through the bridge.

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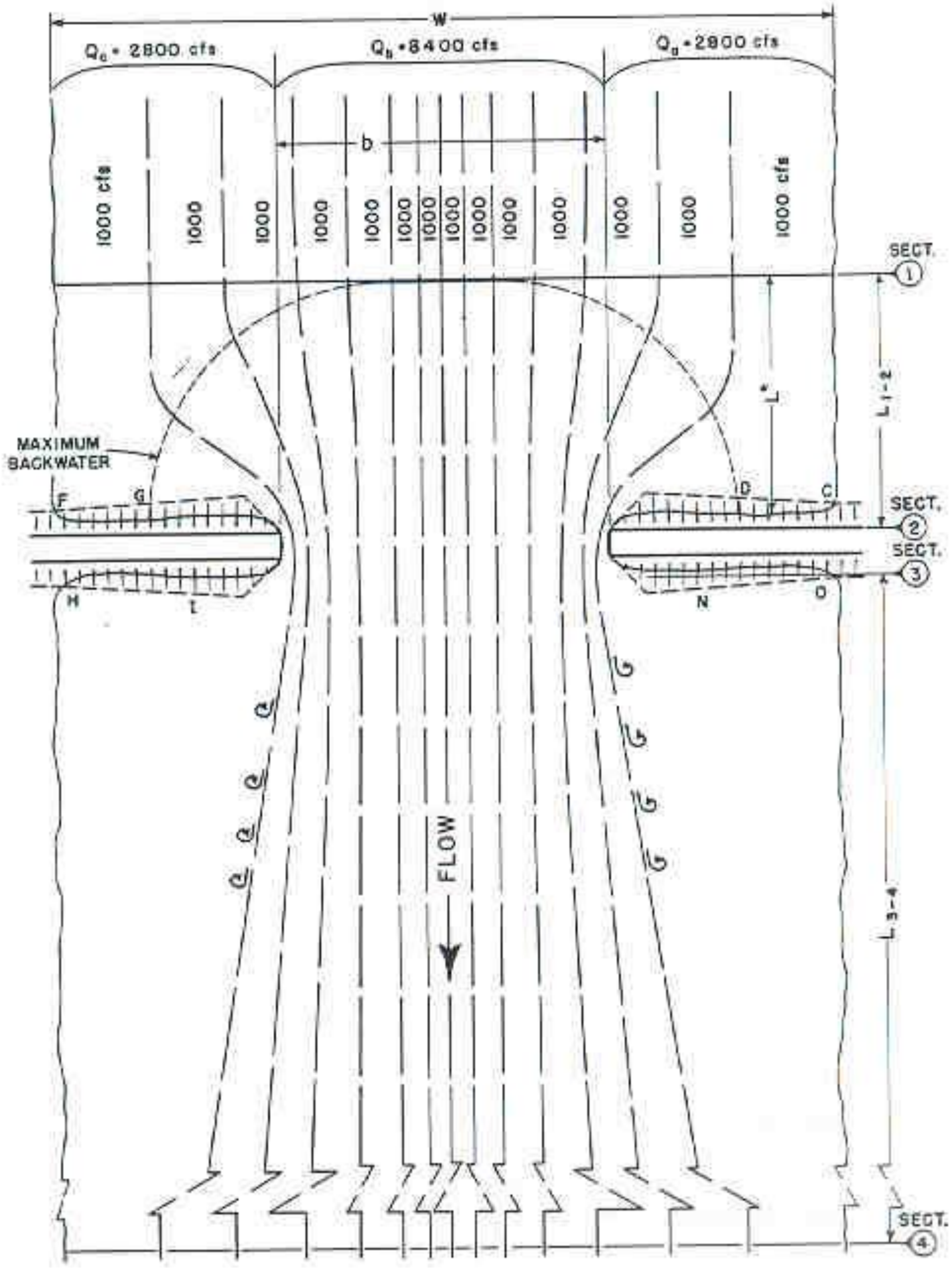

Figure 1
A Typical Pattern of Converging and Diverging Flow Lines
That Occur at a Bridge that Causes a Constriction
To the Flow

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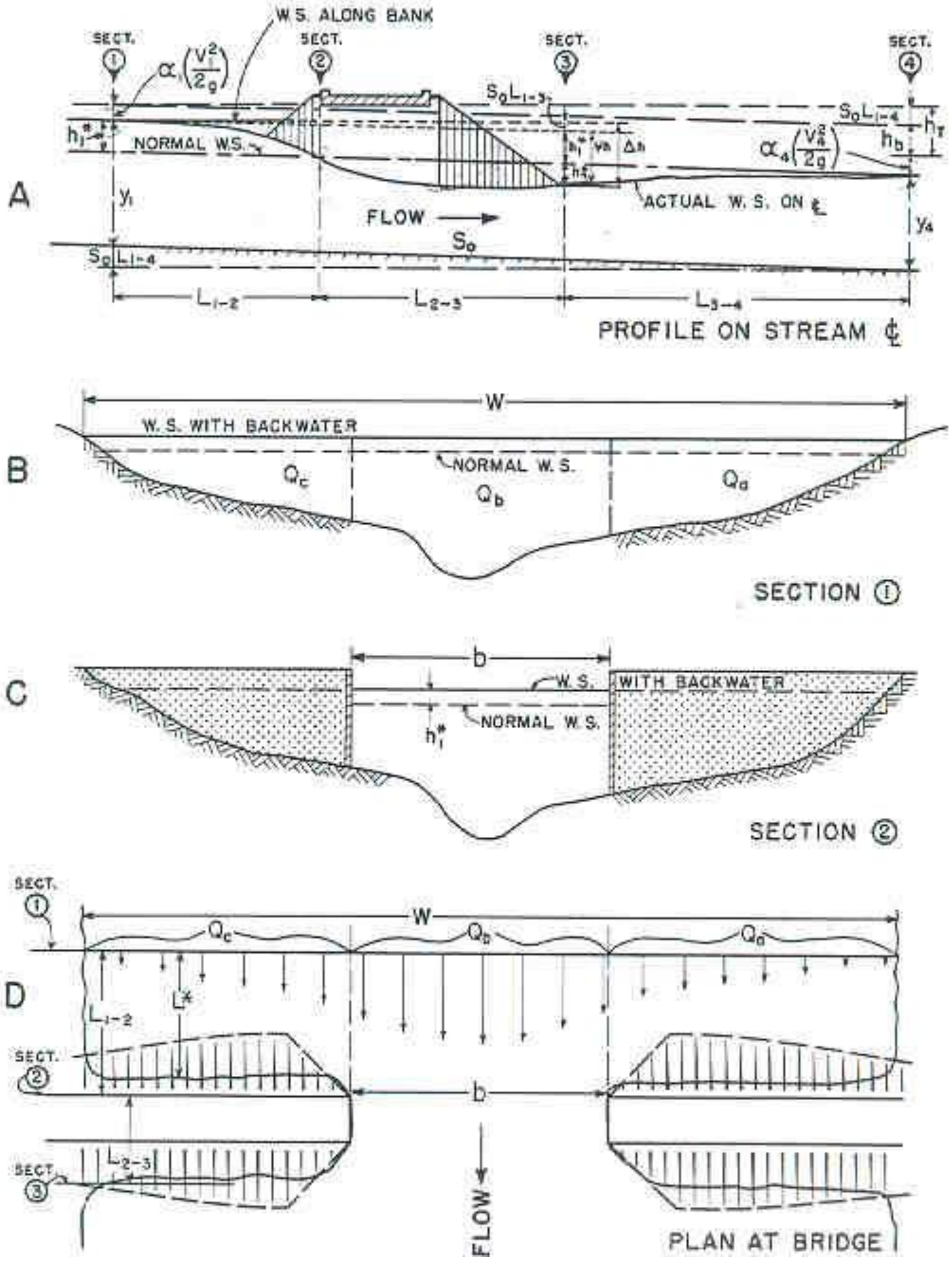

Figure 2
Schematic representation of how bridge backwater is measured <u>at the approach section</u>
for a bridge with wingwall abutments

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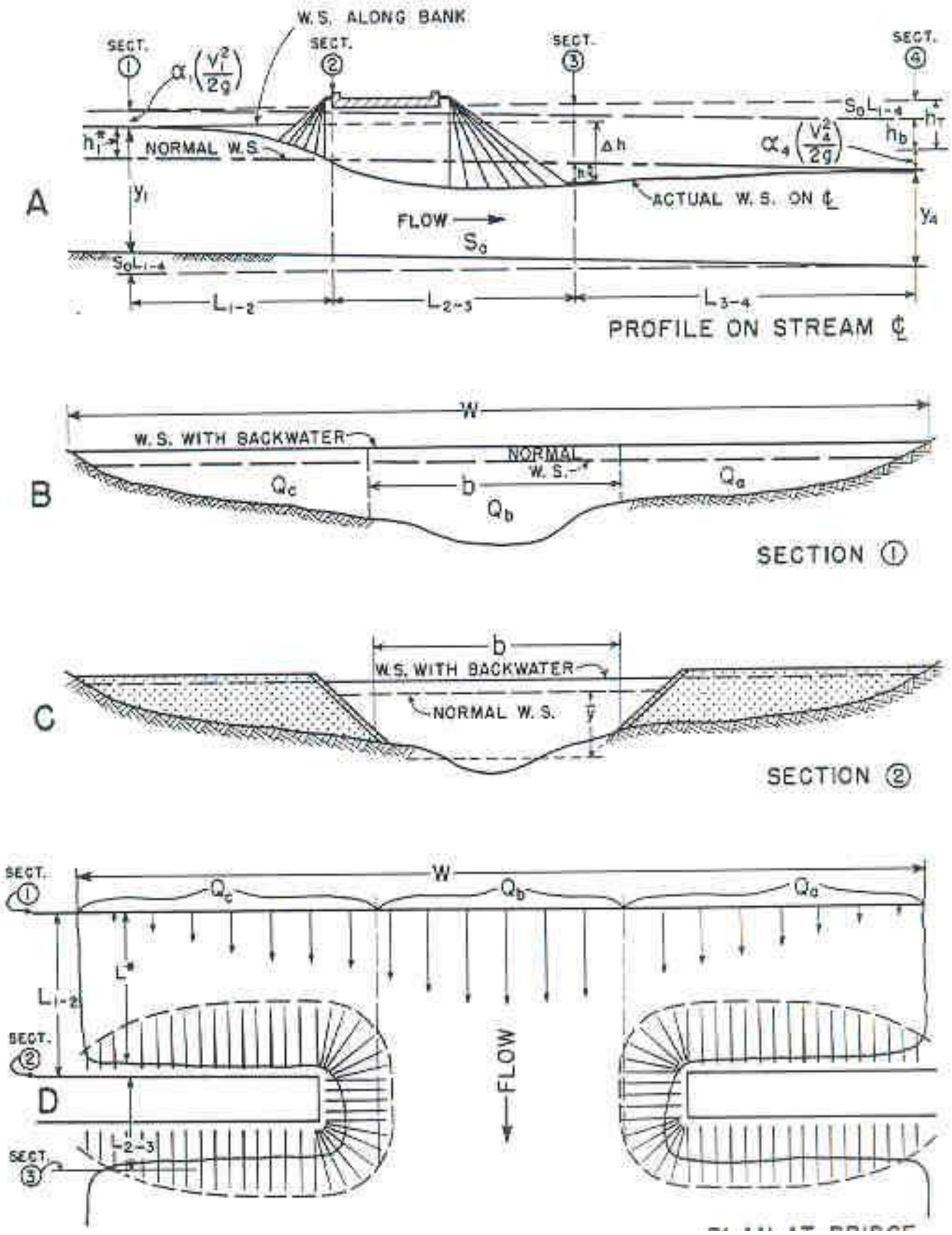

Figure 3

Schematic representation of how bridge backwater is measured <u>at the approach section</u> for a bridge with spillthrough slopes at the abutments.

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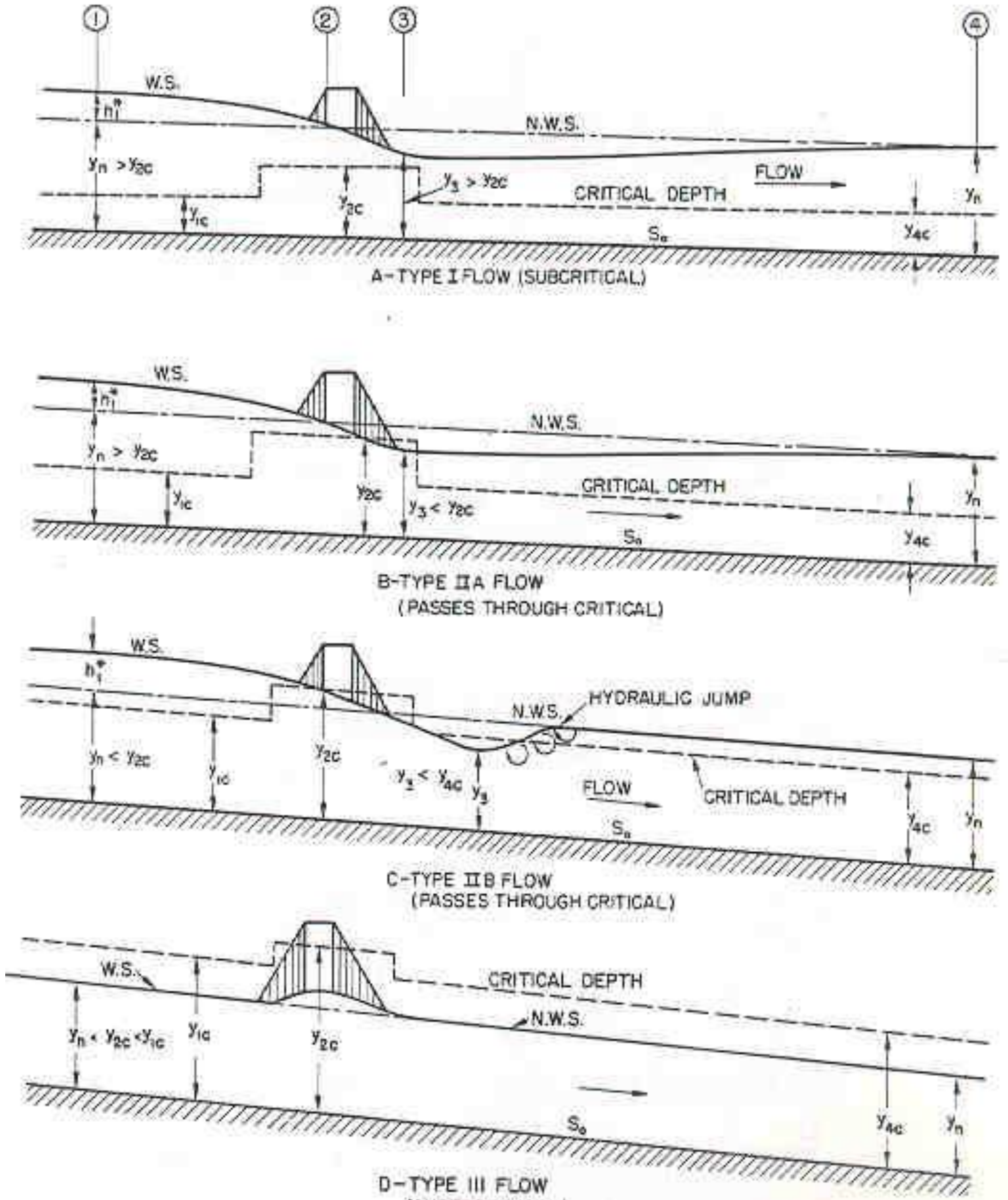

SUPERCRITICAL FLOW

Figure 4
Types of Flow Which May Occur At a Bridge Constriction.

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10.4.4 One-Dimensional Analysis

10.4.4.1 The U.S. Army Corps of Engineers HEC-RAS Model

The HEC-RAS model, Version 4.1, is the standard model in Maryland for computing water surface profiles. HEC-RAS has been developed by the Corps of Engineers Hydraulic Engineering Center at Davis, California (HEC). The initials RAS stand for River Analysis System.

HEC-RAS is menu driven and provides the user with simple, flexible procedures and choices for the input and output data. Graphical and bridge coding routines are improved and simplified. The following items highlight some of the main features of the HEC-RAS program:

- HEC-2 files can be imported to and run in HEC-RAS; however some modification of the HEC-2 files may be required to convert to the HEC-RAS format, especially for structures.
- Subcritical, supercritical and "mixed flow" regimes can be accommodated without requiring separate runs for the input data,
- A limited scour analysis using the FHWA HEC-18 methods can be performed by HEC-RAS; however, the Office of Structures uses ABSCOUR 9 for scour computations (See Chapter 11).
- The user is given the opportunity to evaluate bridge hydraulics using the concepts of (1) energy, (2) momentum, or (3) the Yarnell equations. The user also has a choice of running all three methods and allowing the program to select the method that gives the highest energy loss through the structure,
- The program handles pressure flow through use of orifice equations, and overtopping flows through use of weir equations,
- Culvert analysis procedures are improved to handle multiple culverts of various types. The program uses energy computations for outlet control conditions and the FHWA equations for inlet control conditions.
- The program can accommodate up to seven multiple openings, representing combinations of bridges, culverts and open channels,
- The cross-section output can be divided into as many as 45 segments or "slices" for purposes of determining the flow distribution in the channel and overbank areas.
- Improved graphic capabilities provide rapid, simple procedures for viewing cross-section plots to detect input errors.
- Various alternative bridge and culvert designs can be analyzed individually and then compared in a summary table.
- Output files can be presented in a wide variety of report formats
- The user can address problems of split flow or divided flow through a trial and error process. At present, this option requires the user to have an understanding of how the flow will be distributed and will require a flow separation between openings.

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Appendix A of Chapter 3 provides an extensive checklist of items to evaluate in setting up and running HEC-RAS water surface profiles. It also contains lists of items to consider in the preparation of the hydraulic report for the structure.

When a bridge project affects an existing flood plain management study of FEMA, the use of HEC-RAS is acceptable to FEMA if it is applied to the entire flood plain management study. FEMA may also accept a HEC-RAS study of a portion of the flood plain management study if it can be tied in to the existing HEC-2 model study at both the downstream and upstream ends of the reach in which the bridge is located. Either of these alternatives may require extensive work in order to get the HEC-RAS model to match precisely the existing HEC-2 FEMA study. The FEMA study could involve several miles of a stream whereas the bridge study may involve only a small portion of this distance. Therefore, the preferred method of analysis normally is to obtain the electronic file data for the original HEC-2 run and modify it to account for the changes due to the highway project.

The HEC-RAS, Version 4.1is recommended for both preliminary and final analyses of bridge hydraulics. It provides for a "template" method whereby a single section can be reproduced at several locations along a stream reach for purposes of preliminary analysis. Because of the many advantages of HEC-RAS, the single section energy models, such as those presented in the FHWA publications for Hydraulic Design Series No. 1 and HY-4 are no longer recommended either for design or preliminary analysis

10.4.4.2 The U.S. Army HEC-2 Water Surface Profile Model

The HEC-2 model has been used for the majority of the flood insurance studies performed nation-wide and in Maryland under the National Flood Insurance Program. However, it has now been superseded by HEC-RAS, Version 4.1. In some cases, it may be advantageous to utilize an existing HEC-2 study by converting it to a HEC-RAS model.

10.4.4.3 FHWA/USGS Water Surface Profile (WSPRO) Model

The FHWA/USGS WSPRO model was developed by the FHWA and the USGS and is used by these agencies as well as some highway agencies**.** It is not recommended for use in Maryland, since the HEC-RAS Program has been adopted as the SHA standard method. WSPRO combines step-backwater analysis with bridge backwater calculations. The WSPRO method analyzes pressure flow through the bridge, embankment overtopping, and flow through multiple openings and culverts. The bridge hydraulics routine relies on a one-dimensional application of 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 floodplains. The program also greatly facilitates the hydraulic analysis of alternative bridge lengths.

10.4 Design Procedures

10.4.4.4 FHWA Culvert Program HY-8

The HY-8 Culvert program developed by the FHWA is an excellent model for analyzing the complex flow patterns in culverts and multiple culvert installations and for calculating the upstream water surface elevation and energy gradient required to pass the design flow through a culvert installation. This program is approved for use in Maryland. However, the HEC-RAS program should be used (1) to determine the tailwater elevation to be used in the HY-8 program and (2) to continue the water surface profile in the stream reach upstream of the culvert.

10.4.4.5 Tidal Flows

Analysis of flow through tidal structures is also based on the principles of conservation of energy and mass as expressed by the Bernoulli and continuity equations, respectively. One of the concepts often used in the preliminary hydraulic evaluation of a structure is that the tide controls the water surface elevation on the ocean side of the structure, and therefore controls the tidal discharge through the structure. This assumption must be verified during detailed design, particularly for small structures with shallow depths, since normal depth or critical depth conditions may represent an exception to this general rule. The discharge through the structure can be determined by balancing the flow through the structure for a given time period with the change in the volume of the tidal prism (taking into consideration any upland runoff) for the same time period.

The degree of analysis required for a tidal structure depends upon the complexity of the location. Appendix A depicts the classification scheme used in Maryland for tidal structures. The computer program developed by the Office of Structures, TIDEROUT 2, can be used to analyze tidal flow for bridges where the elevation of the water surface is controlled by tides. This program also serves to evaluate the combined effect of riverine and storm tide flow, and accounts for the effect of overtopping flows. Chapter 11, Appendix B Tiderout 2 Users Manual provides guidance on the use of the TIDEROUT 2 Program.

In some cases, riverine flow will predominate and the water surface elevation will be controlled by the energy of the flow; consequently, the HEC-RAS program is used to evaluate the flow. The Woodrow Wilson Bridge is an example of this case.

Special studies may need to be carried out by persons experienced in tidal hydraulics to analyze flow through structures with certain conditions (structures that span passages between islands or an island and the mainland; structures with multiple inlets, etc). In some cases, the differences in water surface elevations across a structure may be created by wind shear forces, and this situation requires a different approach to evaluating the tidal flow. Examples of special cases of tidal flow are presented in Appendix A.

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10.4.4.6 Considerations in One-Dimensional Flow Analysis

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 the bridge opening is assumed to occur when the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening. The flow is then calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined.

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. The help files for HEC-RAS serve as a source for more detailed information on using the computer model.

10.4.5 Two-Dimensional Flow Models (The FHWA FESWMS Model)

For one-dimensional models the computed water surface profiles and velocities in a section of river are based on the premise that the elevation of the energy line is constant across the width of each cross-section, and that the individual velocity vectors within the sub-areas of the cross-section are oriented parallel to each other. In practice, most analyses are performed using one-dimensional methods such as HEC-RAS or HEC-2. While one-dimensional methods are adequate for many applications, these methods cannot describe changes or differences in water surface elevations or flow velocity vectors which occur within a cross-section. For complex conditions involving one or more features such as wide flood plains, river bends or river confluences, the inherent limitations of the one-dimensional model may produce results that contain significant errors.

Until recently, two-dimensional models were seldom used because of the added time and costs required to set up, calibrate and verify the model grid system. However the FESWMS (Finite Element Surface Water Model System) 2-DH/SMS Model published by the FHWA is now evolving into a more practical design tool. Refinements to the program may eventually permit construction of a two-dimensional grid pattern by the model using standard cross-sectional input data for 1-D models.

The SMS initials used in the FESWMS title above relate to a computer program developed by the Brigham Young University in Provo, Utah. It is used for constructing, editing and displaying finite element networks (meshes) used in the hydraulic modeling. It has an interface that is specifically designed to interact with the FESWMS-2dh model.

The FESWMS model is flexible and may be applied to many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, floodplain encroachments, multiple channels, and tidal flow around islands or in estuaries.

The SHA has utilized the FESWMS Model in several instances. Our experience has been that use of the model requires additional time and expense as compared to the use of HEC-RAS. Application of the model requires the services of persons who have received extensive training and experience with the program. At this time, the selection of the FESWMS Model has been limited to those locations where the HEC-RAS model will not be able to do a satisfactory job of developing a water surface profile.

10.4.6 Physical and Numerical Models

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

- Hydraulic performance data is needed that cannot be reliably obtained from standard design models, such as HEC-RAS

- Risk of failure or excessive over-design is unacceptable, and
- Research is needed.

Constraints on physical modeling which need to be considered include size or scale of the site as compared to the model, cost, time and the availability of facilities and qualified personnel to accomplish the model study.

The SHA developed a major physical modeling study in cooperation with the

10.4 Design Procedures

FHWA in regard to the design of the river piers for the new Woodrow Wilson Bridge. The problem involved in estimating scour at these piers met all of the criteria noted above.

There has been considerable recent progress in developing the use of numerical models to augment the use of physical models. A major consideration in the use of numerical models is to obtain adequate information from physical models or other sources to calibrate the numerical models for design conditions.

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OFFICE OF STRUCTURES
MANUAL ON HYDROLOGIC AND HYDRAULIC DESIGN

CHAPTER 10 APPENDIX A
HYDRAULICS OF TIDAL BRIDGES

September 2011

Hydraulics of Tidal Bridges

Introduction

The Maryland SHA conducts hydraulic studies for proposed new and replacement structures over tidal waters. In addition, the SHA frequently evaluates the adequacy of existing tidal bridges for vulnerability to scour damage. This presentation outlines the methods recommended for use in conducting the hydraulic analyses of existing and proposed tidal bridges.

Recent studies by the Corps of Engineers predict the likelihood of significant sea level rise along the Maryland shoreline over the next century. The Office of Structures is in the process of evaluating how the SHA should respond to this potential in the design of highways and structures. Futures guidance will be included in Chapter 10 Appendix B.

The following general principles have evolved as the SHA, Office of Structures, has gained experience in evaluating tidal bridges:

- The SHA concurs with the observations of C.R. Neill (Reference 4) that "rigorous analysis of tidal crossings is difficult **and is probably unwarranted in most cases,** but in important cases consideration should be given to enlisting a specialist in tidal hydraulics".

- New structures over tidal waters typically are designed to span the tidal channel and adjacent wetlands. Such designs do not significantly constrict the tidal flow, and consequently minimize the extent of contraction scour. A primary concern about scour for these bridges is the extent of local pier scour, and in some cases protection of abutments and approach roads from local scour and/or wave ride-up.

- However, many existing tidal bridges or replacement in kind structures may have smaller waterway openings with resulting high velocities and significant contraction, pier and abutment scour during storm tides.

- Currents of storm tides in unconstricted channels are usually about 1 to 3 feet per second.

Most of the tidal bridges in Maryland are located on the Chesapeake Bay or on estuaries or inlets tributary to the Bay. Previous studies commissioned by FEMA (Reference 12) have defined the elevation of the 100-year and 500-year storm tide elevations throughout the bay area. Studies by the SHA have identified a storm tide period of 24 hours, based on measured historic storm tides on the bay.

With this information, and the hydrologic study of flood runoff from upland drainage areas, the SHA conducts hydraulic studies of tidal bridges following Neill's method as outlined in FHWA Hydraulic Engineering Circular 18 (Reference 13). There are several judgments that need to be made in this regard:

1. If riverine flow prevails, HEC-RAS should be used to make the hydraulic analysis
2. If tidal flow prevails (that is, if the elevation of the flow through the bridge is determined by downstream tide elevations) the procedure described in this chapter can be used through the application of the computer program TIDEROUT 2.
3. There are cases, discussed later on in this chapter, such as tidal flow between an island and the mainland, where special procedures must be used to conduct the study.

Datum for use in Tidal Studies

The old FEMA studies that OBD uses to obtain storm tide elevations were based on the NGVD datum of 1929. SHA has adopted the NAVD datum of 1988 for the design of its facilities. In conducting tidal studies, it is important to convert the FEMA data (NGVD datum) to the SHA data (NAVD datum) prior to running the TIDEROUT2 analyses. Typically, the NAVD datum is lower than the NGVD datum for tidal areas tributary to the Bay. The following methodology described below and illustrated in Table 1 is suggested for making this conversion:

TIDE CHARACTERISTIC	LOCAL DATUM (ELEV.)	NGVD DATUM (ELEV.)	NAVD DATUM (ELEV.)
100-YR STORM TIDE		6	5.2
			0
		0	0.85
MLLW	0	0.05'	

Table 1 Example to Illustrate Use of Tidal Station Data

Table 1 reflects the conversion process for a bridge site on the Eastern Shore. To understand the conversion process, it is helpful to think of there being three separate gages at a tidal gauging station. The first gauge is the local datum for the station. The second gauge is for the NGVD datum and the third gauge is for the NAVD datum. The conversion from the NGVD datum to the NAVD datum involves the following steps.

For this example bridge site, the latitude is N38.33 degrees and the longitude is West 76.21 degrees

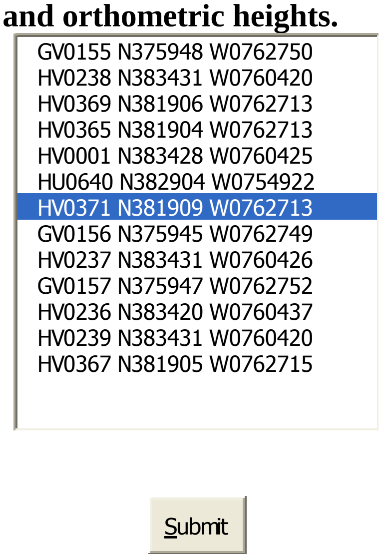
1. Obtain the 100-year storm tide elevation for the site from FEMA maps. In this

case it is 6.0 feet

2. Go to web page (http://geodesy. noaa.gov/TOOLS/Vertcon/vertcon.html. Input the latitude and longitude and the NGVD storm tide elevation of 6.0 feet. Read directly the conversion from the NGVD elevation to the NAVD elevation. In this case, the NAVD elevation corresponding to the NGVD elevation of 6.0 feet is 5.24 feet. These elevations are depicted for the 100-year storm tide in Table 1. They are used to define the high tide for the tidal hydrograph. Please note that tide data is rounded off to the nearest one-tenth of a foot.

3. The next step is to determine the low tide for the tidal hydrograph. This can be obtained from the following web site: http://www.ngs.noaa.gov/newsys-cgi-bin/ngs_opsd.prl
Go to the bottom of the web page, insert the latitude and longitude for the bridge site and hit submit. A number of gaging station sites will be listed. Go to the second set of Stations (PID stations that have all necessary information and select a station:

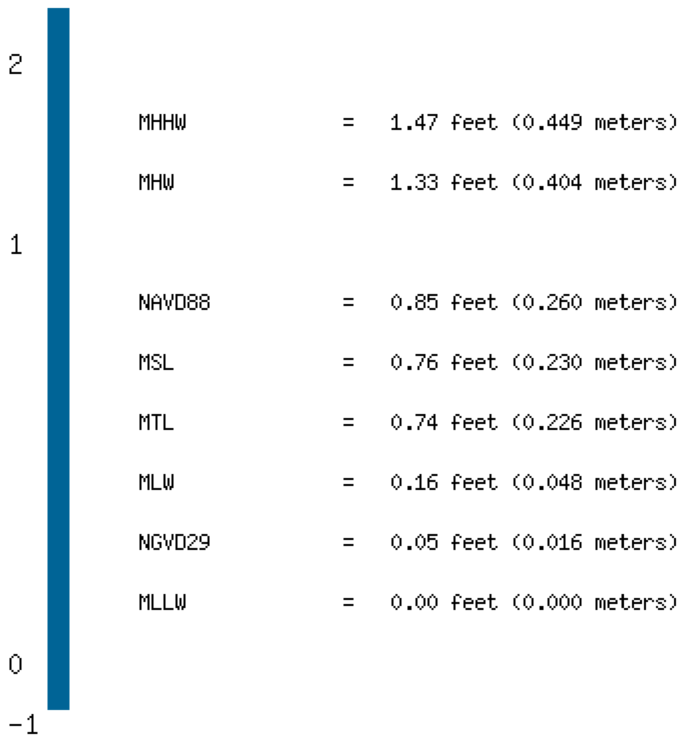
The PIDs below do have the necessary information required to create an image of the tidal and orthometric heights.



For this example, Station HVO 371 was selected at random. The print out of the station data is listed below:

Elevation Information

PID:	HV0371
VM:	450
Station ID:	8577330
Epoch:	1983-2001
Date:	Mon Mar 31 15:37:27 EDT 2008



The NAVD 88 and the NGVD 29 elevations related to MLLW were computed from Bench Mark, TIDAL 10 STA 89, at the station.

Displayed tidal datums are Mean Higher High Water(MHHW), Mean High Water (MHW), Mean Tide Level(MTL), Mean Sea Level (MSL), Mean Low Water(MLW), and Mean Lower Low Water(MLLW) referenced on 1983-2001 Epoch.

Plot the information provided on the Table 1 format. Note that the NGVD datum is 0.05 feet above the local datum and the NAVD Datum is 0.85 feet above the local datum. Therefore there is a difference of 0.8 feet between the two datums and NAVD elevations must be lowered 0.8 feet to match the NGVD elevations.

As noted above, there will be minor differences between the statistical information obtained from different NOAA stations and we recommend rounding all values to the nearest tenth of a foot.

Once the information in Table 1 is compiled, the information on the tidal hydrograph can be computed as outlined below in Table 2

TABLE 2

	NGVD ELEVATION (ft)	NAVD ELEVATION (ft)
100-YEAR PEAK TIDE ELEVATION	6.0	6- 0.8 = 5.2
LOW TIDE ELEVATION (MLLW)	-0.05	-0.85
TIDAL RANGE	-0.05 TO 6.0	-0.85 TO 5.2
TIDAL AMPLITUDE	3.0 ‘	3.0
MEAN TIDE ELEVATION	(6 + 0.05)/2 = 3.0	(5.2-0.85)/2 =2.2

ACCESSING THE NOAA WEB SITE FOR NAVD88 ELEVATIONS AT TIDAL STATIONS.

An alternative approach to the procedure discussed above is to obtain NAVD 88 elevations at tidal stations directly from the NOAA Web site. The procedure is discussed below using as an example the tidal station for Solomon's Island:

1) **Go to NOAA map that displays tidal stations:**
   http://tidesandcurrents.noaa.gov/gmap3/

2) **Zoom in or zoom out, pan etc. to get a better view**

3) **Click on the station marker(In this case Solomon's Island). You should see a "balloon" listing various current and tide links to other reports.**

4) **Click on datums You should see a report with NAVD elevations on various station data - See below:**

==

Aug 26 2011 17:57 GMT ELEVATIONS ON STATION DATUM
National Ocean Service (NOAA)

Station: 8577330
T.M.: 0 W
Name: Solomons Island, MD
Units: Feet
Status: Accepted (Apr 17 2003)
Epoch: 1983-2001

Datum:

STND

Datum	Value	Description
MHHW	5.20	Mean Higher-High Water
MHW	5.05	Mean High Water
NAVD88	4.57	North American Vertical Datum of 1988
MSL	4.48	Mean Sea Level
MTL	4.47	Mean Tide Level
DTL	4.46	Mean Diurnal Tide Level
MLW	3.88	Mean Low Water
MLLW	3.72	Mean Lower-Low Water
STND	0.00	Station Datum
GT	1.47	Great Diurnal Range
MN	1.17	Mean Range of Tide
DHQ	0.15	Mean Diurnal High Water Inequality
DLQ	0.16	Mean Diurnal Low Water Inequality
HWI	6.90	Greenwich High Water Interval (in Hours)
LWI	0.93	Greenwich Low Water Interval (in Hours)
Maximum	8.00	Highest Observed Water Level
Max Date	19550813	Highest Observed Water Level Date
Max Time	03:48	Highest Observed Water Level Time
Minimum	0.00	Lowest Observed Water Level
Min Date	19621231	Lowest Observed Water Level Date
Min Time	23:00	Lowest Observed Water Level Time
HAT	5.58	Highest Astronomical Tide
HAT Date	20010820	Highest Astronomical Tide Date
HAT Time	07:36	Highest Astronomical Tide Time
LAT	3.21	Lowest Astronomical Tide
LAT Date	19960121	Lowest Astronomical Tide Date
LAT Time	13:30	Lowest Astronomical Tide Time

Tidal Datum Analysis Period: 01/01/1983 - 12/31/2001

Click HERE for further station information including New Epoch products.

Evaluating Existing Tidal Bridges

In order to develop a cost-effective method of rating tidal bridges, the SHA developed a screening process to identify low risk bridges. The basic tool used in this screening process is the classification system outlined below:

Classification of Tidal Bridges

Following the guidance presented by Neill (Reference 4), tidal bridges are categorized into three main types based on geometric configurations of bays and estuaries and the flow patterns at the bridges:

1. bridges in enclosed bays or lagoons,
2. bridges in estuaries, and
3. bridges across islands or an island and the mainland.

Please Refer to the FHWA Hydraulic Engineering Circular 18, May 2001, Evaluating Scour at Bridges or Neill's Guide to Bridge Hydraulics, Second edition, June 2001 for further discussion of these categories.

SHA has also classified tidal waterways to take into account whether:
•there is a single inlet or multiple inlets,
•there is a planned or existing channel constriction at the bridge crossing,
•river flow or tidal flow predominates for the anticipated worst-case condition for scour, and
•tidal flow or wind establishes the anticipated worst-case condition for scour for Category 3 bridge crossings.

Category 1. Bridges in enclosed bays or across bay inlets.

In tidal waterways of this type, runoff from upland watersheds is limited, and the flow at the bridge is primarily tidal flow.

For an enclosed bay with only one inlet, the tidal flow must enter and exit through the inlet, and the hydraulic analysis is relatively straightforward using an SHA modification of Neill's tidal prism method. If there are multiple inlets to the bay, special studies must be made to determine the portion of the tidal prism that flows through each inlet for the design conditions.

If a highway crossing constricts a tidal waterway, there is a significant energy loss (head differential) at the structure. SHA has developed a program called TIDEROUT 2 to route the tidal flow through the bridge (Reference 14) for conditions of no constriction as well as significant constriction. This software is included in the Office of Structures Manual for Hydrologic and Hydraulic Design.

The purpose of the analysis is to (1) determine the maximum velocity of flow through the bridge and the corresponding flow depth and (2) determine anticipated maximum high water for storm tides. These values are then used in the bridge design and scour estimating procedures.

Category 2. Bridges in Estuaries.

Flow in estuaries consists of a combination of riverine (upland runoff) flow and tidal flow. The ratio of these flows varies depending upon the size of the upland drainage area, the surface area of the tidal estuary, the magnitude and frequency of the storm tide and the magnitude, frequency, shape and lag time of the flood hydrograph.

Group A includes those bridges over channels where the flow is governed primarily by riverine flow (90% or more of the total flow).

Group B includes bridges on estuaries where the flow is affected by both riverine and tidal flow.

Group C includes bridges over estuaries where 90% or more of the flow consists of tidal flow.

The hydraulic analysis of bridges in Category 2 (Groups A, B and C) is similar to the analysis used for Category 1, with the additional consideration of the upland flow. Where the flow conditions are controlled by the tide, TIDEROUT 2 can be used for the analysis. However, if riverine flow predominates and establishes the water surface profile at the bridge for worst-case conditions, HEC-RAS should be used to conduct the hydraulic analysis. In some cases, the engineer may determine by inspection which flow condition predominates. Such examples include;

- The Woodrow Wilson Bridge at Alexandria, VA. where the riverine flow from the huge 11,000 square mile Potomac River watershed is many times greater than the tidal flow in the Potomac River above the bridge. HEC-RAS was used here to evaluate the flow conditions at the bridge.
- The Wallace Creek crossing described in Example 1 of this chapter where the riverine flow is small in comparison with the riverine flow and TIDEROUT 2 is used to evaluate the hydraulic flow conditions.

It is not always obvious as to which hydraulic flow condition (tidal or riverine) will control and judgment must be used to select the appropriate method. In some cases, it may be necessary to use both methods to analyze the flow for worst-case conditions. The table below provides guidance in regard to selection of the appropriate hydraulic model.

TABLE 3 Selection of Hydraulic Variables for Tidal Analysis

Flow Conditon	Model	Qmax	Tailwater
Q riverine	HEC-RAS and/or TIDEROUT2 as appropriate	Q max = Q riverine	If tidal data available at bridge use MLLT datum If HEC-RAS tailwater > MLLT datum, use HECRAS

			Tailwater.
Q riverine + Q tidal	HEC-RAS and/or TIDEROUT 2 As appropriate	Q max = Q riverine +Q tidal max	If tidal data available at bridge use Mean storm tide elevation If HEC-RAS tailwater > Mean storm tide elevation, use HECRAS Tailwater

<u>Category 3 Bridges connecting two islands or an island and the mainland.</u>

The hydraulic analysis of bridges in this category is almost entirely dependent on the site conditions, and no general guidelines have been developed for such locations. The effect of wind often becomes a primary consideration at these locations. The analysis of such tidal problems should be undertaken by Engineers knowledgeable about tidal hydraulics.

<u>Category 4 Bridges where the bridge creates a constriction in the tidal flow and the site conditions are also vulnerable to wind set up at the bridge</u>

Guidance on evaluating this condition is presented later on in the Appendix and in Example 2.

The SHA Screening Process.

The SHA is using the following process to rate tidal bridges for Item 113, Scour Critical Bridges:

1. The location of each bridge is plotted on USGS topographic maps or NOAA navigation charts. Preliminary information is collected on the tidal waterway, upland drainage basin the highway crossing using the Tidal Bridge Data and Analysis Worksheet (Figure 2).

2. A preliminary estimate is made of the depths and velocities of storm tides, taking into account the expected contribution to the flow of flood runoff from the upland drainage basin. (TIDEROUT 2 can be used to conduct this analysis)

3. An SHA "Phase 2" study is made of each bridge. The bridge plans and files are reviewed, along with the Phase 1 Channel Stability Study conducted by the U.S. Geological Survey. This step may or may not include another bridge site inspection by the hydraulic engineers/interdisciplinary team.

4. The structure is rated for Item 113 based on the foregoing information. Generally,

structures on deep foundations with no history of scour will be rated as low risk when the preliminary hydraulic analysis indicates that the velocity of flow and anticipated scour is low. In those locations where estimated velocities are high, additional studies are made to determine the degree of risk of scour damage.

DESIGN PROCEDURES FOR EVALUATING TIDAL FLOW THROUGH BRIDGES

The steps for evaluating tidal flow through bridges are outlined below for each of the categories of tidal waterways introduced above. Examples and Case Histories are presented later in this Appendix to illustrate the application of each of the design approaches..

Hydraulic analysis of tidal waterways can be complex due to its unsteady, nonlinear and three-dimensional nature. The complexity is further enhanced by the uncertainty surrounding the interaction of tidal flows and runoff events. Several numerical, analytical and physical modeling techniques are available in the literature to address the hydraulic complexity of tidal waterways. However, SHA has determined that it is not generally cost-effective to utilize such sophisticated methods to evaluate tidal bridges in Maryland, particularly where tidal currents are low and resulting scour is minimal.

Hydraulic Analysis of Category I Tidal Bridges

The following approach is recommended for structures over tidal waterways with insignificant riverine flow.

The tidal flow rate through a channel that is relatively unconstricted by a bridge opening depends on the rate at which the bay side of the bridge is "filled" or "emptied", since the head differential between the ocean and bay sides of the bridge is expected to be small, the maximum discharge through the bridge opening is computed as follows:

$$Q_{\max} = \frac{3.14VOL}{T} \qquad \textbf{(I.1)}$$

where

Q_{max} = maximum discharge in a tidal cycle, cu. ft./sec
VOL = volume of water in the tidal prism between high and low tide levels, ft^3
T = tidal period, seconds

Using the maximum tidal flow rate, Q_{max} , the velocities for scour evaluation can be determined using a hydraulic model, or by simply dividing this flow rate by the area of the bridge opening at the mean elevation of the tidal flow being analyzed. (Neill's concept utilizes an ideal tide cycle represented by a cosine curve for a tidal basin upstream of the bridge with vertical sides.) For this condition, the maximum discharge (in an unconstricted channel) occurs at an elevation halfway between high tide and low tide. Flow velocities and depths can be determined from this information, and scour depths can be estimated using information from the soils investigations.

TIDEROUT 2 can also be used to analyze tidal bridges in this category by inputting a value of zero for riverine flow.

SHA uses a different form of this equation:

$$Q_{max} = \frac{3.14\, As * H}{T} \qquad \textbf{(I.2)}$$

where

- Q_{max} = maximum discharge in a tidal cycle, cu. ft./sec
- As = surface area of the tidal basin at mid tide.
- H = difference in elevation between high and low tide levels, ft^3
- T = tidal period, seconds

Equations I.1 and I.2 are based on the same principle. The only difference is Eq. I.1 requires the tidal basin volume between high tide and low tide, and Eq. I.2 requires the tidal basin water surface area at mid tide elevation. TIDEROUT 2 can also be used in this category by inputting a value of zero for riverine flow.

Hydraulic Analysis of Category II Tidal Bridges

Tidal flow through a contracted bridge waterway opening may be treated as flow through an orifice, in which an energy loss is encountered. Generally, the flow through an orifice is expressed in terms of the area of the waterway opening and the difference in the water-surface elevations across the contracted section as:

$$Q_0 = C_d\, A_c \sqrt{2g\,(H_s - H_t)} \qquad \textbf{(II.1)}$$

where

Q_o = flow through the bridge (*cfs*),
C_d = discharge coefficient,
A_c = bridge waterway cross-sectional area, (ft^2),
H_s = water-surface elevation upstream of the bridge (*ft*),
H_t = tidal elevation downstream of the bridge (ft), and
g = 32.2 (ft/s^2).

Using the principle of continuity of flow, the discharge through a contracted section of a tidal estuary can be analyzed as follows:

- The amount of tidal flow is determined from the change in the volume of water in the tidal basin over a specified period. This is calculated by multiplying the surface area of the upstream tidal basin (A_s) by the drop in elevation over the specified time.

$Q_{tide} = A_s\, dH_s/dt$ **(II.1)**

- The total flow approaching the bridge is equal to the sum of the tidal flow and the riverine flow, and the total flow passing through the bridge is calculated from Equation II.1. Equation II.2 is derived by setting these flows equal to each other:

$$Q + A_s \frac{dH_s}{dt} = C_d\, A_c \sqrt{2g\,(H_s - H_t)} \qquad \textbf{(II.2)}$$

where
Q = riverine flow (*cfs*), and
A_s= surface area of tidal basin upstream of the bridge (ft^2).

Equation II.2 is solved by routing the combined tidal flow and riverine flow through the bridge. This involves a trial and error process that has been incorporated into the TIDEROUT program.

$$\frac{Q_1 + Q_2}{2} + \frac{A_{S1} + A_{S2}}{2}\frac{H_{S1} - H_{S2}}{\Delta t} = C_d\left(\frac{A_{C1} + A_{C2}}{2}\right)\sqrt{2g\left(\frac{H_{S1} + H_{S2}}{2} - \frac{H_{t1} + H_{t2}}{2}\right)} \qquad \textbf{(II.3)}$$

For a given initial condition, t_1, all terms with subscript 1 are known. For $t=t_2$, the downstream tidal elevation (H_{t2}), riverine discharge (Q_2), and waterway cross-sectional area (A_{c2}) are also known or can be calculated from the tidal elevation. Only the water-surface elevation (H_{s2}) and the surface area (A_{s2}) of the upstream tidal basin remain to be determined. Since the surface area of the tidal basin is a function of the water-surface elevation, the elevation of the tidal basin at time t_2 (H_{s2}) is the only unknown term in Equation II.3. Its value can be determined by trial-and-error to balance the values on the right and left sides of Equation II.3.

The change of the water-surface elevation with time for the downstream side of the bridge due to the storm tide is determined from Equation II.4 (See Equation 75 of Section 4.6.4 in Reference 13) and illustrated in Figure 3.

$y = A\mathrm{Cos}\ 2\pi(t\text{-}tp)/T + MEL$ **(II.4)**

where
T = tidal period, selected as 24 hours for Maryland,
A = one-half of the tidal range, ft.

y = tidal elevation (*ft*), and
t = time (*hr*).
t_p = peak time (hrs), and
MEL = midtide elevation (ft.)

TIDEROUT2 uses the following method for computing discharge.

The discharge coefficient, C_d, is the product of the coefficient of contraction, C_c, and the velocity coefficient, C_v: $C_d = C_c * C_v$. The velocity coefficient is assumed to be 1.0 for this analysis. The area of flow in the downstream contracted section of the bridge is then equal to the area of the flow as it enters the bridge times the coefficient of contraction, C_c.

$$Q_o = C_d\, A_{upstream}\sqrt{2g\Delta H} = A_{downstream}\sqrt{2g\Delta H} \qquad \textbf{(II.5)}$$

The downstream area of flow corresponding to the tidal elevation is used in the routing procedure for the orifice flow condition.

If the difference in hydraulic grade line across the contracted section exceeds one-third of the flow depth, upstream of the bridge (*d*), the flow will pass through critical depth. The discharge then will be limited to that corresponding to the critical flow condition, which can be expressed as:

$$Q_{cr} = A_{cr}\sqrt{g d_{cr}} = A_{cr}\sqrt{\frac{2}{3}\, g d} \qquad \textbf{(II.6)}$$

where
Q_{cr} = critical discharge (*cfs*).
A_{cr} = critical flow area (ft^2)
d_{cr} = critical depth (*ft*)
d = flow depth upstream of bridge ft.
g = 32.2 ft/s^2
If (Q_c - Q) is negative, it means that more water is flowing into the tidal basin than is flowing out through the bridge, and the water-surface elevation will rise in the tidal basin.

<u>Hydraulic Analysis of Category III Tidal Bridges</u>

The hydraulic analysis of bridges in this category is almost entirely dependent on the site conditions, and no general guidelines have been developed for such locations. The effect of wind often becomes a primary consideration at these locations. The analysis of such tidal problems should be undertaken by Engineers knowledgeable about tidal hydraulics. An example of a the analysis of a Category III tidal bridge is provided in the case history section of this Appendix

<u>Hydraulic Analysis of Category IV Tidal Bridges Affected by Wind</u>

Wind Effects on Tidal Basin Water Level

In a large tidal basin in flat coastal areas, steady wind causes a rise in water level on the leeward side of the basin. A corresponding fall in the water surface occurs on the windward side. The rise in water level is called wind set-up and the corresponding fall is called wind set-down.

Estimation of Wind Setup and Set Down

The TIDEROUT2 Program was designed to compute a combination of tidal flow and riverine flow through a bridge without regard to the effect of the wind. However, wind conditions can have a significant effect on the velocity of flow through the bridge, and therefore on the extent of scour. This section presents a method for taking wind conditions into effect in running the TIDEROUT2 program. Wind setup refers to the rise or piling up of water (measured in feet) at the highway/bridge facility due to a sustained wind blowing towards the highway. Wind setdown refers to a drop in the water surface elevation (measured in feet) of a waterway on the downwind side of the bridge

Design Wind

The design wind needs to be selected in order to estimate wind setup and wind set down. Reference 5 presents information regarding wind speeds 30 ft above the ground for various recurrence intervals for the Maryland area. This reference depicts isolines of the highest winds associated with return periods of 50, 100 and 500-years as determined from this study.

The return period corresponds to the average interval of time for which a given event will occur (Reference 5). When the return period (Tr) is given, the probability of encounter (Ep) can be obtained for a given period of time, such as design life (L). using Equation S-1

$$Ep=1-(1-1/Tr)^L \qquad \text{(S-1)}$$

Recommendations for selecting the design wind are presented in Table 4 below. These values were computed using Equation S-1 and assuming a design service life of 80 years for typical SHA structures.

Table 4 Recommended Design Wind
(Data obtained from Reference 5)

DESIGN EVENT / DESIGN LIFE	RECOMMENDED DESIGN WIND (MPH)			
	20-YEAR OR LESS FLOOD	50-YEAR FLOOD EVENT	100-YEAR FLOOD EVENT	500-YEAR FLOOD (Estimated)
50 OR LESS	63	67	71	76
80 **RECOMMENDED**	**64**	**71**	**77**	**85**
100	64	74	79	88

Selection of the Fetch
Most of the bridges in Maryland are situated in waterways that have both a deep section (over 10 feet) and a shallow section (10 feet or less). When estimating the fetch of water to use in the design calculations for wind setup, as described below, the fetch for the deep water and the fetch for shallow water should be measured separately. The wind creates independent circulation patterns in the waterway for the different depths so that the setup and fetch for the deep water and shallow water portions of the waterway would be different, The fetch most representative of the waterway in the vicinity of the bridge should be selected for the calculation of the wind setup.

Estimation of Wind Setup in Shallow Water (average water depths of 10 feet or less)

Wind setup and set down are unsteady phenomena. They vary with the time and direction of the wind. The simplified equation presented below for wind setup in a shallow basin assumes that magnitude of the wind velocity is constant , and continues to blow in the same direction. Actually, the wind direction can be expected to shift, especially for hurricanes that travel through Maryland in a generally Northerly direction. Assuming the wind direction is a constant and is in alignment with the direction of the fetch provides for a worst-case analysis.

The equation from Reference 6 is presented below.

$$\mathbf{S = 0.00117*(F*\cos\theta)/D)*V\char`^2} \qquad \textbf{(S-2)}$$

Where

S= setup (ft) which is the difference in water level between the two ends of the fetch. The set-up is used in the TIDEROUT 2 program to determine flow quantities and velocities through the bridge.

F= Fetch (miles); The recommended fetch length for equation S-2 is the length of the shallow water portion (depth of ten feet or less) of the waterway

Θ=angle between the wind and the fetch. Assume θ = zero

D= average depth of the shallow water fetch (ft); obtained from navigational charts

V= design wind velocity (mile per hour) from Table A1.

Estimation of Wind Setup in Deep Water (average water depths of 10 feet or more)
The USACE Shore Protection Manual (Reference 6) presents the general equation for the slope of the water surface due to a wind stress in a steady state as:

$$dz/dx = (\tau_s + \tau_b)/(\gamma\ d) \qquad \text{(S-3)}$$

where

dz/ds = water surface slope

τ_s = wind shear stress

τ_b = bottom shear stress
γ = unit weight of water
d = mean water depth

This equation was further simplified by substituting shear stresses in terms of wind velocity to:

$$dz/dx = 0.00000178*(V_{30})^{2.22}/(\gamma\ d) \qquad \text{(S-4)}$$

where V_{30} = wind velocity at 30 ft above the water surface (Table A1), in ft/sec

Set-up for deep-water channels is then calculated as:

$$\mathbf{S = (dz/dx)*F = 0.00000178*F*(V^{2.22})/(\gamma*d)} \qquad \textbf{(S-5)}$$

where

S= setup (ft) which is the difference in water level between the two ends of the fetch. The set-up is used in the TIDEROUT 2 program to determine flow quantities and velocities through the bridge.

F= Fetch (miles); the recommended fetch length for equation S-5 is the length of the deep-water portion (depth of ten feet or more) of the waterway.

V= design wind velocity (miles per hour) from Table A1.

γ = unit weight of water = 62.4 lbs/ cu. ft.

d= average depth of the deep water fetch (ft); obtained from navigational charts

DESIGN PROCEDURE

The following examples and case histories illustrate the methods discussed above for the different conditions encountered at a highway crossing of a tidal waterway. The examples present methodologies for analyzing tidal flow, with and without consideration of the effects of winds. The case histories provide insight into special conditions requiring a more detailed analysis of the hydraulic conditions existing at the bridge.

Example 1: Analysis of Tidal Flow at a Type 1 Bridge Waterway Crossing . The bridge

and its approaches lie between an enclosed low wetland and the open sea. Wind effect is not considered in this example.

Background

The Route 335 bridge over Wallace creek is a typical example of the many State highways located in low lying tidal marsh areas. The drainage area of the tidal basin is a marsh of about 0.68 square miles (19,000,000 sq. ft.) bordered by a water divide on the west, a slightly higher land elevation on the north, and Rt. 335 on the south and the east. This is a Type 1 crossing (after Neill) between an enclosed low wetland and the open water of the Bay. The roadway is designed to accommodate traffic for normal day weather. The elevations along the roads range from 3 to five ft (NAVD) except that the approaches near the bridge and the bridge are raised to an elevation of 6 ft.. Please refer to page 2 for a discussion of the conversion of an NGVD datum to a NAVD datum.

The TIDEROUT 2 program is used to analyze flow through the bridge. The following data are required:

1. Tidal Data- The storm tide elevation may be obtained from FEMA maps. For this location, the 100-year tidal storm elevation is 6 feet (NGVD). Information from a nearby gage is needed to convert the NGVD elevations to NAVD elevations, and Station HVO239 (about seven miles from Wallace Creek) is used for this purpose.
    - The station information indicates that the difference between the NGVD datum and the NAVD datum is 1.02 – 0.26 = 0.76 feet. Therefore the 100-year storm tide elevation of 6.0 feet NGVD will be 6.0 – 0.76 = 5.24 NAVD.
    - The Mean Low Low Tide elevation will be – 1.02 ft. NAVD
    - Based on this information, the following 100-year tide data can be computed for the NAVD Datum:
        - Tidal range = 5.24 - (- 1.02) = 6.26
        - Tidal amplitude = ½ range = 3.13
        - Mean tide elevation = 6.26 – 3.13 = 2.11
2. A 12-hour tidal period is typically used for daily tides and a 24-hour period for storm tides. The unsteady tidal flow is analyzed as a cosine curve using the tidal amplitude and period as described previously in this chapter.
3. Routing Time. The routing period is a variable selected by the user, but a typical value of 0.1 or 0.2 hour is recommended. Making this period too long will cause problems in the solving of the routing equations and lead to inaccurate answers.
4. Roadway elevations are needed to evaluate overtopping flow. These are normally available from SHA maps drawn to a scale of 1" = 200 feet. The typical weir flow coefficient for a broad crested weir (highway) as obtained from HEC-RAS is 2.5

```
Roadway Data:
Weir Flow Coefficient For Overtopping Flow: 2.5
Roadway Profile:
Data#       Station (ft)  Elevation (ft)
1               100            4
2               720         4.46
3              1640         4.09
4              2340            5
5              2780          3.2
6              3060         3.61
7              3789          4.2
8              4780          2.5
9              5000         3.23
10             6000          2.8
11             6500         2.62
12             7500         2.35
```

5. Surface area of the tidal basin at different elevations (Figure 1).
   Tidal basin data can be obtained from contour maps. For low and flat wetland areas, they may have to be obtained from larger scale maps of 1:2400 of which the contour interval is 2 ft or smaller. For the Wallace Creek bridge a 1:2400 scale contour map was used to measure the surface areas for the tidal basin for elevations 0, 2 and 5 ft (NAVD).

   The deepest elevation of the tidal basin is at the bridge where the channel bottom is at the elevation of -6.8 ft.(NAVD) The water surface area of the basin at this point will be zero. The surface areas of the tidal basin at 0, 2, and 8 ft elevations were obtained by planimetering a 1 in=200 ft contour map to be 551,000; 10,600,000 and 19,000,000 ft^2 respectively. Above 4 feet, the basin water surface area is assumed to be enclosed.

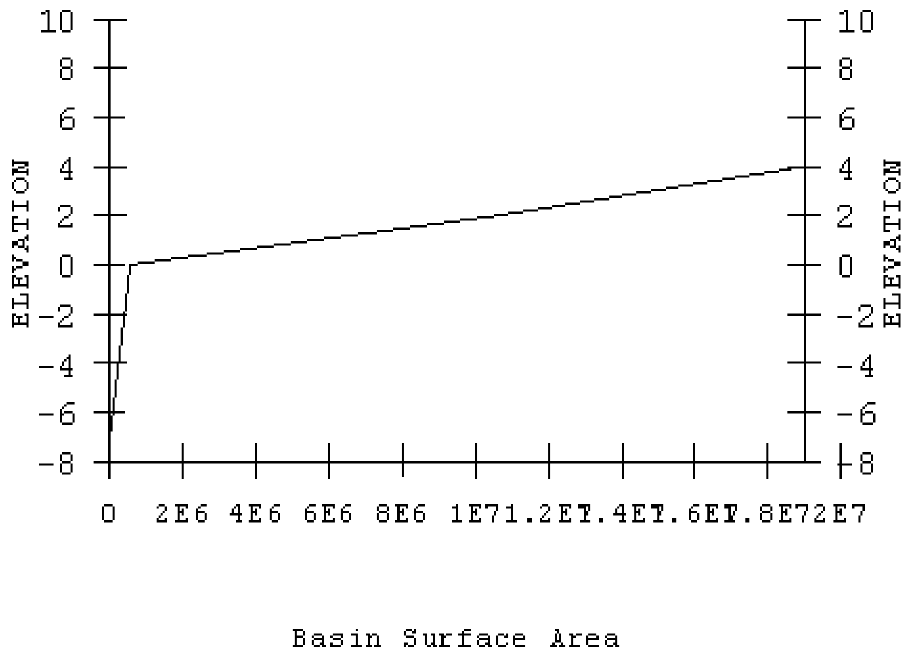


Figure 1 Plot of Tidal Basin Surface Area (ft)Vs Elevation (NAVD)
(Note that 2E6 = 2,000,000 square feet,

6 Bridge opening areas for various water surface elevations can be obtained from a field survey of the bridge or from the plans for the bridge. Figure 2.depicts the relationship between the water surface elevation and the cross-sectional area of the bridge opening. The cross-sectional area of 224 ft^2 for the elevation of 3 ft (the top of the bridge opening) was measured. Above this elevation, the bridge opening and the flow area will stayed the same. Suggested values for the orifice equation for the bridge, as presented in HEC-RAS are presented below:

TABLE 5 SUGGESTED ORIFICE FLOW COEFFIFICENTS (FROM HEC-RAS)

	UPSTREAM CONDITION	DOWNSTREAM CONDITION	Cd AVERAGE VALUE
FREE FLOW	FREE FLOW	FREE FLOW	W2/W1*
PRESSURE FLOW	SUBMERGED	FREE FLOW	0.4
PRESSURE FLOW	SUBMERGED	SUBMERGED	0.8
*NOTE: W2 = Net bridge opening width; W1 = Upstream flow width. For free flow, Use a minimum value of Cd =0.6			

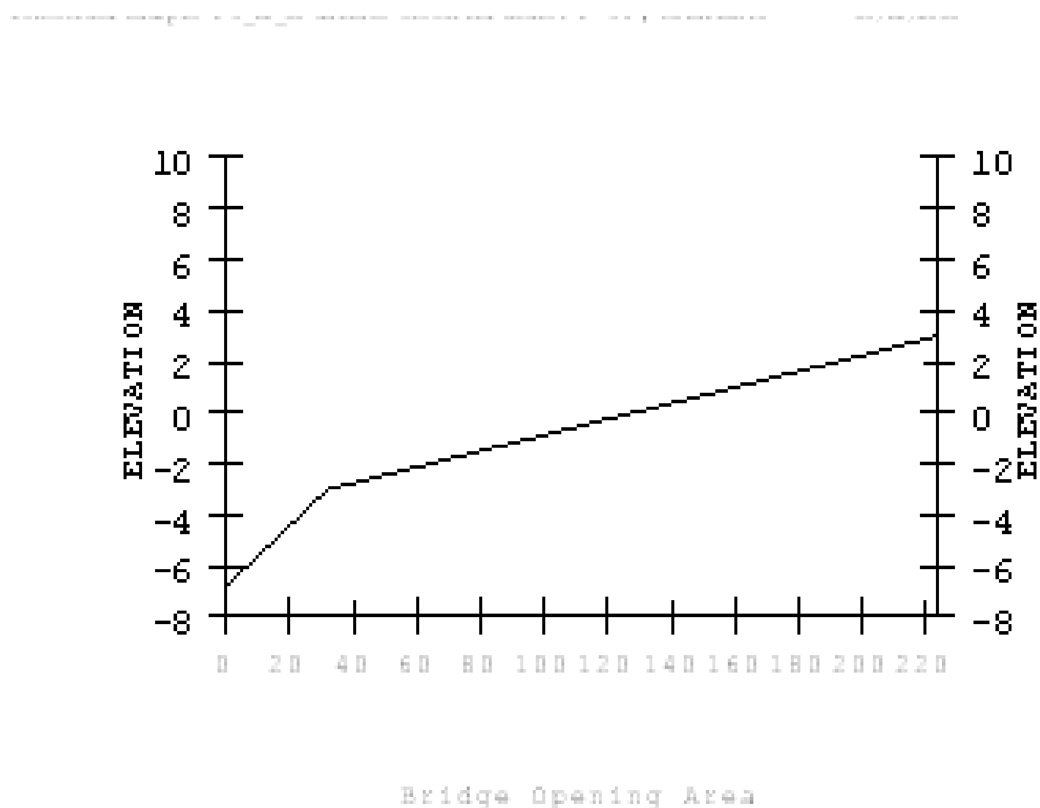


Figure 2 Elevation (NAVD) Vs Bridge Waterway Area (square feet)

Assumed Starting Condition: 100-yr storm tide; (Neither wind setup nor wind set down will be considered for this discussion) The following tidal information is used as computed in the previous section for TIDAL DATA

.

1. Starting bridge headwater elevation for the tidal basin: The User has the flexibility of selecting this value. Typically, a starting elevation is selected equal to the elevation of the 100-year storm tide as determined from the FEMA maps. For Wallace Creek the 100-year storm tide elevation is 5.24 feet NAVD. (In some cases, a different elevation may be selected if the User desires to evaluate different peaking times for the tidal hydrograph and the riverine hydrograph).

2. Mean tide elevation:. The mean storm tide elevation is 3.13 ft NAVD (See page 16 tidal data)..

3. Stream Flow Data: No inflow is expected from other basins because the basin is enclosed.. However, the hydrology of the basin is complex, and some flow may occur into the basin as a result of the variation in the tidal flow between basins. Therefore, a constant discharge of 50 cfs is assumed for this example, For crossings of estuaries with larger riverine flows, the user has the option of inputting a hydrograph or using the TIDEROUT2 program to generate a hydrograph.

Discussion: The data described above is determined and entered into the TIDEROUT2 Program. TIDEROUT2 will then route the tidal prism through the structure. The output table lists average bridge velocity and flow depth for each of the time steps selected for analysis. The worst-case hydraulic condition (typically the flow condition with the highest velocity) is then selected for the hydraulic analysis

TIDEROUT 2 PRINTOUT
Wallace Creek with no wind setup

```
**********************************************************
*          Maryland State Highway Administration         *
*                   TideRout2 Program                    *
*      Tidal Flow Through A Contracted Bridge Opening    *
*           Version 2 Build 1.22, June 29, 2006          *
**********************************************************

Project:Wallace Creek Example 1, 03/12/08; No Wind;32ft span;c

Time stamp: 03/12/2008 1:54:44 PM

Input Data:

Unit: English Units
Analysis starting time (hr.): 0
Analysis ending time (hr.): 12
Time step (hr.): .2
Starting bridge headwater elevation (ft): 5.24
Tidal amplitude (ft): 3.13
Mean tidal elevation (ft): 2.11
Tidal period (hr.): 24
Tidal Peak Time (hr):0
Stream flow is of constant discharge
Constant flow discharge (cfs): 50
```

Figure 1 Input Data

Upstream Tidal Basin Area rating Table:

Data#	Elevation (ft)	Area (sf)
1	-6.8	0
2	0	551000
3	2	10600000
4	4	19000000
5	10	19000000

Figure 2 Tidal Basin Data

Bridge Opening Data:
Discharge Coefficient: .6
Bridge Opening Area rating Table:

Data#	Elevation (ft)	Area (sf)
1	-6.8	0
2	-3	32
3	2	192
4	3	224
5	10	224

Figure 3 Bridge Opening Data

Roadway Data:
Weir Flow Coefficient For Overtopping Flow: 2.5
Roadway Profile:

Data#	Station (ft)	Elevation (ft)
1	100	4
2	720	4.46
3	1640	4.09
4	2340	5
5	2780	3.2
6	3060	3.61
7	3789	4.2
8	4780	2.5
9	5000	3.23
10	6000	2.8
11	6500	2.62
12	7500	2.35
13	8200	2.95
14	9400	3.2
15	9700	4.3

Figure 4 Roadway Data

Output Results:

Note: Remark show critical depth for critical flow, with # indicates fail to converge after 100

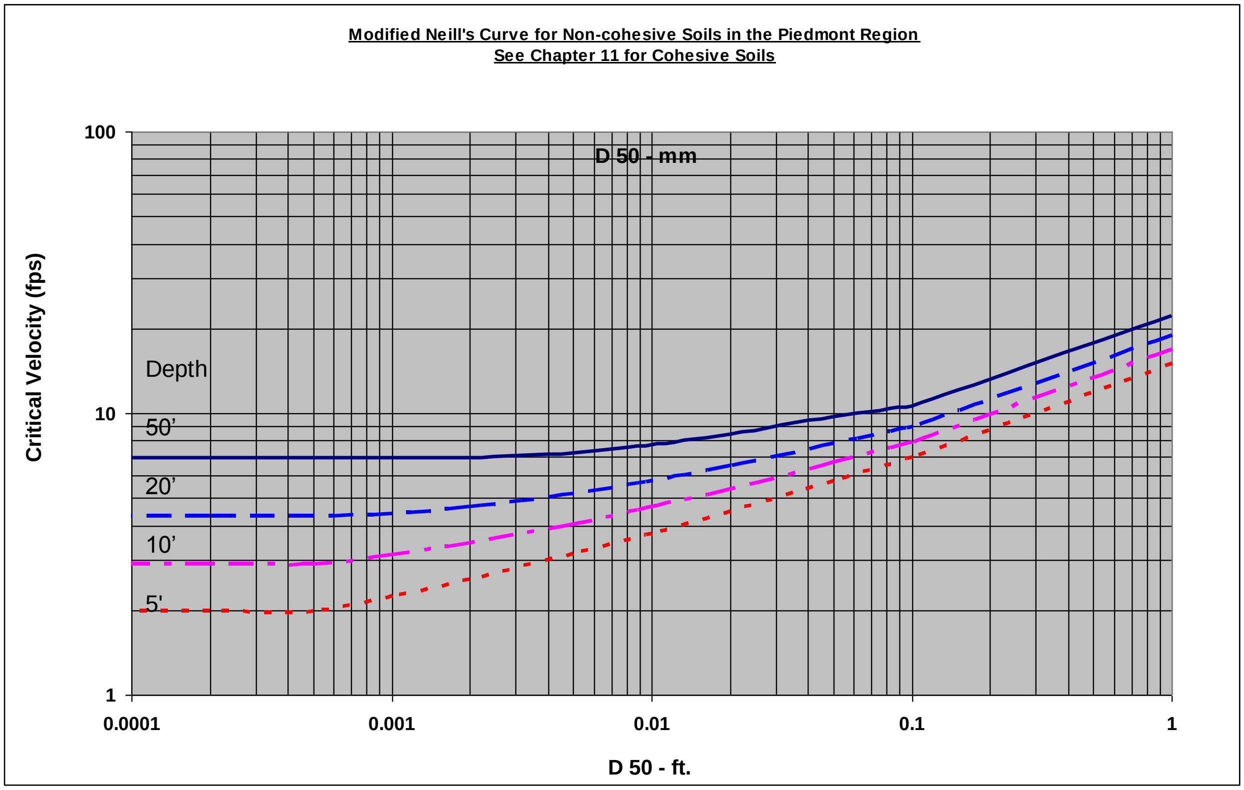
Time (hrs)	Tide EL. (ft)	Basin EL. (ft)	Bridge Q av.(cfs)	Weir Q av.(cfs)	Bridge V (ft/s)	Basin Area (sf)	Flow Area av.(sf)	Remark/ dcr(ft)
0.00	5.240	5.240	0.00	0.00	0.000	19000000.0	224.00	
0.20	5.236	5.240	49.54	2.33	0.369	19000000.0	224.00	
0.40	5.223	5.237	103.61	21.28	0.771	19000000.0	224.00	
0.60	5.201	5.230	157.99	75.43	1.176	19000000.0	224.00	
0.80	5.172	5.218	208.38	173.08	1.550	19000000.0	224.00	
9.80	-0.515	1.740	813.91	0.00	12.004	9292577.2	113.00	
10.00	-0.601	1.681	798.75	0.00	12.086	8996364.3	110.15	
10.20	-0.679	1.621	783.62	0.00	12.146	8696359.0	107.53	
10.40	-0.749	1.561	768.65	0.00	12.184	8392081.6	105.15	
10.60	-0.812	1.499	753.97	0.00	12.198	8082919.3	103.02	
10.80	-0.867	1.436	739.68	0.00	12.189	7768110.7	101.14	
11.00	-0.913	1.372	725.83	0.00	12.156	7446724.8	99.52	
11.20	-0.952	1.307	712.44	0.00	12.097	7117636.0	98.16	
11.40	-0.981	1.240	699.50	0.00	12.010	6779484.6	97.07	

Figure 5 Printout of TIDEROUT 2 Run
For Wallace Creek – No Wind Setup
(Highest velocity occurs at time 10.6: Q = 754 cfs; V = 12.2)

SCOUR ANALYSIS

Note that all elevations are based on the NAVD Datum.
Given: Worst case scour conditions occurs at time 10.6 hours

- **Tide elevation = - 0.812: flow discharge is 754 cfs. Bridge width = 32 feet.**
- **Unit discharge (q) = 754/32 =23.6 cfs/ft.**
- **From soil samples, D50 = 0.1 mm = 0.00033 ft.**
- **For clear water scour, scour will continue until flow velocity (V) = critical veloci ty (Vc)**
- **Yo = hydraulic depth = area/top width = 103.02/32 = 3.22**
- **Unit discharge = q = V * yo = Vc * y2; Since q = 23.6, then Vc*y2 must = 23.6**
- **Vc depends of the depth of flow and the D50 particle size and can be determined from the chart presented below of critical velocities developed by the Office of Structures as a modification of Neill's curves.**
- **y 2 and Vc are both unknown, so the solution requires a trial and error approach. as indicated below.**
- **All elevations based on NAVD datum**



trial number	Q	assumed y2	Vc	calculated	Comment
	unit discharge From TIDEROUT 2	Total contraction scour depth	Critical velocity from chart	unit discharge (=) Y2*Vc	
1	23.6	10	2.8	28	y2 too high
2	23.6	8	2.5	20	y2 too low
3	23.6	9	2.7	24.3	close enough

Trial and Error Solution to determine total scour depth, y2
(flow depth plus contraction scour depth)

The total contraction scour depth (y2) is computed as 9 feet = Elevation – 9.8 (NAVD)

The contraction scour depth (ys) is then computed as:
ys = y2 – yo = 9 – 3.22 = 5.8 feet.

The ABSCOUR equation for the total depth of abutment scour (y2a) is:

$y2a = Kv * Kv^{k2} (y2)$

where Kf = vortex factor for turbulence ~ 1.4 for tidal waterways
Kv^{k2} = velocity factor ~ 1.0 for tidal waterways

y2a = 1.4 *1.0* 9 = 12.6 feet = Elevation -13.4 (NAVD)

The abutment scour depth (y2s) is computed as:

y2s = y2a – yo = 12.6 – 3.2 = 9.4 feet.

Example 2: Analysis of Tidal Flow at a Bridge and Its Approaches for a Secondary Road through a Low Wetland Area. Wind effect is considered

This example uses the same information as Example 1. The conditions in Example 1 are modified to account for the potential for wind setup at the bridge.

ESTIMATION OF WIND SETUP

Location: MD 335 over Wallace Creek, Dorchester County

Given:

- Wind speed for 100 yr for a bridge designed for 80-yr life is 77 MPH. (from H&H Manual, Chapter 10, Appendix A, Table A1)
- Fetch length of the tidal basin upstream of the bridge is approximately 5,000 ft (0.95 mi)
- Average water depth is 3 ft (Shallow depth)
- Use a value of θ equal to zero. This is the worst case because it assumes that the wind is blowing straight down the fetch in the direction of the bridge

Estimate the wind setup:

Use Equation S5 in the above-mentioned manual

Total setup S=0.00117*(F*Cosθ)/D)V^2
=0.00117*(0.95Cos 0)/3)77^2=2.2 ft

(This value is the difference in elevation between the upper end of the fetch and the bridge.)

The total wind setup is the difference in water levels between the two ends of the fetch. **This total wind setup is divided in the following manner between the wind setup at the bridge and the wind set down at the upstream end of the fetch;** If the total setup is evenly divided, the setup will be 1.1 ft at the bridge and the set down will be 1.1 ft at the upstream end of the fetch. However, considering that the water will pile up like a wave against the roadway, a more conservative approach is recommended. A judgment is made to use the wind setup at the bridge of 1.3 ft (by adding 0.2 ft to 1.1 ft) and a set down of 0.9 ft at the upwind start of the fetch (by subtracting 0.2 ft from 1.1 ft). Please note that this difference of 1.3 - (- 0.9) adds up to the total setup calculated By Equation S5.

In order to incorporate these values in the TIDEROUT2 program, the following procedure is recommended (See Figure 3, Wind Setup and Setdown).

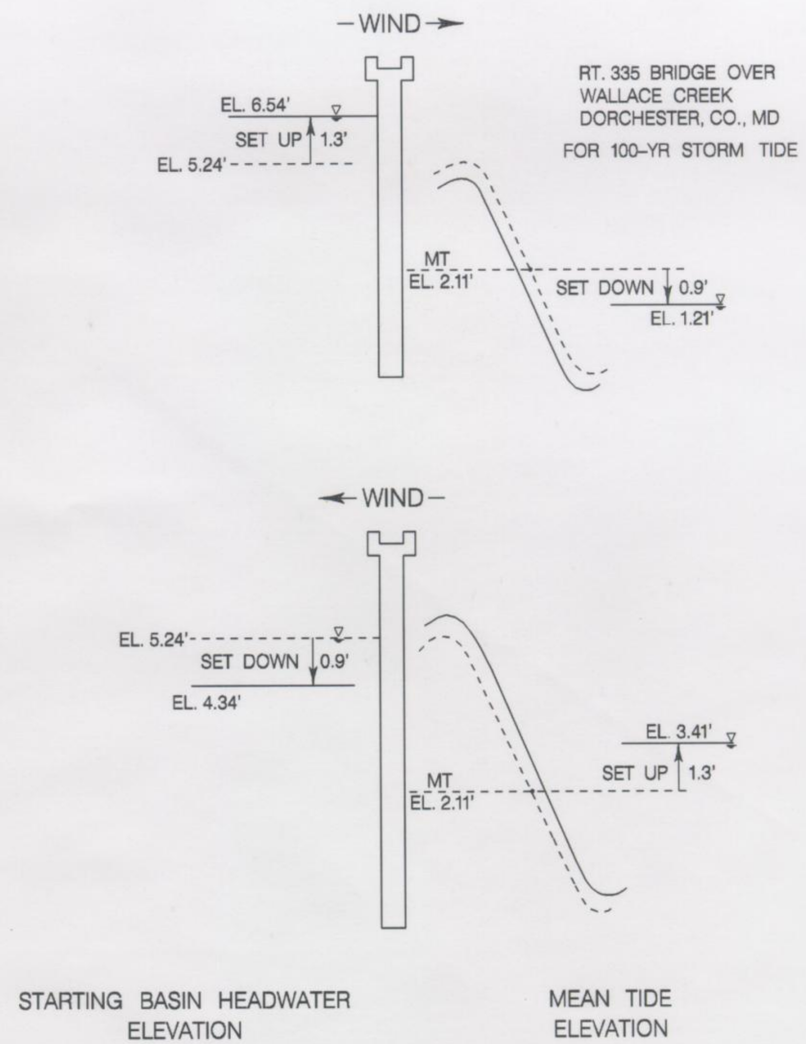


FIGURE 3. WIND SET UP AND WIND SET DOWN

<u>Wind is blowing from the tidal basin to the Bay, creating wind set up and wind set down.</u>

1. Assume that the ebb tide is to be analyzed, starting at the elevation of the high tide in the basin (This is the typical case)

2. Compute wind setup at the bridge (1.3 feet as indicated above)
3. Add the value of the setup to the value of the storm tide elevation at high tide. Input this value as the (modified) starting bridge headwater elevation on the project data card. For this example, we add 1.3 to the high tide elevation of 5.24 NAVD (Example 1)for a tide elevation of 6.54
4. Use the mean tide elevation input on the project data card one-half of the storm tide elevation as computed for Wallace Creek Example 1. This value is 3.8 feet NAVD)
5. Compute the setdown for the fetch on the downwind side of the bridge. (This is assumed to be zero due to the great volume of water in the bay)
6. Subtract the setdown from the mean tide elevation. For the Wallace Creek example, the downwind fetch is the Chesapeake Bay itself and it is likely that a body of water this large will have a setdown value of zero. Subtract the value of the set down from the mean tide elevation to obtain the modified mean tide elevation. Use a zero setdown value.

- Modified mean tide elevation = 2.11 – 0.0 = 2.11

7. Run the program using these modified values and indicate that the analysis incorporates wind setup and setdown

TIDEROUT 2 PRINTOUT FOR EXAMPLE 2
Wallace Creek with wind setup of 1.3 feet; no wind set down

```
*           Maryland State Highway Administration           *
*                     TideRout2 Program                     *
*       Tidal Flow Through A Contracted Bridge Opening      *
*            Version 2 Build 1.22, June 29, 2006            *
*************************************************************

Project:WALLACE Creek EXAMPLE 2; 4_01_2008, wind setup 1.3 ft &  wind setdown 0;100-yr

Time stamp: 04/14/2008 10:40:30 AM

Input Data:

Unit: English Units
Analysis starting time (hr.): 0
Analysis ending time (hr.): 12
Time step (hr.): .2
Starting bridge headwater elevation (ft): 6.54
Tidal amplitude (ft): 3.13
Mean tidal elevation (ft): 2.11
Tidal period (hr.): 24
Tidal Peak Time (hr):0
Stream flow is of constant discharge
Constant flow discharge (cfs): 50
```

Figure 1 Input Data

Upstream Tidal Basin Area rating Table:

Data#	Elevation (ft)	Area (sf)
1	-6.8	0
2	0	551000
3	2	10600000
4	4	19000000
5	10	19000000

Bridge Opening Data:
Discharge Coefficient: .6
Bridge Opening Area rating Table:

Data#	Elevation (ft)	Area (sf)
1	-6.8	0
2	-3	32
3	2	192
4	3	224
5	10	224

Figure 2 Tidal Basin and Bridge Opening Data

Roadway Data:
Weir Flow Coefficient For Overtopping Flow: 2.5
Roadway Profile:

Data#	Station (ft)	Elevation (ft)
1	100	4
2	720	4.46
3	1640	4.09
4	2340	5
5	2780	3.2
6	3060	3.61
7	3789	4.2
8	4780	2.5
9	5000	3.23
10	6000	2.8
11	6500	2.62
12	7500	2.35
13	8200	2.95
14	9400	3.2
15	9700	4.3

Figure 3 Roadway Elevation Data

Time (hrs)	Tide EL. (ft)	Basin EL. (ft)	Bridge Q av.(cfs)	Weir Q av.(cfs)	Bridge V (ft/s)	Basin Area (sf)	Flow Area av.(sf)
0.00	5.240	6.540	1229.74	35573.47	9.150	19000000.0	224.00
0.20	5.236	5.730	1021.64	20397.75	7.602	19000000.0	224.00
0.40	5.223	5.492	666.54	5664.60	4.959	19000000.0	224.00
0.60	5.201	5.379	509.70	2532.95	3.792	19000000.0	224.00
0.80	5.172	5.308	427.16	1490.94	3.178	19000000.0	224.00
1.00	5.133	5.253	386.27	1102.41	2.874	19000000.0	224.00
1.20	5.087	5.204	371.60	981.57	2.765	19000000.0	224.00
1.40	5.032	5.154	373.13	993.72	2.776	19000000.0	224.00
1.60	4.969	5.101	383.80	1081.39	2.856	19000000.0	224.00
1.80	4.899	5.041	399.01	1211.89	2.969	19000000.0	224.00
2.00	4.821	4.976	416.08	1366.86	3.096	19000000.0	224.00
2.20	4.735	4.903	433.56	1532.61	3.226	19000000.0	224.00
2.40	4.642	4.823	450.66	1705.77	3.353	19000000.0	224.00
2.60	4.542	4.736	466.97	1879.34	3.474	19000000.0	224.00
2.80	4.436	4.642	482.36	2049.33	3.589	19000000.0	224.00
3.00	4.323	4.542	497.17	2197.80	3.699	19000000.0	224.00
3.20	4.204	4.438	512.72	2293.43	3.815	19000000.0	224.00
3.40	4.080	4.330	530.24	2359.41	3.945	19000000.0	224.00
3.60	3.950	4.220	550.15	2403.31	4.093	19000000.0	224.00
9.80	-0.515	1.740	813.91	0.00	12.004	9292577.2	113.00
10.00	-0.601	1.681	798.75	0.00	12.086	8996364.3	110.15
10.20	-0.679	1.621	783.62	0.00	12.146	8696359.0	107.53
10.40	-0.749	1.561	768.65	0.00	12.184	8392081.6	105.15
10.60	-0.812	1.499	753.97	0.00	12.198	8082919.3	103.02
10.80	-0.867	1.436	739.68	0.00	12.189	7768110.7	101.14
11.00	-0.913	1.372	725.83	0.00	12.156	7446724.8	99.52
11.20	-0.952	1.307	712.44	0.00	12.097	7117636.0	98.16
11.40	-0.981	1.240	699.50	0.00	12.010	6779484.6	97.07
11.60	-1.003	1.170	686.94	0.00	11.895	6430626.4	96.25
11.80	-1.016	1.098	674.65	0.00	11.749	6069054.1	95.70
12.00	-1.020	1.023	662.46	0.00	11.570	5692286.0	95.43
Maximum Summary:			1229.74	35573.47	12.198		
Maximum total outflow dischrage (cfs):				36803.21			

Figure 4 Printout of TIDEROUT 2 run
For Wallace Creek. Wind Setup of 1.3 feet
(Maximum discharge = 829.7 cfs at time 10.8 hrs; flow area = 159.14 sq. ft.)

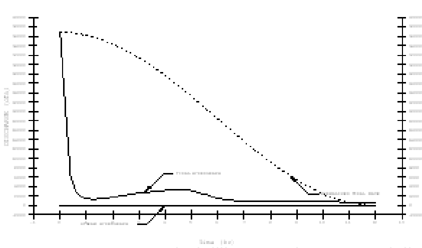


Figure 6
Plot of Storm Hydrograph Considering Wind Setup

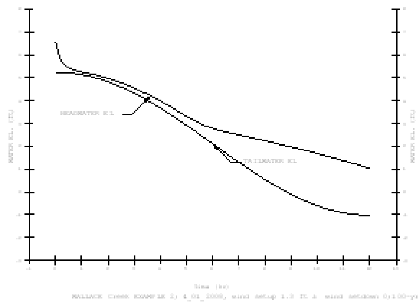


Figure 7
Tailwater- Headwater Relationship at the Bridge

Discussion

For this particular comparison of the tidal flow at Wallace Creek for conditions of no wind Example 1) and wind (Example 2), the wind effect is not significant with regard to the maximum velocity of flow through the bridge and resulting scour depths. The reason for this is that Route 335 is a low road and is overtopped by high tides. Most of the tidal flow goes over the road so the effect of the wind setup is small. This would not be the case for a high road built to an elevation above the 100-year storm tide. In this case, the tide would pile up along the roadway embankment within the tidal waterway and create a greater head differential across the bridge with a resulting greater velocity of flow through the bridge.

The effect of wind set up and set down may be important for highway crossings of tidal waters tributary to the Chesapeake Bay and should be considered in the analysis. Judgment is needed in the applications of these values because of (1) the many variables involved in computing the setup and setdown and (2) the application of these values to the tidal analysis. It is unlikely that the theoretical condition predicted by the wind setup equations will actually occur at the bridge. Nevertheless, it is reasonable to make the estimate and to consider the wind in the hydraulic design.

A comparison of the output tables will show that the worst-case scour conditions for Examples 1 and 2 are the same; therefore the scour analysis for Example 2 will be the same as Example 1 for this particular set of conditions.

CASE HISTORIES

A. Maryland Route 33 over Knapp's Narrows

At the confluence of the Choptank River and the Chesapeake Bay lies a 13-mile long peninsula stretching southward into the bay, Its southern tip is separated from the rest of the peninsula by a 200-ft wide channel, called Knapp's Narrows. This lower island, 3.5-mile long, is called Tilghman Island. MD Route 33 Bridge crosses the Knapp's Narrows, connecting the peninsula with the island.

The flow through the Knapp's Narrows is controlled by the difference in the water surface elevations on the eastern and western shores of the Tilghman Island. The water surfaces are influenced by the tide, wind setup, and wave setup. Since the peninsula intrudes into a wide bay, the tides affect the waters on both sides of the island to an equal degree; consequently, the water surface difference is small. On this basis, it is concluded that the flow through the Knapp's Narrows caused by the tides, including storm tides, will be insignificant. The difference in water surface elevations between the eastern and western shores of Tilghman Island is affected primarily by winds.

1. Wind Setup

Wind blowing over the water exerts a drag force on the surface and causes a pile-up of water on the shore, often called a wind setup. The height of wind setup depends on the wind velocity, water depth, and fetches distance. For steady, 2-D cases, the general equation for the slope of the water surface due to wind can be expressed in the following form (Reference 6)

$$\frac{dz}{dx} = \frac{T_s + T_B}{62.4d} \qquad \textbf{(III.1)}$$

where $\frac{dz}{dx}$ = water surface slope, ft/ft

T_s = wind shear stress, lb/ ft^2

T_B = bottom shear stress , lb/ ft^2

d = water depth, ft

For ($T_S + T_B$), Keulegan (Reference 7) gave a simplified equation:

$$T_S + T_B = 1.25 T_S \qquad \textbf{(III.2)}$$

The value of T_S can be approximated from the relation experimentally obtained by Sibul and Johnson (Reference 8) as:

$$T_S = 1.4x10^{-6} V_{30}^{2.22} \qquad \textbf{(III.3)}$$

Where V_{30} = wind velocity measured at 30 ft above sea surface, ft/s.

The values of V_{30} can be obtained from various sources. For this case history, it was extracted from Thom's (Reference 10) study of extreme winds in the U.S.

Combining Equations III.1 through III.3 yields the following equation:

$$\frac{dz}{dx} = 2.8x10^{-8} \frac{V_{30}^{2.22}}{d} \qquad \textbf{(III.4)}$$

By selecting design wind velocity and using numerical, finite difference techniques, wind setup can be estimated from Equation III.4. For finite difference techniques, the left side of Equation III.4 may be converted from dz/dx to $\Delta z/\Delta x$, where Δz is wind setup within a subsection Δx. With this conversion, Equation III.4 can be solved for Δz by assigning the values of Δx and water depth, d.

2. Wind Setdown

Winds blowing away from the shore cause the water surface level to drop in relation to the still water elevation. This condition is called wind setdown. The factors affecting wind setdown are the same as those for wind setup. Equations III.1 through III.4 may be used to determine the extent of the drop in water surface elevation on the leeward side of the island due to the wind setdown.

3. Wave Setup

Waves breaking along a shoreline will cause an additional increase in the water surface elevation. The Army Shore Protection Manual (Reference 6) gives the following equation for wave setup:

$$Z_W = 0.19\, H_b \left(1 - 2.82 \sqrt{\frac{H_b}{gT^2}} \right) \qquad \textbf{(III.5)}$$

where Z_W = wave setup, ft
H_b = breaker height, ft
T = incidental wave period, sec.

The breaker height can be determined from Figure 3-24, which was extracted from the US Army, CERC, SPM (Reference 6). The incidental wave period can be determined from Figure 4 (Reference 6).

For most design conditions, this equation will give wave setup values of about 0.15 H_b.

4. <u>Hydraulics of Flow in Knapp's Narrows</u>

The flow in the Knapp's Narrows is controlled by the difference in the water surface elevations on the eastern and western shores of the Tilghman Island. The difference in the water-surface elevation is the sum of wind setup, wind setdown, and wave setup.

The following step-by step method was used in calculating (1) the wind setup on the eastern shore and the wind setdown on the western shore of the Tilghman Island, and (2) hydraulic parameters in the Knapp's Narrows for the storm conditions:

Step 1 Determination of Design Wind and Check Wind

Wind setup is an unsteady phenomenon affected by wind speed and duration. The setup increases with an increase in time and ultimately reaches its maximum height. Equations III.1 through III.4 deal with the wind setup in its final stage when the setup becomes steady. Therefore, in estimating wind setups design wind speed as well as the sustain time of the wind need to be determined. The distribution of extreme winds in the United States (Reference 11) and the magnitude of maximum hurricane winds (Reference 10) were reviewed. Based on this review, the storm winds were selected to be 80 and 110 mile/hr, respectively, for the 100-and 500-year storms with a sustained time of 12 hours (Reference 11).

(Note: This case history was analyzed in 1993 for which the wind speeds for the analysis was set slightly higher than those suggested in Table A1 of this manual.)

Step 2 Computation of Wind Setup

Wind setup increases with the fetch over which the wind blows. The fetch measured to the east of the Tilghman Island is longer than the fetch to the west. Therefore, storm wind blowing from the east toward the Tilghman Island was used for the calculation of the maximum wind setup and setdown. Wind from the east would pile up the water on the eastern shore and lower the water surface on the western shore. The Choptank River estuary is about four miles wide with an average water depth of about 30 ft at the confluence with the Chesapeake Bay south of the Tilghman Island. Due to this large estuary opening, some water in the estuary will move to the south into the Chesapeake Bay and not contribute to the water piling-up against the eastern shore of the island. Based on this supposition, the flow pattern of the water out of the estuary was estimated from a NOAA Chart, and the effective fetch distance was determined as 25,000 ft.

The fetch distance was divided into ten equal sections and the water depth in each section was read from NOAA Sounding Map. Then, the wind setup was determined for each section by using Equation III.4. Finally, the total wind setup was calculated by taking the summation of all the section values. The total wind setup was found to be 2.27 ft and 4.25 ft, respectively, for the wind velocities of 80 mph and 110 mph.

Step 3 Computation of Wind Setdown

The east wind causes the water in the Chesapeake Bay to move from the eastern shore (which is the western shore of the Tilghman Island) toward western shore. The water on the eastern shore experiences setdown and that on the western shore experiences setup. The total water-surface differential between the eastern and western shores of Chesapeake Bay can be determined in the same way as described in Step 2. Since the water from the eastern shore would be moved to pile up on the western shore, the rise of water surface from the still water surface will be approximately the average value of setups. More accurate estimates of the setup and setdown can be made by finding the average water surface as illustrated in Figure 5. Values of setup and setdown are then measured from this average water surface. The wind setdown on the western shore of the Tilghman Island is estimated as 1.3 ft and 2.6 ft, respectively, for 80 mph and 110 mph storm winds.

Step 4 Computation of Wave Setup

The procedures described in the US. Army Shore Protection Manual was used in determining the wave setup. A wave setup of 0.6 ft and 0.7 ft, respectively, was calculated for the 100-year wind of 80 mph and the 500-year wind of 110 mph.

Step 5 Calculation of Total Water-Surface Difference

The estimated total water-surface difference between the eastern shore and the western shore of the Tilghman Island is determined by summing the wind setup and wave setup on the eastern shore and the wind setdown on the western shore:

	For 80 mph wind	For 110 mph wind
Wind Setup	2.27 ft	4.25 ft
Wind Setdown	1.30	2.60
Wave Setup	0.60	0.70
Total	4.17 ft	7.55 ft

Step 6 Determination of Flow Velocity

To determine the flow velocity in the channel of the Knapp's Narrows, the water-surface difference between the eastern shore and the western shore of the Tilghman Island was set equal to the total energy loss of the flow through the channel. The 200-ft wide channel has been dredged to an average depth of about 10 ft. The channel is narrowed to a width of 100 feet at the bridge with an average water depth of about 17 feet. The total length of channel is 2,400 ft. The total energy loss includes the entrance loss at the channel inlet, the contraction and expansion losses at the bridge, the exit loss at the outlet of the channel, and the friction loss. For the friction loss, the Manning equation with the coefficient of $n = 0.025$ was used.

The analysis resulted in the flow velocities in the channel to be 5.4 ft/s and 7.3 ft/s, respectively, for the 100- and 500-year storm winds.

The above noted velocities and depth were used to evaluate the scour potential at the bridge.

B. Route 445 Bridge onto Eastern Neck Island.

The Eastern Neck Island consists of a three-mile delta formed in the Chesapeake Bay by the Chester River estuary, Figure 8. The island stretches southward from the mainland. The Chester River flows from the Northeast toward the island and then turns southward at Ringgold Point near the northeast corner of the island. At the southern tip of the island, the river makes a 180 degree turn and discharges into Chesapeake Bay at Love Point. The island is separated from the mainland at the north by a waterway. The Route 445 Bridge crosses this waterway at the narrowest opening. This channel connects the water of the Chester River on the east side of the bridge at Ringgold Point to the water in the Chesapeake Bay on the west side to the river at Love Point. Therefore, the flow at the bridge is controlled by the difference in the water surface levels of the Chester River between the Ringgold Point and Love Point. This unusual geometric configuration of the area surrounding the bridge creates an interesting but complex hydraulic condition that requires special attention in evaluating the extent of scour to be expected at the bridge.

The following approach was used in the hydrologic and hydraulic analysis of the flow at the bridge:

A. Hydrology

As the flow in the Chester River estuary is the combination of the storm runoff from the river basin and tidal flow, the storm runoffs and tides need to be investigated.

Step 1. Determination of The 100-Year Flood

The USGS regression equation (Reference 1)was used to estimate the magnitude of the 100-year flood in the Chester River. The 100-year flood was determined as 29,000 cfs, and the 500-year flood of 49,000 cfs was determined by multiplying the 100-year flood by a factor of 1.7.

Step 2. Determination of Storm Tides

Tidal information at Love Point of Kent Island, compiled by NOAA, was used to determine the heights of storm tides. Since Love Point is located only about 4 miles west of the bridge in the same water, the tidal information of Love Point was considered adequate for this investigation. According to this compiled report, the extreme storm tide was estimated equal to be 7.2 ft above the Mean Sea Level (MSL) at Love Point. The 500-year storm tide was estimated from studies of Davis (Reference 2) and Ho (Reference 3) to be 9.3 ft above mean low water.

B. Hydraulics

The waterway at the bridge is sharply contracted and the flow is similar to the flow through an orifice. Therefore, the orifice equation was used in determining the flow velocity at the bridge. The following procedures were followed:

Step 1. Determination of Surface Area of Tidal Prism.

Using a 4,000-scale US Army Corps map and a NOAA Sounding map, the surface areas of the Chester River estuary tidal prism, at the elevations of 0 and -6 feet (NGVD), were obtained for three locations along the river. These locations included Love Point, Cedar Point, and Ringgold Point. The results are shown in Table 1.

TABLE 1 SURFACE AREA OF TIDAL PRISM

	Surface area, in Billion square Feet		
At Elevation	-6 ft.	0 ft.	+6 ft.*
Love Point	1.295	1.955	2.681
Cedar Point	1.128	1.538	1.989
Ringgold Point	0.97	1.22	1.49

* Estimated

No suitable map was available to determine accurately the surface area for the +6 ft elevation. Therefore, the surface area at the elevation of + 6 ft was estimated by extrapolation.

Step 2. Estimation of Tidal Flow Velocity and Discharge

The tidal flow velocities and discharges for the 100-year and 500-year high tides were determined using Neill's method (Reference 4). The velocity of the tidal flow in the estuary can be computed using the following equation:

$$V = R\left(\frac{A_s}{A_c}\right) \qquad \textbf{(6)}$$

Where R = time rate of tidal rise or fall, ft/s.

A_s 1 = surface area of tidal prism, ft^2.

A_c 2 = channel cross section,ft^2.

V = flow velocity, ft/sec.

The rate of tidal rise changes with time. The maximum rate of tidal rise generally occurs at mid-tide. If a cosine curve is assumed for a tide height vs time curve, the maximum rate will be (3.14/2) times the average rate of tidal rise. A storm tide usually takes longer than 12 hours to reach its maximum height or to reach its ebb from the maximum height. In this study, however, a half-period of 12 hours was used as the storm tide period to determine the average rate of tidal rise for a conservative estimation as assumed by Davis (Reference 2).

The average rates of tidal rise were calculated by dividing the tidal heights by the tidal period (12 hours). The maximum rate of tidal rise was then determined by multiplying the average rate by 1.57. The maximum velocities and the tidal flow rates in the Chester River at Love Point, Cedar Point, and Ringgold Point for the 100- and 500-year storm tide were then determined using Equation III.1. The results are presented in Tables 2 and 3.

TABLE 2. VELOCITIES AND FLOW RATES IN CHESTER RIVER FOR 100-YEAR STORM TIDE CONDITION

Location	Surface Area * As,bil.sq.ft.	Cross Sectional Area* Ac, mil. sq.ft.	Velocity V, ft/s	Flow Rate Q,mil,cfs
Love Point	2.39	0.354	1.77	0.617
Cedar Point	1.81	0.240	1.95	0.467
Ringgold Point	1.39	0.277	1.27	0.354

* at El 3.55 ft. (mid-tide level)

TABLE 3. VELOCITIES AND FLOW RATES IN CHESTER RIVER FOR 500-YEAR STORM TIDE CONDITION

Station	Surface Area* As,bil. sq. ft	Cross Sectional Area* Ac. mil. sq. ft	Velocity V. ft/s	Flow Rate Q, mi. cfs
Love point	2.52	0.372	2.29	0.851
Cedar Point	1.89	0.252	2.51	0.638
Ringgold Point	1.43	0.291	1.60	0.484

* at El 4.65 ft (mid-tide level)

<u>Step 3.</u> Determination of the Difference in Water-Surface Elevations across the Bridge.
Since the flow under the Eastern Neck Island bridge is influenced by surface runoff and tidal flow, the combined effects of these flows need to be considered for the investigation of the bridge scour. The surface runoff from the drainage area and the flows from storm tides were compared,

and the surface runoff was found to be less than 10% of the tidal flows. In view of the unlikely possibility that the two peak discharges would coincide and considering the insignificant amount of surface runoff, the surface runoff was subsequently neglected from further analysis.

Using the HEC-2 program, the Chester River flow was routed from Love Point to Ringgold Point for the 100- and 500-year high tide conditions to determine the water surface differences between these two points. Tidal flow is of a non-uniform nature. The flow increases along the river toward the point of discharge into the bay. For each section, the corresponding tidal discharge estimated in Step 2 (Tables 2 and 3) was used as input discharge in executing the HEC-2 program. The starting water-surface elevation at the Love Point was set at the mid-tide elevation.

The following results were obtained. The water surface differences between the Ringgold Point and the Love Point for the storm tide conditions were:

100-year high tide condition: h = 0.51 ft.
500-year high tide condition: h = 0.74 ft.

<u>Step 4.</u> Determination of Flow Velocity

The flow at the bridge is sharply contracted to form a flow condition similar to that of orifice flow; therefore, to determine the flow velocity at the bridge, the orifice equation below was used:

$$V = C\sqrt{2gh} \qquad \textbf{(7)}$$

where V = Flow velocity, ft/s
C = Velocity Coefficient
g = 32.2 ft/sec$_2$
h = Water Surface Difference, ft.

The difference in water surface elevations across the bridge is approximately the same as the water surface difference in the Chester River between Love Point and Ringgold Point as calculated in Step 3.

The velocity coefficients for various orifices can be found in any fluid mechanics text. For a streamlined orifice with a minimum energy loss, the velocity coefficient may be as high as 0.98. For the flow at the Eastern Neck Island Bridge, considering energy losses attributed to the bents, the velocity coefficient was assumed to be 0.9. With this assumption, the velocities of the flow at the bridge were determined as:

100-year high tide condition: v = 5.16 ft/s
500-year high tide condition: v = 6.21 ft/s.

The above noted velocities were used to evaluate the scour potential at the bridge.

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CHAPTER 10 APPENDIX B
HYDRAULICS OF TIDAL BRIDGES

GUIDELINES FOR CONSIDERING THE EFFECT OF FUTURE SEA-LEVEL RISE IN MARYLAND

OFFICE OF STRUCTURES

September 2009

U.S. Army Corps of Engineers
CECW-CE Washington, DC 20314-1000
Circular
No. 1165-2-211 1 July 2009
EXPIRES 1 JULY 2011

WATER RESOURCE POLICIES AND AUTHORITIES
INCORPORATING SEA-LEVEL CHANGE CONSIDERATIONS IN CIVIL WORKS PROGRAMS

1. Purpose. This circular provides United States Army Corps of Engineers (USACE) guidance for incorporating the direct and indirect physical effects of projected future sea-level change in managing, planning, engineering, designing, constructing, operating, and maintaining USACE projects and systems of projects. Recent climate research by the Intergovernmental Panel on Climate Change (IPCC) predicts continued or accelerated global warming for the 21st Century and possibly beyond, which will cause a continued or accelerated rise in global mean sea-level. Impacts to coastal and estuarine zones caused by sea-level change must be considered in all phases of Civil Works programs.

2. Applicability. This Circular applies to all USACE elements having Civil Works responsibilities and is applicable to all USACE Civil Works activities. This guidance is effective immediately, and supersedes all previous guidance on this subject. Districts and Divisions shall inform CECW of any problems with implementing this guidance.

3. Distribution Statement. This publication is approved for public release; distribution is unlimited.

4. References. Required and related references are at Appendix A. A glossary is included at the end of this document.

5. Geographic Extent of Applicability.

a. USACE water resources management projects are planned, designed, constructed and operated locally or regionally. For this reason, it is important to distinguish between global mean sea level (GMSL) and local (or "relative") mean sea level (MSL). At any location, changes in local MSL reflect the integrated effects of GMSL change plus changes of regional geologic, oceanographic, or atmospheric origin as described in Appendix B and the Glossary.

b. Potential relative sea-level change must be considered in every USACE coastal activity as far inland as the extent of estimated tidal influence. Fluvial studies (such as flood studies) that include backwater profiling should also include potential relative sea-level change in the starting water surface elevation for such profiles, where appropriate. The base level of potential relative

sea-level change is considered the historically recorded changes for the study site. Areas already experiencing relative sea-level change or where changes are predicted should analyze this as part of the study.

6. Incorporating Future Sea-Level Change Projections into Planning, Engineering Design, Construction, and Operating Projects.

a. Planning, engineering, and designing for sea level change must consider how sensitive and adaptable 1) natural and managed ecosystems and 2) human systems are to climate change and other related global changes. To this end, consider the following two documents:

(1) The Climate Change Science Program (CCSP) Synthesis and Assessment Product 4.1 (SAP 4.1) *Coastal Sensitivity to Sea-Level Rise: A Focus on the Mid-Atlantic Region* details both

how sea-level change affects coastal environments and what needs to be addressed to protect the environment and sustain economic growth. SAP 4.1 represents the most current knowledge on implications of rising sea levels and possible adaptive responses.

(2) The National Research Council's 1987 report *Responding to Changes in Sea Level: Engineering Implications* recommends a multiple scenario approach to deal with key uncertainties for which no reliable or credible probabilities can be obtained. In the context of USACE planning, multiple scenarios address uncertainty and help us develop better riskinformed alternatives.

b. Planning studies and engineering designs should consider alternatives that are developed and assessed for the entire range of possible future rates of sea-level change. These alternatives will include structural and nonstructural solutions, or a combination of both. Evaluate alternatives using "low," "intermediate," and "high" rates of future sea-level change for both "with" and "without" project conditions. Use the historic rate of sea-level change as the "low" rate. Base "intermediate" and "high" rates on the following:

(1) Estimate the "intermediate" rate of local mean sea-level change using the modified NRC Curve I and equations 2 and 3 in Appendix B (see Figures B-9 and B-11). Consider both the most recent IPCC projections and modified NRC projections and add those to the local rate of vertical land movement.

(2) Estimate the "high" rate of local sea-level change using the modified NRC Curve III and equations 2 and 3 in Appendix B (see Figures B-9 and B-11). Consider both the most recent IPCC projections and modified NRC projections and add those to the local rate of vertical land movement. This "high" rate exceeds the upper bounds of IPCC estimates from both 2001 and 2007 to accommodate for the potential rapid loss of ice from Antarctica and Greenland.

c. Determine how sensitive alternative plans and designs are to these rates of future local mean sea-level change, how this sensitivity affects calculated risk, and what design or operations and maintenance measures should be implemented to minimize adverse consequences while maximizing beneficial effects. Consider sensitivity relative to human health and safety, economic costs and benefits, environmental impacts, and other social effects. Address risks for each alternative and each potential future rate of sea-level change ("low," "intermediate," and "high"). For those alternatives sensitive to sea-level change, evaluate the potential timing and cost consequences during the plan formulation process.

FOR THE COMMANDER:

ALEX C. DORNSTAUDER
Colonel, Corps of Engineers
Executive Director of Civil Works

4 Appendices:
APPENDIX A: References
APPENDIX B: Technical Supporting Material
APPENDIX C: Flowchart to Account for Changes in Mean Sea Level
Glossary

CHAPTER 10 APPENDIX C

Passage of Floating Debris

OFFICE OF STRUCTURES

September 2009

CHAPTER 10 – APPENDIX C

Passage of Floating Debris

The problem of passing floating debris should always be a consideration in the design of bridges over waterways. Facilitating passage of debris may involve additional costs and compromises with other design objectives. These other factors need to be considered, along with the degree of the debris problem, in the overall bridge design. General guidance on design features is presented below. Additional guidance is available in the FHWA Hydraulic Engineering Circular No. 9 dated October, 2005.

1. FREEBOARD. Current OOS policy in setting the design elevation of bridges and approach roads is set forth in the main body of Chapter 10. Unless there is some exceptional debris problem which must be addressed at a particular site, the standard OSS policy governs in regard to freeboard at bridges

2. PIER TYPE It is desirable to use bridge piers that are solid with rounded noses and which are aligned with the flow. If pile bents or multiple columns are used, consider including a solid web wall between the columns to an elevation above the design storm to reduce the potential for the entrapment of debris between the columns. An example of such a pier design is Bridge No. 3015, MD 7 over White Marsh Run.

3. PIER LOCATION. Piers should be placed outside of the main path of the floating debris. For a straight reach of a stream, avoid locating the piers near the thalweg where the flow is deepest and fastest. For a curved channel, avoid the area near the bank toe on the outside bend.

4. PIER SPACING. It is desirable to provide for adequate spacing of piers to accommodate debris, consistent with other design elements. A concept referred to as the “design log length” is helpful in determining support spacing. The objective is to keep the spans long enough to keep logs from becoming lodged between supports. Support spacing should be slightly greater than the design log length. The FHWA HEC-9 guideline on estimating the design log length for a given site is provided below. For the purpose of limiting debris collection only, use the smallest of the three values in support spacing.
   - Width of the upstream channel
   - Maximum length of sturdy logs, suggested as 80 feet for Maryland.
   - One fourth of the upstream channel plus 30 feet.

   The Office of Structures has established separate stream stability criteria for pier spacing that will typically provide for longer spans than the HEC-9 criteria. SHA prefers to span stream channels with widths of 80 feet or less in order to comply with Maryland flood plain regulations, and provide for a minimum 10 foot setback between the channel bank and the abutment or pier. The following typical examples, comparing the SHA criteria and the FHWA HEC-9 criteria, indicate that current SHA design criteria provides for adequate span lengths to limit collection of debris at most sites.

Comparison of SHA Support Spacing Criteria (Stream Stability) VS FHWA HEC-9 Support Spacing Criteria (Passage of Debris).

CHANNEL WIDTH (FT)	SHA SUPPORT SPACING CRITERIA	FHWA HEC-9 SUPPORT SPACING CRITERIA (FT)
40	60 foot single span	40-50
80	100 foot single span	50-60
> 80	Fit the bridge to the stream channel to the extent practicable in locating supports • Avoid the area of the thalweg, • Avoid the area of the banks, especially the outside bank on curved channels, • Maintain a 10 foot berm beyond the channel bank.	60-70

5. SUPERSTRUCTURE. Design the superstructure to withstand extreme floods and overtopping conditions. Streamline the superstructure to minimize lateral forces on the bridge and to avoid features which will collect debris. There are trade-offs which need to be considered. The standard open rail (5” structural tubing rail) provides minimal obstruction to flood flows, but is more likely to intercept debris. On the other hand, the solid parapet and low rail presents a more streamlined shape for passing debris, but is likely to increase backwater due to the increased height of the solid parapet. The design of the connection of the pier cap with the superstructure should be streamlined to the extent practicable to minimize features that may collect debris.

CHAPTER 10 APPENDIX D

DESIGN STORM FOR PEDESTRIAN BRIDGES AND OTHER NON-HIGHWAY STRUCTURES

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CHAPTER 10 APPENDIX D

DESIGN STORM FOR PEDESTRIAN BRIDGES AND OTHER NON-HIGHWAY STRUCTURES

The Federal Highway Administration has no standards in regard to evaluating pedestrian structures for scour and hydraulic forces. Accordingly, the Office of Structures (OOS) recommends that a flood event with a recurrence interval of 10 years or greater be used when designing structures to serve only pedestrians such as bridges, piers or other such public facilities that are funded wholly or in part with Federal or State funds. The low chord of the structure should be above the water surface elevation of the design flood.

- For design floods of less than the 100-year flood, the structure should be evaluated to assure that it meets State regulations and will not create a flooding hazard due to any upstream backwater.
- If the pedestrian bridge is designed alongside an existing highway structure, the waterway area of the pedestrian bridge should match the waterway area of the highway structure.
- If the pedestrian structure is a significant and costly structure, such as a crossing of a large river, it is reasonable to design the structure for the 100-year flood.

1. An evaluation of scour should be prepared in accordance with the OOS H&H Manual. As a minimum, the overtopping flood (if less than 10 years) and the 10-year floods should be used for the scour evaluation, except for the particular cases cited above where a larger flood would be appropriate.

2. Consideration should be given to the potential for debris build-up.

3. In most cases, public safety should not be an over-riding concern since pedestrians are not expected to be on such facilities during a flood or storm event. Selection of the design storm should be based on the value of the structure, considering such factors as costs of removal of the damaged structure and replacement of a new structure. Other considerations should include the amount of pedestrian traffic, inconvenience to pedestrians during the time the facility would be closed to repair flood damage, available detour routes to accommodate pedestrian traffic and so forth.

4. Normally, the additional cost of designing a structure to resist damage from a major storm event will be small in comparison to the cost of a structure designed for a minimal storm event. SHA encourages designs which minimize the potential for flood damage (i.e. the 100-year flood). The selection of hydrologic and hydraulic factors for use in structure designs should be based on the designer's best estimate of what may occur.

5. Pedestrian structures are not as sturdy as those designed for vehicles and are therefore more likely to be damaged from a major storm.