

**OFFICE OF BRIDGE DEVELOPMENT  
MANUAL ON HYDROLOGIC AND HYDRAULIC DESIGN**

**CHAPTER 10 APPENDIX A  
HYDRAULICS OF TIDAL BRIDGES**



**JULY 16 2007**

# 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.

The following general principles have evolved as the SHA 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 are normally 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. The primary concern about scour is normally the extent of local pier scour, and in some cases protection of abutments and approach roads from local scour and/or wave ride-up.
- Currents of storm tides in unconfined channels are usually about 1 to 3 feet per second.

Almost all 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 most tidal bridges following Neill's method as outlined in FHWA Hydraulic Engineering Circular 18 (Reference 13). Special cases where this method does not apply are addressed later in this presentation.

## **Datum for use in Tidal Studies**

The old FEMA studies available for use 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 higher than the NGVD datum. The following method is suggested for making this conversion:

1. Go to the NOAA web page ([http://www.ngs.noaa.gov/cgi-bin/ngs\\_opsd.pri](http://www.ngs.noaa.gov/cgi-bin/ngs_opsd.pri)), select the July 16, 2007

- tidal station nearest to the waterway under consideration.
2. Print out the Elevation Data Graphics for the station.
  3. Compare the NGVD elevation and the NAVD elevation and note the difference.
  4. Convert the NGVD elevation to the NAVD elevation by adding the difference. For example, consider the crossing of MD 335 over Wallace Creek in Dorchester County. This bridge will be use as an example later to illustrate the application of TIDEROUT2 to tidal analysis.

The peak 100-year NGVD (FEMA) tidal elevation for Wallace Creek is 6 feet, and the peak NAVD 100-year tidal elevation is 6.8 feet. The correction factor is 0.8 feet. The mid-tide elevation for the 100-year NAVD storm tide is computed as illustrated in the table below:

NGVD ELEVATION	NAVD ELEVATION
100-YEAR PEAK TIDE ELEVATION = 6	100-YEAR PEAK TIDE ELEVATION = 6.8
MEAN TIDE ELEVATION = $6/2 = 3$	MEAN TIDE ELEVATION = $6/2 + 0.8 = 3.8$
LOW TIDE ELEVATION = 0	LOW TIDE ELEVATION = 0.8

### **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.

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### 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 Bridge Development 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. TIDEROUT 2 can be used to evaluate the flow conditions for these Category 2 bridges.

### Category 3 Bridges connecting two islands or an island and the mainland.

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.

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

Guidance on evaluating this condition is presented later on in the Appendix

### **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:

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

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<sup>3</sup>
- 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:

$$Q_{\max} = \frac{3.14 A_s * H}{T} \quad (I.2)$$

where

- $Q_{\max}$  = maximum discharge in a tidal cycle, cu. ft./sec
- $A_s$  = surface area of the tidal basin at mid tide.
- H = difference in elevation between high and low tide levels, ft<sup>3</sup>
- 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.

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### **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:

$$Q_o = C_d A_c \sqrt{2g (H_s - H_t)} \quad \text{(II.1)}$$

where

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

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.

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

- 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:

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

where

$Q$  = riverine flow (*cfs*), and  
 $A_s$  = surface area of tidal basin upstream of the bridge (*ft<sup>2</sup>*).

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

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

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.

$$y = A \cos(2\pi(t-t_p)/T) + \text{MEL} \quad (\text{II.4})$$

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,  $C_c$ .

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

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:

$$Q_{cr} = A_{cr} \sqrt{gd_{cr}} = A_{cr} \sqrt{\frac{2}{3}gd} \quad \text{(II.6)}$$

where

$Q_{cr}$  = critical discharge (*cfs*).

$A_{cr}$  = critical flow area (*ft<sup>2</sup>*)

$d_{cr}$  = critical depth (*ft*)

$d$  = flow depth upstream of bridge *ft*.

$g$  = 32.2 *ft/s<sup>2</sup>*

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.

### **Hydraulic Analysis of Category III Tidal Bridges**

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

### **Hydraulic Analysis of Category IV Tidal Bridges Affected by Wind**

#### **Wind Effects on Tidal Basin Water Level**

In large tidal basin in flat coastal arrears, steady wind causes a rise in water level on the leeward side of the basin. The corresponding fall occurs on the windward side. The rise in water level is called wind setup 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 setback 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

$$E_p = 1 - (1 - 1/T_r)^L \tag{S-1}$$

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

**Table A1 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
<b>80 RECOMMENDED</b>	<b>64</b>	<b>71</b>	<b>77</b>	<b>85</b>
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  
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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.

$$S = 0.00117*(F*\text{Cos } \theta)/D)*V^2 \quad (\text{S-2})$$

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)**

Also in USACE Shore Protection Manual (Reference 6), the general equation for the slope of the water surface due to a wind stress in steady state is shown as:

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

where

$dz/ds$  = water surface slope

$\tau_s$  = wind shear stress

$\tau_b$  = bottom shear stress

$\gamma$  = unit weight of water

$d$  = mean water depth

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

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

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:

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

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 (mile per hour) from Table A1.

$\gamma$  = 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 procedure illustrates the methods discussed above for the different conditions encountered at a highway crossing of a tidal waterway.

### **Example 1: Analysis of Tidal Flow at a Bridge and Its Approaches for a Secondary Road through a Low Wetland Area. Wind effect is not considered**

#### **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. The roadway is designed to accommodate traffic for normal day weather. The elevations along the roads are mostly lower than 4 ft (NAVD) except that the approaches near the bridge and the bridge are raised to an elevation of 5 ft. The roadway is not overtopped by daily tides of which the tidal range is about 1.5 ft (based on NOAA Solomon Is. Gage No. 3577330). The 100-year storm tide is 6.8 feet(NAVD) and it will overtop the road and bridge. Please refer to page 2 for a discussion of the conversion of a 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- amplitude; mean tide elevation; tidal period. Daily tide heights can be obtained from NOAA. Storm tide heights are available from FEMA maps. We

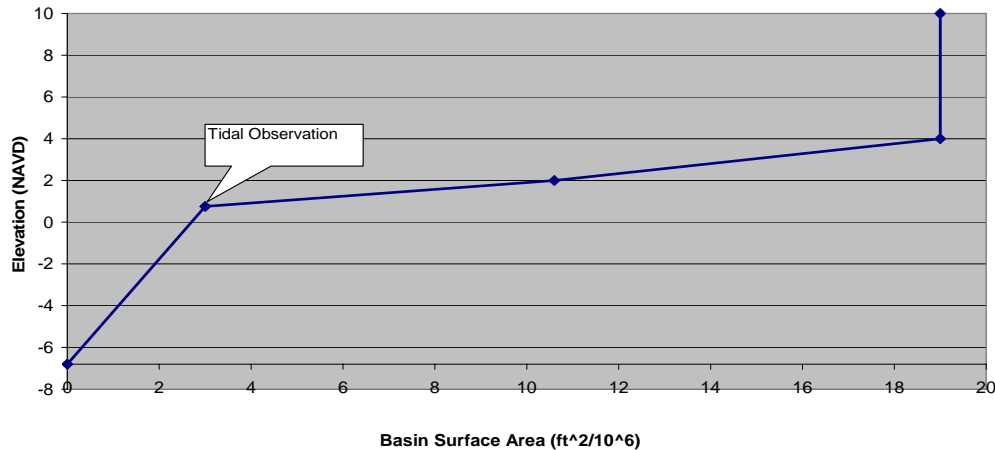
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typically use a 12-hour tidal period for daily tides and a 24-hour period for storm tides. The unsteady tidal flow is analyzed as a cosine curve using the tidal amplitude and period as described previously in this chapter. The FEMA 100-year storm tide elevation is 6.0 feet.

2. 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.
3. Roadway elevations are needed to evaluate overtopping flow. These are normally available from SHA maps drawn to a scale of 1" = 200 feet.
4. 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 2 and 4 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. The water surface area of the basin at this point will be zero ft<sup>2</sup>. The surface areas of the tidal basin at 2 and 4 ft water surface elevations were obtained by planimetry on a 1 in=200 ft contour map to be 10,600,000 and 19,000,000 ft<sup>2</sup> respectively. (Above 4 feet, the basin water surface elevation is assumed to be constant. Actually tidal basin flows would mix with other basins above this point and therefore the flows cannot be defined.) For a 10-ft elevation, an estimate of 19,100,000 ft<sup>2</sup> was made for the basin. (See Figure 1)

It is not usually feasible to obtain accurate water surface areas of the drainage basin for elevations below 2.0 from contour maps. However, if a surface area for a lower elevation is needed, the surface area at the daily mid-tide elevation can be estimated from tidal measurements as described in the following section,  
Estimation of Surface Area of Tidal Basin at Lower Level



**Figure 1 Plot of Tidal Basin Surface Area Vs Elevation**

Figure 1 identifies a point on the curve of Basin Surface Area Vs Elevation that was obtained by a tidal survey. The water surface area of the tidal basin at elevation 0.76 ft was measured as 3,000,000 ft<sup>2</sup>. The following section describes how this tidal survey was conducted.

### **ESTIMATION OF SURFACE AREA OF TIDAL BASIN AT LOWER LEVEL**

The contour intervals of most maps are usually 2 or higher. For tidal flow analysis in flat, low coastal lands sometimes, it requires the estimation of surface area of the tidal basin at the level lower than 2 ft. This section discusses the procedure in obtaining such information.

The daily tide ranges along the shore line of the Chesapeake Bay in Maryland are about one to three feet. If a tidal survey of daily tide can be made at a bridge site, the surface area of the tidal basin at a tide elevation within the tide range can be calculated by analyzing the survey data for the tide elevations and average flow velocity under the bridge.

The tidal basin surface area can be obtained from the following equation. This equation is a continuity equation, equating the displaced volume of water in the tidal basin with the flow through the bridge in the same period.

$$A_s * V_y = A * V \quad (T.1)$$

where

$A_s$  = surface area of tidal basin, ft<sup>2</sup>

$V_y$  = vertical velocity of tide, ft/s

$A$  = flow cross sectional area under the bridge, ft<sup>2</sup>

$V$  = aver flow velocity of flow under the bridge, ft/s

Step-by-Step procedures:

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4-1 Survey tide elevation and flow velocity under the bridge for a half cycle ( 6 hr.) of a daily tide from low tide to high tide or vice versa. Measurements are recommended to be taken at hourly interval. Also survey the cross sectional area of the channel at the bridge.

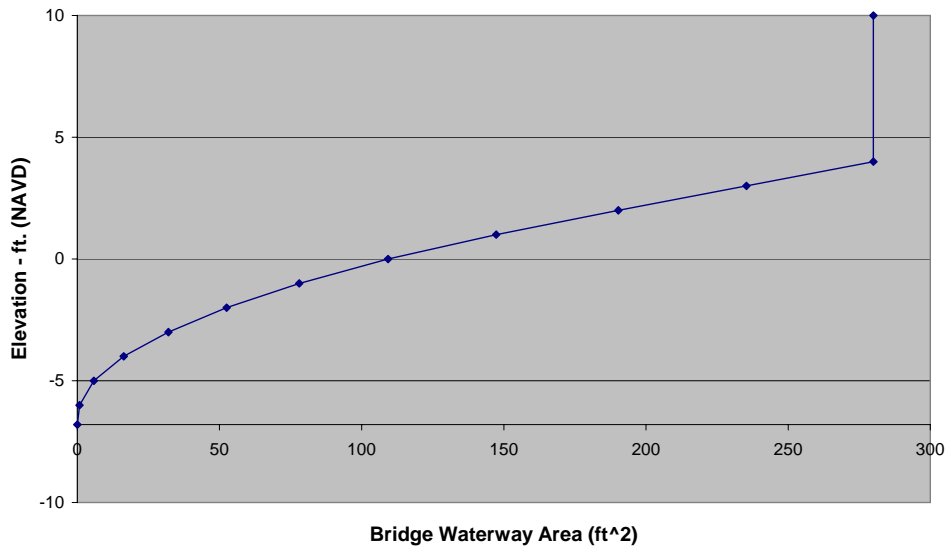
4-2 Determine the flow cross sectional area, A, and velocity, V, at the mid tide elevation.

4-3 Calculate tide range (high tide elevation minus low tide elevation), H and compute Vy at the mid tide elevation by:

$$V_y = 1.57 * H / 21,600 \quad (T.2)$$

4-4. Use results from Step2 and Equation (T.1) to compute the surface area of the basin at mid tide elevation.

5 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<sup>2</sup> 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 stay the same.



**Figure 2 Elevation Vs Bridge Waterway Area**

Assumed Starting Condition: 100-yr storm tide; (Neither wind setup nor wind set down will be considered for this discussion)

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 6.8 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 6/2 or 3 feet NGVD or  $6/2 + 0.8 = 3.8$  ft NAVD.
3. Stream Flow Data: A constant discharge of 50 cfs is assumed for this example, because riverine flow is very small and has no definable hydrograph. 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**

```
Project:WALLACE Creek, no wind setup,32ft span
Time stamp: 05/23/2007 1:32:56 PM
Input Data:
Unit: English Units
Analysis starting time (hr.): 0
Analysis ending time (hr.): 24
Time step (hr.): .2
Starting bridge headwater elevation (ft): 6.81
Tidal amplitude (ft): 3
Mean tidal elevation (ft): 3.81
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          0.76         3000000
3          2           10600000
4          4           19000000
5          10          19100000

```

**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 10

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	6.810	6.810	0.00	0.00	0.000	19046833.3	224.00	
0.20	6.806	6.810	48.70	2.21	0.362	19046832.8	224.00	
0.40	6.794	6.807	101.65	20.09	0.756	19046787.6	224.00	
0.60	6.773	6.801	154.85	71.03	1.152	19046676.7	224.00	
0.80	6.744	6.789	204.23	162.95	1.520	19046476.9	224.00	
1.00	6.708	6.770	248.73	294.37	1.851	19046166.2	224.00	
1.20	6.663	6.744	288.11	457.44	2.144	19045728.0	224.00	
1.40	6.611	6.709	322.56	641.95	2.400	19045151.8	224.00	
1.60	6.551	6.666	352.56	838.29	2.623	19044433.0	224.00	
1.80	6.483	6.614	378.72	1039.02	2.818	19043571.1	224.00	
2.00	6.408	6.554	401.61	1239.04	2.988	19042568.8	224.00	
2.20	6.326	6.486	421.77	1435.19	3.138	19041430.0	224.00	
2.40	6.237	6.410	439.66	1625.67	3.271	19040159.9	224.00	
2.60	6.141	6.326	455.64	1809.47	3.390	19038763.8	224.00	
2.80	6.039	6.235	470.00	1986.02	3.497	19037247.3	224.00	
3.00	5.931	6.137	482.97	2154.96	3.594	19035615.9	224.00	
3.20	5.817	6.033	494.71	2315.98	3.681	19033875.5	224.00	
3.40	5.698	5.922	505.36	2468.83	3.760	19032031.8	224.00	
3.60	5.573	5.805	515.03	2613.22	3.832	19030090.8	224.00	
3.80	5.444	5.684	523.79	2748.88	3.897	19028058.6	224.00	

-----Data Break-----

7.80	2.448	2.935	686.71	1449.36	5.486	14528621.9	208.61
8.00	2.310	2.848	703.61	1095.55	5.745	14159869.7	204.13
8.20	2.176	2.775	725.10	738.52	6.049	13854686.2	199.78
8.40	2.047	2.712	748.72	495.96	6.381	13591428.6	195.56
8.60	1.922	2.656	771.38	327.60	6.714	13355998.3	191.50
8.80	1.803	2.605	791.59	209.45	7.033	13138903.9	187.59
9.00	1.689	2.556	808.68	126.22	7.331	12933637.1	183.86
9.20	1.581	2.508	822.43	69.38	7.602	12735293.5	180.31
9.40	1.479	2.462	832.87	32.61	7.845	12540164.1	176.95
9.60	1.383	2.416	840.11	11.36	8.057	12345382.5	173.78
9.80	1.294	2.369	844.34	1.75	8.238	12148815.5	170.83
10.00	1.212	2.321	845.80	0.00	8.386	11949089.8	168.09
10.20	1.137	2.273	844.74	0.00	8.503	11746240.3	165.58
10.40	1.069	2.224	841.47	0.00	8.588	11540682.3	163.30
10.60	1.009	2.174	836.23	0.00	8.643	11332793.3	161.26
10.80	0.957	2.125	829.22	0.00	8.667	11122925.0	159.46
11.00	0.912	2.074	820.57	0.00	8.661	10911418.6	157.90
11.20	0.876	2.023	810.35	0.00	8.624	10698618.4	156.60
11.40	0.847	1.972	798.58	0.00	8.556	10431234.8	155.56
11.60	0.826	1.921	785.16	0.00	8.455	10115448.7	154.77
11.80	0.814	1.869	769.92	0.00	8.319	9796346.9	154.25
12.00	0.810	1.816	752.68	0.00	8.147	9474526.2	153.99

**Figure 4 Printout of TIDEROUT 2 Run  
For Wallace Creek – No Wind Setup  
(Highest velocity occurs at time 10.8: Q = 829.2 cfs; A = 159.5)**

**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 is approximately 5,000 ft (0.95 mi)
- Average water depth is 3 ft (Shallow depth)

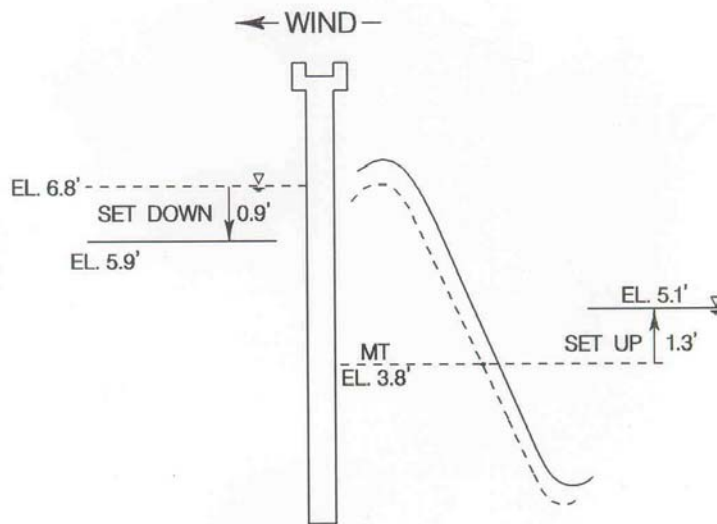
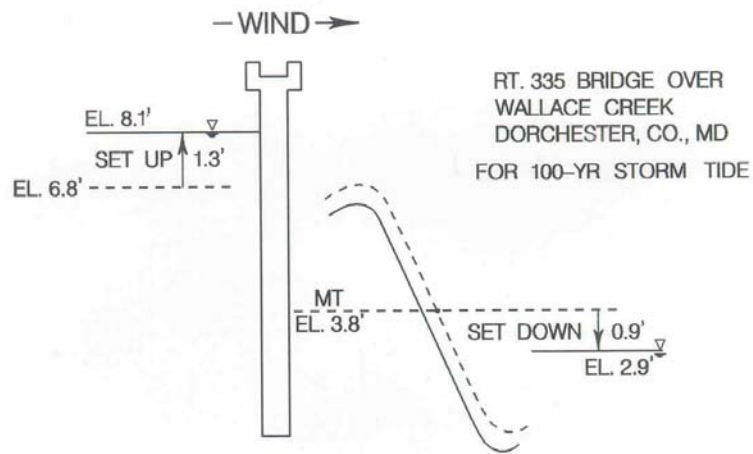
Estimate the wind setup:

Use Equation S5 in the above-mentioned manual

$$\begin{aligned}\text{Total setup } S &= 0.00117 * (F * \cos\theta) / D * V^2 \\ &= 0.00117 * (0.95 \cos 0) / 3 * 77^2 = 2.2 \text{ ft}\end{aligned}$$

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) add 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).



STARTING BASIN HEADWATER  
ELEVATION

MEAN TIDE  
ELEVATION

FIGURE 3. WIND SET UP AND WIND SET DOWN

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

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 6.8 NAVD for a tide elevation of 8.1
4. Assume that the mean tide elevation input on the project data card was previously computed as one-half of the storm tide elevation (For the Wallace Creek example, this value is 3.8 feet NAVD)
5. Compute the setback for the fetch on the downwind side of the bridge.
6. Subtract the setback 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 setback 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 setback value.
  - Modified mean tide elevation =  $3.8 - 0.0 = 3.8$
  - For a bridge over an estuary some distance from the bay, it may be reasonable to assume that the downwind estuary will have a setback that affects the tidal flow at the bridge. As an example, assume that the setup/setback of this downwind estuary is the same as the setup/setback for the upwind estuary calculated above or 1.8 feet. Using a judgment that the setup and setback is evenly distributed, the setback at the bridge would be estimated as  $1.8/2$  or 0.9 feet. The adjusted mean tide elevation (MEL) is calculated as  $3.8 - 0.9 = 2.9$  feet
7. Run the program using these modified values and indicate that the analysis incorporates wind setup and setback

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

```

Input Data:

Unit: English Units
Analysis starting time (hr.): 0
Analysis ending time (hr.): 24
Time step (hr.): .2
Starting bridge headwater elevation (ft): 8.1
Tidal amplitude (ft): 3
Mean tidal elevation (ft): 3.8
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	0.76	3000000
3	2	10600000
4	4	19000000
5	10	19100000

**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 Elevation Data**

M:\2007\CHAPTER 10 TIDAL\wallace creek with wind setup only.tid											
File Run Draw Tools Help											
Project Data		Stream Flow data		Tidal Basin Data		Bridge Opening Data		Roadway Data		Output	Graphic
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	6.800	8.100	1229.74	35573.47	9.150	19068333.3	224.00				
0.20	6.796	7.292	1022.06	20422.88	7.605	19054864.4	224.00				
0.40	6.784	7.054	667.48	5688.41	4.966	19050892.8	224.00				
0.60	6.763	6.940	510.01	2537.64	3.795	19049004.5	224.00				
0.80	6.734	6.870	426.48	1483.85	3.173	19047832.5	224.00				
1.00	6.698	6.816	384.36	1086.15	2.860	19046937.6	224.00				
1.20	6.653	6.768	368.42	956.60	2.741	19046134.3	224.00				
1.40	6.601	6.720	368.82	959.70	2.744	19045328.8	224.00				
1.60	6.541	6.668	378.58	1037.91	2.817	19044467.7	224.00				
1.80	6.473	6.611	393.04	1161.43	2.924	19043519.7	224.00				
2.00	6.398	6.548	409.34	1312.02	3.046	19042466.5	224.00				
2.20	6.316	6.478	425.87	1477.48	3.169	19041298.6	224.00				
2.40	6.227	6.401	441.80	1649.51	3.287	19040012.1	224.00				
2.60	6.131	6.316	456.74	1822.57	3.398	19038607.0	224.00				
2.80	6.029	6.225	470.56	1993.05	3.501	19037085.7	224.00				
3.00	5.921	6.127	483.24	2158.65	3.596	19035451.8	224.00				
3.20	5.807	6.023	494.84	2317.88	3.682	19033710.1	224.00				
3.40	5.688	5.912	505.43	2469.78	3.761	19031865.8	224.00				
3.60	5.563	5.795	515.06	2613.68	3.832	19029924.5	224.00				
3.80	5.434	5.674	523.80	2749.09	3.897	19027892.0	224.00				
-----Page Break-----											
7.40	2.725	3.142	666.07	2040.78	5.102	15396781.6	217.57				
7.60	2.580	3.030	675.08	1732.01	5.285	14926662.3	212.87				
7.80	2.438	2.929	688.03	1429.35	5.506	14501782.7	208.29				
8.00	2.300	2.842	705.34	1069.82	5.768	14137466.7	203.81				
8.20	2.166	2.770	727.15	718.99	6.076	13835609.4	199.46				
8.40	2.037	2.708	750.86	482.46	6.410	13574510.7	195.24				
8.60	1.912	2.652	773.47	318.03	6.743	13340479.0	191.18				
8.80	1.793	2.601	793.53	202.65	7.062	13124248.6	187.27				
9.00	1.679	2.552	810.44	121.48	7.359	12919445.0	183.54				
9.20	1.571	2.505	824.01	66.14	7.630	12721275.9	179.99				
9.40	1.469	2.459	834.25	30.64	7.872	12526069.5	176.63				
9.60	1.373	2.412	841.31	10.31	8.083	12331027.6	173.46				
9.80	1.284	2.365	845.37	1.40	8.263	12134058.7	170.51				
10.00	1.202	2.318	846.67	0.00	8.411	11933865.1	167.77				
10.20	1.127	2.269	845.48	0.00	8.527	11730561.9	165.26				
10.40	1.059	2.220	842.10	0.00	8.611	11524560.3	162.98				
10.60	0.999	2.171	836.77	0.00	8.666	11316231.8	160.94				
10.80	0.947	2.120	829.69	0.00	8.689	11105924.5	159.14				
11.00	0.902	2.070	820.98	0.00	8.683	10893974.9	157.58				
11.20	0.866	2.019	810.72	0.00	8.646	10680722.1	156.28				

**Figure 5 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.)**

## Discussion

For this particular comparison of the tidal flow at Wallace Creek for conditions of wind and no wind, 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

### Scour Analysis for tidal flow through the Route 335 bridge for the conditions of Example 2 assuming a wind setup:

Refer to Chapter 11 for guidance on estimating scour depths. The following example is provided only to present a method to obtain the hydraulic variables necessary for the scour estimate.

Assume that the scour in this sand channel will be clear water scour and the average particle size is  $D_{50} = 0.1$  mm (sand/silt). For a flow depth of 5 ft., the critical velocity obtained from Neill's curve is  $V_c = 2.0$  ft/s. Top width of the bridge,  $T = 32$  feet

For Wallace Creek with a wind setup of 1.3 feet, the TIDEROUT 2 output indicates a maximum discharge = 829.7 cfs at time 10.8 hrs; flow area = 159.14 sq. ft. (Figure 5)

The average velocity through the bridge  $V = Q/A = 829.7/159.14 = 5.21$  fps.

Hydraulic flow depth before scour ( $y_0$ ) =  $A/T = 159.14/32 = 5.0$  feet

Unit discharge before scour  $q = V y_0 = 26.01$  cfs/ft

And

Unit discharge after scour =  $V_c y_2$ , where  $V_c$  = critical velocity and  $y_2$  = total flow depth after scour.

Solving for  $y_2$ :

$V y_0 = V_c y_2$  and  $y_2 = V y_0 / V_c = 5.21 * 5 / 2 = 13$ . ft.

Contraction scour depth =  $y_2 - y_0 = 13 - 5 = 8$  feet.

Computations for abutment scour should be made following the guidance set forth in the ABSCOUR Users Manual in Chapter 11 of the OBD H&H Manual

## 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

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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)

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

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

$T_s$  = wind shear stress, lb/ ft<sup>2</sup>

$T_B$  = bottom shear stress, lb/ ft<sup>2</sup>

$d$  = water depth, ft

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

$$T_s + T_B = 1.25 T_s \quad \text{(III.2)}$$

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

$$T_s = 1.4 \times 10^{-6} V_{30}^{2.22} \quad \text{(III.3)}$$

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:

$$\frac{dz}{dx} = 2.8x 10^{-8} \frac{V_{30}^{2.22}}{d} \quad \text{(III.4)}$$

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  $\Delta z$  is wind setup within a subsection  $\Delta x$ . With this conversion, Equation III.4 can be solved for  $\Delta z$  by assigning the values of  $\Delta x$  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:

$$Z_w = 0.19 H_b \left( 1 - 2.82 \sqrt{\frac{H_b}{gT^2}} \right) \quad \text{(III.5)}$$

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. Hydraulics of Flow in Knapp's Narrows

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:

	<u>For 80 mph wind</u>	<u>For 110 mph wind</u>
Wind Setup	2.27 ft	4.25 ft
Wind Setdown	1.30	2.60
Wave Setup	<u>0.60</u>	<u>0.70</u>
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.

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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

At Elevation	Surface area, in Billion square Feet		
	-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:

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

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

$A_s$  = surface area of tidal prism, ft<sup>2</sup>.

$A_c$  = channel cross section, ft<sup>2</sup>.

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)

**Step 3.** 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.

#### **Step 4.** 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:

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

where  $V$  = Flow velocity, ft/s

$C$  = Velocity Coefficient

$g = 32.2$  ft/sec<sup>2</sup>

$h$  = 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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