Bridge Scour in the Coastal Regions

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Bridge scour and waterway instability in the coastal region where waterways are subjected to tidal flow can be subjected to mass density stratification, water salinity, sedimentation (littoral drift or riverine transport), and unsteady reversible flows from astronomical tides and storm surges, as well as riverine flows. Nevertheless, bridge foundation scour depths can be determined and waterway instability can be countered by using existing scour equations and geomorphology techniques. A major difference for nontidal (riverine) streams is that the design discharge (50-, 100-, or 500-year return period flows) has a constant value, whereas with tidal waterways the design discharge for the same return periods may increase because it is dependent on the design storm surge elevation, the volume of water in the tidal prism upstream of the bridge, and the area of the waterway under the bridge at mean tide. If there is erosion of the waterway from the constant daily flow from the astronomical tides that increase the area of the waterway, the discharges can increase. An existing clear-water scour equation can be used to predict the magnitude of this scour, but not its time history. Recent experience indicates that this long-term degradation can be as large as 0.2 to 0.9 m/year.

Scour (erosion) of the foundations of bridges over tidal waterways in the coastal region that are subjected to the effects of astronomical tides and storm surges is a combination of long-term degradation, contraction scour, local scour, and waterway instability. These are the same scour mechanisms that affect nontidal (riverine) streams. Although many of the flow conditions are different in tidal waterways, the equations used to determine riverine scour are applicable if the hydraulic forces are carefully evaluated.

Bridge scour in the coastal region results from the unsteady diurnal and semidiurnal flows resulting from astronomical tides, large flows that can result from storm surges (hurricanes, northeasters, and tsunamis), and the combination of riverine and tidal flows. Also, the small size of the bed material (normally fine sand) as well as silts and clays with cohesion and littoral drift (transport of beach sand along the coast resulting from wave action) affect the magnitude of bridge scour. Mass density stratification and water salinity have a minor effect on bridge scour. The hydraulic variables (discharge, velocity, and depths) and bridge scour in the coastal region can be determined with as much precision as riverine flows. These determinations are conservative, and research is needed for both cases to improve scour determinations. Determining the magnitude of the combined flows can be accomplished by simply adding riverine flood flow to the maximum tidal flow, if the drainage basin is small, or routing the design riverine flows to the crossing and adding them to the storm surge flows.

Although tidal flows are unsteady, peak flows from storm surges have durations long enough that the time is sufficient for fine sand in most coastal zones to reach scour depths determined from existing scour equations.

Astronomical tides, with their daily or twice-daily inflows and outflows, can and do cause long-term degradation if there is no source of sediment except at the crossing. This has resulted in long-term degradation of 0.2 to 0.9 m (0.8 to 3.0 ft) per year with no indication of stopping (*1,2*). The Indian River inlet in Delaware went from a depth of 3.7 m (12 ft) in 1938 to 15.8 m (52 ft) in 1986 (*1*).

Mass density stratification (saltwater wedges), which can result when the denser, more saline ocean water enters an estuary or tidal inlet with significant freshwater inflow, can result in larger velocities near the bottom than the average velocity in the vertical. With careful evaluation the correct velocity can be determined for use in the scour equations. With storm surges, mass density stratification will not normally occur. The density difference between saltwater and freshwater, except as it causes saltwater wedges, is not significant enough to affect scour equations. Density and viscosity differences between freshwater and sediment-laden water can be much larger in riverine flows than their differences between saltwater and freshwater. Salinity can affect the transport of silts and clays by causing them to flocculate and possibly deposit, which may affect stream stability and which must be evaluated. Salinity may affect the erodibility of cohesive sediments, but this will affect only the rate of scour, not the ultimate scour. Littoral drift is a source of sediment to a tidal waterway (*3*), and its availability can decrease contraction and possible local scour and may result in a stable or aggrading waterway. The lack of sediment from littoral drift can increase long-term degradation, contraction scour, and local scour. Evaluating the effect of littoral drift is a sediment transport problem involving historical information, future plans (dredging, jetties, etc.) for the waterway or the coast, sources of sediment, and other factors.

One major difference exists between riverine scour at highway structures and scour resulting from tidal forces. In determining scour depths of riverine conditions, a design discharge is used (discharge associated with a 50-, 100-, and 500-year return period). For tidal conditions a design storm surge elevation is used, and from that the discharge is determined. That is, for the riverine case the discharge is fixed, whereas for the tidal case the discharge may not be. In the riverine case, as the area of the stream increases the velocity and shear stress on the bed decrease because of the fixed discharge. In the tidal case, as the area of the waterway increases the discharge may also increase and the velocity and shear stress on the bed may not decrease appreciably. Thus, long-term degradation and contraction scour can continue until sediment inflow equals sediment outflow or the discharge driving force (difference in elevation across a highway crossing an inlet, estuary, or channel between islands or islands and the mainland) reduces to a value that the discharge no longer increases (*4,5*). Hydraulic Engineering Circular 18 (HEC-18) (*4*) and Richardson et al. (*5*) present a method for determining the potential long-term degradation, but not its time history, when the sediment supply is cut off or decreased at an inlet.

An overview of tidal hydraulics, a three-level method of scour analysis for tidal waterways, and scour equations for determining scour depths as given by FHWA is presented in the following section (*4*). Level 1 is a qualitative evaluation of the stability of a tidal waterway, estimating the magnitude of the tides, storm surges, littoral drift, and flow in the tidal waterway and attempting to determine whether the hydraulic analysis depends on tidal or river conditions, or both. Level 2 represents the engineering analysis necessary to obtain the velocity, depths, and discharge for tidal waterways to be used in determining long-term aggradation or degradation, contraction scour, and local scour by using existing scour equations. Level 3 analysis is for complex tidal situations that require physical or one- or two-dimensional computer models.

Overview of Tidal Processes

Glossary

Bay: a body of water connected to the ocean with an inlet.

Estuary: tidal reach at the mouth of a river.

Flood or flood tide: flow of water from the ocean into the bay or estuary.

Ebb or ebb tide: flow of water from the bay or estuary to the ocean.

Littoral drift: transport of beach material along a shoreline by wave action.

Run-up: height to which water rises above still-water level when waves meet a beach or wall.

Storm surge (hurricane surge, storm tide): tidelike phenomenon resulting from wind and barometric pressure changes.

Tidal amplitude: generally, half of tidal range.

Tidal cycle: one complete rise and fall of the tide.

Tidal inlet: a channel connecting a bay or estuary to the ocean.

Tidal passage: a tidal channel connected with the ocean at both ends.

Tidal period: duration of one complete tidal cycle.

Tidal prism: volume of water contained in a tidal bay, inlet, or estuary between low and high tide levels.

Tidal range: vertical distance between specified low and high tide levels.

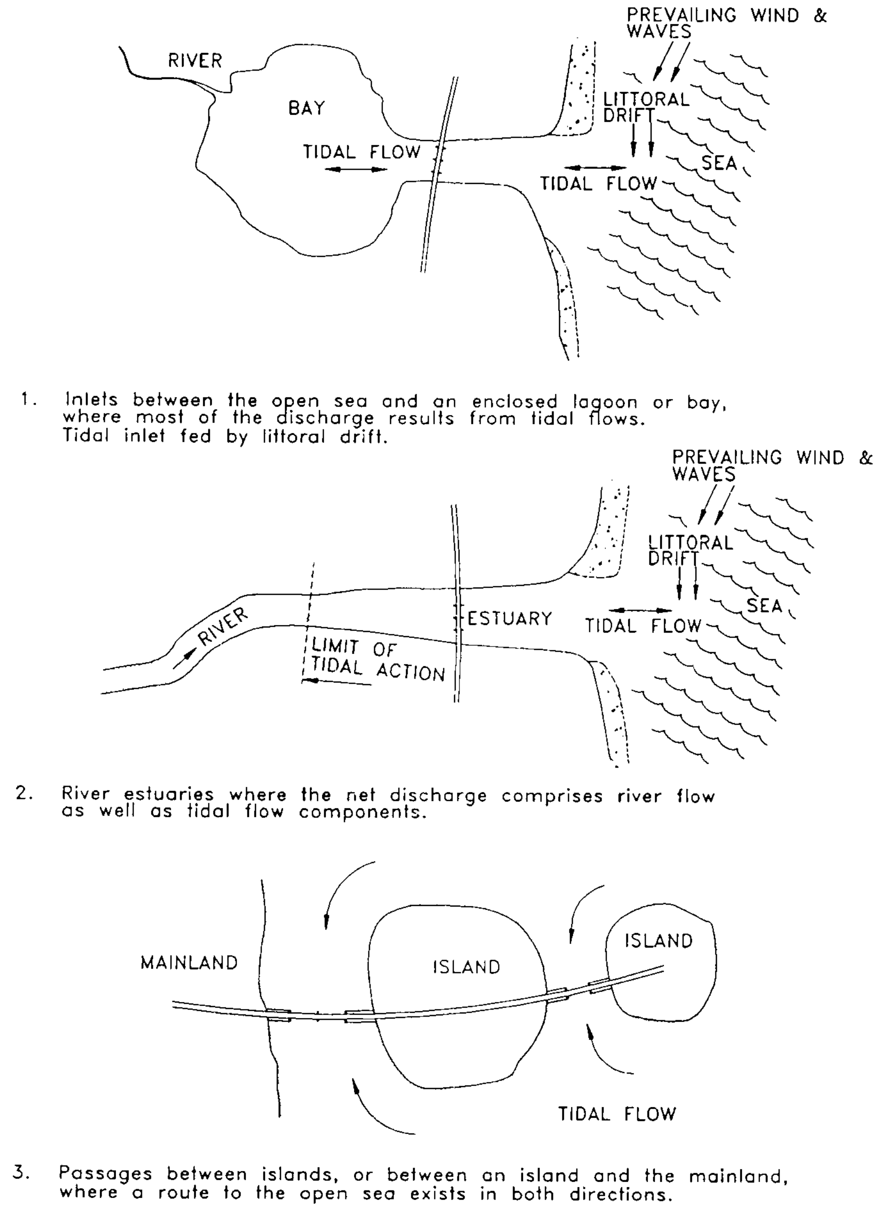


FIGURE 1 Types of tidal waterway crossings (6).

Tidal waterways: a generic term that includes tidal inlets, estuaries, bridge crossings to islands or between islands, crossings between bays, tidally affected streams, and so forth.

Tides, astronomical: rhythmic diurnal or semidiurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating earth.

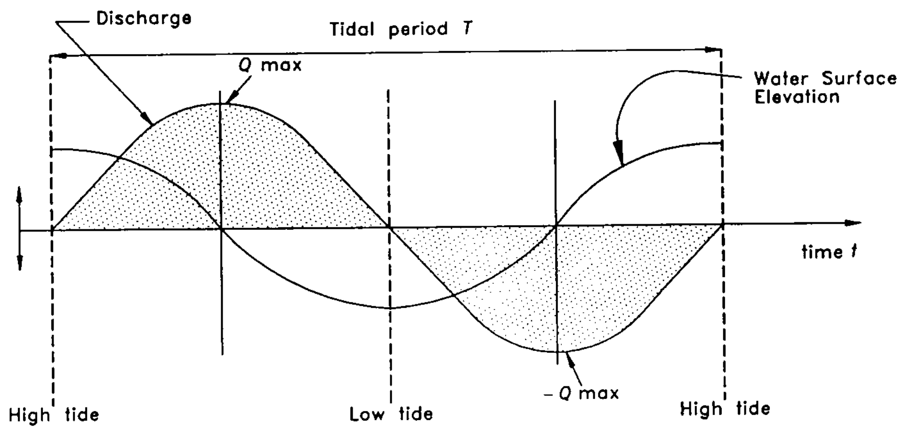
Tsunami: long-period ocean wave resulting from earthquake, other seismic disturbances, or submarine landslides.

Waterway opening: width or area of bridge opening at a specific elevation, measured normal to principal direction of flow.

Wave period: time interval between arrivals of successive wave crests at a point.

Definition of Tidal and Coastal Processes

Typical bridge crossings of tidal waterways are diagrammed in Figure 1 (6). Tidal flows are defined as being between the ocean and a bay (or lagoon), from the ocean into an estuary, or through passages between islands or islands and the mainland. Idealized astronomical tidal conditions and tidal terms are illustrated in Figure 2 (6).



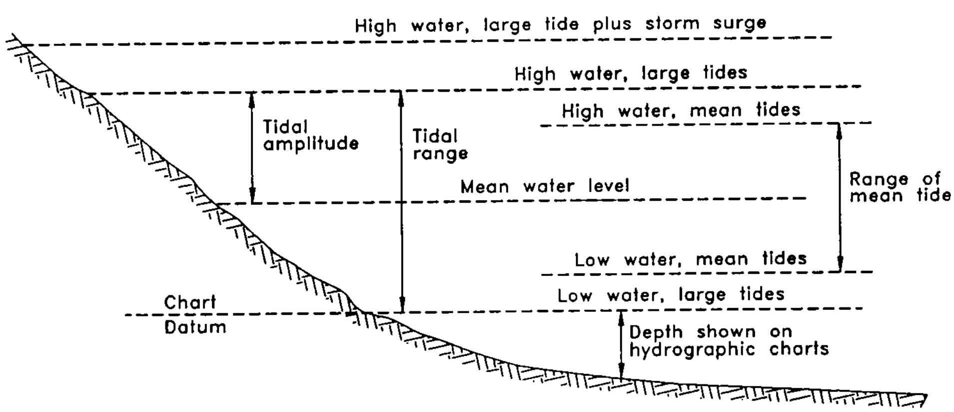


FIGURE 2 Principal tidal terms (6).

The forces that drive tidal fluctuations are primarily the result of the gravitational attraction of the sun and moon on the rotating earth (astronomical tides), wind and storm setup or seiching (storm surges), and geologic disturbances (tsunamis). As illustrated in Figure 2 the maximum discharge (Q_{max}) at the flood or ebb tide occurs often (but not always) at the crossing from high to low or low to high tide. The continuous rise and fall of astronomical tides will usually influence long-term trends of aggradation and degradation. Conversely, when storm surges or tsunamis occur, the short-term contraction and local scour can be significant. These storm surges and tsunamis are infrequent events with much longer tidal periods, elevations, and discharges than astronomical tides. Storm surges and tsunamis are a single-event phenomenon that, because of their magnitudes, can cause significant scour at a bridge crossing.

Although the hydraulics of flow for tidal waterways is complicated by the presence of two-directional flow, the basic concept of sediment continuity is valid. Consequently, a clear understanding of the principle of sediment continuity is essential for evaluating scour at bridges spanning waterways influenced by tidal fluctuations. The sediment continuity concept states that the sediment inflow minus the sediment outflow equals the time rate of change of sediment volume in a given reach.

In addition to sediments from upland areas, littoral drift (Figure 3) is a source of sediment supply to the inlet, bay or estuary, or tidal passage. During the flood tide these sediments can be transported and deposited into the bay or estuary. During the ebb tide these sediments can be remobilized and transported out of the inlet or estuary and can be either deposited on shoals or moved farther down the coast as littoral drift.

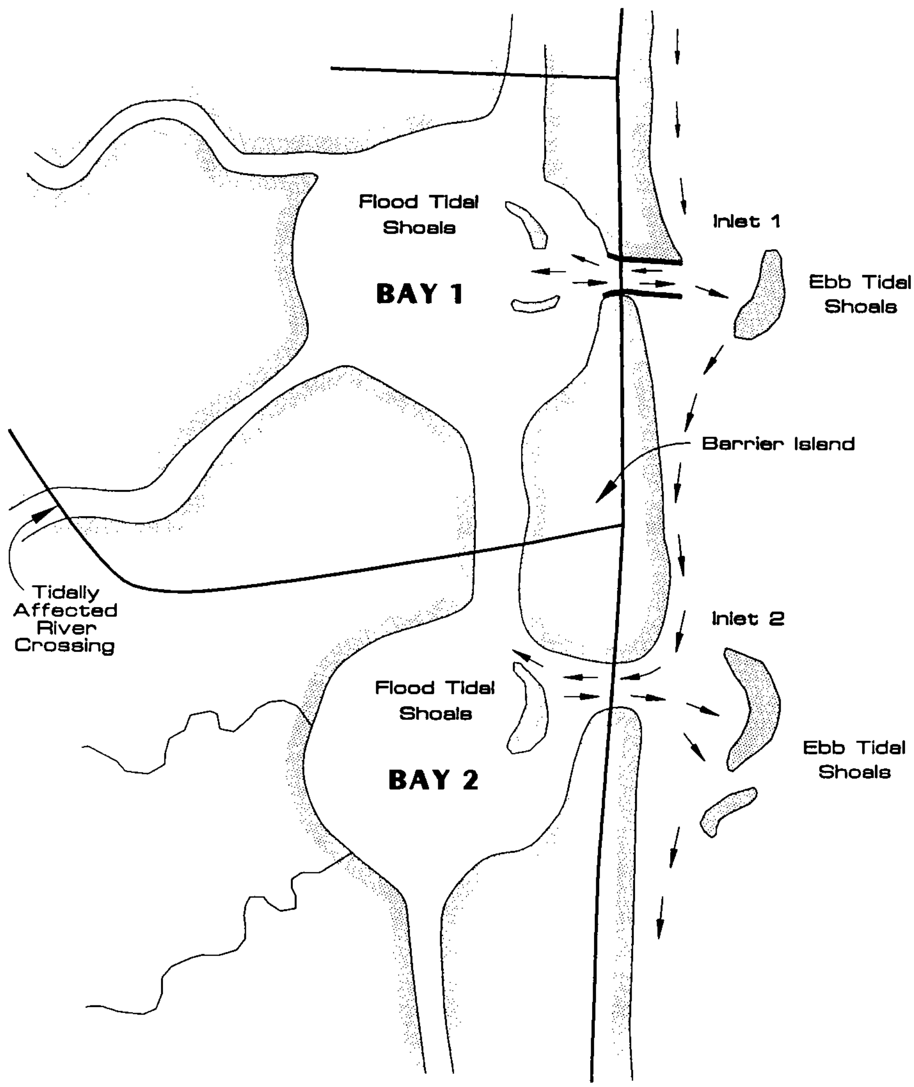


FIGURE 3 **Sediment transport in tidal inlets (3).**

Sediment transported to the bay or estuary from the upland river system can also be deposited in the bay or estuary during the flood tide and can be remobilized and transported through the inlet or estuary during the ebb tide. However, if the bay or estuary is large, sediments derived from the upland river system can deposit in the bay or estuary in areas where the velocities are low and may not contribute to the supply of sediment to the bridge crossing. The result is clear-water scour unless sediment transported on the flood tide (ocean shoals, littoral drift) is available on the ebb. Sediments transported from upland rivers into an estuary may be stored there on the floor and transported out during ebb tide. This would produce live-bed scour conditions unless the sediment source in the estuary was disrupted. Dredging, jetties, or other coastal engineering activities can limit sediment supply to the reach and influence live-bed and clear-water scour conditions.

A net loss of sediment discharge into the tidal waterway could be the result of cutting off littoral drift by means of a jetty projecting into the ocean (Figure 3) or dredging. Because the availability of sediment for transport into the bay or estuary is reduced, highway crossing degradation could result. As discussed earlier, as the cross-sectional area of the crossing increases, the flow velocities during the ebb and flood tides may not decrease, resulting in further degradation of the inlet.

Level 1 Analysis

Level 1 analysis is the qualitative determination of the (a) classification of the tidal crossing, (b) tidal characteristics, (c) lateral, vertical, and overall stabilities of the waterway and bridge foundations, and (d) characteristics of the riverine and tidal flows. Design plans; boring logs; inspection and maintenance reports; fluvial geomorphology; historical flood, scour, and tidal information; 100- and 500-year return period storm surge elevations; riverine flows; and so forth are collected and analyzed. Field reconnaissance, office review, and contact with relevant agencies such as the Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), U.S. Geological Survey, U.S. Coast Guard, U.S. Corps of Engineers (USCOE), state agencies, and so forth are used.

The crossing is classified as an inlet, bay, estuary, or passage between islands or islands and the mainland (Figure 1). The crossing may be tidally affected or tidally controlled. Tidally affected crossings do not have flow reversal, but the tides act as a downstream control. Tidally controlled crossings have flow reversal. The limiting case for a tidally affected crossing is when the magnitude of the tide is large enough to reduce the discharge through the bridge to zero.

The objectives of a Level 1 analysis are to determine the magnitudes of the tidal effects on the crossing, the overall long-term vertical and lateral stabilities of the waterway and bridge crossing, and the potential for the waterway and the crossing to change.

Level 2 Analysis

Level 2 analysis is the basic engineering assessment of scour and stream stability at an existing bridge for the design of a new or replacement bridge or the design of countermeasures for waterway instability or bridge scour. The general procedure is to determine (a) design flows (100- and 500-year storm tides and riverine floods) and (b) hydraulic characteristics (discharge, velocity, and depths) and scour components (depths of degradation, contraction scour, pier scour, and abutment scour) and (c) to evaluate the results.

Design Flows and Hydraulic Variables

The riverine 100- and 500-year return period storm discharge is determined by standard hydrology frequency analysis procedures. The magnitude of the 100- and 500-year return period discharge for a tidal surge depends on the elevation of the surge at the crossing, the volume of water in the tidal prism above the crossing, the area of the bridge waterway at the water surface elevation between high and low tides (ebb) or low and high tides (flood), and the tidal period (time between successive high or low tides).

The elevation of the 100- and 500-year storm surge, tidal period, and surge hydrographs for storm surges can be obtained from FEMA, NOAA, and USCOE. From this information the volume of the tidal prism above the crossing, the area of the waterway at the bridge and the elevation of the crossing between high and low tides, the design storm surge discharges, and hydraulic variables for use in the scour equations can be determined for an unconstricted waterway by a method given by Neill (6) and for a constricted waterway by a method given by Chang et al. (7).

Design Flows and Hydraulic Variables for Unconstricted Waterways

FHWA's HEC-18 (4) presents an example problem by Neill's method (6). The steps are as follows:

Step 1. Determine and plot the net waterway area at the crossing as a function of elevation. Net area is the gross waterway area between abutments minus the area of the piers.

Step 2. Determine and plot tidal prism volumes as a function of elevation. The tidal prism is the volume of water between low- and high-tide levels or between the high-tide elevation and the bottom of the tidal waterway.

Step 3. Determine the elevation-versus-time relation for the 100- and 500-year storm tides. The relation can be approximated by a sine curve, which starts at the mean water level, or a cosine curve, which starts at the maximum tide level. The cosine equation is

$$y = A \cos \theta + Z \tag{1}$$

where

y = amplitude or elevation of the tide above mean water level (m) at time t;
A = maximum amplitude of the tide or storm surge (m), defined as half the tidal range or half the height of the storm surge; and
θ = angle subdividing the tidal cycle (degrees); one tidal cycle is equal to 360 degrees.

$$\theta = 360 \left(\frac{t}{T}\right)$$

where

t = time from beginning of total cycle (min),
T = total time for one complete tidal cycle (min), and
Z = vertical offset to datum (m).

To determine the elevation-versus-time relation for the 100- and 500-year storm tides, the tidal range and period must be known. FEMA, USCOE, NOAA, and other federal or state agencies compile records that can be used to estimate the 100- and 500-year storm surge elevation, mean sea level elevation, low-tide elevation, and time period.

Tides, and in particular storm tides, may have periods different from those of astronomical semidiurnal and diurnal tides, which have periods of approximately 12.5 and 25 hr, respectively. This is because storm tides are influenced by factors other than the gravitational forces of the sun, moon, and other celestial bodies. Factors such as the wind, the path of the hurricane or storm creating the storm tide, freshwater inflow, and shape of the bay or estuary influence the storm tide amplitude and period.

Step 4. Determine the discharge, velocities, and depth. The maximum discharge, in an ideal tidal estuary, may be approximated by the following equation (6):

$$Q_{max} = \frac{3.14\ VOL}{T} \tag{2}$$

where

Q_{max} = maximum discharge in the tidal cycle (m^3/sec),
VOL = volume of water in the tidal prism between high- and low-tide levels (m^3), and
T = tidal period between successive high or low tides (sec).

In the idealized case Q_{max} occurs in the estuary or bay at the mean water elevation and at a time midway between high and low tides, when the slope of the tidal energy gradient is steepest (Figure 2). In many cases in the field Q_{max} occurs 1 or 2 hr before or after the crossing, but any error caused by this is diminutive. The corresponding maximum average velocity in the waterway is

$$V_{max} = \frac{Q_{max}}{A'} \tag{3}$$

where V_{max} is the maximum average velocity in the cross section at Q_{max} (m/sec) and A' is the cross-sectional area of the waterway at mean tide elevation, halfway between high and low tides (m^2).

The average velocity must be adjusted to determine the velocities at individual piers to account for the nonuniformity of velocity in the cross section. As for inland rivers local velocities can range from 0.9 to approximately 1.7 times the average velocity, depending on whether the location in the cross section is near the bank or near the flow thalweg. The calculated velocities should be compared with any measured velocities for the bridge site or adjacent tidal waterways to evaluate the reasonableness of the results.

The discharge at any time t in the tidal cycle (Q_t) is given by

$$Q_t = Q_{max} \sin\left(360 \frac{t}{T}\right) \tag{4}$$

Step 5. Include any riverine flows. This may range from simply neglecting the riverine flow into a bay (which is so large that the riverine flow is insignificant in comparison with the tidal flows) to routing the riverine flow through the crossing.

Step 6. Evaluate the discharge, velocities, and depths that were determined in Steps 4 and 5.

Step 7. Determine scour depths for the bridge by using the values of the discharge, velocity, and depths determined from the earlier analysis.

Design Flows and Hydraulic Variables for Constricted Waterways

To determine the hydraulic variables at a constricted waterway (constricted either by the bridge or the channel) the tidal flow may be treated as orifice flow and the following equation taken from van de Kreeke (8) and Bruun (9) can be used:

$$V_{max} = C_d(2g\ \Delta H)^{1/2} \tag{5}$$

$$Q_{max} = A'\ V \tag{6}$$

where

V_{max} = maximum velocity in the inlet (m/sec),
Q_{max} = maximum discharge in the inlet (m^3/sec),
C_d = coefficient of discharge ($C_d < 1.0$),
g = acceleration due to gravity (9.81 m/sec^2),
ΔH = difference in water surface elevation between the upstream and downstream sides of a crossing or channel for the 100- and 500-year return period storm surge as well as for the normal astronomical average tides; the latter is used to determine the average normal discharge on a daily basis to determine potential long-term degradation at the crossing of a tidal waterway if it becomes unstable (4) (m), and
A' = net cross-sectional area at the crossing, at the mean water surface elevation (m^2).

The coefficient of discharge (C_d) is

$$C_d = (1/R)^{1/2} \tag{7}$$

where

$$R = K_u + K_d + \frac{2g\, n^2\, L_c}{h_c^{4/3}} \tag{8}$$

and

R = coefficient of resistance,
K_d = velocity head loss coefficient on downstream side of the waterway,
K_u = velocity head loss coefficient on upstream side of the waterway,
n = Manning's roughness coefficient,
L_c = length of the waterway or bridge opening (m), and
h_c = average depth of flow at the bridge at mean water elevation (m).

If ΔH is not known, the following method developed by Chang et al. (7), which combines the orifice equation with the continuity equation, can be used. The total flow approaching the bridge crossing at any time (t) is the sum of the riverine flow (Q) and tidal flow. The tidal flow is calculated by multiplying the surface area of the upstream tidal basin (A_s) by the drop in elevation (H_s) over the specified time ($Q_{tide} = A_s\, dH_s/dt$). This total flow approaching the bridge is set equal to the flow calculated from the orifice equation.

$$Q + A_s \frac{dH_s}{dt} = C_d A_c \sqrt{2g\, \Delta H} \tag{9}$$

where A_c is the bridge waterway cross-sectional area (m^2) and the other variables have been defined previously.

Equation 9 may be rearranged into the form of Equation 10 for the time interval $\Delta t = t_2 - t_1$, where subscripts 2 and 1 represent the end and beginning of the time interval, respectively:

$$\frac{Q_1 + Q_2}{2} + \frac{A_{S1} + A_{S2}}{2}\frac{H_{S1} - H_{S2}}{\Delta t} = C_d\left(\frac{A_{C1} + A_{C2}}{2}\right)\sqrt{2g\left(\frac{H_{S1} + H_{S2}}{2} - \frac{H_{t1} + H_{t2}}{2}\right)} \tag{10}$$

For a given initial condition, t_1, all terms with the 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. Because 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 10, which can be determined by trial and error to balance the values on the right and left sides.

Chang et al. (7) suggest the following steps for computing the flow:

Step 1. Determine the period and amplitude of the design tide(s) to establish the time rate of change of the water surface on the downstream side of the bridge.

Step 2. Determine the surface area of the tidal basin upstream of the bridge as a function of elevation by planimetering successive contour intervals and plotting the surface area versus the elevation.

Step 3. Plot bridge waterway area versus elevation.

Step 4. Determine the quantity of riverine flow that is expected to occur during passage of the storm tide through the bridge.

Step 5. Route the flows through the contracted waterway by using Equation 10 and determine the maximum velocity of flow. Chang et al. (7) and Richardson et al. (4) give an example problem using a spreadsheet and have developed a computer program to aid in using this method.

Level 3 Analysis

Level 3 analysis involves the use of physical models or computer programs for complex situations in which a Level 2 analysis appears to be inadequate. Many computer programs are available. A study of computer models (*10*) for analyzing the hydraulic conditions of tidal streams at highway structures recommended the one-dimensional unsteady flow model entitled UNET (*11*) and two two-dimensional models entitled FESWMS-2D (*12*) and the TABS/FastTABS system with RMA-2V (*13*).

Scour Calculations

By using the information and hydraulic variables developed in the Level 2 or 3 analysis, long-term degradation, contraction scour, and local scour at piers and abutments are determined. The methods and equations given in HEC-18 (*4*) are summarized in the following sections.

Long-Term Aggradation or Degradation

From a study of site conditions, fluvial geomorphology, historical data of changes in waterway bed elevation, and potential future changes in the tidal waterway or coastal conditions, determine if the waterway is aggrading or degrading. If the waterway from that study is degrading an estimate of the amount of degradation that will occur in the future is made and is added to the other scour components. Historical data sources could be maps, soundings, tide gauge records, and bridge inspection reports for the site and in the area. Determine if there are plans to construct jetties or breakwaters, dredge the channel, construct piers, and so forth that could affect waterway stability. Also, determine changes in the riverine environment, such as dams, which would change flow conditions.

Long-term degradation can occur if there is little or no sediment supply to an inlet or estuary or if it is decreased (*1,2*). HEC-18 (*4*) presents an example problem that estimates potential long-term degradation.

Contraction Scour

Contraction scour can occur at a tidal inlet, estuary, or passage between islands or islands and the mainland. It may be live-bed or clear-water scour. It would be considered live-bed scour if there is a substantial quantity of bed material moving in contact with the bed. Contraction scour can occur if the tidal waterway constricts the flow or if only the bridge constricts the flow. Also, because the discharge in a contracted tidal waterway depends on the area of the waterway for a given tidal or storm surge amplitude, the discharge will need to be recalculated after the area has increased from contraction scour.

Live-Bed or Clear-Water Scour

To determine if the flow in the tidal waterway is transporting bed material, compare the critical velocity for beginning of motion (V_c) with the mean flow velocity (V) in the tidal waterway. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed scour may exist. To calculate the critical velocity Equation 17 can be rearranged into the following:

$$V_c = \frac{K_s^{1/2}(S_s - 1)^{1/2}\, D^{1/2} y^{1/6}}{n} \tag{11}$$

By using S_s equal to 2.65 and n equal to 0.041 $D^{1/6}$, Equation 11 for critical velocity (V_c) for fine bed material ($D_{50} < 2$ mm, $K_s = 0.047$) becomes

$$V_c = 6.79\ y^{1/6} D^{1/3} \tag{12}$$

for medium coarse-bed material (2 mm $< D_{50} <$ 40 mm, $K_s = 0.03$) Equation 11 becomes

$$V_c = 5.43\ y^{1/6} D^{1/3} \tag{13}$$

and for coarse-bed material ($D_{50} > 40$ mm, $K_s = 0.02$) Equation 11 becomes

$$V_c = 4.43\ y^{1/6} D^{1/3} \tag{14}$$

where

V_c = critical velocity above which bed material of size D and smaller will be transported (m/sec),
K_s = shields parameter,
S_s = specific gravity of the bed material,
y = depth of flow (m),
D = partial size for V_c (m), and
n = Manning's n roughness coefficient.

Live-Bed Scour

To calculate live-bed contraction scour a modified Laursen's equation (*14*) is recommended in HEC-18 (*4*):

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{K_1} \tag{15}$$

$$y_s = y_2 - y_1 = \text{(average scour depth)} \tag{16}$$

where

y_1 = average depth in the upstream main channel (m),
y_2 = average depth in the contracted section (m),
W_1 = bottom width of the upstream main channel (m),
W_2 = bottom width of the main channel in the contracted section (m),
Q_1 = flow in the upstream channel transporting sediment (m³/sec),
Q_2 = flow in the contracted channel or bridge opening (m³/sec),
k_1 = exponent determined from below,
$V_* = (\tau_0/\rho)^{1/2} = (gy_1S_1)^{1/2}$, shear velocity in the upstream section (m/sec),
ω = fall velocity of bed material based on the D_{50} (m/sec),
g = acceleration of gravity (9.81 m/sec²),
S_1 = slope of energy grade line of main channel (m/m),
τ_0 = shear stress on the bed (N/m²), and
ρ = density of water (freshwater = 1,000 kg/m³).

Exponent k_1 is as follows:

TABLE 1 Correction Factor K_1 for Pier Nose Shape

	Shape of Pier Nose	K_1
(a)	Square nose	1.1
(b)	Round nose	1.0
(c)	Circular cylinder	1.0
(d)	Sharp nose	0.9
(e)	Group of cylinders	1.0

TABLE 3 Correction Factor K_4 for Armoring by D_{90} Size (Froude number ≤ 0.8)

	D_{50} (mm)	$D_{90}/D_{50} \geq$	Maximum V_1	K_4
Sand	< 2.0	--	--	1.0
Gravel	2 - 32	--	--	1.0
Gravel	32 - 64	4 - 3	$V_1 \leq 0.7\ V_c$	0.95
Cobbles	64 - 250	3 - 2	$V_1 \leq 0.8\ V_c$	0.90
	250 - 500	2 - 1	$V_1 \leq 0.8\ V_c$	0.85
	> 500	1	$V_1 \leq 0.9\ V_c$	0.80

V_c from equations 11 through 14 using the D_{90} particle size.

V_*/w	k_1	*Mode of Bed Material Transport*
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

If the bed material is moving as suspended sediment discharge or if there are large particles in the bed material, the use of the clear-water scour equation should be investigated. If the bed material is moving mostly in suspension, clear-water scour may occur, which could increase contraction scour. Large particles in the bed material may decrease the contraction scour by armoring the bed.

Clear-Water Scour

To calculate clear-water scour, the following equations based on Laursen's method (*15*) for relief bridge scour were developed:

$$y = \left[\frac{n^2 V^2}{K_s(S_s - 1)D}\right]^3 \tag{17}$$

In terms of discharge (Q) the depth (y) is

$$y = \left[\frac{n^2 Q^2}{K_s(S_s - 1)D_m W^2}\right]^{3/7} \tag{18}$$

With Manning's n given by Stricklers in metric form as n equal to 0.040 $D_m^{1/6}$, with S_s equal to 2.65, and the indicated values for Shields coefficient (K_s) for the indicated bed material size range, the equations are as follows:

For fine bed material (D_{50} < 2 mm, K_s = 0.047):

$$y = \left(\frac{0.0206\ Q^2}{D_m^{2/3} W^2}\right)^{3/7} \tag{19}$$

For medium coarse-bed material (2 mm < D_{50} < 40 mm, K_s = 0.03):

$$y = \left(\frac{0.0323\ Q^2}{D_m^{2/3} W^2}\right)^{3/7} \tag{20}$$

For coarse-bed material (D_{50} > 40 mm, K_s = 0.02):

$$y = \left(\frac{0.0485\ Q^2}{D_m^{2/3} W^2}\right)^{3/7} \tag{21}$$

TABLE 2 Increase in Equilibrium Pier Scour Depths (K_3) for Bed Condition

Bed Condition	Dune Height (m)	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	3> H > 0.6	1.1
Medium Dunes	9> H >3	1.1 to 1.2
Large Dunes	H >9	1.3

$$y_s = y - y_o = \text{(average scour depth)} \quad (22)$$

where

y = average depth in the contracted section (m),
n = Manning's roughness coefficient,
Q = discharge through the bridge associated with the width W (m^3/sec),
K_s = Shield's coefficient,
S_s = specific gravity (2.65 for quartz),
D_m = effective mean diameter of the smallest non-transportable particle in the bed material ($1.25\ D_{50}$) in the contracted section (m),
D_{50} = median diameter of bed material (m),
W = bottom width of the contracted section less pier widths (m),
y_s = depth of scour in the contracted section (m), and
y_o = original depth in the contracted section before scour (m).

For stratified bed material the depth of scour can be determined by using the appropriate clear-water scour equation sequentially with successive D_m values of the bed material layers.

Local Scour at Piers

HEC-18 (*4*) on the basis of a study by Jones (*16*), recommends the following equation for computing local live-bed and clear-water scour at piers in tidal waterways:

$$\frac{y_s}{a} = 2.0\ K_1 K_2 K_3 K_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \quad (23)$$

where

y_s = scour depth (m),
y_1 = flow depth directly upstream of the pier (m),
K_1 = correction factor for pier nose shape (Table 1),
K_2 = correction factor for angle of attack of flow, $(\cos\theta + L/a \sin\theta)^{0.65}$,
K_3 = correction factor for bed condition (Table 2),
K_4 = correction factor for bed material size (Table 3),
a = pier width (m),
L = length of pier (m),
Fr_1 = Froude number = $V_1/(gy_1)^{1/2}$,
V_1 = mean velocity of flow directly upstream of the pier (m/sec), and
θ = angle between velocity vector and pier.

Scour depths are limited to y_s/a equal to 2.4 when the Froude number is less than 0.8 and to y_s/a equal to 3.0 when the Froude number is greater than 0.8.

HEC-18 (*4*) gives methods and recommendations on determining pier scour for exposed footings, exposed pile groups, pile caps in the flow, multiple columns, pressure flow, and debris on a pier.

Abutment Scour

Abutment scour equations are based almost entirely on laboratory data, and experience has indicated that they predict excessive scour depths. This results because the equations use abutment and approach embankment length as a major variable. Richardson and Richardson (*17*) state, "The reason the equations in the literature predict excessively conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to the abutment length; whereas, in the field, this is rarely the case." Thus, "predictive abutment scour equations, based soley on the available laboratory studies are flawed" (*18*). Therefore, foundations can be designed with shallower depths than predicted by the equations when the foundations are protected with rock riprap or a guide bank placed upstream of the abutment (*4*). Design of riprap and guide banks is given in HEC-18 (*4*), and design of guide banks is given in HEC-20 (*19*).

Conclusions

Bridge scour at tidally affected waterways is very complex because of unsteady diurnal and semidiurnal flows resulting from astronomical tides, large flows from storm surges (hurricanes, northeasters, and tsunamis), mass density stratification, water salinity, sand-size bed material as well as silts and clays with cohesion, littoral drift, and the combination of riverine and tidal flows. However, by using the available methods to determine the hydraulic variables of discharge, velocity, and depth resulting from the tides and storm surges, the total scour at bridges can be calculated by using the available scour equations. A major difference between scour at a riverine highway crossing and that at a tidal bridge crossing is that the flow at a riverine crossing has a fixed discharge for a given return period, whereas a tidal bridge crossing may have an increase in the discharge for a given return period because it is based on the storm surge elevation and period, the volume of water in the tidal prism, and the cross-sectional area of the waterway opening. If the area of the waterway opening increases, the design discharge may increase for a given storm surge elevation and period. Thus, there is no (or only a small decrease in) velocity or boundary shear stress in the tidal crossing with an increase in area due to scour. In such a case an equilibrium condition between the erosional and resisting forces is not reached.

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