CHAPTER 10

Bridge Scour Evaluation

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10.1 INTRODUCTION

Scour at highway bridges is the result of the erosive action of flowing water removing bed material from around the abutments and piers that support the bridge and erosion of stream bed and bank material which the bridge crosses. The latter results from stream migration and degradation. Both stream migration and degradation (stream instability) and scour at highway bridges can cause bridge failure.

Bridge failures cost millions of dollars each year as a result of both direct costs necessary to replace and restore bridges, and indirect costs related to disruption of transportation facilities. However, of even greater consequence is loss of life from bridge failures (Richardson et al. 1989). In the United States there are over 575,000 bridges in the National Bridge Inventory. These numbers include federal highway system, state, county, and city bridges. Approximately 84% of these bridges are over water. Erosion of the foundations of the bridges resulting from stream instability, long-term degradation, contraction scour, and local scour cause 60% of bridge failures. There have been 25 fatalities from bridge failures in the United States since 1987 (Richardson and Lagasse 1999).

Chang's study for the Federal Highway Administration (Chang 1973) indicated that about $75 million was expended annually to repair flood damage to roads and bridges. Rhodes and Trent (1993) document that $1.2 billion was expended for restoration of flood damaged highway facilities during the 1980s. They state that this amount is conservative because (1) they only include the amount funded by the U.S. government, which ranges from 75 to 100% of the total restoration costs, and (2) the funds were only for disasters that are very large and do not include the hundreds of smaller events that occur every year. These costs do not include the additional indirect costs to highway users for fuel and operating costs resulting from temporary closures and detours and to the public for costs associated with higher tariffs, freight rates, additional labor costs, and time. Rhodes and Trent (1993) also demonstrate that the indirect cost (operating a vehicle over a detour and time lost traveling when a bridge fails) exceed by several times the direct cost of bridge replacement or repair.

Research efforts have developed a large body of knowledge on bridge scour, mostly from laboratory studies. This bridge scour research started in the early 1950s through Carl Izzard's efforts to have the U.S. Bureau of Public Roads (predecessor agency to the Federal Highway Administration) and the Iowa State Highway Department fund Emmett Laursen's research on bridge scour (Laursen and Toch 1956; Laursen 1958; 1960; 1963). However, field data and measurements of scour at bridges, which are necessary to better understand the problem of stream instability and scour and to evaluate analytical methods for scour prediction, are extremely limited. In addition, many of the problems of stream instability and bridge scour have not been studied in depth. Many analytical techniques are recommended for use simply because they are the best currently available, and are overly conservative. For example, many equations for determining local scour depths at bridge abutments use abutment and roadway approach length as a variable instead of the flow they intercept (Richardson and Richardson 1993; Richardson and Davis 2001). In the field case, this is a spurious correlation.

All material on the stream bed and banks at a bridge crossing will erode. It is just a matter of time. Some material, such as granite, may take hundreds of years, Whereas sand-bed streams will erode to the maximum depth of scour in hours. Sandstones, shales, and other sedimentary bed rock material do not erode in hours or days but will, over time, if subjected to the erosive force of water, erode to the extent that a bridge will be in danger unless the substructure is founded deep enough. Cohesive bed and bank material such as clays, silty clays, silts, and silty sand or material such as glacial tills, which are cemented by chemical action or compression, will erode. The erosion of these materials is slower than that of sand-bed material, may take the erosive action of several major floods, but ultimately the scour hole will be equal to the depth with a noncohesive sand-bed material (Jackson et al. 1991; Briaud et al. 1999).

Scour at bridge crossings is a sediment transport process. Long-term degradation, contraction scour, and local scour at piers and abutments result from the fact that more sediment is removed from these areas than is transported into them. If there is no transport of bed material into the bridge crossing, *clear-water* scour exists. Transport of appreciable bed material into the crossing results in *live-bed* scour. In this latter case the transport of the bed material may limit scour depth. With clear-water scour the scour depths are limited by the critical velocity or critical shear stress of a dominant size in the bed material at the crossing.

Major floods tend to scour the material at a bridge crossing during the rising limb of the flood and refill the scour holes during the recession limb. Often the redeposited material in the scour hole is more easily eroded by subsequent floods. Postflood inspection of the bridge crossing may indicate that the material around the foundations is adequate when, in fact, the bridge is in jeopardy of failing during the next flood. This infilling also makes it difficult to obtain field measurements of scour depths because the measurements have to be made during a flood.

The magnitude of the scour depth depends on the flow variables of the stream (discharge, flow velocity and depth, angle of the flow to the bridge, etc.), bed and bank material characteristics (bed rock, alluvial or nonalluvial, cohesive or noncohesive, size distribution, etc.), and bridge characteristics (size and shape of the pier and abutments, elevation of the deck, etc).

The magnitude of the flow variable depends on the selection of a design discharge. The design discharge selected for a bridge is based on the design life of the bridge, bridge importance, consequences of failure, etc. The design discharge for a divided highway with large average daily traffic (ADT) (interstate highway, autobahns, etc.) would be larger than that for a farm-to-market or logging road. Some engineers advocate a maximum possible flood for important bridges (Laursen 1998); others recommend risk analysis. Important bridges are those with large ADT, interstate highways, school bus and ambulance routs, etc.

For important highways the Federal Highway Administration in HEC 18 (Richardson and Davis 2001) recommends that bridges should be designed to resist the flood event(s) that are expected to produce the most severe scour conditions. HEC 18 recommends the 100-year flood or the overtopping flood when it is less than the 100-year flood. Overtopping refers to flow over the approach embankment(s), the bridge itself, or both. Also, investigate other flood events if there is evidence that such events would create deeper scour than the 100-year or overtopping floods. In addition, HEC 18 states, "Bridges should be designed to withstand the effects of scour from a super-flood (a flood exceeding the 100-year flood) with little risk of failing. This requires careful evaluation of the hydraulic, structural, and geotechnical aspects of bridge foundation design. It is recommended that this super-flood or check flood be on the order of a 500-year event. "The bridge design for the 100 year or overtopping flood should be designed with the normal safety factors but checking the design for the super flood is made with safety factors of 1.0." Also, "The foundation should be designed by an interdisciplinary team of engineers with expertise in hydraulic, geotechnical, and structural design."

10.2 TOTAL SCOUR

Total scour at a highway crossing is composed of long-term degradation, general scour (contraction and other general scour), and local scour. The components are assumed to be additive. In addition, lateral shifting of a stream can cause or increase the scour of bridge foundations. Each of the three types of scour and stream instability are introduced separately below.

10.2.1 Long-Term Aggradation and Degradation

Aggradation is the deposition of sediment in the bridge reach of a stream, whereas, degradation is the erosion of the sediment in the bridge reach. The former causes the bed elevation to increase and the latter causes the bed elevation to decrease. These riverbed elevation changes are over long lengths and times due to natural or man-made changes. These changes can be in controls, such as dams or bed rock, in sediment discharge, and in river form, such as from a meandering to a braided stream. Long-term degradation is defined as long-term scour and is added to the other scour components to obtain total scour, but long-term aggradation is not usually considered because over time it could stop or change to degradation.

10.2.2 General Scour

General scour is a uniform or nonuniform lowering of the waterway bed as a result of the passage of high flow. It may result from contraction of the flow (contraction scour) or flow around a bend (other general scour).

- *Contraction scour* is erosion of the stream bed under a bridge that results from the acceleration of the flow due to either a natural oraman-made contraction. It may occur during the passage of a flood, scouring during the rising stage and refilling on the falling limb of the runoff.
- *Other general scour* may result from flow around a bend, variable downstream control, or other stream changes that decrease the bed elevation.

- *General scour* is different from *long-term degradation* in that it may be cyclic and/or related to the passage of a flood.

10.2.3 Local Scour

Erosion of the stream bed around a pier or abutment as the result of the pier or abutment obstructing the flow is *local scour*. These obstructions accelerate the flow and create vortexes that remove bed material around them.

10.2.4 Lateral Shifting of the Stream

In addition to the above, lateral shifting of a stream (stream instability) may erode the approach roadway and abutments of a bridge and/or change the angle of the flow to the piers and abutments (angle of attack). This latter can increase local scour at the piers or abutments.

10.3 CLEAR-WATER AND LIVE-BED SCOUR

There are two conditions for contraction and local scour. These are clear-water and live-bed scour. Clear-water scour occurs when there is no transport of bed material in the flow upstream of the bridge. Live-bed scour occurs when there is transport of bed material from upstream of and into the bridge cross-section. However, clear-water scour may occur if the material being transported in the upstream reach or floodplain is transported in suspension through the bridge cross-section.

Typical clear-water scour situations include (1) coarse bed material streams, (2) flat gradient streams during low flow, (3) local deposits of bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation), (4) armored stream beds where the only locations with tractive forces adequate to penetrate the armor layer are at piers and/or abutments, and (5) vegetated channels where, again, the only locations where, cover is penetrated are at piers and/or abutments.

During a flood event, bridges over streams with coarse bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges, and then clear-water scour in the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (see Fig. 10-1). In fact, local clear-water scour may not reach a maximum until after several floods.

Equations given later for determining the velocity or shear stress associated with initiation of motion can be used as indicators for clear-water or live-bed scour. If the mean velocity (V) or average shear stress (τ_0) in the upstream reach is less than the critical velocity (V_c) or critical shear stress (τ_c) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be clear-water scour.

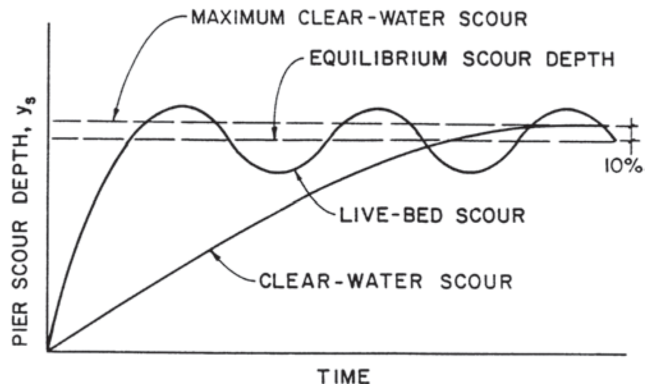


Fig. 10-1. Illustrative pier scour depth in a sand-bed stream as a function of time (not to scale) (Richardson and Davis 2001).

10.4 LONG-TERM BED ELEVATION CHANGES

Long-term bed elevation changes (aggradation or degradation) may be the natural trend of a stream or may be the result of some modification to the watershed condition of the stream. The stream bed may be aggrading, degrading, or not changing in the bridge crossing reach. When the bed of the stream is neither aggrading or degrading, it is considered to be in equilibrium with the sediment discharge supplied to the bridge reach. It is the long-term trends, not the cutting and filling of the bed of the stream that might occur with contraction scour, that must be determined. The engineer must assess the present state of the stream and watershed and determine future changes in the river system, and from this, determine the long-term stream bed elevation changes.

Factors that affect long-term bed elevation changes are dams and reservoirs upstream and downstream of a bridge, changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoff of meander bends (natural or man-made), changes in the downstream base level (control) of the bridge reach, gravel mining from the stream bed, diversion of water into or out of the stream, natural lowering of the total system, movement of a bend, bridge location in reference to stream plan form, and stream movement in relation to the crossing (Keefer et al., 1980). Richardson et al. (1990; 2001) provide examples of long-term bed elevation changes.

Analysis of long-term stream bed elevation changes must be made using the principles of river mechanics in the context of a fluvial system analysis. Such an analysis of a fluvial system requires consideration of all influences upon the bridge crossing, i.e., runoff from the watershed to the channel (hydrology), sediment delivery to the channel (erosion), sediment transport capacity of the channel (hydraulics), and response of the channel to these factors (geomorphology and river mechanics). Many of the stream impacts are from human activities, in either the past, present, or future. Analysis requires a study of the past history of the river and human activities on it; a study of present water and land use and stream control activities; and finally contacting all agencies involved with the river to determine future changes to the river system.

A method for organizing such an analysis is to use a three-level fluvial system approach. This method provides three levels of detail in an analysis, (1) qualitative determination based on general geomorphic and river mechanics relationships, (2) engineering geomorphic analysis using established qualitative and quantitative relationships to establish the probable behavior of the stream system in various scenarios of future conditions, and (3) quantifying the changes in bed elevation using available physical process mathematical models such as BRI-STARS (Molinas 1993), HEC-6 (USACE 1993), or SAMwin (Ayres Associates 2003), extrapolation of present trends, and engineering judgment to assess the result of the changes in the stream and watershed. Recent FHWA reports, such as "Stream Channel Degradation and Aggradation: Analysis of Impacts to Highway Crossings" (Brown et al. 1980), "Stream Stability at Highway Structures" (Lagasse et al. 2001a), and "River Engineering for Highway Encroachments—Highways in the River Environment" (Richardson et al. 2001) discuss methodologies to determine long-term elevation trends. Vanoni (1975) discusses degradation and aggradation in Section 21, pp. 64 and 65. The general discussion of sediment transport in Vanoni (1975) is also very useful in understanding and determining long-term degradation.

10.5 GENERAL SCOUR

10.5.1 Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or by a bridge and/or its approach embankments. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached. That is, either the quantity of bed material that is transported into the contraction is equal to that removed from the reach, ***live-bed scour,*** or the mean velocity (V) or average shear stress (τ_0) in the contraction is less than the critical velocity (V_c) or critical stress (τ_c) of the median diameter (D_{50}) of the bed material, ***clear-water scour.***

In coastal streams that are affected by tides, as the cross-sectional area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets that experience clear-water or live-bed scour, contraction scour may result in continual lowering of the bed (long-term degradation) (Richardson et al 1993; 1995; Richardson and Davis 2001).

Live-bed contraction scour is typically cyclic. That is, the bed scours during the rising stage of a runoff event and fills in the falling stage. The contraction of flow due to a bridge can be caused either by a natural decrease in the flow area of the stream channel or by abutments projecting into the channel and/or the piers blocking a large portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section and/or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments.

Other factors that can cause contraction scour are (1) ice formation or jams, (2) natural berms along the banks due to sediment deposits, (3) island or bar formations upstream or downstream of the bridge opening, (4) debris, and (5) growth of vegetation in the channel or floodplain.

10.5.2 Other General Scour

In a natural channel, the depth of flow and the velocity are always greater on the outside of a bend. In fact there may well be deposition on the inner portion of the bend at the point bar. Other general scour at a bridge located on or close to a bend will be concentrated on the outer part of the bend. Also, in bends, the thalweg (the part of the stream where the flow is deepest and, typically, the velocity is the greatest) may shift toward the center of the stream as the flow increases. This can increase scour and the nonuniform distribution of the scour in the bridge opening (chute channel).

10.5.3 Contraction Scour Equations

Contraction scour equations are based on the principle of conservation of sediment transport. In the case of live-bed scour, this simply means that the fully developed scour in the bridge cross-section reaches equilibrium when sediment transported into the contracted section equals sediment transported out and the conditions for sediment continuity are in balance. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material.

To determine if the contraction scour at a bridge is *clear-water* or *live-bed,* determine if the critical velocity (V_c) or critical shear stress (τ_c) of the median diameter (D_{50}) of the bed material in the channel upstream from the bridge opening is greater than the average velocity or shear stress (clear-water scour) or smaller (live-bed scour). Or calculate the contraction scour using both equations and take the smaller scour depth (Richardson and Davis 2001).

10.5.4 Live-Bed Contraction Scour Equation

Laursen (1958,1962) derived the following equation for live-bed contraction scour. It is based on a simplified transport function (Laursen and Toch 1956) to obtain equilibrium sediment transport in a long contraction. In short contractions, such as at a bridge, it slightly overestimates the scour depth,

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k_1}\left(\frac{n_2}{n_1}\right)^{k_2} \tag{10-1}$$

$$y_s = y_2 - y_0 = \text{(average scour depth, m, ft)}$$

where

y_1 = average depth in the upstream main channel, m, ft;
y_2 = average depth in the contracted section, m, ft;
y_0 = average depth in the contracted section before contraction scour, m, ft;
W_1 = bottom width of the upstream main channel, m, ft;
W_2 = bottom width of main channel in the contracted section, m, ft;
Q_1 = flow in the upstream channel transporting sediment, m^3 s, cms, cfs;
Q_2 = flow in the contracted channel, m^3/s, cfs (often this is equal to the total discharge unless the total flood flow is reduced by relief bridges or water overtopping the approach roadway);
n_2 = Mannings n for contracted section;
n_1 = Mannings n for upstream main channel;
k_1,k_2 = exponents determined depending on the mode of bed material transport;
V_* = $(gy_1S_1)^{1/2}$ shear velocity in the upstream section, m/s, ft/s;
ω = median fall velocity of the bed material based on the D_{50} (see Fig. 10-2);
g = acceleration of gravity (9.81 m/s^2, 32.2 ft/s^2);
S_1 = slope of energy grade line of main channel, m/m, ft/ft;
D_{50} = median diameter of the bed material, m, ft.

V_* / w	k_1	k_2	Mode of Bed Material Transport
<0.50	0.59	0.066	Mostly contact bed material
0.50–2.0	0.64	0.21	Some suspended bed material discharge
>2.0	0.69	0.37	Mostly suspended bed material discharge

The value of y_0 may be difficult to determine because of residual contraction scour from previous floods or other factors. Nevertheless, y_0 must be determined. A reasonable value can be determined by a study of the channel using cross sections and longitudinal profiles from upstream, through the bridge, and downstream.

Richardson and Davis (2001) recommend that the Manning n ratio in Eq. (10-1) be eliminated. The Manning n ratio can be significant for a condition of dune bed in the main channel and a corresponding plane bed, washed out dunes or antidunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planing out (which decreases resistance to flow and increases the velocity and the transport of bed material at the bridge). That is, Laursen's equation indicates a decrease in scour for this case, whereas in reality, there would be an increase in scour depth. In addition, in flood flows, a plane bedform will usually exist upstream and through the contracted waterway, and the values of Manning's n will be equal.

10.5.5 Clear-Water Contraction Scour Equations

Clear-water contraction scour occurs in a long contraction when (1) there is no significant bed material transport in the upstream reach into the downstream bridge reach or (2) the material being transported in the upstream reach is transported through the downstream bridge reach mostly in suspension. With clear-water contraction scour, the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_0) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain large size (D) in the bed material. The width (W) of the contracted section is constrained and the depth (y) increases until the limiting conditions are reached.

Following a development proposed by Laursen (1963), Richardson and Davis (2001) developed the following equation for determining the clear-water contraction scour in a long contraction:

$$\tau_0 = \tau_c \tag{10-2}$$

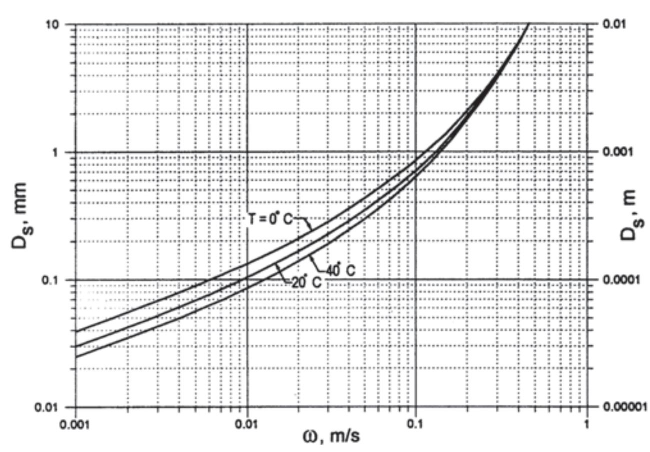


Fig. 10-2. Fall velocity of sand-sized particles.

where

τ_0 = average bed shear stress, contracted section, N/m^2, lb/ft^2
τ_c = critical bed shear stress at incipient motion, N/m^2, lb/ft^2.

The average bed shear stress using y for the hydraulic radius (R) and the Manning equation to determine the slope (S_f) can be expressed as

$$\tau_0 = \gamma\, y\, S_f = \frac{\rho g V^2 n^2}{y^{1/3}} \qquad (10\text{-}3)$$

For noncohesive bed materials and for fully developed clear-water contraction scour, the critical shear stress can be determined using the Shields (Vanoni 1975) relation,

$$\tau_c = K_s\,(\rho_s - \rho) g D \qquad (10\text{-}4)$$

The bed in a long contraction scours until $\tau_0 = \tau_c$, resulting in

$$\frac{\rho g n^2 V^2}{y^{1/3}} = K_s\left(\rho_s - \rho\right) g D \qquad (10\text{-}5)$$

Solving for the depth (y) in the contracted section gives

$$y = \left[\frac{n^2 V^2}{K_s\left(S_s - 1\right) D}\right]^3 \qquad (10\text{-}6)$$

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

$$y = \left[\frac{n^2 Q^2}{K_s\left(S_s - 1\right) D\, W^2}\right]^{3/7} \qquad (10\text{-}7)$$

where

V = average velocity in the contracted section, m/s, ft/s;
Q = discharge, m^3/s or cms, *cf*s;
D = diameter of smallest nontransportable bed material particle, m, ft;
γ = the unit weight of water (9,800 N/m^3 62.4 lb/ft^3);
n = Manning roughness coefficient;
K_s = Shield's coefficient;
S_s = specific gravity (2.65 for quartz);
ρ = density of water (999 kg/m^3, 1.94 slugs/ft^3);
ρ_s = density of sediment (quartz-2,647 kg/m^3, 5.14 slugs/ft^3);
g = acceleration of gravity (9.81 m/s^2, ft/s^2).

Equation (10-7) is the basic equation for the clear-water scoured depth (y) in a long contraction. Laursen (1963), in English units, used a value of 4 for $K_s\,(S_s - 1)\gamma$ in Eq. (10-4); D_{50} for the size (D) of the smallest nonmoving particle in the bed material, and Strickler's approximation for Manning's n ($n = 0.034\, D_{50}^{1/6}$). Laursen's value for Shield's coefficient, K_s is 0.039. Froehlich (1995) gives equations for Manning's n and Shield's coefficient, taking into account size distribution of the bed material and the fact that the bed material increases in size as the section scours.

Shield's coefficient for initiation of motion ranges from 0.03 to 0.1 (Vanoni 1975). Strickler's equation for n given by Laursen, in metric units, is $n = 0.041\, D^{1/6}$. Research discussed in Richardson et al. (1990; 2001) recommends the use of the effective mean bed material size (D_m) in place of the D_{50} size. The use of D_m would also be in accordance with the work of Froehlich (1995). D_m is approximately 1.25 $D_{50.}$ Using Laursen's value for Shield's coefficient K_s of 0.039, $n = 0.04$ D$m^{1/6}$, and $S_s = 2.65$ in Eq. (10.7) results in

$$y = \left[\frac{0.025 Q^2}{D_m^{2/3}\; W^2}\right]^{3/7} \qquad (10\text{-}8)$$

$$y_s = y - y_0 \text{ (average scour depth)} \qquad (10\text{-}9)$$

where

D_m = effective mean diameter of the bed material (1.25 D_{50}) in the contracted section, m;
y_s = depth of scour in the contracted section, m;
y_0 = original depth in the contracted section before scour, m;

other variables are as previously defined.

Clear-water contraction scour equations assume homogeneous bed materials. However, with clear-water scour in stratified materials, assuming the layer with the finest D_{50} would result in the most conservative estimate of contraction scour. Alternatively, the clear-water contraction scour equations could be used sequentially for stratified bed materials.

Both the live-bed and clear-water contraction scour equations are the best that are available and should be regarded as a first level of analysis. If a more detailed analysis is warranted, a sediment transport model such as BRI-STARS (Molinas 1993) or HEC 6 (USACE 1993) could be used.

10.6 CRITICAL VELOCITY FOR MOVEMENT OF BED MATERIAL

The velocity and depth given in Eq. (10-6) are associated with initiation of motion of the indicated size (D). Rearranging Eq. (10-6) to give the critical velocity for the beginning of motion of bed material of size D results in

$$V_c = \frac{K_s^{1/2}\;(S_s - 1)^{1/2}\; D^{1/2}\; y^{1/6}}{n} \qquad (10\text{-}10)$$

Using $K_s = 0.039$, $S_s = 1.65$, and $n = 0.041\, D^{1/6}$,

$$V_c = K u y^{1/6}\; D^{1/3} \qquad (10\text{-}11)$$

where

V_c = critical velocity above which bed material of size D and smaller will be transported, m/s, ft/s;

K_s = Shield's parameter;
S_s = specific gravity of the bed material;
D = size of bed material, m, ft;
y = depth of flow, m, ft;
n = Manning's roughness coefficient;
Ku = 6.19 SI units and 11.17 English units.

Additional discussion of beginning of motion is given in Vanoni (1975, pp. 91–107 for noncohesive sediments and 107–114 for cohesive sediments).

10.7 LOCAL SCOUR

The basic mechanism causing local scour at piers or abutments is the formation of vortices at their bases (known as the horseshoe vortex at a pier, Fig. 10-3, and horizontal vortex at an abutment, Fig. 10-4). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier, called the wake vortex (Fig. 10-3). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of wake vortices diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

At abutments, in addition to the horizontal vortex that forms around and erodes their bases there is a vertical vortex that results from flow separation at the downstream side of the abutment (Fig. 10-4). This vortex erodes the approach embankment and the abutment foundations on the downstream corner and side. Thus, there are two scour problems at abutments, (1) a scour hole at the abutment base resulting from the horizontal vortex and (2) erosion of the downstream approach embankment and abutment foundation by the vertical vortex caused by the flow separation.

Factors that affect the magnitude of local scour at piers are (1) width of the pier; (2) length of the pier if skewed to flow; (3) depth and (4) velocity of the approach flow upstream of the pier; (5) size and gradation of bed material; (6) angle of attack of the approach flow; (7) shape; (8) bed configuration; (9) ice formation or jams; and (10) debris. The scour results from free surface flow unless the bridge is submerged or overtopped; then the scour results from pressure flow. The shape of many piers is complex. The piers may rest on footings or pile caps on piles. The footings or pile caps may be in the flow or at the mean water elevation by design or erosion.

Factors that affect the magnitude of local scour at abutments are (1) discharge intercepted by the abutment and returned to the main channel at the abutment (in laboratory flumes this discharge is a function of projected length of an abutment and approach roadway into the flow); (2) depth of flow; (3) velocity of flow at the upstream and downstream ends of the abutment; (4) size and gradation of bed material; (5) angle of attack of the approach flow; (6) shape;

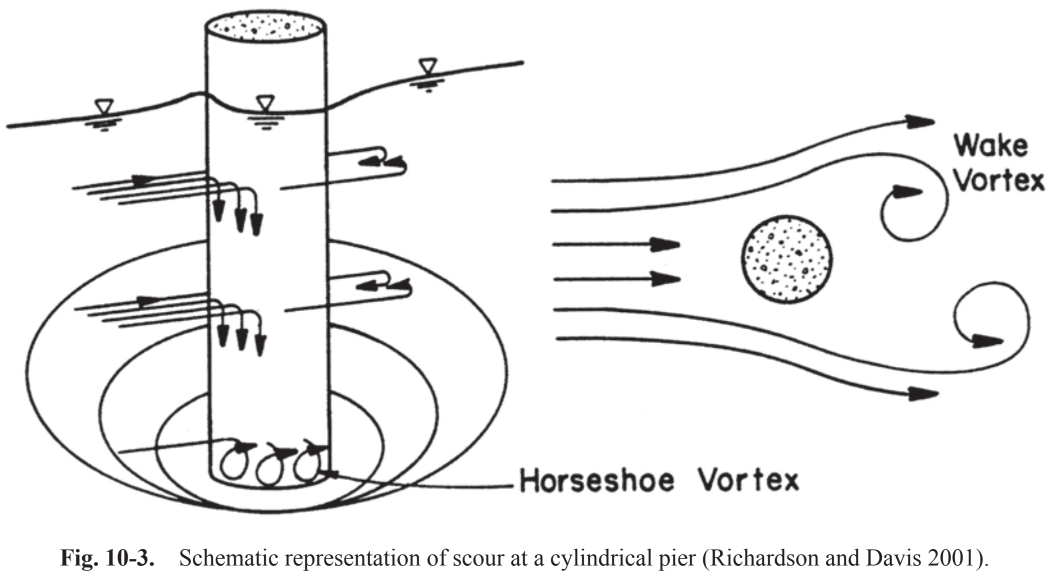


Fig. 10-3. Schematic representation of scour at a cylindrical pier (Richardson and Davis 2001).

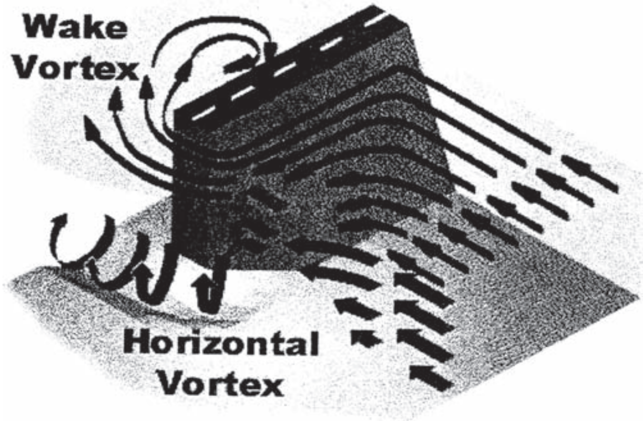


Fig. 10-4. Schematic representation of scour at an abutment.

(7) bed configuration; (8) ice formation or jams; and (9) debris. The scour results from free surface flow unless the bridge is submerged or overtopped. Then the scour results from pressure flow. As with piers, abutments may have complex shape.

10.8 LOCAL SCOUR AT PIERS

Local scour at piers has been studied extensively since the late 1940s (Loungen and Toch 1956; Laursen 1958; 1960; 1963; Richardson and Lagasse 1999). As a result of the many studies there are many equations. In general the equations are for ultimate (maximum) scour in sand beds. Jones (1983) compared the more common equations, Fig. 10-5. An equation developed by Melville and Sutherland in 1988 has been added to the figure. Many of the equations have velocity of the flow just upstream of the pier as a variable, normally in the form of a Froude number. However, some equations, such as Laursen's do not include velocity. As can be seen from Fig. 10-5, the Colorado State University (CSU) (Richardson et al. 1990) equation envelops all the points, but gives lower values of scour than Laursen's (1960), Jain and Fischer's (1979), Melville and Sutherland's (1988), and Neill's (Blench 1989) equations. Fred Chang (Richardson and Davis 2001) pointed out that Laursen's 1960 equation is essentially a special case of the CSU equation with the $F = 0.4$.

In Fig. 10-6, from flume studies in sand bed material, the ratio of scour depth to pier width (y_s / a) as a function of the ratio of approach velocity to critical velocity (V / V_c) for different-sized bed material is given. Nondimensional scour (y_s / a) starts when the mean approach velocity is approximately half of V_c the critical velocity for the beginning of motion of the bed material ($V / V_c = 0.5$) and reaches a maximum when this ratio equals 1.0. This maximum value of the nondimensional scour depth decreases with decreased bed material size. The scour that takes place from $V / V_c = 0.5$ to 1.0 is clear-water scour. For values of $V / V_c > 1.0$ the scour is live-bed. As can be seen from Fig. 10-6 after $V / V_c = 1.0$, the nondimensional scour depth decreases and then increases. The bed configuration after $V / V_c = 1.0$ in the flumes is either ripples or dunes. When the live-bed scour nondimensional depth starts to increase with an increase in V / V_c the bed configuration changes to plain bed and antidunes. The increase in nondimensional scour depth results because

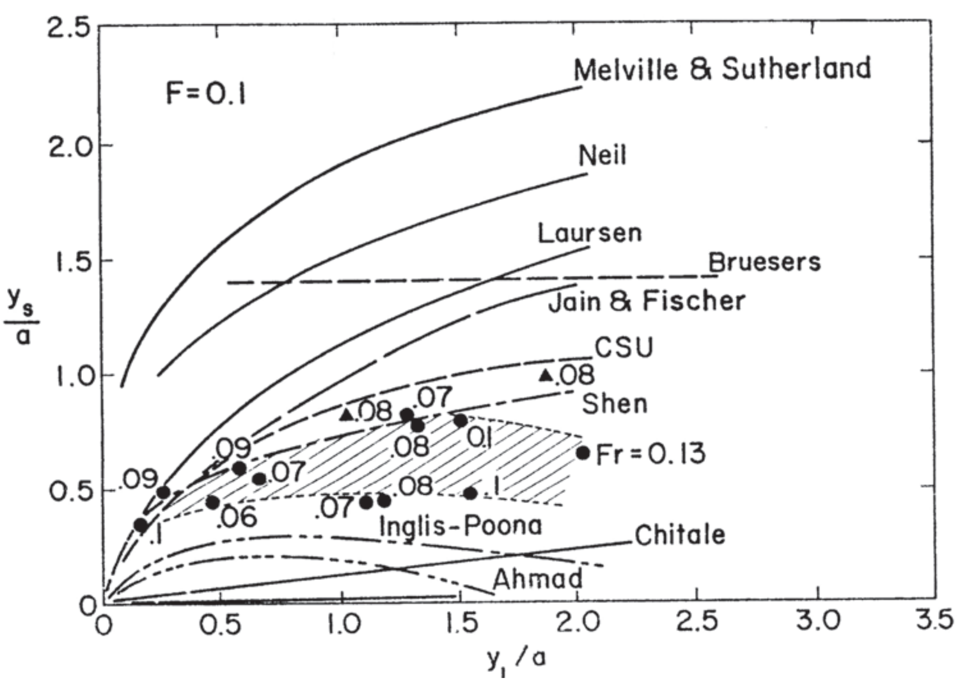


Fig. 10-5. Comparison of scour formulas for variable depth ratios (y/a) after Jones (1983).

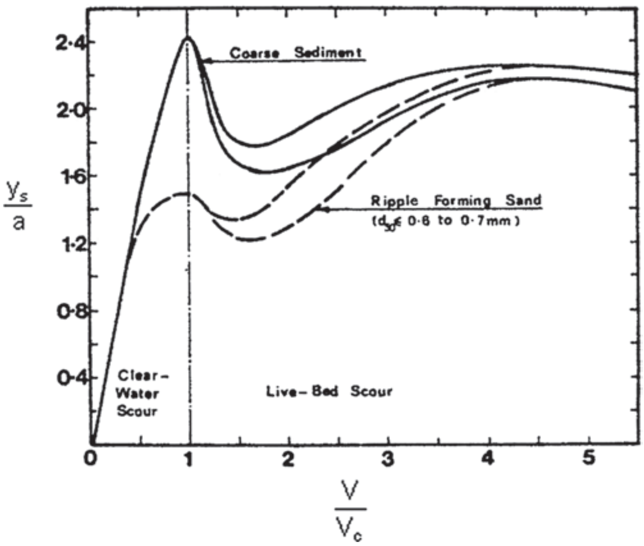


Fig. 10-6. Nondimensional local scour depth as a function of nondimensional velocity and bed material size (Melville 1984).

during plain bed and antidune flow conditions some of the sediment in transport washes through the scour hole. At high values of V/V_c the scour condition is similar to clear-water scour. That is, the bed material that is being transported upstream of the pier is swept through the scour hole and takes no part in the scouring process.

Chang (Richardson and Davis 2001) noted that in all the data he studied, there were no values of the ratio of scour depth to pier width (y_s/a) larger than 2.3. Melville and Sutherland (1988) reported 2.4 as an upper limit ratio for cylindrical piers. In these studies, the Froude number was less than 1.0. Values of y_s/a around 3.0 were obtained by Jain and Fischer (1979) for chute-and-pool flows with Froude numbers as high as 1.5. Their largest value of y_s/a for antidune flow was 2.5 with a Froude number of 1.2. These upper limits were derived for circular piers and were uncorrected for pier shape and for skew. Also, pressure flow or debris can increase the ratio.

From the above discussion, the ratio of y_s/a can be as large as 3 at large Froude numbers. Therefore, Richardson and Davis (2001) recommended that the maximum value of the ratio be taken as 2.4 for Froude numbers less than or equal to 0.8 and as 3.0 for larger Froude numbers. These limiting ratio values apply only to round nose piers that are aligned with the flow.

Over 30 equations have been developed for pier scour (Jones 1983; McIntosh 1989; Landers and Mueller 1996). In the following, three of the equations given in the literature are presented.

10.9 HEC 18 PIER SCOUR EQUATION

To determine pier scour, an equation based on the CSU equation (Richardson et al. 1990; 2001) was recommended by the Federal Highway Administration in HEC 18 (Richardson and Davis 2001) for both live-bed and clear-water pier scour. A study of 22 scour equations using field data presented by Landers et al. (1996) indicated that the HEC 18 equation was good for design because it rarely underpredicted measured scour depth, but frequently grossly overpredicted the observed scour (Mueller 1996). The data contained 384 measurements of scour at 56 bridges. The Landers and Mueller data are also given by Richardson and Lagasse (1999). The HEC 18 equation slightly underpredicted 6 of the 384 scour measurements. The maximum deviation was 3 ft when the scour depth was 25 ft (7.62m). The HEC 18 equation overestimated scour in coarse bed streams because of restrictions placed on a correction factor K_4 for coarse bed material. A K_4 factor for coarse bed material developed by Mueller (1996) decreased the overprediction without altering the underprediction.

The HEC 18 pier scour equation is

$$\frac{y_s}{y_1} = 2.0\, K_1\, K_2\, K_3\, K_4\, K_w \left(\frac{a}{y_1}\right)^{0.65} \mathsf{F}_1^{0.43} \tag{10-12}$$

In terms of y_s/a, Eq. (10-12) is

$$\frac{y_s}{a} = 2.0\, K_1\, K_2\, K_3\, K_4\, K_w \left(\frac{y_1}{a}\right)^{0.35} \mathsf{F}_1^{0.43} \tag{10-13}$$

$$\begin{aligned} y_s &\le 2.4\, a \qquad \mathsf{F} < 0.8 \\ y_s &\le 3.0\, a \qquad \mathsf{F} > 0.8 \end{aligned} \tag{10-14}$$

where

y_s = scour depth, m, ft;
y_1 = flow depth directly upstream of the pier, m, ft;
K_1 = correction factor for pier nose shape from Fig. 10-7 and Table 10-1;
K_2 = correction factor for angle of attack of flow from Eq. (10-15) or Table 10-2;
K_3 = correction factor for bed condition from Table 10-3;
K_4 = correction factor for size of bed material;
K_w = correction factor for very wide, piers;
a = pier width, m, ft;
L = length of pier, m, ft;
F_1 = Froude number = $V_1/(gy_1)^{1/2}$;
V_1 = mean velocity of flow directly upstream of the pier, m/s, ft/s.

The correction factor for angle of attack of the flow K_2 given in table 10-2 can be calculated using the equation:

$$K_2 = (\mathrm{Cos}\,\theta + L/a\ \mathrm{Sin}\,\theta)^{0.65} \tag{10-15}$$

If L/a is larger than 12, use $L/a = 12$ as a maximum in Eq. (10-15).

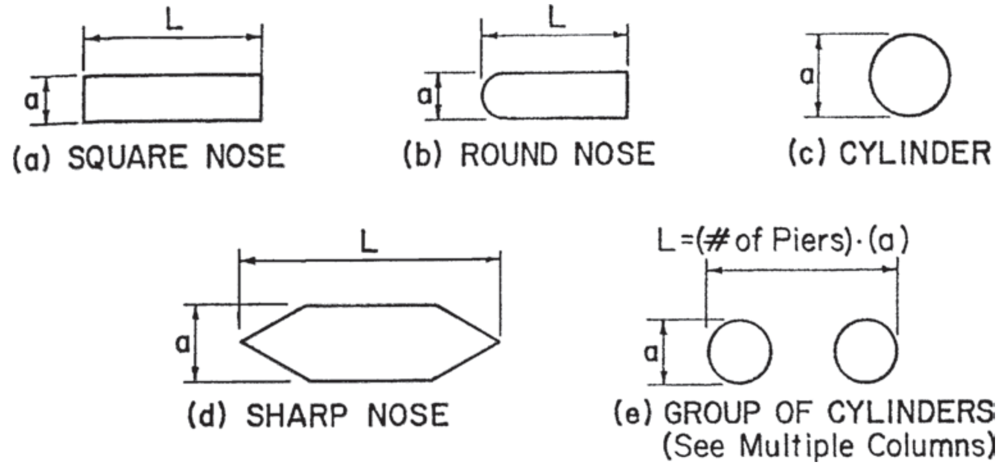


Fig 10-7. Common pier shapes (Richardson, and Davis 2001).

10.9.1 Mueller (1996) K_4 Correction Coefficient

Mueller (1996) developed a K_4 correction coefficient from a study of 384 field measurements of scour at 56 bridges. It is as follows:

$$\begin{aligned} &K_4 = 1 \text{ if } D_{50} < 2 \text{ mm or } D_{95} < 20 \text{ mm} \\ &K_4 = 0.4\left(K_5\right)^{0.15} \text{ if} \\ &D_{50} \geq 2 \text{ mm and } D_{95} \geq 20 \text{ mm} \end{aligned} \qquad (10\text{-}16)$$

where

$$K_5 = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0 \qquad (10\text{-}17)$$

V_{icDx} = the approach velocity corresponding to critical velocity for incipient scour in the accelerated flow region at the pier for the grain size D_x, m/s;

$$V_{icD_x} = 0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x} \qquad (10\text{-}18)$$

V_{cDx} = the critical velocity for incipient motion for the grain size D_x, m/s, ft/s.

Mueller (1996) used a variable Shield's parameter to define the critical velocity for incipient motion. However, for the coarser size of bed material to which K_4 is applicable, it can be determined using Eq. (10-11). It is as follows:

$$V_{cDx} = Ku\, y_1^{1/6} D_x^{1/3} \qquad (10\text{-}19)$$

y_1 = depth of flow just upstream of the pier, excluding local scour, m, ft;
V_1 = velocity of the approach flow just upstream of the pier, m/s, ft/s;
D_x = grain size for which x% of the bed material is finer, m, ft;
Ku = 6.19 SI units and 11.17 English units.

Although this K_4 provides a good fit to the field data the velocity ratio terms are so formed that if D_{50} is held constant and D_{95} increases the value of K_4 increases rather than decreases (Mueller and Jones 1999). For field data an increase in D_{95} was always accompanied by an increase in D_{50}. A minimum value for K_4 is 0.4.

10.9.2 Correction Factor for Very Wide Piers

Field and flume studies of scour depths at wide piers in shallow flows indicate that existing scour equations over estimate scour depths. Johnson and Torrico (1994) suggest the following equations for a K_w to correct for wide piers in shallow flows.

The correction factor should be used when the ratio of depth of flow to pier width is less than 0.8; the ratio of the pier width to the median diameter of the bed material is greater than 50; and the Froude number of the flow is subcritical:

$$K_w = 2.58\,(y/a)^{0.34}\,\mathsf{F}^{0.65} \quad \text{for} \quad V/V_c < 1 \qquad (10\text{-}20)$$

$$K_w = 1.0\,(y/a)^{0.13}\,\mathsf{F}^{0.25} \quad \text{for} \quad V/V_c \geq 1 \qquad (10\text{-}21)$$

Engineering judgment should be used in applying K_w because it is based on limited data.

10.9.3 Scour for Complex Pier Foundations

10.9.3.1 Introduction The piers of many bridges may not be solid single shafts as shown in Figs. 10-3 and 10-7 but may be composed of a combination of elements. In the general case, the flow could be obstructed by three

Table 10-1 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 10-2 Correction for Angle of Attack θ of the Flow

Angle	$L/a=4$	$L/a=8$	$L/a=12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Note: = Angle = skew angle of flow; L = length of pier, m, ft.

Table 10-3 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

Note: The correction factor K_1 for pier nose shape should be determined using Table 9-2 for angles of attack up to 5°. For greater angles, K_2 dominates and K_1 should be considered as 1.0. If L/a is larger than 12, use the values L/a = 12 as a maximum. The correction factor K_3 results from the fact that for plane-bed conditions, which are typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10% greater than computed with the CSU equation (Richardson et al. 1990). In the unusual situation where a dune bed configuration with large dunes exists at a site during flood flow, the maximum pier scour may be 30% greater than the predicted value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller and the maximum scour may be only 10 to 20%, greater than equilibrium scour. For antidune bed configuration the maximum scour depth may be 10% greater than the computed equilibrium pier scour depth.

substructural elements, which include the pier stem, the pile cap or footing, and the pile group. The three types of exposure to the flow may be by design or by scour (long-term degradation, general (contraction) scour, and local scour, in addition to stream migration).

Ongoing research has determined methods and equations to determine scour depths for complex pier foundations (Jones 1989; Salim and Jones 1995; 1996; 1999; Jones and Sheppard 2000). The results of this research are given in HEC 18 (Richardson and Davis 2001) and are given in the following sections. Physical model studies are still recommended for complex piers with unusual features such as staggered or unevenly spaced piles or for major bridges where conservative scour estimates are not economically acceptable (Richardson et al. 1987). However, the methods presented in this section provide a good estimate of scour for a variety of complex pier situations.

The following steps are recommended for determining the depth of scour for any combination of the three substructural elements exposed to the flow. However, engineering judgment is an essential element in applying the design graphs and equations presented in this section, as well as in deciding when a more rigorous level of evaluation is warranted. Engineering judgment should take into consideration the volume of traffic, type of traffic (school bus, ambulance, fire trucks, local road, interstate, etc.), importance of the highway, cost of a failure (potential loss of life and dollars), and increase in cost that would occur if the most conservative scour depth were used. The stability of the foundation should be checked for the following:

Determine the scour depths for the 100-year flood or smaller discharge if it causes deeper scour and the superflood, i.e., the 500-year flood, as recommended in this manual.

If needed, use computer programs such as HEC-RAS (USACE 2001), FESWMS (Froehlich 1996), or RMA2 (USACE 1997) to compute the hydraulic variables.

Determine total scour depth by separating the scour-producing components, determining the scour depth for each component and adding the results. The method is called "superposition of the scour components."

Analyze the complex pile configuration to determine the components of the pier that are exposed to the flow or will be exposed to the flow, which will cause scour.

Determine the scour depths for each component exposed to the flow using the equations and methods presented in the following sections.

Add the components to determine the total scour depths.

Plot the scour depths and analyze the results using an interdisciplinary team to determine their reliability and adequacy for the bridge, flow and site conditions, and safety and costs.

Conduct a physical model study if engineering judgment determines that it will reduce uncertainly, increase the safety of the design, and/or reduce cost.

10.9.3.2 Superposition of Scour Components Method of Analysis The components of a complex pier are illustrated in Fig. 10-8. Note that the pile cap can be above the water surface, at the water surface, in the water, or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group. In more complex pile group arrangements, this methods of analysis may give smaller or larger scour depths.

The variables illustrated in Fig. 10-8 and others used in computations are as follows:

f = distance between front edge of pile cap or footing and pier, m (ft);
h_0 = height of the pile cap above bed at beginning of computation, m (ft);
$h_1 = h_0 + T$ = height of the pier stem above the bed before scour, m (ft);
$h_2 = h_0 + y_{s\ \text{pier}} / 2$ = height of pile cap after pier stem scour component has been computed, m (ft);
$h_3 = h_0 + y_{s\ \text{pier}} / 2 + y_{s\ \text{pc}} / 2$ = height of pile group after the pier stem and pile cap scour components have been computed, m (ft);
S = spacing between columns of piles, pile center to pile center, m (ft);
T = thickness of pile cap or footing, m (ft);
V_1 = approach velocity used at the beginning of computations, m/s (ft/s);
$V_2 = V_1(y_1 / y_2)$ = adjusted velocity for pile cap computations, m/s (ft/s);
$V_3 = \mathrm{V}_1(y_1 / y_3)$ = adjusted velocity for pile group computations, m/s (ft/s).
y_1 = approach flow depth at the beginning of computations, m (ft);
$y_2 = y_1 + y_{s\ \text{pier}} / 2$ = adjusted flow depth for pile cap computations, m (ft);
$y_3 = y_1 + y_{s\ \text{pier}} / 2 + y_{s\ \text{pc}} / 2$ = adjusted flow depth for pile group computations, m ;(ft)

Total scour from superposition of components is given by

$$y_s = y_{s\,\text{pier}} + y_{s\,\text{pc}} + y_{s\,\text{pg}} \tag{10-22}$$

where

y_s = total complex pier scour depth, m (ft);
$y_{s\ \text{pier}}$ = scour component for the pier stem in the flow, m (ft);
$y_{s\ \text{pc}}$ = scour component for the pier cap or footing in the flow, m (ft);
$y_{s\ \text{pg}}$ = scour component for the piles exposed to the flow, m (ft).

Each of the scour components is computed from the basic pier scour by Eq. (10.12) using an equivalent-sized pier to represent the irregular pier components, adjusted flow depths, and velocities as described in the list of variables for Fig. 10-8 and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. In the following sections, guidance for calculating each of the components is given.

10.9.3.3. Determination of the Pier Stem Scour Depth Component The need to compute the pier stem scour depth component occurs when the pier cap or the footing is in the flow and the pier stem is subjected to sufficient flow depth and velocity to cause scour. The first computation is the scour estimate, $y_{s\ \text{pier}}$, for a full-depth pier that has the width and length of the pier stem using the basic pier equation

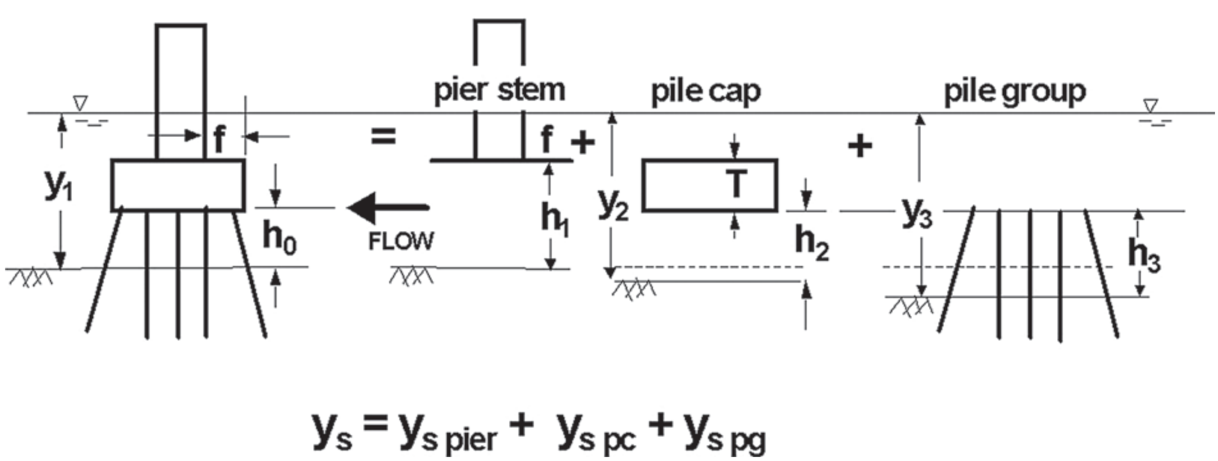


Fig. 10-8. Definition sketch for scour components for a complex pier (Richardson and Davis 2001).

(Eq. (10-12)). In Eq. (10-12), a_{pier} is the pier width and other variables in the equation are as defined previously. This base scour estimate is multiplied by $K_{h\ \text{pier}}$, given in Fig. 10-9 as a function of h_1/a_{pier} and f/a_{pier}, to yield the pier stem scour component

$$\frac{y_{s\,\text{pier}}}{y_1} = K_{h\ \text{pier}}\left[2.0\ K_1\ K_2\ K_3\ K_4\ K_w \times \left(\frac{a_{\text{pier}}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)^{0.43}\right] \tag{10-23}$$

where

$K_{\text{h pier}}$ = coefficient to account for the height of the pier stem above the bed and the shielding effect by the pile cap overhang distance f in front of the pier stem (from Fig. 10-9).

The quantity in the square brackets in Eq. (10-23) is the basic pier scour ratio as if the pier stem were full depth and extended below the scour.

10.9.3.4 Determination of the Pile Cap (Footing) Scour Depth Component The need to compute the pile cap or footing scour depth component occurs when the pile cap is in the flow by design, or as the result of long-term degradation, contraction scour, and/or by local scour attributed to the pier stem above it. As described below, there are two cases to consider in estimating the scour caused by the pile cap (or footing). Eq. (10-12) is used to estimate the scour component in both cases, but the conceptual strategy for determining the variables to be used in the equation is different (partly due to limitations in the research that has been done to date). In both cases the wide pier factor, K_w, may be applicable for this computation.

Case 1: The bottom of the pile cap is above the bed and in the flow, either by design or after the bed has been lowered by scour caused by the pier stem component. The strategy is to reduce the pile cap width, a_{pc}, to an equivalent full depth solid pier width, a^*_{pc}, using Fig. 10-10. The equivalent pier width, an adjusted flow depth, y_2, and an adjusted flow velocity, V_2, are then used in Eq. (10-12) to estimate the scour component.

Case 2: The bottom of the pile cap or footing is on or below the bed. The strategy is to treat the pile cap or exposed footing like a short pier in a shallow stream of depth equal to the height to the top of the footing above the bed. The portion of the flow that goes over the top of the pile cap or footing is ignored. Then, the full pile cap width, a_{pc}, is used in the computations, but the exposed footing height, y_f (in lieu of the flow depth), and the average velocity, V_f, in the portion of the profile approaching the footing are used in Eq. (10-12) to estimate the scour component.

An inherent assumption in this second case is that the footing is deeper than the scour depth, so it is *not necessary*

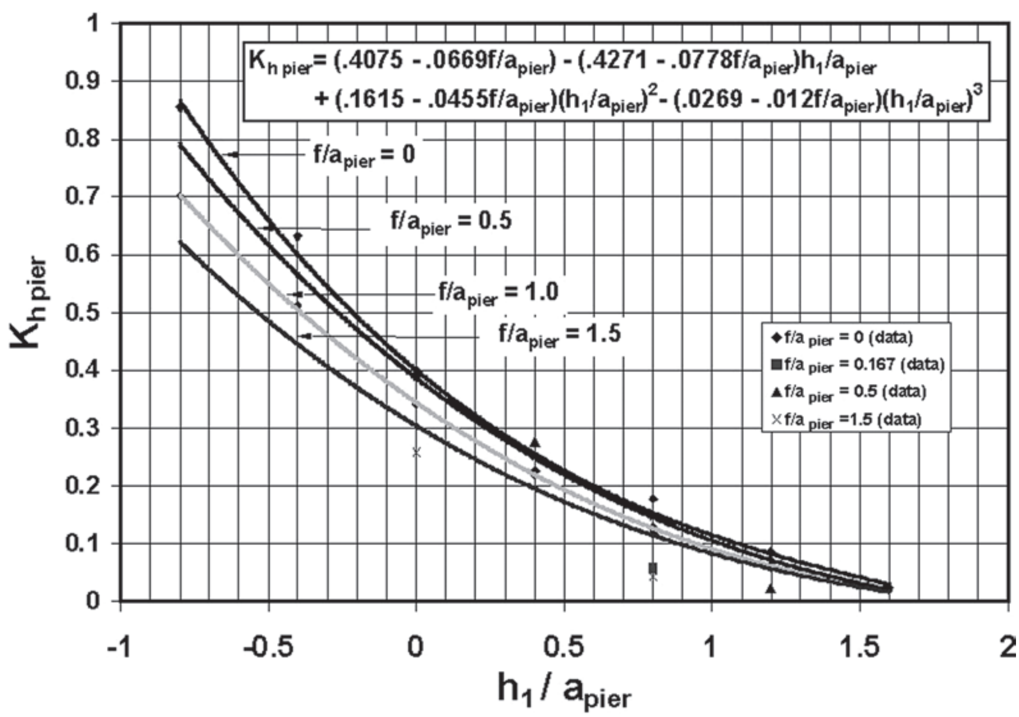


Fig. 10-9. Suspended pier scour ratio (Jones and Sheppard 2000; Richardson and Davis 2001).

to add the pile group scour as a third component in this case. If the bottom of the pile cap happened to be right on the bed, either the case 1 or case 2 method could be applied, but they would not necessarily give the same answers. If both methods are tried, then engineering judgment should dictate which one to accept.

Details for determining the pile cap or footing scour component for these two cases are described in the following paragraphs.

10.9.3.4.1 Case 1. Bottom of the Pile Cap (Footing) in the Flow above the Bed

T = thickness of the pile cap exposed to the flow, m (ft);
$h_2 = h_0 + y_{s\ pier} / 2$, m (ft);
$y_2 = y_1 + y_{s\ pier} / 2$, = adjusted flow depth, m (ft);
$V_2 = V_1 (y_1 / y_2)$ = adjusted flow velocity, m/s (ft/s),

where

h_0 = original height of the pile cap above the bed, m (ft);
y_1 = original flow depth at the beginning of the computations before scour, m (ft);
$y_{s\ pier}$ = pier stem scour depth component, m (ft);
V_1 = original approach velocity at the beginning of the computations, m/s (ft/s).

Determine a^*_{pc} / a_{pc} from Fig. 10-10 as a function of h_2 / y_2 and T / y_2 (note that the maximum value of $y_2 = 3.5\ a_{pc}$).

Compute $a^*_{pc} = (a^*_{pc} / a_{pc})\ a_{pc}$ where a^*_{pc} is the width of the equivalent pier to be used in Eq. (10-12) and a_{pc} is the width of the original pile cap. Compute the pile cap scour component, $y_{s\ pc}$, from Eq. (10-12) using a^*_{pc}, y_2, and V_2 as the pier width, flow depth, and velocity parameters, respectively. The rationale for using the adjusted velocity for this computation is that the near-bottom velocities are the primary currents that produce scour and they tend to be reduced in the local scour hole from the overlying component. For skewed flow use the L/a for the original pile cap as the L/a for the equivalent pier to determine K_2. Apply the wide pier correction factor, K_w, if (1) the total depth $y_2 < 0.8\ a^*_{pc}$, (2) the Froude number $V_2 / (g\ y_2)^{1/2} < 1$, and (3) $a^*_{pc} > 50\ D_{50}$. The scour component equation for the case 1 pile cap can then be written

$$\frac{y_{s\ pc}}{y_2} = 2.0\ K_1\ K_2\ K_3\ K_4\ K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43}. \quad (10\text{-}24)$$

Next, the pile group scour component should be computed. This is discussed later.

10.9.3.4.2 Case 2. Bottom of the Pile Cap (Footing) Located on or below the Bed One limitation of the procedure described above is that the design chart in Fig. 10-10 has not been developed for the case of the bottom of the pile cap or footing being below the bed (i.e., negative values of h_2).

As for case 1,

$$y_2 = y_1 + y_{s\ pier} / 2, \text{ m (ft)};$$
$$V_2 = V_1 (y_1 / y_2), \text{ m/s (ft/s)}.$$

The average velocity of flow at the exposed footing (V_f) is determined using the following equation

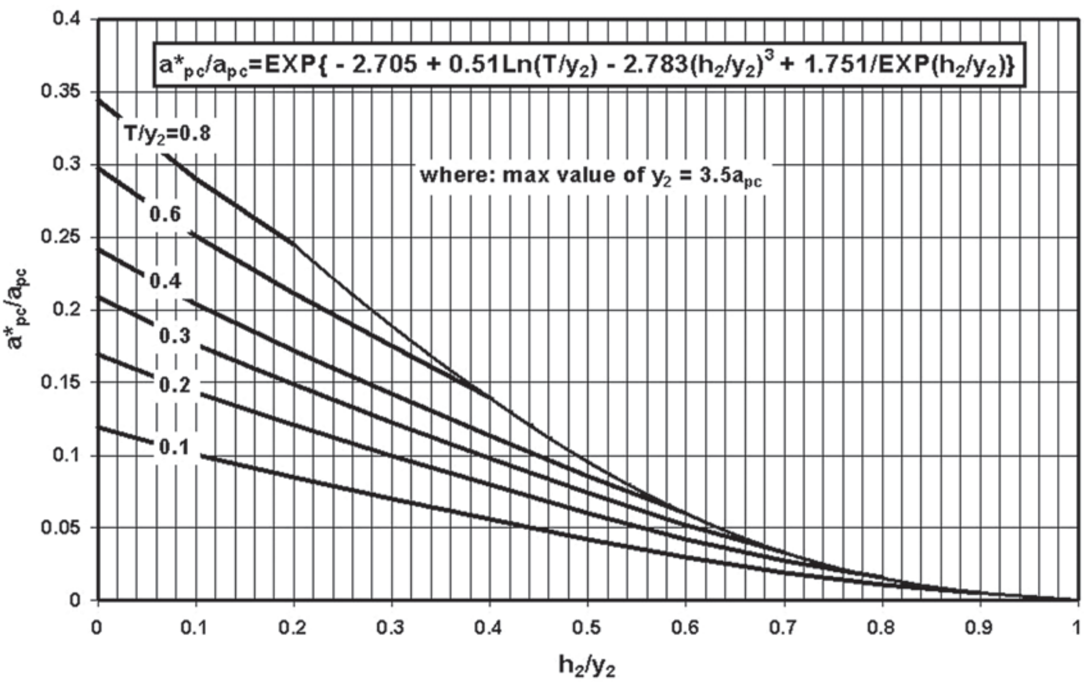


Fig. 10-10. Pile cap (footing) equivalent width (Jones and Sheppard 2000; Richardson and Davis 2001).

$$\frac{V_f}{V_f} = \frac{\ln\left(10.93\dfrac{y_f}{k_s} + 1\right)}{\ln\left(10.93\dfrac{y_2}{k_s} + 1\right)} \qquad (10\text{-}25)$$

where

V_f = average velocity in the flow zone below the top of the footing, m/s (ft/s);
V_2 = average adjusted velocity in the vertical of flow approaching the pier, m/s (ft/s);
In = log to the base e (natural log);
y_f = $h_1 + y_{s\,\text{pier}}/2$ = distance from the bed (after degradation, contraction scour, and pier stem scour) to the top of the footing, m (ft);
k_s = grain roughness of the bed (normally taken as D_{84} for sand-size bed material and 3.5 D_{84} for gravel and coarser bed material), m (ft);
y_2 = adjusted depth of flow upstream of the pier, including degradation, contraction scour, and half the pier stem scour, m (ft).

See Fig. 10-11 for an illustration of variables.

Compute the pile cap scour depth component $y_{s\,\text{pc}}$ from Eq. (10-12) using the full pile cap width a_{pc}, y_f, and V_f as the width, flow depth, and velocity parameters, respectively. The wide pier factor K_w should be used in this computation if (1) the total depth $y_2 < 0.8\ a_{\text{pc}}$, (2) the Froude, number $V_2 / (gy_2)^{1/2} < 1$, and (3) $a_{\text{pc}} > 50\ D_{50}$. Use y_2 / a_{pc} to compute the K_w factor if it is applicable. The scour component equation for the case 2 pile cap or footing can then be written

$$\frac{y_{s\,\text{pc}}}{y_f} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a_{\text{pc}}}{y_f}\right)^{0.65} \left(\frac{V_f}{\sqrt{g y_f}}\right)^{0.43} \qquad (10\text{-}26)$$

In this case assume the pile cap scour component includes the pile group scour and compute the total scour depth as

$$y_s = y_{s\,\text{pier}} + y_{s\,\text{pc}} \text{ (for case 2 only)} \qquad (10\text{-}27)$$

10.9.3.5 Determination of the Pile Group Scour Depth Component Research by Salim and Jones (1995; 1996; 1999) and by Smith (1999) has provided a basis for determining pile group scour depth by taking into consideration the spacing between piles, the number of pile rows, and a height factor to account for the pile length exposed to the flow. Guidelines are given for analyzing the following typical cases:

Piles aligned with each other and with the flow. No angle of attack.

Pile group skewed to the flow, with an angle of attack, or pile groups with staggered rows of piles.

The strategy for estimating the pile group scour component is the same for both cases, but the technique for determining the projected width of piles is simpler for the special case of aligned piles. The strategy is as follows:

Project the width of the piles onto a plane normal to the flow.

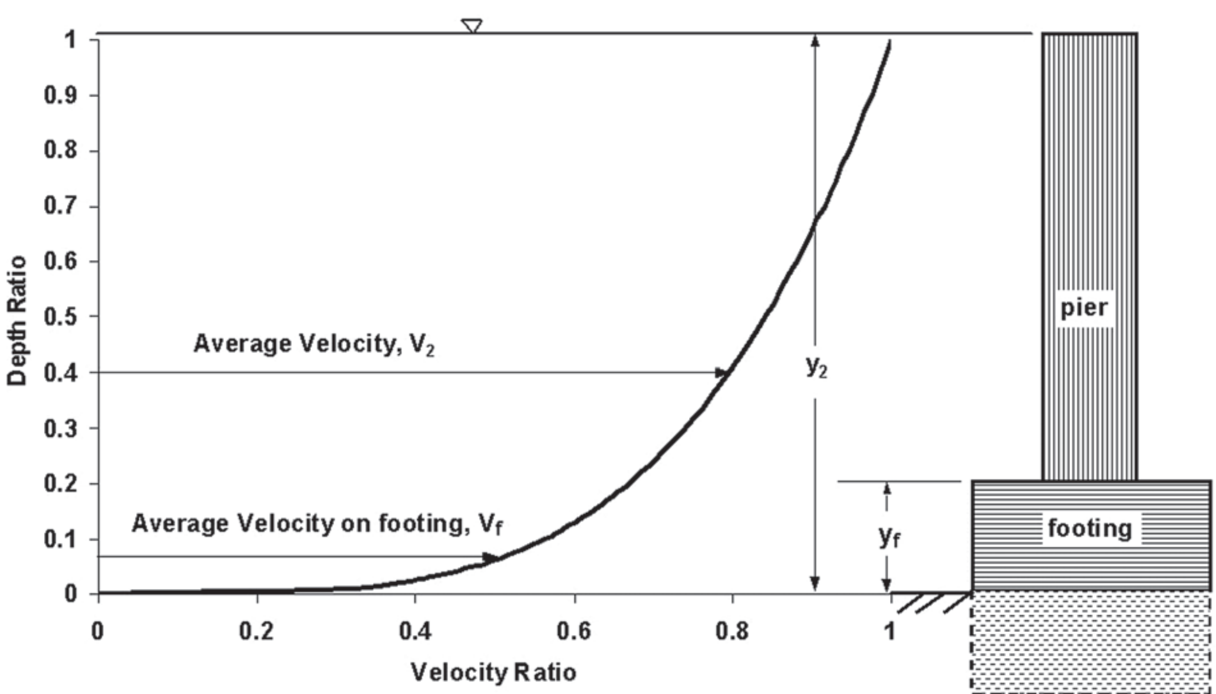


Fig. 10-11. Definition sketch for velocity and depth on exposed footing (Richardson and Davis 2001).

Determine the effective width of an equivalent pier that would produce the same scour if the pile group penetrated the water surface.

Adjust the flow depth, velocity, and exposed height of the pile group to account for the pier stem and pile cap scour components previously calculated.

Determine the pile group height factor based on the exposed height of the pile group above the bed.

Compute the pile group scour component using a modified version of Eq. (10-12).

10.9.3.5.1 Projected Width of Piles For piles aligned with the flow, the projected width, a_{proj}, onto a plane normal to the flow is simply the width of the collapsed pile group as illustrated in Fig. 10-12.

Pile groups not aligned to the flow are represented by an equivalent solid pier that has an effective width, a^*_{pg}, equal to a spacing factor multiplied by the sum of the nonoverlapping projected widths of the piles onto a plane normal to the flow direction (Smith 1999). The projected width can be determined by sketching the pile group to scale and projecting the outside edges of each pile onto a projection plane as illustrated in Fig. 10-13 or by systematically calculating coordinates of the edges of each pile along the projection plane. The coordinates are sorted in ascending order to facilitate inspection to eliminate double counting of overlapping areas. Additional experiments are being conducted at the FHWA hydraulics laboratory to test simpler techniques for estimating the effective width, but currently Smith's summation technique is a logical choice.

Smith attempted to derive weighting factors to adjust the impact of piles according to their distance from the projection plane, but concluded that there were not enough data and the procedure would become very cumbersome with weighting factors. A reasonable alternative to using weighting factors is to exclude piles other than the two rows and one column closest to the plane of projection, as illustrated in Fig 10-13.

10.9.3.5.2 Effective Width of an Equivalent Full Depth Pier The effective width for an equivalent full depth pier is the product of the projected width of piles multiplied by a spacing factor and a number of aligned rows factor (used for the special case of aligned piles only),

$$a^*_{pg} = a_{proj}\, K_{sp}\, K_m \tag{10-28}$$

where

a_{proj} = sum of nonoverlapping projected widths of piles (see Figs 10-12 and 10-13);
K_{sp} = coefficient for pile spacing (Fig 10-14)
K_m = coefficient for number of aligned rows, m, (Figure 10-15 — note that K_m is constant for all S/a values when there are more than six rows of piles)
K_m = 1.0 for skewed or staggered pile groups.

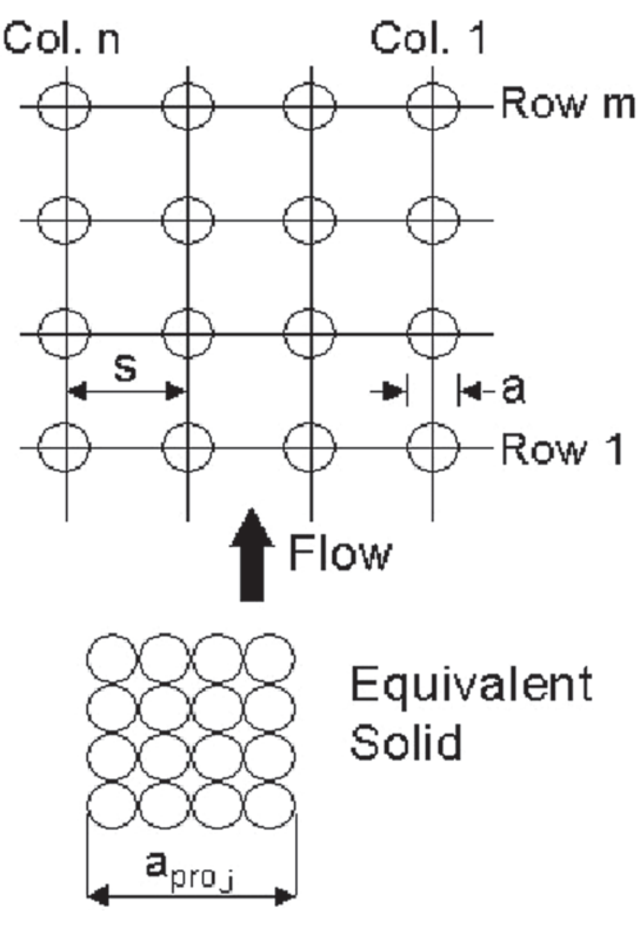


Fig. 10-12. Projected width of piles for flow aligned with the piles (Richardson and Davis 2001).

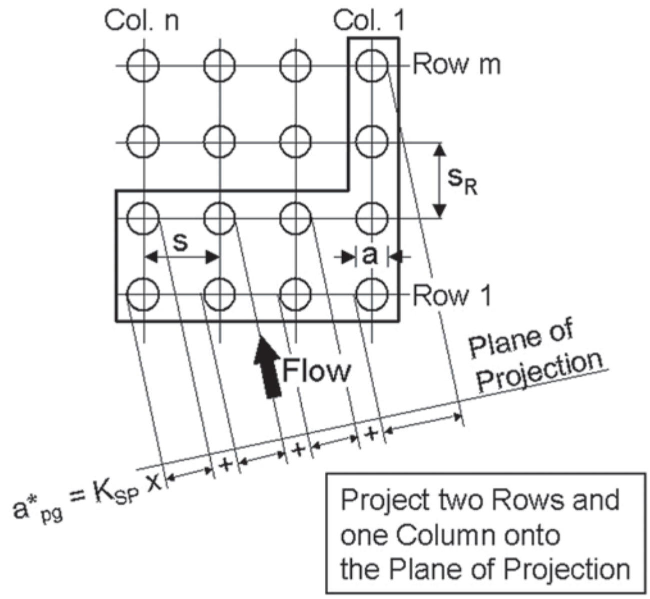


Fig. 10-13. Projected width of piles for skewed flow (Richardson and Davis 2001).

The number of rows factor, K_m, is 1.0 for the general case of skewed or staggered rows of piles because the projection technique for skewed flow accounts for the number of rows and is already conservative for staggered rows.

10.9.3.5.3 Adjusted Flow Depth and Velocity The adjusted flow depth and velocity to be used in the pier scour equation are as follows:

$$y_3 = y_1 + y_{s\,\text{pier}} / 2 + y_{s\,\text{pc}} / 2, \text{ m (ft)} \tag{10-29}$$

$$V_3 = V_1 (y_1/y_3), \text{ m/s (ft/s)} \tag{10-30}$$

The scour equation for a pile group can then be written as

$$\frac{y_{s\,\text{pg}}}{y_3} = K_{h\,\text{pg}} \left[2.0 K_{10} K_3 K_4 K_w \left(\frac{a^*_{\text{pg}}}{y_3} \right)^{0.65} \left(\frac{V_3}{\sqrt{g y_3}} \right)^{0.43} \right] \tag{10-31}$$

where

$K_{h\,\text{pg}}$ = pile group height factor given in Fig 10-16 as a function of h_3 / y_3 (note that the maximum value of $y_3 = 3.5\, a^*_{\text{pg}}$);

$h_3 = h_0 + y_{s\,\text{pier}} / 2 + y_{s\,\text{pc}} / 2$ = height of pile group above the lowered stream bed after pier and pile cap scour components have been computed, m (ft).

K_2 from Eq. (10-12) has been omitted because pile widths are projected onto a plane that is normal to the flow. The quantity in the square brackets is the scour ratio for a solid pier of width a^*_{pg}, if it extended to the water surface. This is the scour ratio for a full depth pile group.

In many complex piers, the pile groups have different numbers of piles in rows or columns, the spacing between piles is not uniform, and the widths of the piles may not all be the same. An estimate of the scour depth can be obtained using the methods and equations in this section. However, again it is recommended that a physical model study be conducted to arrive at the final design and to determine the scour depths.

Engineering judgment must be used if debris is considered a factor, in which case it would be logical to treat the pile group and debris as a vertical extension of the pile cap and to compute scour using the case 2 pile cap procedure described previously.

In cases of complex pile configurations where costs are a major concern or where significant savings are anticipated, and/or for major bridge crossings, physical model studies are still the best guide. Nevertheless, the equations and methods described in this section provide a good calculation of the scour depth.

10.9.4 Multiple Columns Skewed to the Flow

Scour depth for multiple columns skewed to the flow (as illustrated as a group of cylinders in Fig. 10–7) depends on the spacing between the columns. The correction factor for angle of attack would be smaller than that for a solid pier. How much smaller is not known. Raudkivi (1986), in discussing effects of alignment, states that “the use of cylindrical

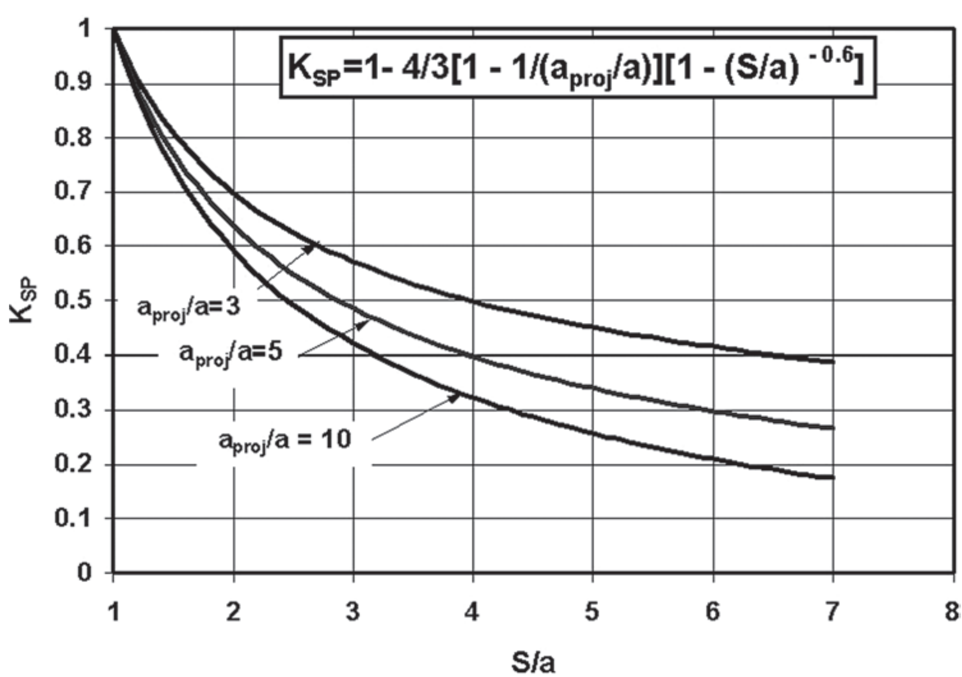


Fig. 10-14. Pile spacing factor (D.M. Sheppard, unpublished design procedure, University of Florida, 2001).

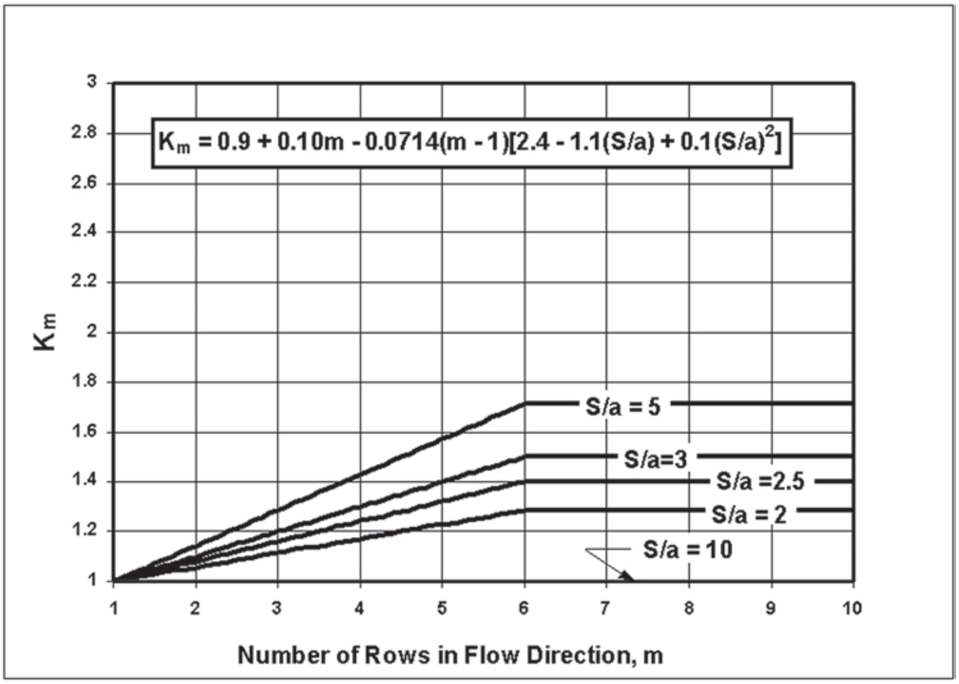


Fig. 10-15. Adjustment factor for number of aligned rows of piles (D.M. Sheppard, unpublished design procedure, university of Florida, 2001).

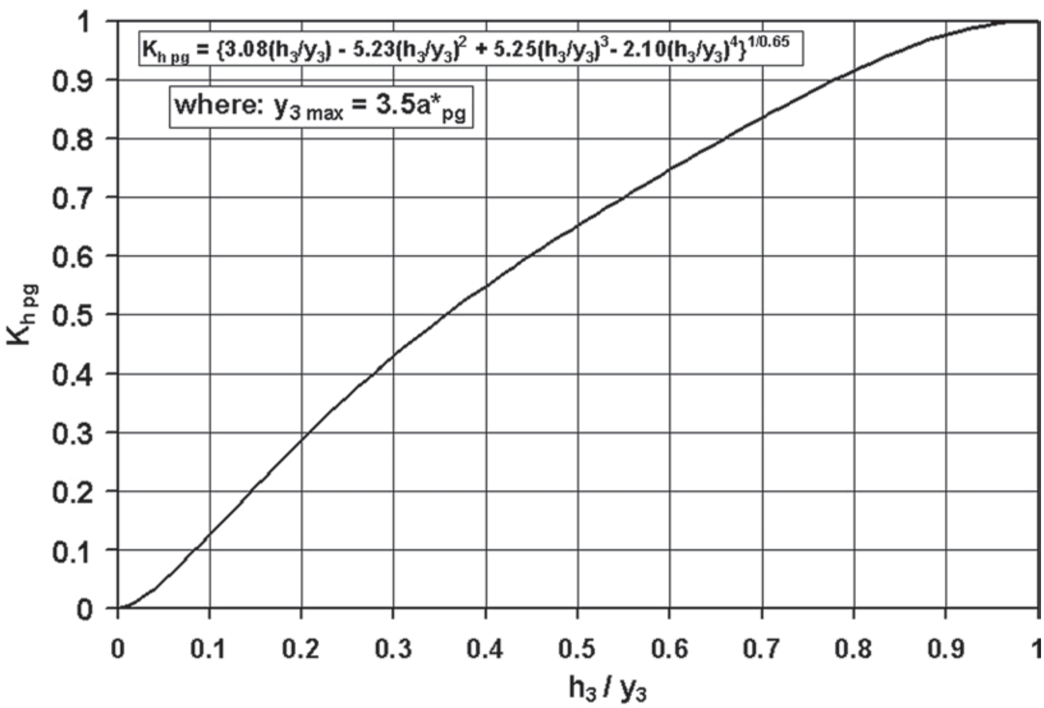


Fig. 10-16. Pile group height adjustment factor (D.M. Sheppard, unpublished design procedure, University of Florida, 2001).

columns would produce a shallower scour; for example, with five-diameter spacing between columns the local scour can be limited to about 1.2 times the local scour at a single cylinder." Thus for multiple columns spaced five diameters or more apart and at an angle, Richardson and Davis (2001) recommend that the local scour depth can be taken as 1.2 times the local scour depth at a single column.

For multiple columns spaced less than five pier diameters apart, the pier width "a" is the total projected width of all the columns in a single row, normal to the flow angle of attack. This composite pier width would be used in Eq. (10-12) to determine depth of pier scour. The correction factor K_1 would be 1.0 regardless of column shape. The coefficient K_2 would also be equal to 1.0 because the effect of skew would be accounted for by the projected area of the piers normal to the flow (Richardson and Davis 2001).

The depth of scour for a multiple column bent will be analyzed in this manner except in addressing the effect of debris lodged between columns. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier.

Additional laboratory studies are necessary to provide guidance on the limiting flow angles of attack for a given distance between multiple columns, beyond which multiple columns can be expected to function as solitary members with minimal influence from adjacent columns.

10.9.5 Pressure Flow Scour

Pressure flow, which is also denoted as orifice flow, occurs when the water surface at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure and the water is in significant contact with the bridge deck. At higher approach flow depths, the bridge can be entirely submerged, with the resulting flow being a complex combination of plunging flow under the bridge (orifice flow) and flow over the bridge (weir flow). In many cases, when a bridge is submerged, flow will also overtop adjacent approach embankments. Hence, for any overtopping situation, the total weir flow can be subdivided into weir flow over the bridge and weir flow over the approach. Weir flow over approach embankments and the bridge reduces the discharge that passes under the bridge.

With pressure flow, the local scour depths at a pier or abutment may be larger than those for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subjected to pressure flow results from vertical contraction scour and local pier scour caused by the horseshoe vortex (Jones et al. 1993). However, sometimes when a bridge becomes submerged, the average velocity under the bridge is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of the discharge that passes under the bridge due to weir flow over the bridge and approach embankments. As a consequence scour depths are reduced.

Abed (Abed 1991; Abed et al. 1991), from a limited clear-water flume study at Colorado State University, stated that pressure flow could increase pier scour depths by 2.3 to 10 times. These results were obtained by comparison of scour depths for free surface and pressure flow simulations with similar hydraulic characteristics.

Jones (Jones et al. 1993; 1996; Richardson and Lagasse 1999, p. 288), in clear-water pressure flow studies at FHWA's Turner-Fairbank Research Center, found that (1) local pier scour with pressure flow has two components; (2) one component is vertical contraction scour caused by the bridge superstructure and the other is local pier scour caused by the pier obstructing the flow; (3) the magnitude of the local pier scour with pressure flow is approximately the same as for free surface flow; and (4) the two components are additive.

Arneson (1997; Arneson and Abt 1998), in a comprehensive live-bed flume study of pressure flow scour sponsored by the FHWA, verified Jones's findings. Equation (10-12) is used to determine the local pier scour component caused by the pier obstructing the flow. For the vertical contraction pier scour component additional research is needed.

10.10 SCOUR DEPTHS WITH DEBRIS ON PIERS

Debris lodged on a pier usually increases local scour at the pier. The debris may increase pier width, and local velocity and deflect the flow downward. This increases the transport of sediment out of the scour hole. When floating debris is lodged on the pier, the scour depth is estimated by assuming that the pier width is larger than the actual width. The problem is in determining the increase in pier width to use in the pier scour equation. Furthermore, at large depths, the effect of the debris on the scour depths should diminish. Also, debris lodged on piers and abutments can deflect the flow against another pier or abutment, resulting in very large angles of attack and larger velocities. This may be worse than the scour at the pier or abutment with the debris.

As with estimating local scour depths with pressure flow, only limited research has been done on local scour with debris. Melville and Dongol (1992) have conducted a limited quantitative study of the effect of debris on local pier scour and have made some recommendations. However, additional laboratory studies will be necessary to better define the influence of debris on local scour.

10.11 JAIN AND FISHER'S EQUATION

Jain and Fisher (1979) studied local pier scour at large Froude numbers in the laboratory. They found that live-bed scour at a circular pier first slightly decreased and then increased with the increase in the Froude number. Live-bed scour depths at high Froude numbers are larger than the maximum clear-water scour. The contribution of bed-form scour to the total scour depth in the upper flow regime becomes significant with higher flow velocities. They developed the following two equations:

For live-bed scour $(\mathsf{F} - \mathsf{F}_c) > 0.2$,

$$y_s/a = 2.0(\mathsf{F}-\mathsf{F}_c)^{0.25}(y_1/a)^{0.5}. \quad (10\text{-}32)$$

For clear-water scour, $(\mathsf{F} - \mathsf{F}_c) \leq 0.2$,

$$y_s/a = 1.84(\mathsf{F}_c)^{0.25}(y_1/a)^{0.3} \quad (10\text{-}33)$$

where

F_c = Froude number for beginning of motion, $V_c/(gy_1)^{½}$ of the D_{50} size of the bed material.

The other variables are as defined previously.

They determined the critical velocity for the beginning of motion using a procedure based on Einstein's (1950) logarithmic velocity equations. His equations are given by Richardson et al, (1990) as follows:

1. Determine the median diameter, D_{50}, of the bed material, m, ft;
2. Determine τ_c from Shield's relation, N/m^2, lb/ft^2;
3. Compute $U_*c = (\tau_c/p)^{0.5}$, m/s, ft/s;
4. Compute $V_c = U_* c\,[(2.5 \ln(12.27\, y\, X/D_{65})]$, m/s, ft/s;
5. Assume χ is 1.0, i.e., hydraulically rough flow;
6. Compute $\mathsf{F}_c = V_c/(gy_1)^{0.5}$.

The equation given in Section 10.6 can also be used to determine the critical velocity.

They also recommended that the scour depth for $0 < (\mathsf{F} - \mathsf{F}_c) < 0.2$ can be assumed equal to the larger of the two values of scour obtained from Eqs. 10-29 and 10-30. For shapes different from circular piers and pier alignment other than parallel with the flow direction, multiply the results given by Jain and Fisher's equations by the coefficients given in Tables 10-1 and 10-2.

10.12 MELVILLE'S EQUATION

Mellville (1997) gave the following equation for computing local scour depths at piers

$$y_s = K_1 K_2 K_{ya} K_i K_D \quad (10\text{-}34)$$

where

y_s = depth of scour, m;
K_1 = correction for pier nose shape from Figure 10-7 and Table 10-1;
K_2 = correction for flow angle of attack from Eq. (10-15);
K_{ya} = flow depth-pier size expression

$$K_{ya} = 2.4a, \text{ if } y/a < 0.7$$
$$K_{ya} = 2\sqrt{y_1 a},\ 0.7 < \frac{a}{y_1} < 5 \quad (10\text{-}35)$$
$$K_{ya} = 4.5y_1, \frac{a}{y_1} > 5$$

K_i = flow intensity factor

$$K_i = \frac{V_1 - (V_a - V_c)}{V_c}, \text{ if } \frac{V_1(V_a - V_c)}{V_c} < 1$$
$$K_i = 1, \text{ if } \frac{V_1 - (V_a - V_c)}{V_c} \geq 1 \quad (10\text{-}36)$$

K_D = sediment size factor

$$K_D = 1.0, \quad \text{if } a/D_{50} > 25$$
$$K_D = 0.57\log(2.24a/D_{50}), \quad \text{if } a/D_{50} \leq 25 \quad (10\text{-}37)$$

V_1 = mean approach velocity, m/s;
V_a = mean approach velocity at the armor peak = 0.8 V_{ca}, m/s;
V_c = critical velocity at beginning of motion, m/s.

Melville gives the equation

$$\frac{V_c}{V_{*c}} = 57.5 \log\left(5.53\frac{y_1}{D_{50}}\right) \quad (10\text{-}38)$$

where

V_{ca} = maximum mean approach velocity for armoring of the channel bed to occur, m/s.

Mellville gives the equation

$$\frac{V_{ca}}{V_{*ca}} = 57.5 \log\left(5.53\frac{y_1}{D_{50a}}\right) \quad (10\text{-}39)$$

where

V_{*c} = critical shear velocity for the D_{50} defined by the Shield's relation, m/s;
V_{*ca} = critical shear velocity for the D_{50a} defined by the Shield's relation, m/s;
D_{50a} = median armor size, m, where $D_{50a} = D_{max}/1.8$;
D_{max} = maximum bed material size, m.

Melville gives as an approximation to the Shield's diagram for quartz sediment in water at 20ºC the following,

$$
\begin{aligned}
V_{*c} &= 0.0115 + 0.0125D^{1.4}, 1\text{mm} < D < \phi.0.1\text{mm} \\
V_{*c} &= 0.305D^{0.5} - 0.0065D^{-1}, 1\text{mm} < D < 100\text{mm}
\end{aligned}
\tag{10-40}
$$

where V_{*c} or (V_{*ca}) is in m/s and $D = D_{50}$ or D_{50a} in mm.

10.13 OTHER PIER SCOUR EQUATIONS

Other pier scour equations and data sets are given by Jones (1983); Froehlich (1988); Johnson and Torrico (1994); Landers and Mueller (1996); Landers, et al. (1996); Mueller (1996); and Richardson and Lagasse (1999). Vanoni (1975) discusses pier scour and gives Laursen's equation.

10.14 TOP WIDTH OF PIER SCOUR HOLES

The top width of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the equation (Richardson and Abed 1993; Richardson and Lagasse 1999; Richardson and Davis 2001)

$$W = y_s \left(K + \cot\theta\right) \tag{10-41}$$

where

- W = top width of the scour hole from each side of the pier or footing, m, ft;
- y_s = scour depth, m, ft;
- K = bottom width of the scour hole as a fraction of scour depth;
- θ = angle of repose of the bed material which ranges from about 30° to 44°.

If the bottom width of the scour hole is equal to the depth of scour y_s ($K = 1$), the top width in cohesionless sand will vary from 2.07 to 2.80 y_s. At the other extreme, if $K = 0$, the top width will vary from 1.07 to 1.8 y_s. Thus, the top width could range from 1.0 to 2.8 y_s and would depend on the bottom width of the scour hole and the composition of the bed material. In general, the deeper the scour hole, the smaller the bottom width. A top width of 2.0 y_s is suggested for practical application.

10.15 LOCAL SCOUR AT ABUTMENTS

Local scour at abutments has two components (see Fig. 10-4). One component is caused by a horizontal vortex that forms at the upstream end of the abutment and runs along the abutment toe. The other component is a vertical vortex that forms at the downstream end of the abutment when the flow separates and starts to expand. This vertical vortex erodes the downstream corner of the abutment and the downstream approach roadway. There are no equations available to determine the erosion caused by this downstream vortex. The abutment is protected from erosion caused by this vertical vortex by riprap or a short guidebank (Lagasse et al. 2001). The available equations are for the scour caused by the horizontal vortex.

Equations for predicting local scour depths at abutments are almost all based entirely on laboratory data. For example, equations by Laursen and Toch (1956), Liu et al. (1961), Laursen (1980), Froehlich (1989; 1989b), and Melville (1992; 1997) are based entirely on laboratory data. The problem is that few field data on abutment scour exist. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line drawn through the laboratory data. Laursen's equations are based on inductive reasoning on the change in transport relations due to the acceleration of the flow caused by the abutment. Froehlich's equation was derived from dimensional analysis and regression analysis of the available laboratory data. Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.

All equations in the literature, prior to 1993, were developed using the abutment and roadway approach length (L) as one of the variables and result in excessively conservative estimates of scour depth. As Richardson and Richardson (1992) and Richardson and Richardson (1998) point out in a discussion of Melville's (1992; 1997) papers and in a 1993 paper, 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.

Figure 10-17 illustrates the difference. Thus, using the abutment length in the equations instead of the discharge returning to the main channel at the abutment results in a spurious correlation between abutment lengths and scour depth at the abutment end.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. Also, abutment scour depth depends on abutment shape, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and the depth of the overbank flow at the abutment), velocity in the main channel and in the flow returning to the main channel at the abutment, and alignment. In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. Much of the research up to 1993 failed to replicate these field conditions. However, since 1993, research by Sturm et al., Young et al., Kouchakzadeh and Townsend, Chang and Davis, and Molinas et al. (Richardson and Lagasse

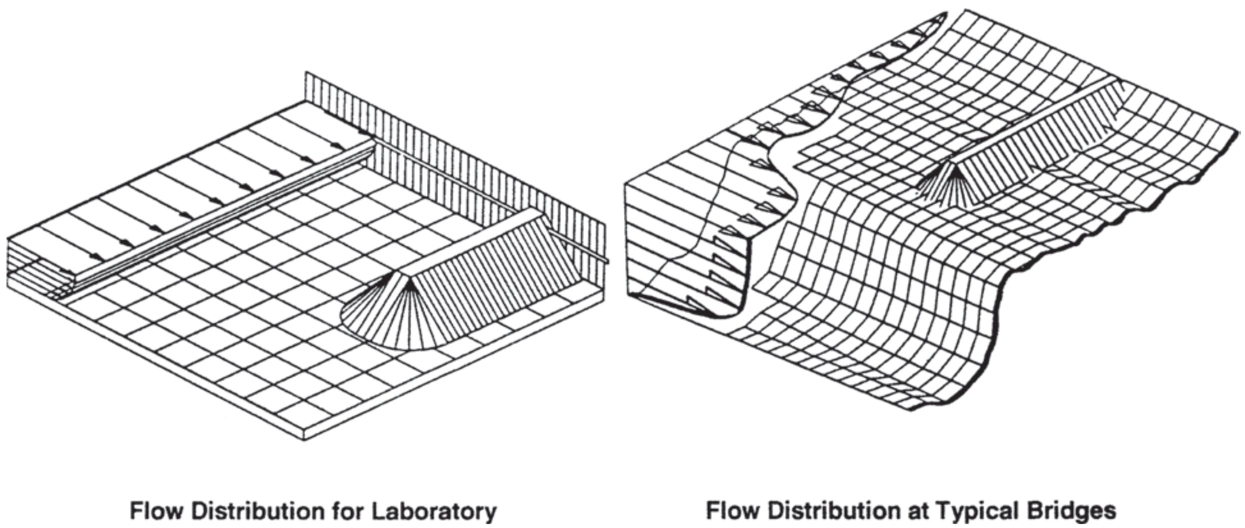


Fig. 10-17. Comparison of laboratory flow characteristics to field conditions (Richardson and Richardson 1998).

1999) has addressed the problem of using abutment length as the primary variable for the discharge intercepted by the abutment.

Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when the foundations are protected with rock riprap placed below the streambed and/or a guide bank placed upstream of the abutment (Richardson and Davis 2001). The design of guide banks is given by Lagasse et al. (2001).

10.15.1 Abutment Site Conditions

Abutments can be at the channel bank, be set back from the natural stream bank, or project into the channel. They can have various shapes and can be set at varying angles to the flow. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

10.15.2 Abutment Shape

There are three general shapes for abutments: (1) spill-through abutments, (2) vertical-wall abutments with wing walls, and (3) vertical walls without wing walls (Fig. 10-18). Depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments. In Table 10-4 coefficients for correcting scour equations for abutment shape (Froehlich, 1989) is given. However, recent research by Sturm (1999) on abutment scour in compound channels demonstrated that abutment shape is important for shorter abutments but detected no abutment shape effects as abutments increased in length and caused more contraction with encroachment on the main channel.

10.15.3 Skew Adjustment of Abutment Scour Depths

Figure 10-19 shows the effect of flow angle of attack on abutment scour (Ahmad 1953). As shown, an abutment or spur angled downstream decreases scour depth, whereas an abutment angled upstream into the flow increases scour depth.

10.15.4 Design for Scour at Abutments

The lack of adequate abutment scour equations (some equations are fundamentally wrong and/or overconservative) lead the Federal Highway Administration to recommend that in setting abutment foundation depths the potential for lateral migration, long-term degradation, and contraction scour should be considered. It is recommended that foundation depths for abutments be set at least 1.8 m below the stream bed, including long-term degradation and contraction scour, and rock riprap and/or guide banks should be used to protect the abutment. As a check on the potential scour depth they gave two equations to aid in design and placement of rock riprap (Richardson and Davis 2001).

In the following sections four equations are given. These equations are the result of recent research that properly uses the discharge obstructed by the abutment rather than abutment length. These are

- The Chang and Davis equation (Richardson and Lagasse 1999), which is based on the Laursen live-bed contraction scour equation.
- The Sturm (1999) equation for abutments in compound channels with variable setbacks from the main channel.

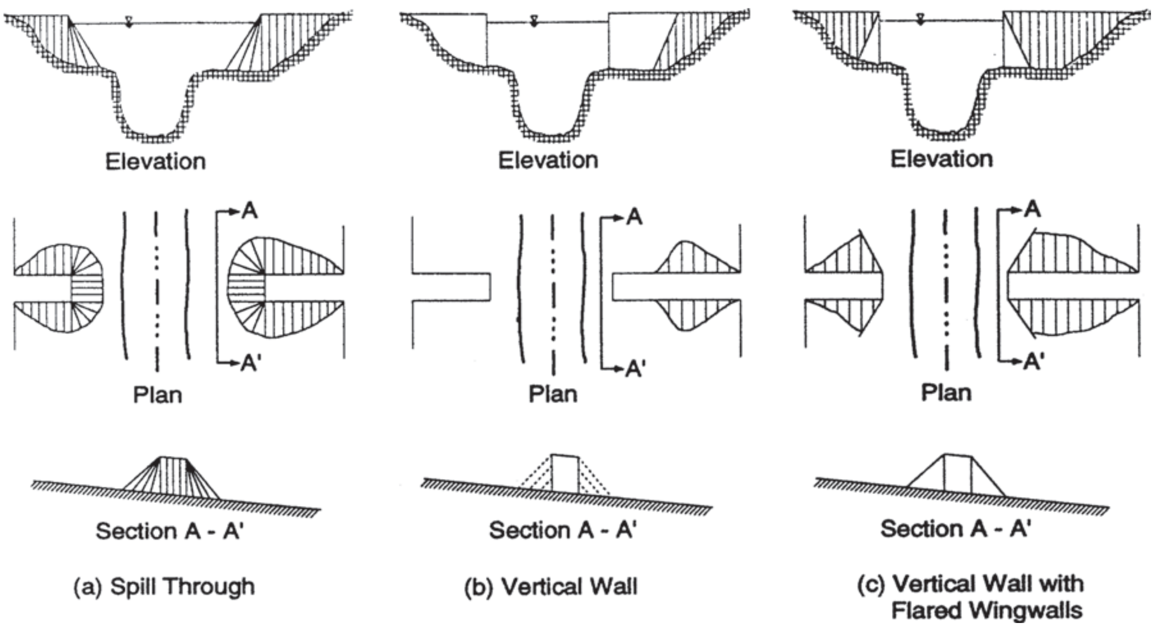


Fig. 10-18. Abutment shape.

- The Richardson and Trivino (1999) equation, based on momentum exchange.
- The Richardson et al. (1990) equation, based on Corps of Engineers data on scour at the end of spur dikes in the Mississippi River. It is recommended for use when abutment length divide by flow depth is greater than 25 ($L / y > 25$).

10.16 CHANG AND DAVIS ABUTMENT SCOUR EQUATION

Chang and Davis (Richardson and Davis 2001) present methods for computing local scour at abutments, developed for the Maryland Department of Transportation. Different equations and methods are given for live-bed and clear-water scour. The equations are adjustments to live-bed and clear-water contraction scour for the increase in local scour caused by the horizontal vortex at the abutment. Both equations are nondimensional and can be used for either English or SI units. In the process a computer program titled ABSCOUR was developed. Their equations are given in the following.

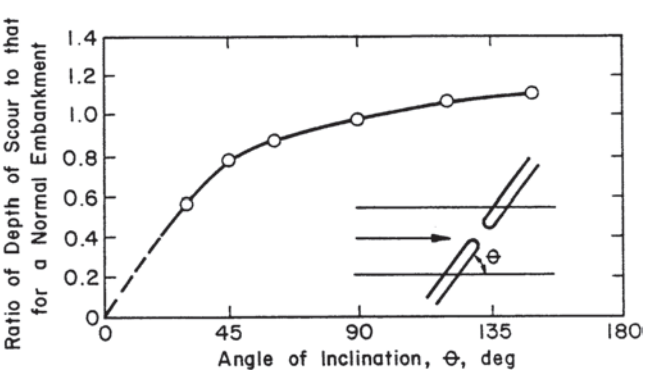


Fig. 10-19. Adjustment of abutment scour estimate for skew (Ahmad 1953).

10.16.1 Live-Bed Abutment Scour

The equation is

$$\frac{y_{2a}}{y_1} = k_f \left(\frac{k_v\ q_2}{q_1}\right)^{K_2} \tag{10-42}$$

where

y_{2a} = total flow depth in the abutment scour hole after scour has occurred, measured from the water surface to the bottom of the scour hole, m (ft);

y_1 = approach flow depth, m (ft);

Table 10-4 Abutment Shape Coefficients (Froehlich 1989)

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

q_1 = flow rate per unit width in the approach section, $m^3/s/m$ ($ft^3/s/ft$);
q_2 = flow rate per unit width in contracted section, $m^3/s/m$ ($ft^3/s/ft$) (determination of q_1 and q_2 is explained in a section);
k_v = $0.8 (q_1 / q_2)^{1.5} + 1$;
k_f = $0.1 + 4.5 F_1$ for clear-water scour;
k_f = $0.35 + 3.2 F_1$ for live-bed scour.

Equation (10-42) applies to live-bed scour. It can be used for clear-water scour only for the condition where the shear stress in the approach section (Section 1) is at the critical value.

Values of k_v should range from 1.0 to 1.8. If the calculated value is smaller or larger than this range, use the limiting value.

Values of k_f should range from 1.0 to 3.3. If the calculated value is smaller or larger than this range, use the limiting value.

The Froude number in the approach section, Section 1, $F_1 = V_1/(gy_1)^{0.5}$, where V_1 = average flow velocity in the approach floodplain or channel section (m/s or ft/s) and y_1 = average flow depth in the approach floodplain or channel section (m or ft).

Laursen's sediment transport function for K_2 is

$$K_2 = 0.11 (\tau_c / \tau_1 + 0.4)^{2.2} + 0.623 \qquad (10\text{-}43)$$

where

τ_c = critical shear stress of soil, N/m^2 (lb/ft^2);
τ_1 = shear stress at approach section, Section 1, N/m^2 (lb/ft^2), $\tau_1 < \tau_c$. The value of K_2 varies from 0.637 to 0.857. $\tau_c \geqslant \tau_1$, select a value of K_2 equal to 0.857.

Chang (personal communication 2000) determined that, although K_2 in Eq. (10-35) is based on a concept similar to K_1 in the table accompanying the live-bed contraction scour equation, (10-1), the values of these coefficients are derived in different ways and cannot be mathematically correlated.

10.16.2 Clear-Water Abutment Scour

Clear-water scour occurs if the shear stress in the approach section, Section 1, is less than critical, or if the approach section is armored. The clear-water abutment scour equation is

$$y_{2a} = k_f \ (k_v)^{0.857} \ y_{2c} \qquad (10\text{-}44)$$

where

y_{2a} = total depth of flow at the abutment, measured from the water surface down to the bottom of the abutment scour hole, m (ft);
y_{2c} = clear-water contraction scour depth in the channel or on the floodplain (beyond the abutment scour hole) at critical velocity $y_{2c} = q_2 / V_c$, m (ft) (Eq. 10-11) or similar equations can be used to compute V_c. Another approach would be to compute y_{2c}, the clear-water contraction scour, from Eq. (10-7) or (10-8);
k_f and k_v = dimensionless coefficients as defined in the discussion of live-bed scour.

10.17 STURM ABUTMENT SCOUR EQUATION

Sturm (1999) evaluated abutment scour using a flume with a compound channel. He determined that a discharge distribution factor (M) is a better measure of the effect of abutment length on the flow redistribution and abutment scour. His resurch resulted in an equation for clear-water scour around setback and bankline abutments and for live-bed scour around bankline abutments. His equations are given in the following discussions.

10.17.1 Clear-Water Scour

Sturm's clear-water abutment scour equation is

$$y_s / y_{fo} = 8.14 \ K_{st} \ (q_{fl} / MV_{xc} \ y_{fo} - 0.47) + \text{FS} \qquad (10\text{-}45)$$

where

y_s = depth of scour at the abutment, m;
y_{fo} = average depth of flow on the floodplain at the approach section for existing conditions based on normal flow conditions in the river without backwater from the proposed bridge, m;
K_{st} = abutment shape factor given below;
q_{fl} = unit flow rate on the approach floodplain section that will be blocked by the embankment at Section 2 (The conditions are based on the proposed structure in place and creating backwater effects at the approach section), $m^3/s/m$;
M = discharge distribution factor $= (Q_{1/2 \text{ channel}} + Q_{\text{floodplain}} - Q_{\text{blocked flow}}) / (Q_{1/2 \text{channel}} + Q_{\text{floodplain}})$— $Q_{1/2 \text{ channel}}$ is the discharge from the centerline to the bank of the main channel in the approach section, $Q_{\text{floodplain}}$ is the floodplain discharge in the approach section, and $Q_{\text{blocked flow}}$ is the floodplain discharge blocked by the embankment in the approach section;
V_{xc} = critical velocity at the approach floodplain section for existing conditions based on normal flow conditions in the river without backwater from the

proposed bridge, m/s (use Eq. (10-10) or (10-11) and the D_{50} of the bed material);

FS = factor of safety, with a recommended value of 1.0;

K_{st} = 1.0 for vertical wall abutments.

For spillthrough abutments K_{st} is as follows:

K_{st} = 1.52 $(K_a - 0.67)/(K_a - 0.40)$ where $0.67 \leqslant K_a \leqslant 1.2$
1.0 where $K_a > 1.2$
0.0 where $K_a < 0.67$

$$K_a = q_{fl} / (M \times V_{xc} \times y_{f0}) \qquad (10\text{-}46)$$

10.17.2 Live-Bed Scour around Bankline Abutments

Sturm's live-bed abutment scour equation around bankline abutments is

$$y_s / y_{f0} = 2.0K_{st}\,[q_{ml} \;/(MV_{m0c}\, y_{f0}) - 0.47] + \text{FS} \qquad (10\text{-}47)$$

where

y_s = depth of scour at the abutment, m (ft);

y_{f0} = average depth of flow on the floodplain (Step 5), m (ft);

K_{st} = 1.0;

q_{ml} = unit flow rate in the main channel at the approach section 1 for the approach critical velocity, i.e., $(V_{m1c} \times y_{m1})$, m^3/s/m (cfs/ft);

M = discharge distribution factor as defined above;

V_{m0c} = critical velocity in the main channel for unconstricted flow at depth y_{m0}, m/s (ft/s);

FS = factor of safety, with a recommended value of 1.0.

Note that Eq. (10-47) is based on experimental results for clear-water scour around bankline abutments. Its extension to the live-bed case by assuming threshold live-bed scour is tentative at this time.

10.18 RICHARDSON AND TRIVINO ABUTMENT SCOUR EQUATION

Using a regression technique developed by Box and Tidwell (1962), Richardson and Trivino (1999) regressed approximately 160 clear-water scour data compiled by Froehlich (1989); Lim (1993; 1997); and a field measurement of abutment scour obtained during the 1993 Missouri–Mississippi River flood. The last was an 18.3-m (60-ft)-deep abutment scour hole near the right abutment of the 1-70 Bridge over the Missouri River, near Columbia, MO (Brian Hefner, Hydraulic Section, Missouri Department of Transportation, Bridge Inspection File for Interstate 70 near Rocheport; Missouri, personal communication, 1999). A hydraulic study by Greble (1999) noted that the 2,060-m (6,760-ft)-long approach embankment cut off nearly 80% of the estimated 9,900 m^3/s (349,700 cfs) floodplain discharge. The equation is

$$\frac{y_s}{y_1} = 0.02K_1^{-6.81} + 7.47\mathsf{F}^{1.60} + 1.68\left(\frac{L}{y_1}\right)^{0.41} - 3.32\left(\frac{M_1}{M_2}\right)^{2.46} - 335\left(\frac{D_{50}}{y_1}\right)^{1.66} - 1.41 \qquad (10\text{-}48)$$

where

y_s = the depth of abutment scour, m (ft);

y_1 = the unscoured average flow depth on the overbank (near the abutment end), m (ft);

K_1 = the coefficient for abutment shape (as previously defined), m (ft);

F = the Froude number of the approach flow unobstructed by the abutment;

L = the length the approach embankment projects into the floodplain, m (ft);

D_{50} = the median grain size of the bed material, m (ft);

M_1 = the momentum of the flow intercepted by the abutment and approach (Eq. (10-48));

M_2 = the momentum of the flow in the bridge opening (Eq. (10-49)).

M_1/M_2 is the momentum ratio of the flow that is mixed near the abutment end, which causes the horizontal vortex and abutment scour:

$$M_1 = \rho\, Q_1\, V_1 \qquad (10\text{-}49)$$

$$M_2 = \rho\, Q_2\, V_2 \qquad (10\text{-}50)$$

Where

ρ = mass densıty of water;

Q_1 = overbank discharge cutoff by the abutment and approach one bridge length upstream, m^3/s (cfs);

Q_2 = discharge in the constricted section (bridge section), m^3/s (cfs)—for an abutment set back from the main channel it is the discharge between the end of the abutment and the channel bank, whereas for abutments at the channel bank or projecting into the main channel it is the total discharge in the bridge section;

V_1 = average overbank velocity of the flow cutoff by the abutment and approach embankment one bridge length upstream of the bridge (corresponding to Q_1), m/s (ft/s);

V_2 = average velocity of the flow in the constricted (bridge section corresponding to Q_2), m/s (ft/s).

Equation (10-48), for Froehlich's and Lim's data set, has a computed R^2 equal to 0.895. The standard error, S_e, of estimating d_s/d_1 was 0.48. In comparison, the S_e was computed to be 1.12 and 1.98 for Froehlich's and Lim's equation when applied to the same data set. It accurately predicted the actual y_s/y_1 of 3.9 for the I-70 scour hole. This contrasts with y_s/y_1 values of 6.4 and 17.8 using Froehlich's and Lim's equation respectively. When Eq. (10-47) was applied to a set of 37 complex laboratory channel data documented by Sturm (1998) and Sturm and Janjua (1994), the standard error of estimate (S_e) was 0.89. S_e was likely greater due to the relatively small sample size. Considering all of the data, the S_e was only slightly higher (S_e = 0.56).

A separate Box-Tidwell regression without the L/d_1 term produced good agreement with the Missouri River data, but the correlation with the flume data was poor. This was not surprising because the flume experiments were performed using the approach and abutment length as the primary variable, not the momentum or discharge ratios. Also, at small laboratory scales the momentum ratio is small and has a minimal influence on the resulting dependent variable. At larger scales, the influence of the momentum ratio, M_1 / M_2 is more important than the L / y_1. Because of its importance in the data set, the L / y_1 was retained in the formulation. The sensitivity of the dependent variable (y_s / y_1) to L / y_1 is significantly less than for other formulations involving the ratio of abutment length to flow depth.

Due to the manner in which the equation was formulated, Eq. (10-48) is applicable to conditions in which the abutment is set back from the main channel. All of the data used to develop the equation was for approach embankments normal to the average flow direction, and therefore no correction for abutments angled to the flow is incorporated into the equation. However, for abutments at an angle to the flow, the length L should be adjusted to its normal length and Fig. 10-19 used to correct the scour depths. As with all other existing abutment scour equations, the equation has not been thoroughly verified for field conditions.

10.19 RICHARDSON ET AL. EQUATION FOR $L/y > 25$

Richardson et al. (1990, 2001) give an equation developed using Corps of Engineers field data on scour at the end of spurs in the Mississippi River. This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. This is recommended when the ratio of projected abutment length (L) to flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_1) at an abutment where conditions are similar to the field conditions from which the equation was derived,

$$\frac{y_s}{y_1} = 4F_1^{0.33}\frac{K_1}{0.55} \tag{10-51}$$

where

y_s = scour depth, m, ft;
y_1 = depth of flow at the abutment, on the overbank or in the main channel, m, ft;
F_1 = Froude number based on the velocity and depth adjacent to and upstream of the abutment;
K_1 = abutment shape coefficient, from Table 10-6.

To correct Eq. (10-51) for abutments skewed to the stream use Fig. 10-19.

10.20 COMPUTER MODELS

The hydraulic routines of computer models WSPRO (Shearman 1987) or HEC-RAS (USACE 2001), can determine the one-dimensional flow variable for use in the determination of scour depths at a bridge. These models determine average flow depths and velocities over a roadway and bridge, as well as average velocities and depths approaching and under the bridge.

10.21 STREAM INSTABILITY

Streams are dynamic. Areas of flow concentration continually shift bank lines. In meandering streams having an S-shaped planform, the channel moves both laterally and downstream. A braided stream has numerous channels that are continually changing. In a braided stream, the deepest natural scour occurs when two channels come together or when the flow comes together downstream of an island or bar. This scour depth has been observed to be one to two times the average flow depth (Northwest Hydraulic Consultants Ltd., personal communication, 1973; Richardson and Davis 2001).

A bridge is static. It fixes a stream at one place in time and space. A meandering stream whose channel moves laterally and downstream into the bridge reach can erode the approach embankment and affect contraction and local scour because of changes in flow direction. A braided stream can shift under a bridge and have two channels come together at a pier or abutment, increasing scour. Descriptions of stream morphology are given by Schumm (1977), Lagasse et al., (2001), and Richardson et al. (2001), among others.

Factors that affect lateral shifting of a stream and the stability of a bridge are the geology and geomorphology of the stream, the location of the crossing on the stream, flood characteristics, the characteristics of the bed and bank material, and wash load.

It is difficult to anticipate when a change in planform may occur. It may be gradual with time or the result of a major flood. Also, the direction and magnitude of the movement of the stream are not easily determined. It is difficult to evaluate the vulnerability of a bridge properly due to changes in planform. It is important to incorporate potential planform changes into the design of new bridges and design of countermeasures for existing bridges.

Countermeasures for lateral shifting and instability of a stream may include changes in the bridge design, construction of river control works, protection of the foundations with riprap, or careful monitoring of the river in a bridge inspection program. Richardson and Davis (2001) recommend that foundations of piers and abutments located on floodplains be placed at elevations approximating those for piers located in the main channel.

To control lateral shifting requires river training works, bank stabilization by riprap, and/or guide banks. Design methods are given in publications of the Federal Highway Administration, U.S. Army Corps of Engineers, and American Association of State Highway and Transportation Officials (AASHTO). Of particular importance are “Spurs and Guide Banks” (Richardson and Simons 1974); “The Design of Spurs for River Training” (Richardson et al. 1975); “The Streambank Erosion Control Evaluation and Demonstration Act of 1974” (USACE 1981); “Streambank Protection Guidelines for Landowners and Local Governments” (USACE 1983); “Use of Spurs and Guidebanks for Highway Crossings” (Richardson and Simons, 1984); “Streambank Stabilization Measures for Highway Stream Crossings” (Brown 1985); “Highways in the River Environment” (Richardson et al. 1990); “Hydraulic Analysis for the Location and Design of Bridges,” Volume VII, Highway Drainage Guidelines (AASHTO 1992); “Bridge Scour and Stream Instability Countermeasures” (Lagasse et al. 2001a); “Stream Stability at Highway Structures” (Lagasse et al. 2001b); “River Engineering for Highway Encroachments” (Richardson et al. 2001).

10.22 SCOUR IN TIDE-AFFECTED WATERWAYS

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 (Richardson et al. 1993; 1995; Richardson and Lagasse 1999; Richardson and Davis 2001). 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, nor’easters, and tsunami), 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. In addition, tidal flows are subject to mass density stratification and water salinity, but these have only a minor effect on bridge scour. The hydraulic variables (discharge, velocity, and depth) 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 to improve scour determinations in both cases. Determining the magnitude of the combined flows can be accomplished by simply adding riverine flood flow to the maximum tidal flow or routing the design riverine flows to the crossing and adding them to the storm surge flows.

Some of the similarities and differences between tidal and riverine flows are as follow:

- Tidal flows are unsteady with short-duration peak flows. Riverine flows are also unsteady and many have short-duration peak flows. Existing scour equations predict scour depths for these short-duration peak riverine flows. Also, waterways in the coastal zone are composed of fine sand that erodes easily. Therefore, riverine scour equations will predict scour depths in short-duration tidal flows.
- Astronomical tides, with their daily or twice-daily in- 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 several feet per year with no indication of stopping (Butler and Lillycrop 1993; Vincent et al. 1993). Existing scour equations can predict the magnitude of this scour, but not the time history (Richardson et al. 1993).
- Mass density stratification (saltwater wedges), which can result when 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 (Sheppard 1993). However, with careful evaluation, the correct velocity for use in the scour equations can be determined. With storm surges, mass density stratification will not normally occur. The density difference between salt and fresh water, except when it causes saltwater wedges, is not significant enough to affect scour equations. Density and viscosity differences between fresh and sediment-laden water can be much larger in riverine flows than the differences between salt and fresh water. Salinity can affect the transport of silts and clays by causing them to flocculate and possibly deposit, which may affect stream stability and must be evaluated. Salinity may affect the erodibility of cohesive sediments, but this will only affect the rate of scour, not ultimate scour.
- Littoral drift is a source of sediment to a tidal waterway (Sheppard 1993) 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 and/or the coast, sources of sediment, and other factors.

- *There is one major difference between riverine scour at highway structures and scour resulting from tidal forces*. In determining scour depths for riverine conditions, a design discharge is used (discharge associated with a 50-, 100-, or 500-year return period). For tidal conditions, a design storm surge elevation is used (elevation for the 50-, 100-, or 500-year storm surge return period), and from the storm surge elevation, 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 (Richardson et al. 1993; Richardson and Davis 2001).

The reason the design discharge for the same return periods for tidal waterways may increase is that the discharge 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 or from the storm surge, the discharge may increase as the waterway area increases.

10.22.1 Design Discharge

The design discharge for tidal waterways is determined from the 50-, 100-, and 500-year storm surge return period elevation. From this elevation, tidal prism volume, and waterway area, the design discharge is determined. If the waterway area increases the design discharge may increase. This is a major difference between the tidal and riverine design discharge (see discussion above) (Richardson et al. 1993; Richardson and Davis 2001). Models are available to generate synthetic storm surge hydrographs combined with different periods of the daily tides (Zevenbergen et al. 1997a, b).

Determination of the design discharge for scour analysis for bridges in tidal waterways consists of a three-level approach. First is preliminary qualitative evaluation of the stability of a tidal waterway, estimation of the magnitude of the tides, storm surges, littoral drift, and flow in the tidal waterway, and determination of whether the hydraulic analysis depends on tidal or river conditions or both. Next an engineering analysis is used 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 using existing scour equations. Finally, if necessary for complex tidal situations, one- or two-dimensional computer models or even physical models must be used.

10.23 SCOUR CALCULATIONS FOR TIDAL WATERWAYS

Long-term degradation, contraction scour, and local scour can be determined in tidally affected waterways using methods and equations given previously for riverine flows (Richardson and Davis 2001). A brief summary for long-term degradation and contraction scour follows.

10.23.1 Long-Term Degradation

To determine if long-term degradation is occurring, site conditions, fluvial geomorphology, historical data on changes in waterway bed elevation, and potential future changes in the tidal waterway or coastal conditions must be studied to determine if the waterway is aggrading or degrading. If the waterway is degrading, an estimate of the amount of degradation that will occur in the future is made and 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, etc., which could affect waterway stability. Also, determine changes in the riverine environment, such as dams, which could change flow conditions.

In tidal conditions long-term degradation can occur from the daily tides if there is little or no sediment supply to an inlet or estuary or it is decreased (Butler and Lillycrop 1993; Richardson et al. 1993; Vincent 1993). The potential magnitude but not the time of this long-term degradation can be determined using the clear-water contraction scour equation given previously. Richardson and Davis (2001) present an example of the use of the clear-water contraction scour equation to estimate potential long-term degradation.

10.23.2 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 were a substantial quantity of bed material transport in contact with the bed. Equations given previously can be used to determine contraction scour from the daily tidal or storm surge flows. Because the discharge in a contracted tidal waterway depends on the area of the waterway for a given tidal or storm surge amplitude and tidal prism, the discharge will need to be recalculated after the area has increased from contraction scour.

10.23.3 Local Scour

The equations and method given previously for local scour at piers and abutments are used for tidal waterways.

10.24 OVERVIEW OF TIDAL PROCESSES

10.24.1 Glossary of Terms

Bay: A body of water connected to the ocean by 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: Tidelike phenomenon resulting from wind and barometric pressure changes. Hurricane surge, storm tide.
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.
Tidal waterways: A generic term that includes tidal inlets, estuaries, bridge crossings to islands or between islands, crossings between bays, tidally affected streams, etc.
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 an earthquake, or other seismic disturbance, or a submarine landslide.
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.

10.24.2 Definition of Tidal and Coastal Processes

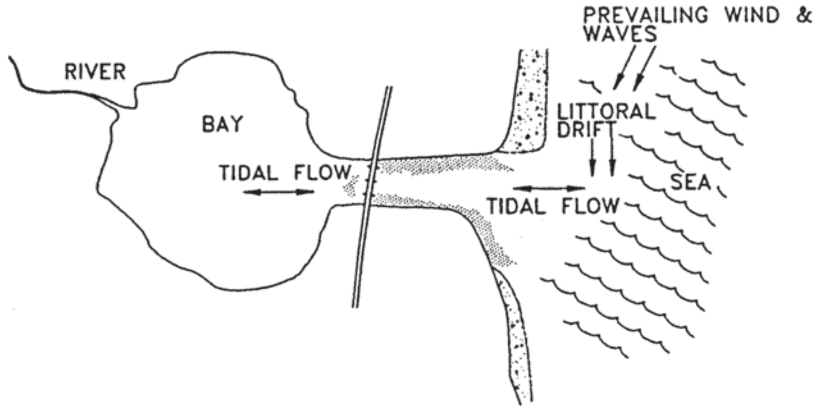
Typical bridge crossings of tidal waterways are diagramed in Fig. 10-20. Tidal flows are defined as being between the ocean and a bay (or lagoon), from the ocean into an estuary, or through passages between islands or between islands and the mainland. Idealized astronomical tidal conditions and tidal terms are illustrated in Fig. 10-21.

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 (tsunami). As illustrated in Fig. 10-21, 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 tsunami occur, the short-term contraction and local scour can be significant. Storm surges and tsunami are single-event phenomena that, due to their magnitude, can cause significant scour at a bridge crossing.

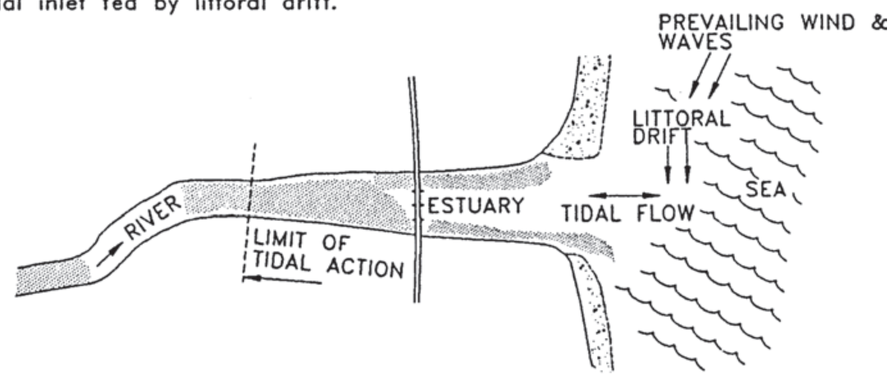
Although the hydraulics of flow for tidal waterways is complicated by the presence of two-directional flow, the basic concept of sediment continuity is valid. Consequently, a clear understanding of the principle of sediment continuity is essential for evaluating scour at bridges spanning waterways influenced 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 (Figs. 10-20 and 10-22) is a source of sediment supply to an inlet, bay estuary, or tidal passage. During flood tide, sediments can be transported and deposited into the bay or estuary. During ebb tide, sediments can be remobilized and transported out of the inlet or estuary and either deposited on shoals or moved further down the coast as littoral drift.

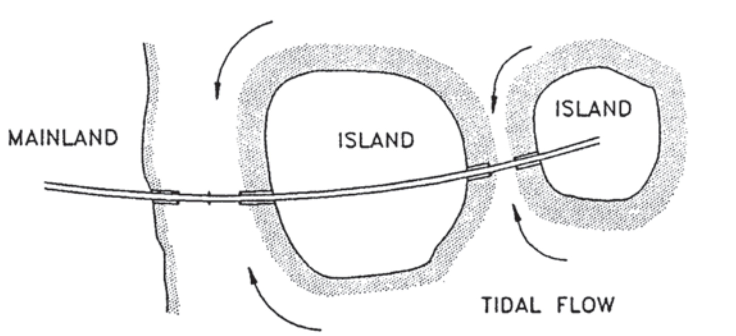
Sediment transported to a bay or estuary from an upland river system can also be deposited in the bay or estuary during flood tide and remobilized and transported through the inlet or estuary during the ebb tide. However, if the bay or estuary is large, sediments derived from the upland river system can be deposited 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 were disrupted. Dredging, jetties, or other coastal engineering activities can limit sediment supply to a reach and influence live-bed and clear-water scour conditions.



1. Inlets between the open sea and an enclosed lagoon or bay, where most of the discharge results from tidal flows. Tidal inlet fed by littoral drift.



2. River estuaries where the net discharge comprises river flow as well as tidal flow **components**



3. Passages between islands, or between an island and the mainland, where a route to the open sea exists in both directions.

Fig.10-20. Types of tidal waterway crossings (after Neill 1973).

10.25 PRELIMINARY ANALYSIS

As a preliminary analysis it is necessary to determine (1) classification of the tidal crossing, (2) tidal characteristics, (3) lateral, vertical, and overall stability of the waterway and bridge foundations, and (4) characteristics of the riverine and tidal flows. In such a 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, etc. are collected and analyzed. In addition, field reconnaissance and contact with agencies such as the Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), U.S. Geological Survey (USGS), U.S. Coast Guard (USCG), U.S. Corps of Engineers (USCOE), state agencies, etc. are used.

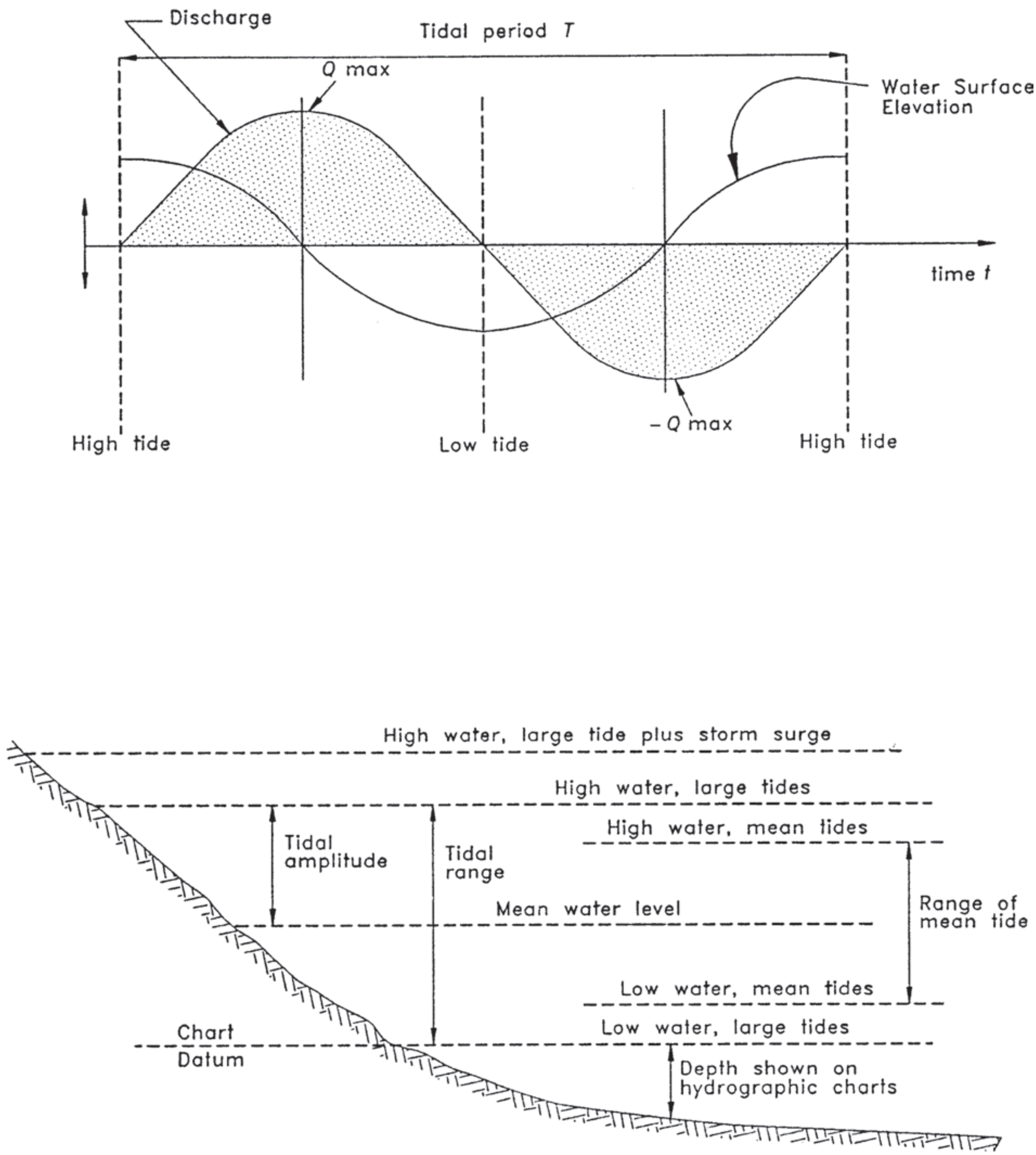


Fig. 10-21. Principal tidal terms (after Neill 1973).

The crossing is classified as an inlet, bay, estuary, or passage between islands or islands and the mainland (Fig. 10-20). 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 the preliminary analysis are to determine the magnitude of the tidal effects on the crossing, the overall long-term vertical and lateral stability of the waterway and bridge crossing, and the potential for waterway and crossing to change.

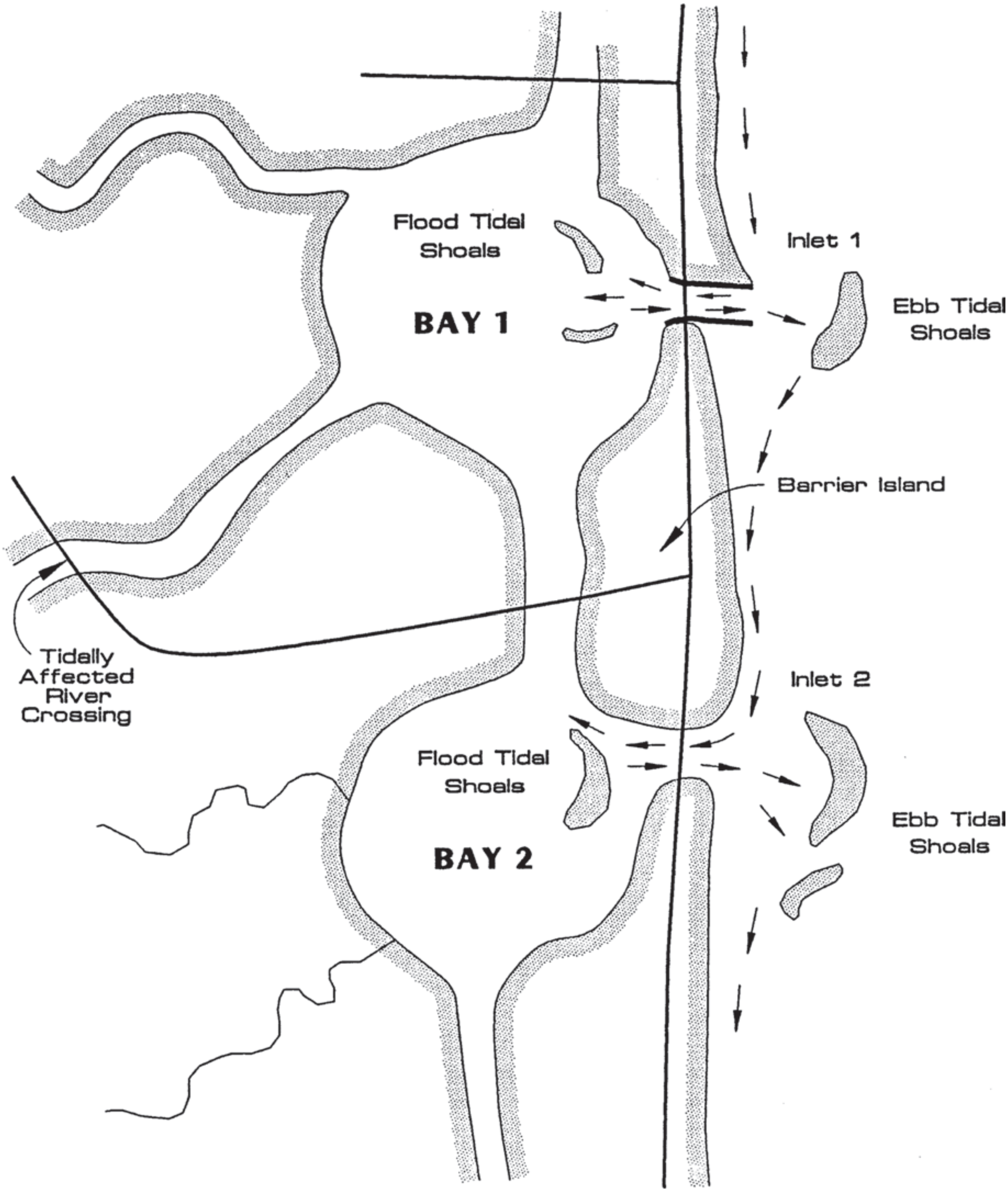


Fig. 10-22. Sediment transport in tidal inlets (after Sheppard 1993).

10.26 DETERMINATION OF HYDRAULIC VARIABLES

The general procedure is to determine (1) design flows (100- and 500-year storm tides and riverine floods), and (2) hydraulic variables of discharge, velocity, and depths. These variables are then used to determine the scour components (depths of degradation, contraction scour, pier scour, and abutment scour) using the equations and methods given previously, followed by (3) evaluation of the results.

10.26.1 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 discharges 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 tide (ebb) or low and high tide (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 tide, 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 (1973) and for a constricted waterway by a method given by Chang et al. (1994).

10.27.2 Hydraulic Variables for Unconstricted Waterways

Richardson and Davis (2001) present Neill's (1973) method as follows:

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 area of the piers.
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.
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 mean water level or a cosine curve which starts at the maximum tide level. The cosine equation is

$$y = A\cos\theta + Z \tag{10-52}$$

where

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

$$\theta = 360\left(\frac{t}{T}\right) \tag{10-53}$$

t = time in minutes from beginning of total cycle;
T = total time for one complete tidal cycle, min;
Z = vertical offset to datum, m, ft.

To determine the elevation versus time relation for the 100- and 500-year storm tides, the tidal range and period must be known. The 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 different periods than astronomical semidiurnal and diurnal tides, which have periods of approximately 12.5 and 25 h, 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, path of the hurricane or storm creating the storm tide, freshwater inflow, shape of the bay or estuary, etc. influence the storm tide amplitude and period.

4. Determine the discharge, velocities, and depth. The maximum discharge, in an ideal tidal estuary, may be approximated by the equation (Neill 1975)

$$Q_{max} = \frac{3.14\,\mathrm{VOL}}{T} \tag{10-54}$$

where

Q_{max} = maximum discharge in the tidal cycle, cms, cfs;
VOL = volume of water in the tidal prism between high and low tide levels, m^3, ft^3;
T = tidal period between successive high or low tides, s.

In the idealized case, Q_{max} occurs in the estuary or bay at mean water elevation and at a time midway between high and low tides when the slope of the tidal energy gradient is steepest (Fig. 10-21). In many field cases, Q_{max} occurs 1 or 2 h before or after the crossing, but any error caused by this is diminutive.

The corresponding maximum average velocity in the waterway is

$$V_{max} = \frac{Q_{max}}{A'} \tag{10-55}$$

where

V_{max} = maximum average velocity in the cross section at Q_{max}, m/s, ft/s;
A' = cross-sectional area of the waterway at mean tide elevation, halfway between high and low tide, m^2, ft^3.

The average velocity must be adjusted to determine velocities at individual piers to account for 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:

$$Q_t = Q_{\max} \sin\left(360\frac{t}{T}\right) \tag{10-56}$$

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 to the tidal flows), to routing the riverine flow through the crossing.
6. Evaluate the discharge, velocities and depths that were determined in Steps 4 and 5.
7. Determine scour depths for the bridge using the values of the discharge, velocity and depths determined from the above analysis.

10.26.3 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 (1967) and Bruun (1990) reported by Richardson and Davis (2001) can be used:

$$V_{\max} = C_d\left(2g\Delta H\right)^{1/2} \tag{10-57}$$

$$Q_{\max} = A'V \tag{10-58}$$

where

$V_{\max}$ = maximum velocity in the inlet, m/s, ft/s;
$Q_{\max}$ = maximum discharge in the inlet, cms, csf;
C_d = coefficient of discharge ($C_d < 1.0$);
g = acceleration due to gravity, 9.81 m/s^2, 32.2 ft/s^2;
ΔH = difference in water surface elevation between the up- and downstream sides of a crossing or channel for the 100- and 500-return period storm surges as well as for the normal astronomical average tides. —this 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 (*3*), m, ft;
A' = net cross-sectional area at the crossing, at mean water surface elevation, m^2, ft^2.

The coefficient of discharge (C_d) is:

$$C_d = \left(\frac{1}{R}\right)^{1/2} \tag{10-59}$$

where

$$R = K_u + K_d + \frac{2g\, n^2\, L_c}{1.49^2\, h_c^{4/3}} \tag{10-60}$$

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, ft;
h_c = average depth of flow at the bridge at mean water elevation, m, ft.

If ΔH is not known, the following method, developed by Chang et al. (1994), 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 ($Q_{\text{tide}} = A_s\, dH_s/dt$). This total flow approaching the bridge is set equal to the flow calculated from the orifice equation,

$$Q + A_s \frac{dH_s}{dt} = C_d A_c \sqrt{2g\Delta H} \tag{10-61}$$

where

A_c = bridge waterway cross-sectional area, m^2, ft^2.

The other variables have been defined previously.

Equation (10-52) may be rearranged into the form of Eq. (10-53) for the time interva, $\Delta t = t_2 - t_1$, subscripts 2 and 1 representing the end and beginning of the time interval, respectively. Then

$$\frac{Q_1+Q_2}{2} + \frac{A_{s1}+A_{s2}}{2}\,\frac{H_{s1}-H_{s2}}{\Delta t} = C_d\left(\frac{A_{c1}+A_{c2}}{2}\right)\sqrt{2g\left(\frac{H_{s1}+H_{s2}}{2} - \frac{H_{t1}+H_{t2}}{2}\right)} \tag{10-62}$$

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. Because 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 Eq. (10-62), which can be determined by trial and error to balance the values on the right and left sides.

Chang et al. (1994) suggest the following for computing the discharge:

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.

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 elevation.
3. Plot bridge waterway area versus elevation.
4. Determine the quantity of riverine flow that is expected to occur during passage of the storm tide through the bridge.
5. Route the flows through the contracted waterway using Eq. 10-62 and determine the maximum velocity of flow.

Chang et al. (1994) give an example problem using a spreadsheet and have developed a computer program to aid in using this method. Richardson and Davis (1995) also give the sample problem and list the computer progress.

10.26.4 Hydraulic Variables Using Computer Programs

A Federal Highway Administration Pooled Fund study funded by the Connecticut, Florida, Georgia, Louisiana, Maine, Maryland, Mississippi, New Jersey, New York, North Carolina, South Carolina, and Virginia Departments of Transportation of computer models to analyze tidal stream hydraulic conditions at highway structures recommended a one-dimensional unsteady flow model entitled UNET (Burkau 1993; USACE 1996) and a two-dimensional unsteady flow model entitled FESWMS (Froehlich 1996). The studies were carried out by Ayres Associates, Inc., and Edge & Associates, Inc, with William H. Hulbert, North Carolina DOT, as project manager (Ayres Associates 1994; Zevenbergen et al. 1997a; 1997b). The use of FESWMS was enhanced by the FHWA-supported development of a graphical user interface called the Surface Water Modeling System (SMS) (Brigham Young University 1997). The interface develops two-dimensional model networks, run control, variable assignment, and output analysis for FESWMS. Both models proved themselves under a wide range of field tidal conditions.

Methods for predicting storm surge hydrographs using peak storm surge elevation and hurricane characteristics (radius of maximum winds and forward speed) for the 50-, 100-, and 500-year hurricanes are included. Zevenbergen et al. (1997a, b) contain methods and procedures for using UNET, FESWMS, and SMS and developing of storm surge hydrographs.

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