Estimating Scour at Bridges

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Methods of estimating scour at bridges and abutments based on recommendations in the FHWA 1988 Technical Advisory on Scour [1] are presented. Estimating abutment scour is discussed based on specific recommendations in FHWA's publication HEC-18. Additionally, research results on pressure flow developed at Colorado State University by Lila Abed, and on the effect of wide footings on scour depths developed at the Fairbanks Turner Hydraulics Laboratory by Sterling Jones are presented. Also discussed are the effect of angle of attack on shape factor of a pier and on rows of columns and the top width of scour holes. Knowledge of the top width of scour holes is important in determining if scour at piers and abutments are independent of each other, and for designing riprap protection.

Total Scour at a highway crossing is comprised of three components. These components are:

1. Aggradation and Degradation. These are long-term streambed elevation changes due to natural or man induced causes within the reach of the river on which the bridge is located.
2. Contraction Scour. Contraction scour results from a contraction of the flow, a change in downstream control of the water surface elevation, or the location of the bridge in relation to a bend. Contraction of the flow by the bridge approach embankments encroaching onto the floodplain and/or into the main channel is the most common form of contraction scour.
3. Local Scour. Local scour occurs around piers, abutments, spurs, and embankments, and is caused by the acceleration of the flow and the subsequent development of vortex systems induced by these obstructions to the flow.

In addition to the types of scour mentioned above, *lateral movement or shifting of the stream* may also erode the approach roadway to the bridge or change the total scour by changing the alignment of the flow in the waterway at the bridge crossing.

In 1988, the Federal Highway Administration issued a Technical Advisory on Scour [1]. In the advisory, interim procedures for evaluating scour at bridges were given. These procedures were based on a detailed evaluation of the best available knowledge of scour. Since issuance of the TA, researchers, as well as state and federal agencies, have been active in the improvement and verification of predictive methods to determine scour depths. Some of this activity is in the form of fundamental research in the laboratory and field. One of the most promising programs is being conducted by various state agencies who have been actively evaluating existing bridges and assessing their vulnerability to scour.

From this resurgence of interest in scour at bridges has come a wealth of new literature concerning bridge scour. These studies have sought to improve upon the equations in the TA, as well as shed new light on factors which could have an aggravating or mitigating influence on the total scour at bridges. In particular, these efforts have:

1. questioned whether the scour depths predicted by the recommended equations in the TA for abutments and piers are too deep;
2. sought to determine whether, and to what extent, larger bed material sizes (cobbles) decrease scour;
3. sought to determine the effect of footings or multiple piles on scour depths when they are exposed by the flow;
4. sought to determine the width of the scour hole around piers and abutments; and to determine to what extent pressure flow influences total scour.

For example, research by Raudkivi and Ettema [2], Raudkivi [3], Copp et. al. [4], and, Melville and Sutherland [5] indicates that large particles in the bed-material decrease scour depths. However, field verification is insufficient to incorporate these studies into a recommended procedure that takes bed-material size into account at this time (FHWA [6] and Richardson and Richardson [7]).

Concerning abutment and pier scour, new studies indicate that the equations for abutment scour are conservative, and can be used to estimate the worst-case scenario. For pier scour, the CSU equation still appears to give the most likely estimate of scour depths. Research by Jones [8] reveals a method to evaluate the effect of wide footings and pile groups, while very recent research by Lila Abed shows that contraction, abutment and pier scour are significantly increased with pressure flow. In some tests, these increases are 3 times what clear water, free surface flow scour would be.

It should be noted that most of these questions still do not have definitive answers. However, the recent research, highlighted above, needs to be brought forward and placed in the context of the original TA. It is for this reason that the discussions of the above topics will be included, where relevant, in a general review of methods and equations for determining total scour at a bridge. This review will begin by outlining the basic design approach, followed by a more detailed discussion of the methods and equations to be used for each of the design steps.

DESIGN APPROACH

Before the various scour forecasting methods for contraction and local scour can be applied, it is first necessary to 1) obtain the fixed-bed channel hydraulics; 2) estimate the long-term profile degradation or aggradation; 3) adjust the fixed-bed hydraulics to reflect these changes and 4) compute the bridge hydraulics. If contraction scour is large (larger than 4 ft.), it may be necessary to adjust the fixed-bed hydraulics before computing local scour.

The steps recommended for estimating scour at bridges as given in the Technical Advisory [1], are summarized below and discussed in greater detail after this general summary.

STEP 1. Determine scour analysis variables.
STEP 2. Analyze long-term bed elevation change.
STEP 3. Compute the magnitude of contraction scour.
STEP 4. Compute the magnitude of local scour at abutments.
STEP 5. Compute the magnitude of local scour at piers.

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STEP 6. Plot the total scour depths.
STEP 7. Evaluate the total scour depths.
STEP 8. Reevaluate the bridge design as necessary.

DETAILED PROCEDURES

Determine Scour Analysis Variables

1. From a flood frequency study, determine the magnitude of the discharges for the floods to be used for design, including the overtopping flood, when applicable. If the magnitude of the 500 year flood is not available, multiply the Q_{100} by 1.7. This multiplier may be modified if hydrology studies indicate another value may be more appropriate. Experience has shown that the incipient overtopping discharge often puts the most stress on a bridge. However, special conditions such as ice jams, debris, and other factors may cause a more severe condition for scour with a flow smaller than the overtopping flood or, floods with a return period of 100 or 500 years.
2. Determine the water surface profiles for the design discharge using the FHWA/USGS program WSPRO or other existing methods that employ the FHWA bridge analysis procedure. Various conditions and discharges should be used to determine the worst combinations of scour conditions that could occur at the bridge. The engineer should anticipate future conditions at the bridge, in the stream's watershed, and with various downstream water surface elevation controls.
3. Determine if there are existing or potential future factors that will produce a combination of high discharge and low tail water downstream. Check for bedrock or other controls (old diversion structures, erosion control checks, other bridges, etc.) that might be removed in the future. In some cases, dams or locks downstream can control the tailwater elevation on a seasonal basis. Dams upstream or downstream can directly influence the water surface elevation at the bridge. As a rule, the lowest reasonable downstream water surface elevation and the largest discharge should be used to estimate the greatest scour potential. Assess the distribution of the velocity and discharge per foot of width for the design flow and other flows through the bridge opening. Consider also the approach flow and the flow distribution downstream (the contraction and expansion of the flow). The designer must also consider conditions and anticipated future changes in the river.
4. From the output of the above computer programs and from other hydraulic studies, determine the critical discharges, velocities, and other hydraulic parameters which are most critical to the structural stability of the bridge. These parameters are needed for computation of the scour depth.
5. Any additional pertinent information concerning the bridge crossing and the stream requires collection and assimilation prior to assessing the scour at the bridge. This information is essential to the proper estimation of scour depth and includes, but is not limited to, the following types of information:
   - Bore hole logs;
   - Bed material size distribution in the bridge reach;
   - Existing stream and floodplain cross section;
   - Stream geomorphic plan form;
   - Watershed characteristics;
   - Scour data on other bridges in the area;
   - Slope of energy grade line upstream and downstream of the bridge;
   - Estimation of the bed material sediment discharge for flood discharges (flood discharges are mean annual, and 5, 10, 20, 50, 100 and 500 year frequencies). Use Colby's method for sand-bed streams and the Meyer-Peter, Muller equation for coarse-bed-material streams [10];
   - History of flooding;
   - Location of bridge site with respect to other bridges in the area, tributaries to the stream close to the site, bed-rock controls, man-made controls (dams, old check structures, river training works, etc.), and downstream confluences with another stream;
   - Character of the stream (perennial, flashy, intermittent, gradual peaks, etc.);
   - Geomorphology of the site (floodplain stream; crossing of a delta, youthful, mature or old age stream; crossing of an alluvial fan, etc.);
   - Erosion history of the stream;
   - Development history (past, present and future) of the stream and watershed. Collect maps, photographs, areal photographs, interviews of local residences, and planned or contemplated water research projects;
   - Sand and gravel mining in the streambed up or downstream from site;
   - The potential for stream movement and its effect on the bridge, and;
   - Any other factors that could affect the bridge.

Analysis Of Long-Term Bed Elevation Change

Using the information collected above, determine qualitatively the long-term trend in the streambed elevation. Where conditions indicate that significant aggradation or degradation is likely, estimate the change in bed elevation over the next 100 years using one or more of the following techniques:

1. Available computer programs such as HEC 6 from The Corps of Engineers;
2. Straight line extrapolation of present trends;
3. Engineering judgment; and
4. Using the worst-case scenarios, i.e., in the case of a confluence with another stream immediately downstream of the bridge, assume the design flood would occur with a low downstream water surface elevation using a qualitative assessment of the joint probability of flood magnitudes and river conditions on the main stream and its tributary.

If the stream is aggrading, and this condition can be expected to affect the crossing, consider countermeasures including a relocation of the bridge. This should be done taking into account contraction scour. If the stream is degrading, then use the change in bed elevation in the calculations of total scour.

Scour Analysis Method

The recommended method to estimate scour is as follows:

- estimate the natural channel's hydraulics for a fixed-bed condition based on existing conditions;
- assess the expected profile and plan form changes;
- adjust the fixed-bed hydraulics to reflect any expected long-term profile or plan-form changes;
- estimate contraction scour using the empirical contraction formula and the adjusted fixed-bed hydraulics;
- estimate local scour using the adjusted fixed-bed channel and bridge hydraulics; and
- add the local scour to the contraction scour to obtain the total scour.

Contraction Scour

Types of Contraction Scour

Contraction scour can be caused by different bridge site conditions. There are four main conditions (cases) which are as follows:

Case 1.Overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge.

1a.The river channel width becoming narrower either because of the bridge abutments projecting into the channel or the bridge being on a narrower reach of the river.

1b.Abutments set at the stream bank.

1c.Abutments set back from the stream channel.

Case 2.The normal river channel width becoming narrower either because of the bridge itself or by the bridge site being on a narrower reach of the river.

Case 3.A relief bridge in the overbank area with little or no bed material transport in the overbank area.

Case 4.A relief bridge over a secondary stream in the overbank area.

Estimating Contraction Scour

Case 1. Contraction Scour, Stream With Over Bank Flow. Laursen's [9] equation (Equation 1) is recommended to predict the depth of scour in the contracted section for case 1a and 1b. Note that the average contracted depth is the difference between the flow depth upstream, y_1 and the depth in the contracted section, y_2.

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{K_1} \left(\frac{n_2}{n_1}\right)^{K_2} \qquad (1)$$

where

y_1 =Average depth in the main channel,

y_2 =Average depth in the contracted section,

W_1 =Width of the main channel,

W_2 =Width of the contracted section,

Q_c =Discharge in the main channel upstream of the bridge,

Q_t= Discharge in the contracted section,

n_1= Manning n in the main channel,

n_2= Manning n in the contracted section

The powers K_1 and K_2 can be related to the transport factor, e, which can be related to the ratio of the shear velocity, V_{*_c}, to the fall velocity of the bed material, w.

For most cases, Table 1 can be used as a guide to determining the appropriate values of K_1 and K_2.

TABLE 1 K_1 and K_2 FOR CONTRACTION SCOUR

$\frac{V_{*_c}}{w}$	e	K_1	K_2	Mode of Bed Material Transport
< 0.5	0.25	0.59	0.066	Mostly Contact Bed Material
1.0	1.0	0.64	0.21	Some Suspended Bed Material
> 2.0	2.25	0.69	0.37	Mostly Suspended Bed Material

For values of e not listed in the table, interpolation to find the appropriate transport factor provides a method of determining K_1 and K_2. This entails computation of the shear velocity using:

$$V_{*_c} = (g y_1 S_1)^{0.5}$$

where

S_1=Slope of the Energy Grade Line in the Main Channel,

g = Acceleration of Gravity (32.2 Ft/s^2).

The exponents K_1 and K_2 can be determined by division of V_{*_c} by w and interpolation to find the proper value of e. Using this value of the transport factor, K_1 and K_2 can be computed using the following relationships.

$$K_1 = \frac{6(2+e)}{7(3+e)}$$

$$K_2 = \frac{6e}{7(3+e)}$$

It should be noted that for this case (Case 1a and 1b):

1.The Manning n ratio can be significant when the main channel is composed of dunes with the contracted channel being plain bed, washed out dunes, or antidunes [10].

2.The average width of the bridge opening is normally taken as the bottom width with the width of the piers subtracted.

3.Laursen's equation for a long contraction will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or if the contraction is the result of the bridge abutments and piers. At this time however, this equation is the best available.

4.The fall velocity for sand-sized material can be estimated given the grain size of the material by using Figure 1.

Case 1c: is complex, and there is no single equation available to determine contraction scour for this case. The depth of contraction scour depends on many factors such as: how far back from the bank line the abutment is set, the condition of the bank (erodibility, vegetation, bank height, etc), width at the bridge in relationship to the upstream section, the amount of the overbank flow returning to the bridge opening and other influences. Therefore, it is recommended that variables to be used in Equation 1 be carefully selected, coupled with engineering judgement, when estimating contraction scour.

For example, contraction scour in the stream bed of the main channel underneath the bridge could be estimated using Equation 1 by substituting the increase in discharge in the main channel under the bridge for Q_t which can be obtained using WSPRO. Furthermore, the width under the bridge in the main channel would be used for W_2.

The contraction scour in the overbank area under the bridge can be estimated using Equation 2 by substituting the ratio of the overbank flow upstream of the bridge to the overbank flow at the bridge for W_1/W_2. These discharges can also be obtained using WSPRO. It should be noted that Laursen's abutment scour equation for the case of the abutment set back from the main channel gives the sum of the contraction and local scour.

Case 2. Contraction Scour, No Overbank Flow. This case applies where there is no overbank flow, but the stream channel narrows either naturally or by the bridge abutments encroaching on the channel. Laursen's equation described for Case 1 using $Q_t = Q_c$ is applicable in this case. If the decrease in width W_2 is less than 10 percent, then neglect any width change.

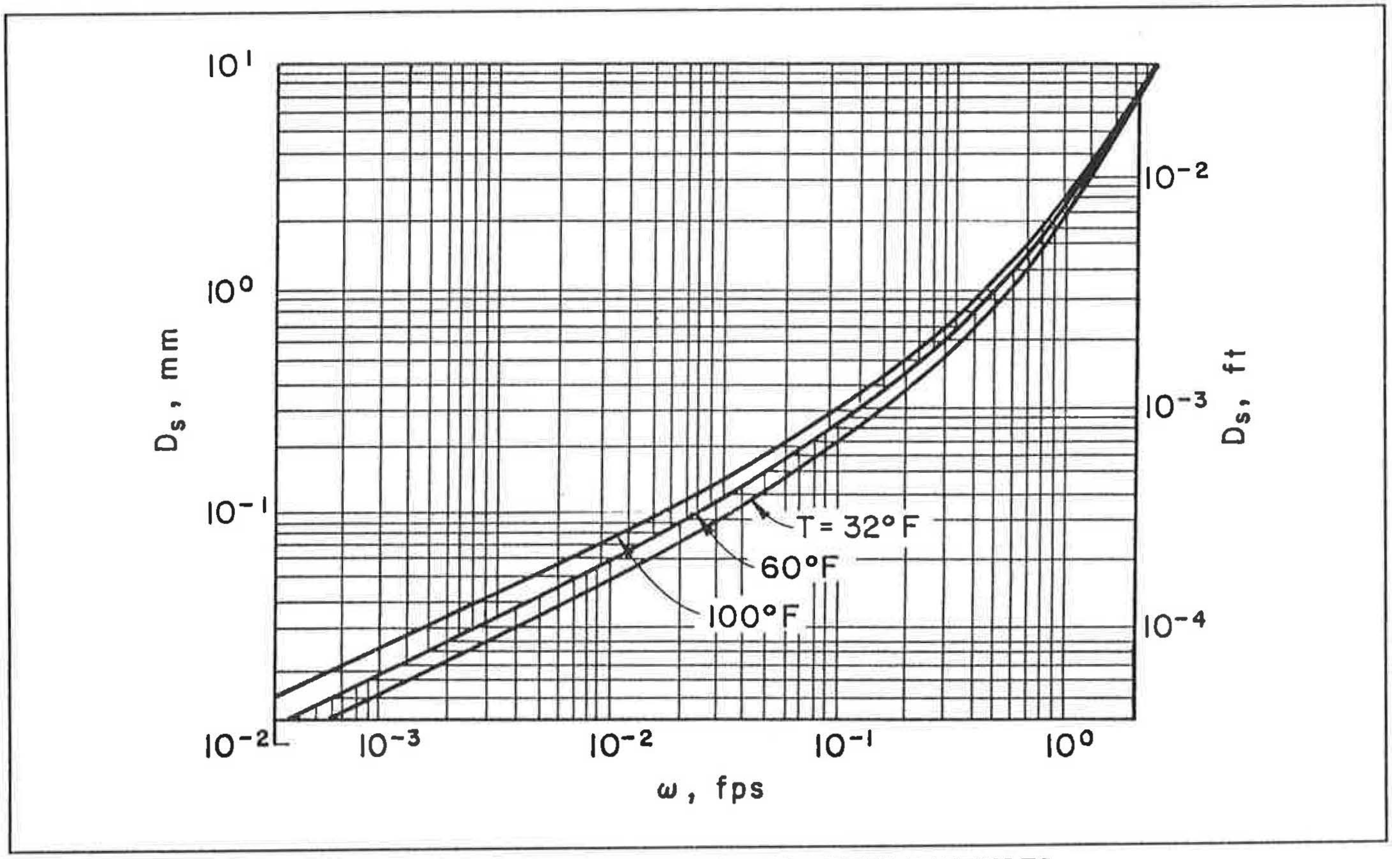

FIGURE 1. FALL VELOCITY OF SAND-SIZED PARTICLES

Case 3. Contraction Scour, Relief Bridge With No Bed-Material Transport. Case 3 applies to a relief bridge on a floodplain where there is no bed-material transport (i.e., clear-water scour). Laursen's [11] equation given below is applicable for this case.

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{\frac{6}{7}} \left[\frac{V_1^{\,2}}{120 y_1^{\frac{1}{3}} D_{50}^{\frac{2}{3}}}\right]^{\frac{3}{7}} \tag{2}$$

In Equation 2, most of the variables were defined previously. However, for this equation, the subscripts 1 and 2 refer to upstream conditions and to the width and depth at the relief bridge respectively. Furthermore:

W_1 = Width upstream of the relief bridge in feet. This can be estimated by assuming a point of stagnation between the main and relief bridge,

V_1 = Average velocity on the floodplain, in ft/s, one bridge length upstream of the river crossing,

D_{50} = Median diameter of the bed material, in feet, at the relief bridge.

Case 4. Contraction Scour, Relief Bridge With Bed-Material Transport. Case 4 applies to a relief bridge with bed-material transport. This case can occur when a relief bridge is over a secondary channel on the floodplain. It is recommended that Equation 1, which was described for Case 1 be used with appropriate adjustments of the variables.

Other Contraction Scour Considerations

Contraction scour resulting from variable water surface downstream of the bridge is analyzed by determining the lowest potential water surface elevation downstream of the bridge in so far as scour processes are concerned. It is recommended that a universally accepted water surface program such as the U.S. Corps of Engineers HEC 2 or the FHWA/USGS WSPRO (WSPRO) program be used to determine the flow variables such as velocity and depth through the bridge. With these variables, determine contraction and local scour depths.

Contraction scour resulting from the flow through the bridge being concentrated in one area is analyzed by determining the superelevation of the water surface on the outside of the bend and estimating the resulting velocities and depths through the bridge. The maximum velocity in the outer part of the bend can be 1.5 to 2 times the mean velocity. A physical model study can also be used to determine the velocity and scour depth distribution through the bridge for this case.

Estimating contraction scour for unusual situations involves particular skills in the application of principles of river mechanics to the specific site conditions, and such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

Local Scour At Abutments

General

Most equations for predicting local scour at abutments, including those by Liu, et al [12], Laursen [11] and Froehlich [13] are based, almost entirely, on laboratory data. The lack of field data for derivation or verification of abutment scour inhibits the accuracy and reliability of these equations. Still, these equations represent the best available technology at the present time.

Liu et al's [12], equations were developed by dimensional analysis of the variables, and a best-fit line was drawn through the laboratory data. Laursen's equations [11] are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment. Froehlich's equations [13], are derived from regression analysis of the available laboratory data. Because of the manner that these equations were developed, and their inherent uncertainties, they are conservative and should be used to consider the worst-case condition of abutment scour.

Because of the methods by which experiments were conducted to obtain these equations, they will only predict the maximum scour that could occur for an abutment projecting out in a stream if the velocities and depth upstream of the abutment are similar to those in the main channel. However, the way the experiments were conducted do not represent many of the conditions typically encountered in the field. Field conditions may have tree lined or vegetated banks with low velocities and shallow depths upstream of the abutment. With overland flow, depths are often shallow in the overbank, with low velocities and little to no bed-material transport.

Because of the discrepancies between the laboratory data and field conditions, engineering judgement is required. In many cases, foundations can be designed with shallower depths than given by the equations provided the foundations are protected with riprap placed below the streambed. Alternatively, a spur dike (guide bank) placed upstream of the abutment can be used instead of riprap.

Abutment Site Conditions

Abutments can be set back from the natural streambank or can project out into the flow. They can have various shapes, and can be set at an angle to the flow. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow that are intercepted by the approaches to the bridge and returned to the stream at the abutment. All of these factors affect abutment scour.

Scour at abutments can be divided into seven cases. These cases are given in Table 2. However only Laursen [11] has developed equations for all these cases. His equations are based on transport relations, but have not as yet been fully verified in the field. Laursen's equations, as well as Liu, et al's [12], Froehlich's [13] and one presented in Highways in the River Environment (HIRE [10]) for case 6, are given in FHWA [10] and HEC-18 [6].

Abutment Shape

There are two general shapes for abutments. These are vertical wall abutments, with or without wing walls, and spill-through abutments. The depth of scour is about double for vertical wall abutments without wing walls, as compared with spill-through abutments (see Liu et al's equation in HIRE [10] or Froehlich's correction term for abutment shape given later in this section).

Design for Scour at Abutments

It is recommended that abutment foundation depths be set by AASHTO [14] and [15] standards, and that they be protected with riprap designed according to HEC-11 [16] or by the methods given in HIRE [10], and/or be protected by spur dikes designed as per FHWA instructions. The reason for this recommendation is as stated at the start of this section. That is, the available equations give the worst-case scour, and there little to no field verification of these equations.

As a check on the potential depth of scour to aid in the design of the foundation and placement of riprap or spur dikes, the equation for live-bed scour developed by Froehlich [13] (Equation 3) is recommended. This equation was developed after analysis of 170 live-bed scour measurements in laboratory flumes.

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{a'}{y_a}\right)^{0.43} Fr_e^{\,0.61} + FS \qquad (3)$$

where

y_a = Depth of Flow at the Abutment

y_s = Abutment Scour Depth

K_1 = Coefficient for Abutment Shape. For vertical abutments, $K_1 = 1.0$, for vertical abutments with wing walls, $K_1 = 0.82$, for spill-through abutments, $K_1 = 0.55$.

$K_2 = \left(\frac{\theta}{90}\right)^{0.13}$ Which is a Coefficient for angle of embankment to flow. Where Θ is equal to the angle between the approach embankment and a line drawn normal to the main flow. As a convention, Θ is less than 90 degrees if the embankment points downstream and greater than 90 degrees if the embankment points upstream.

$a' = \frac{A_e}{y_1}$ = Projected length of Abutment normal to the flow. For this variable, A_e is the flow area of the cross-section which is obstructed by the embankment.

$Fr_e = V_e/(g y_1)^{0.5}$ = Froude number of the approach flow upstream of the abutment; $V_e = Q_e/A_e$ and;

Q_e = The flow obstructed by the abutment and approach embankment.

The factor of safety, FS, is given in Table 3 and is derived based on the measured laboratory abutment scour. As an illustrative example, a factor of safety of 1.0, 98 percent of the measured laboratory abutment scour depths were less than that predicted using Froehlich's equation.

TABLE 2 SUMMARY OF ABUTMENT SCOUR CASES

CASE	ABUTMENT LOCATION	OVERBANK FLOW	$\frac{a}{y_1}$	BED LOAD	ABUTMENT TYPE
1	Projects Into Channel	No	< 25	Live Bed Clear Water	Vertical Wall or Spill through
2	Projects Into Channel	Yes	< 25	Live Bed Clear Water	Vertical Wall or Spill through
3	Set Back from Main Channel	Yes	< 25	Clear Water	Vertical Wall or Spill through
4	Relief Bridge on Floodplain	Yes	< 25	Clear Water	Vertical Wall or Spill through
5	At Edge of Main Channel	Yes	< 25	Live Bed	Vertical Wall or Spill through
6	Not Designated	Yes	> 25	Not Designated	Spill through
7	Skewed to Stream	---	---	---	---

From personal contact with Froehlich, he has suggested that the abutment scour depth be increased by one sixth of y_a if there are dunes in the main channel upstream of the abutment. From the authors experience and from Jain and Fischer's [19] research we recommended that no increase in the abutment scour depth for either plain-bed or antidune regimes.

TABLE 3 FACTOR OF SAFETY FOR ABUTMENT SCOUR

FS	PERCENT OF LABORATORY SCOUR DATA LESS THAN PREDICTED USING EQUATION
0.00	44.6%
0.25	72.4%
0.50	91.4%
0.75	95.2%
1.00	98.0%
1.50	99.4%
2.00	99.4%
2.50	100.0%

Clear-water Scour at an Abutment Froehlich [13] also developed a clear-water abutment scour equation based on dimensional analysis and multiple regression analysis of 164 clear-water scour measurements in laboratory flumes. However, it is not recommended for use because the potential decrease in scour depth at abutments resulting from coarser material is not known. Although not recommended, this equation is given in HIRE [10] AND HEC-18 [6], and may be useful for use by researchers.

Local Scour At Piers

General

Local scour at piers is a function of bed-material size, flow characteristics, fluid properties and the geometry of the pier. There is also some evidence that pile caps, pile groups, or exposed footings influence local scour at piers. The subject has been studied extensively in the laboratory, but there is only limited field data. As a result of these many studies, there are an equal amount of equations. In general, the equations are for live-bed scour in cohesionless, sand-bed streams.

For the determination of pier scour, HEC-18 [6] and the Technical Advisory [1] recommends the use of the Colorado State University equation for both live-bed and clear-water scour. With a dune-bed configuration, the equation predicts equilibrium scour depth. Therefore, when dunes are present, the maximum scour will be 30% greater than that predicted by the CSU equation. For flow with plane-bed configuration or antidunes, from analysis of Jain and Fischer's [19] research, CSU's equation will predict the maximum scour. Furthermore, from a study of Jain and Fischer's [19] research and from the author's field and laboratory experience with plane-bed and antidune flow, the trough of the antidune is not a factor when considering local scour at piers.

CSU's equation, as with most equations, does not take into account the possibility that larger sizes in the bed material could armor the scour hole. Raudkivi and others [2,3,4 and 5] developed equations that take into consideration large particles armoring the bed. However, the significance of armoring of the scour hole over a long time frame and over many floods is not known. Therefore, equations which consider large bed-material armoring are not recommended for use at this time. However, for the researchers needs an equation based on Raudkivi's work is given in HIRE [10], HEC 18 [6] and Richardson, and Richardson [7].

Computing Pier Scour

The Colorado State University's equation (Richardson et al) [10] is as follows:

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\ 0.43} \tag{4}$$

where

y_1 = Depth of Flow directly upstream of the pier,

y_s = Scour Depth at the pier,

K_1 = Correction coefficient for pier type (Table 4),

K_2 = Correction coefficient for angle of piers skewed to the flow (Table 5),

a = Pier width.

$Fr_1 = V_1/(gy_1)^{0.5}$ = Froude number of the approach flow upstream of the pier using, the velocity in front of the pier for V_1.

The subscript 1 used for variables in the CSU equation refers to the depth and velocity in the flow zone in front of the pier. These values will be larger than the average stream velocity in most cases. In a straight, uniform channel, V_1 would probably be about 20% greater than the average velocity in the thalweg. In bends, V_1 can be as much as 50% to 100% greater than the average velocity in the thalweg.

When piers are not skewed to the flow, $K_2 = 1$ and K_1 can be determined using values tabulated in Table 4. For skewed piers, no correction for pier shape should be attempted. Therefore, for skewed piers set $K_1 = 1$ and select K_2 from values tabulated in Table 5. Note that the value of K_2 depends on the ratio of the pier length, L, to the width of the pier, a.

TABLE 4 K_1, FOR PIER TYPE

Pier Type	K_1
Square Nose	1.1
Round Nose	1.0
Cylinder	1.0
Sharp Nose	0.9
Group of Cylinders	1.0

TABLE 5 K_2, FOR ANGLE OF ATTACK

Angle	$\frac{L}{a} = 4$	$\frac{L}{a} = 8$	$\frac{L}{a} = 12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

For a group of cylinders skewed to the flow, the scour depth depends on the spacing of the piers. If the spacing was zero, meaning the piers were touching, the scour would resemble a single solid pier. However, for piers spaced further apart, there is no good method to estimate the scour depth. Reference is made here to Raudkivi [3], who in discussing effects of alignment states *...the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder...* It is clear from this discussion that when groups of columns are skewed to the flow, engineering judgement must be employed in determining the maximum pier scour.

For quick estimation of scour depths, the Technical Advisory [1] recommends equations based upon the CSU equation. Equation 5 is valid for Froude numbers less than or equal to 0.9, and Equation 6 is recommended for Froude numbers larger than 0.9. Note that these equations are identical, except for the coefficient.

$$y_{s_{max}} = 2.3 K_2 a \tag{5}$$

$$y_{s_{max}} = 2.9 K_2 a \tag{6}$$

Pier Scour for Exposed Footings

Pier footings or pile caps can become exposed to the flow by scour. This may occur either from long-term degradation, contraction scour, local scour, or lateral shifting of the stream. In practice, computations of local pier scour depths for exposed footings uses the footing width instead of the pier width to determine the scour depth. This practice is usually too conservative. For example, calculations of scour depths for the Schoharie Cr. bridge failure were closer to the measured model and prototype scour depths when pier width was used rather than footing width [17]. In this case, where the footing top was at the elevation of the bed surface, the calculated depths using the footing width were 47% larger than the measured scour depths.

Furthermore, a recent model study of scour at the Acosta bridge at Jacksonville, Florida, by Jones [8] found that when the top of the footing was flush with the stream, bed local scour was 20 percent less than when; 1) the bottom of the footing was at the bed surface; 2) when the top of the footing was at the water surface with the pile group exposed; and 3) the top of the footing protruded vertically up into the flow approximately half the flow depth. In a generalized study, he found that a footing with a lip extending upstream of the pier, reduced pier scour when the top of the footing was located flush or below the bed. However scour depths were deeper and larger in proportion to the extent that this footing projected up into the flow field. It is clear that stream degradation can expose footings if they are not placed deep enough below the streambed. For new bridges, it is recommended that , as a minimum, the top of footings be set a minimum of three feet below the sum of the long-term bed degradation and the contraction scour.

Based on this study, Jones states:

"It is recommended that the pier width be used for the value of ' a ' in the pier scour equations if the top of the footing is at or below the streambed (taking into account contraction scour). If the pier footing extends above the streambed, make a second computation using the width of the footing for the value of a and the depth and average velocity in the flow zone obstructed by the footing for the ' y ' and ' V ' respectively in the scour equation. Use the larger of the two scour computations"

To determine ' V ' obstructed by the flow, Jones recommends Equation 7. Using this approach, the values of V_f and y_f would be used in the CSU equation presented previously.

$$\frac{V_f}{V_1} = \frac{\ln\left(\frac{10.93 y_f}{k_s} + 1\right)}{\ln\left(\frac{10.93 y_1}{k_s} + 1\right)} \tag{7}$$

Where:

V_f = Average velocity in the flow zone below the top of the footing.

y_f = Distance from the bed to the top of the footing.

k_s = The grain roughness of the bed (usually the D_{84} of the bed material).

Pier Scour for Exposed Pile Groups

Jones [8] also conducted experiments to determine guidelines for specifying the characteristic width of a pile group that is or may be exposed to the flow when the cylinders are spaced laterally as well as longitudinally in the stream flow. He concluded the following:

Pile groups that project above the stream bed can be analyzed conservatively by representing them as a single pier width equal to the projected area of the piles ignoring the clear spaces between piles. Good judgment needs to be used in accounting for debris because pile groups tend to collect debris that could effectively clog the clear spaces between piles and cause the pile group to act as a much larger mass."

Pressure Flow Scour

Pressure flow at a bridge occurs when bridge decks intersect the flow or are submerged. Flume studies at Colorado State University were conducted in the spring of 1990 with a bridge deck partly submerged and a single pier in the flume. For this study, the independent parameters were the distance from the streambed to the bridge deck and the flow velocity. There was no sediment transport upstream of the bridge (clear-water scour). Without the deck submerged, there was no contraction scour and only local scour occurred. With the deck submerged, there was contraction scour and scour depths at the pier were increased by a factor of two or three. The magnitude of the contraction and local scour was as expected, and depended on the velocity of the approach flow and the distance from the deck to the bed. For the same approach velocity, contraction scour and local pier scour increased as the distance from the bed to the deck decreased. These results are preliminary and are only now being analyzed. This research is the subject of a dissertation by Ms. Lila Abed, which should be completed in the near future.

Width of 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 angle of repose of the material (Equation 8).

$$W = y_s(K + Cotan\theta) \tag{8}$$

where

W = The width of the scour hole measured from the side of the pier to the lateral extent of the hole.

y_s = Scour depth.

K = Bottom width of the scour hole as a fraction of the scour depth.

θ = Angle of repose of the bed material (ranges between 30 to 44 degrees)

From the above equation, using K equal to 0, for the range of repose angles, the width of the scour hole will be between 1.1 to 1.8 times the depth of scour. If the bottom width of the scour hole was equal to the scour depth, the range of widths would be between 2.1 to 2.8 times the depth of scour. These extremes will encompass most of the scour widths encountered at bridge piers.

Total Scour Depth And Design Evaluation

Total Scour Depth Plots

Abutment and foundation design parameters and limits can be determined after determining the various scours, which are discussed in previous sections. For this step in the design, it is recommended that the estimate of 1) long-term bed elevation change; 2) contraction scour; and 3) local scour at the piers and abutments be plotted on a cross-section of the stream channel and floodplain at the bridge crossing. It is also recommended that a distorted scale be used so that the total scour can be easily evaluated. Make a sketch of any plan form changes (lateral stream channel movement due to meander migration, etc) that might also be reasonably expected to occur. The plotting of the individual scour depths can be envisioned in the following recommended order.

- Long-term elevation. Note that these may involve aggradation or degradation.
- Contraction scour. Plot these from the aggradation or degradation lines which were determined in the previous step.
- Local scour is then plotted from and below the combined aggradation and contraction scour lines.
- It is important to plot the scour-hole width at abutments and piers, as well as the total depth of scour.

Evaluate The Total Scour Depth

After construction of the total scour depth plots, it is essential to evaluate them. In some cases the results of this step will require the designer to go back and reevaluate the estimations of one or more scour-depth computations discussed previously in this text. The following is a list of many aspects of this evaluation, which may cause to designer to go back and reevaluate various aspects of the design.

- Are the scour depths reasonable and consistent with previous experience?
- Do the local scour holes from the piers or abutments intersect or come close to each other between spans? If so, Method 1 should be considered for the scour analysis. The length of the bridge opening may need to be reevaluated, increasing the opening size, or decreasing the number of piers as necessary.
- Are there additional factors (lateral movement of the stream, scour-hole armoring, stream-flow hydrograph, velocity and discharge distribution, moving of the thalweg, shifting of the flow direction, channel changes, type of stream, etc.) to be considered?
- Do the calculated scour depths appear too deep for the conditions in the field relative to the laboratory conditions? Remember, the abutment scour equations were for the worst-case conditions. Would riprap or spur dikes (guide bank) be a better, more cost-effective solution?
- Evaluate cost, safety, and any other aspect relating to the design. Also, has the influence of debris on scour been adequately accounted for?
- Consider whether countermeasures may be needed. If they are needed, determine whether their use is a cost-effective method of alleviating hazards to the bridge. In some cases they can be incorporated into the design at a later date to prevent total failure of the support system in the future.

Reevaluate The Bridge Design

Reevaluate the bridge design on the basis of the foregoing scour analysis. Revise the design as necessary. This evaluation should consider:

- If the contraction scour is too large, the waterway area may need to be increased.
- If the scour holes from the piers overlap with each other, or with the abutment scour holes, then the number of piers may need to be decreased. Also the alignment of the piers may be modified to minimize pier scour.

- If abutment scour or contraction scour is excessive, relief bridges may be needed to pass the flood flows and to decrease the amount of overland flow returning to the main channel. Alternatively, and in some cases, the main bridge opening can be widened. In some cases, bridge widening and relief bridges may be necessary.

- Check whether the bridge abutments and piers are properly aligned in regards to the stream channel and the flood plain. Adjust as necessary.

- In some cases the bridge crossing of the stream and the flood plain may not be in a desirable location. If the location presents problems, see if it can be realigned or relocated. If not, ascertain whether river-training works, guide banks or relief bridges could be used to develop an acceptable flow pattern at the bridges.

- Determine whether the hydraulic study is adequate to provide the necessary information for foundation design. If the flow patterns are complex, a two-dimensional water surface profile model may be required to adequately describe the flow conditions and provide parameters for the estimation of scour. In some cases, based partly on the costs of an acceptably safe foundation, a physical model study may be required as well.

SUMMARY AND CONCLUSIONS

Equations and methods for predicting and evaluating total scour at bridges for design purposes are presented in this paper. These procedures and recommendations are based on the 1988 Technical Advisory on Bridge Scour and on HEC-18, titled "Evaluating Scour at Bridges". Both publications have been funded and published by the Federal Highway Administration.

In summary, equations derived by Laursen [9 &11], CSU [10], and Froehlich [13] are recommended for computation of contraction, pier, and abutment scour, respectively. It should be noted that the recommended abutment scour equations were developed with little to no field data. Therefore, the estimations of abutment scour represent the maximum scour that might occur at abutments. Because of this, it is recommended that abutments be set at minimal depths and protected with riprap and/or spurs.

The influence of pier footings of local scour at piers was also discussed in this paper. For piers with footings at or below the bed level, local scour depths are decreased. For footings above the bed level, the local scour is increased and depends on the degree of footing penetration into the flow. A method originally determined by Jones [8] for determining local scour for this case was reiterated in this report. Jones [8] also has shown that pile groups that are exposed to the flow can be also be analyzed.

Recent research at Colorado State University on the effect of pressure flow caused by submerged bridge decks show a significant increase in local and contraction scour. Preliminary results indicate that this increase can be as high as two to three times the contracted or local scour depth. In general the degree of increase depends on the approach velocity and the distance from the deck to the bed of the channel.

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