Praktische Berechnungen zu Kolken an Brücken in den USA

United States Practice for Bridge Scour Analysis

Everett V. Richardson, Senior Associate, Ayres Associates, Inc., and Emeritus Professor of Civil Engineering, Colorado State University, Colorado, USA
Jean-Louis Briaud, Spencer J. Buchanan, Professor, Dept. of Civil Engineering, Texas A&M University, College Station, Texas, USA

1 Introduction

In 1988 the Federal Highway Administration (FHWA) of the U. S. Department of Transportation issued a Technical Advisory to the States requiring them to evaluate all bridges over water as to their vulnerability to scour. The Advisory was the result of floods in the New England States in 1987, which destroyed or damaged 17 bridges and cost 10 lives. This required the States to evaluate or have evaluated all private, county, city and state owned bridges. In addition the Federal Government had to evaluate the bridges it owned. This totaled 481,530 bridges in the national bridge management data bank. The only exceptions to the evaluation in 1988 were bridges over tidal waterways (987) and those with unknown foundations (89,611). Later tidal bridges over tidal waterways were added to the list of bridges that had to be evaluated but bridges with unknown foundations are still exempt. But most states are evaluating them. The reason unknown foundations are exempt from a national requirement is the lack of technology to determine unknown foundations depth.

To aid the States to perform their scour evaluations and as part of the advisory, FHWA issued an "Interim Procedure for Evaluating Scour at Bridges." This was the first time in the United States that a cohesive compellation of methods and equations for determining stream instability and scour at the nation's bridges was assembled into a single publication. Prior to this the only advice on stream instability and scour at highway bridges was a publication entitled "Highways in the River Environment – Hydraulic and Environmental Design Consideration" (Richardson et al. 1975). In 1991 the interim procedures were replaced with Hydraulic Engineering Circular No. 18 (HEC–18) (Richardson et al. 1991).The circular (HEC-18) was updated in 1993 (Richardson et al. 1993) in 1995 (Richardson and Davis 1995) and in 2001 (Richardson and Davis 2001). In addition, to provide the States additional information and help, FHWA issued Hydraulic Engineering Circular 20 (HEC-20) in 1991 titled "Stream Stability at Highway Structures" (Lagasse et al. 1991). This document was updated in 1995 (Lagasse et al. 1995) and 2001 (Lagasse et al. 2001). In 1997 FHWA issued Hydraulic Engineering Circular 23 (HEC-23) (Lagasse et al. 1997) entitled "Bridge Scour and Stream Instability Countermeasures – Experience,

1 Einführung

Im Jahr 1988 ließ die Bundesstraßenbehörde der USA (FHWA) die Gefährdung der Brücken über Wasser durch Kolkbildung ermitteln. Diese Maßnahme war das Ergebnis der Überschwemmungen in den Neu-England-Staaten 1987, die 17 Brücken zerstörten oder beschädigten und 10 Menschenleben forderten. Dabei sollten alle privaten, kommunalen landes- und bundeseigenen Brücken erfasst werden. In der nationalen Brückendatenbank wurden schließlich 481530 Brücken erfasst. Eine Ausnahme bildeten Brücken über tidebeeinflusste Wasserstraßen (987) und solche mit unbekannten Gründungen (89611). Erstere wurden später aufgenommen, Brücken mit unbekannter Gründung fehlen noch immer. Sie werden aber von den meisten Ländern beurteilt. Von der nationalen Erfassung sind sie infolge der unbekannten Gründungtiefe ausgeschlossen.

Um den Bundesländern bei der Erfassung und Beurteilung zu helfen, veröffentlichte FHWA ein „Vorläufiges Verfahren zur Beurteilung von Kolken an Brücken". Das war das erste Mal in den USA, dass eine zusammenhänge Aufstellung von Verfahren und Berechnungen zur Bestimmung von Flussveränderungen und Kolkbildung an den Brücken des Landes in einer einzigen Veröffentlichung zusammengefasst wurde. Die in dieser Hinsicht bisher einzige Veröffentlichung war „Schnellstraßen im Bereich von Flüssen – Hydraulische und umweltrelevante Bemessungskriterien" (Richardson et al. 1975). 1991 wurde die FHWA-Schrift ersetzt durch „Hydraulic Engineering Circular (HEC) Nr. 18" (Richardson et al. 1991). Diese Schrift wurde 1993, 1995 und 2001 überarbeitet. Für weitergehende Informationen und Unterstützung wurden von FHWA HEC 20 „Flussstabilität und Straßenbauwerke" (Lagasse et al. 1991) herausgegeben. Diese Schrift wurde 1997 und 2001 überarbeitet. 1997 wurde schließlich HEC 23 „Gegenmaßnahmen für Brückenkolke und Flussveränderungen – Erfahrungen, Auswahl- und Bemessungshilfen" veröffentlicht. Darin waren die Erfahrungen der Bundesländer bei der Auswahl und der Bemessung von Kolkgegenmaßnahmen enthalten. Dieses Merkblatt wurde 2001 um neue Informationen erweitert (Lagasse et al. 2001). Jede der aufeinander folgenden HEC-Publikationen enthält neue Informationen für die Bemessung und die Beherrschung von Flussveränderungen und Kolkbildung an Brücken.

Selection and Design Guidance." This document includes the experience of the States in the selection and design of countermeasures. The document was upgraded with new information in 2001 (Lagasse et al. 2001). Each successive HEC publication includes new information for the analysis design and control of stream instability and scour at bridges.

The three 2001 Hydraulic Engineering Circulars (HEC-18, 20, and 23) form a unit for the evaluation, design and inspection and the selection and design of countermeasures for stream instability and scour at bridges. The methodology and relationship of the three documents is illustrated in the flow chart given in Figure 1.

The purpose of HEC-18 is to

1. provide guidelines for designing new and replacement bridges to resist scour;
2. evaluating existing bridges for vulnerability to scour;
3. inspecting bridges for scour; and
4. improving the state-of-practice of estimating scour at bridges.

The purpose of HEC-20 is to provide guidelines for identifying stream instability problems at highway-stream crossings. HEC-20 gives techniques for stream channel reconnaissance and classification, as well as rapid assessment methods for channel instability. Both qualitative and quantitative geomorphic and engineering techniques for stream channel stability analysis are presented.

The purpose of HEC-23 is to identify and provide design guidelines for bridge scour and stream instability countermeasures that have been implemented by various State Departments of Transportation (DOTs) in the United States. Countermeasure guidance from FHWA publications is included as well as that derived from practice outside of the United States.

The 2001 editions of HEC-18, 20 and 23 use dual units. That is, the publications are in both English and Metric (SI) system of measurement units.

This paper will summarize the guidance given in HEC-18 (Richardson and Davis 2001) for the evaluation of scour at bridges.

Die drei überarbeiteten Fassungen von 2001 bilden eine Einheit für die Beurteilung, Bemessung und Inspektion und Auswahl und Ausbildung von Gegenmaßnahmen gegen Flussveränderungen und Kolkbildung. Die Methodik der Veröffentlichungen und ihre gegenseitige Beziehung zeigt das Flussdiagramm in Bild 1.

Die Ziele von HEC 18 sind

1. Bemessungshilfen für eine ausreichenden Kolkwiderstand zu geben für neue und zu ersetzende Brücken,
2. die Beurteilung der Kolkempfindlichkeit bestehender Brücken zu ermöglichen,
3. Brückeninspektionen hinsichtlich möglicher Kolkbildung durchzuführen,
4. die Kolkabschätzung zu verbessern.

HEC 20 hat Richtlinien für die Erkundung von möglichen Flussveränderungen an Kreuzungen von Fluss und Straße aufgestellt. HEC 20 gibt die Möglichkeit, Flussbettformen zu erfassen und zu klassifizieren, sowie mögliche Instabilitäten schnell zu ermitteln. Dafür werden qualitative und quantitative geomorphologische und ingenieurpraktische Verfahren für die rechnerische Behandlung zur Verfügung gestellt.

HEC 23 liefert Bemessungsrichtlinien für entsprechende Gegenmaßnahmen, die von einer Reihe von Landesverkehrsbehörden der USA (DOT) umgesetzt wurden. Die Hinweise für Gegenmaßnahmen stammen sowohl von FHWA-Veröffentlichungen als auch von außerhalb der USA.

Die Ausgaben des Jahres 2001 enthalten sowohl englische als auch metrische (SI) Einheiten.

Der vorliegende Beitrag fasst die in HEC 18 (Richardson & Davis 2001) enthaltenen Richtlinien zur Beurteilung der Kolkgefahr an Brücken zusammen.

NB: Die drei Empfehlungen waren im Jahr 2001 überarbeitet worden, nachdem eine Delegation amerikanischer Ingenieure bei einer Reise durch Mitteleuropa die hier üblichen Maßnahmen kennen lernen konnten. Im Zuge der Reise fand auch ein intensiver Gedankenaustausch in der BAW statt.

Wegen des Umfangs des Beitrages und der speziellen Ausrichtung auf die Verhältnisse in den USA wird auf eine Übersetzung ins Deutsche verzichtet.

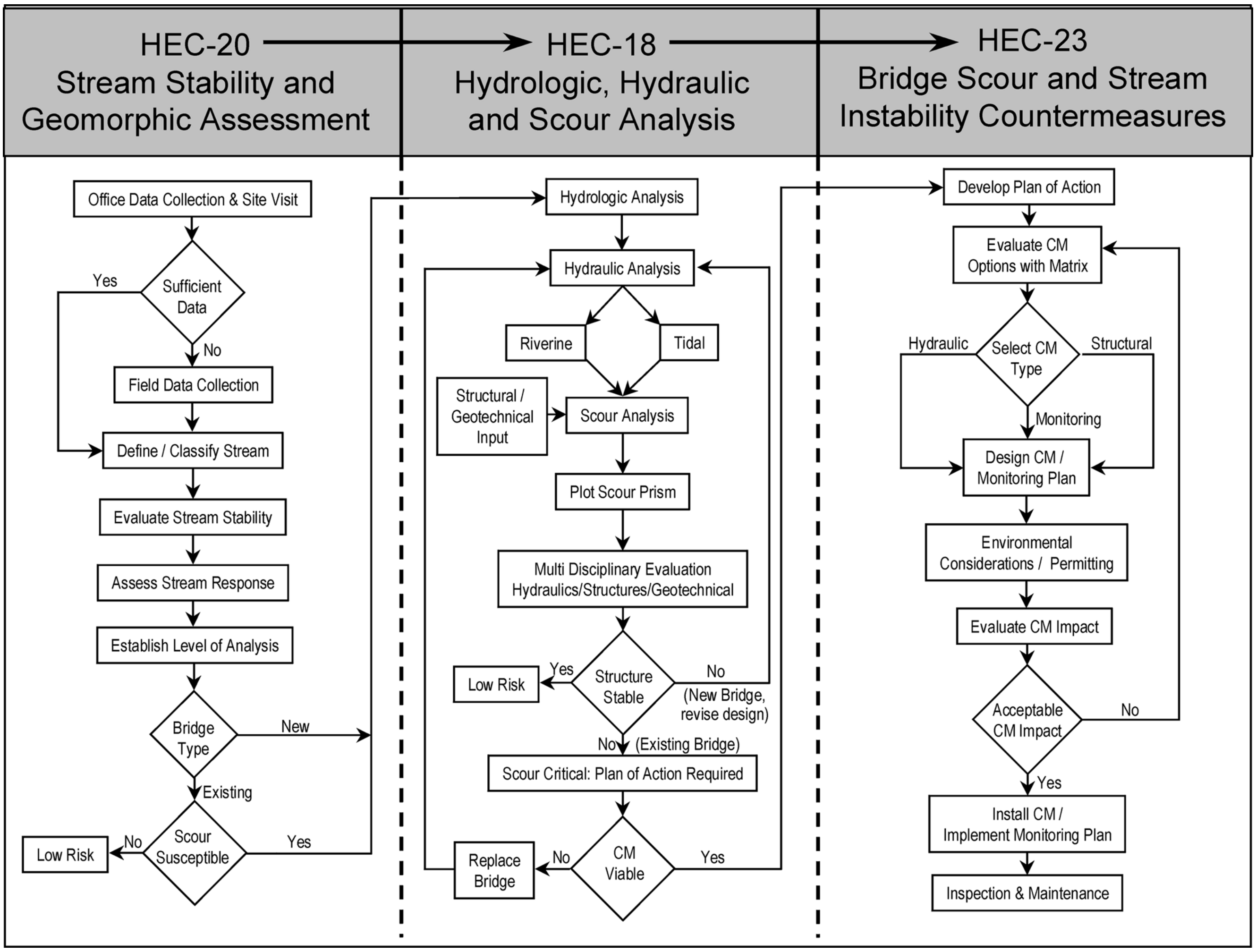

Figure 1: Flow chart for scour and stream stability analysis and evaluation

2 U. S. Design Philosophy and Considerations

2.1 General Considerations

The foundations of bridges should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to the 100-year flood, or a smaller flood, if it will cause scour depths deeper than the 100-year flood. Overtopping floods with a frequency less than the 100-year flood may cause the worse case scour situation. The foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of a superflood in order of magnitude of a 500-year flood. This requires careful evaluation of the hydraulic, structural and geotechnical aspects of bridge foundation design.

Normal geotechnical safety factors should be applied for the 100-year of smaller design floods. Whereas, all foundations should have a minimum factor of safety of 1.0 (ultimate load) under the superflood conditions.

The bridge foundation analysis is to be performed on the basis that all streambed material in the scour prism above the total scour line has been removed and is not available for bearing or lateral support. All foundations should be designed in accordance with the AASHTO Standard Specifications for Highway Bridges (1992). In the case of a pile foundation, the piling should be designed for additional lateral restraint and column action because of the increase in unsupported pile length after scour. In areas where the local scour is confined to the proximity of the footing, the lateral ground stresses on the pile length which remains embedded may not be significantly reduced from the pre-local scour conditions.

2.2 Spread Footings

In the case of spread footings without piles the following applies:

Spread Footings On Soil

Place the top of the footing below the total scour line. That is below the sum of the long-term degradation, contraction scour, local scour and lateral migration.

Spread Footings on Rock Highly Resistant to Scour

Place the bottom of the footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Small embedments (keying) should be avoided since blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour. If footings on smooth massive rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing level.

Spread Footings on Erodible Rock

Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine, if rock or soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life. An important consideration may be the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated and the footing base placed below that depth. Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize overbreak beneath the footing level. Loose rock pieces should be removed and the zone filled with clean concrete. In any event, the final footing should be poured in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level. Guidance on scourability of rock formations is given in FHWA memorandum "Scourability of Rock Formations" dated July 19, 1991.

2.3 Drilled Shafts and Piles

In the case of spread footings with drilled shafts or piles the following applies:

Spread Footings Placed on Tremie Seals and Supported on Soil

Place the top of the footing below the sum of the long-term degradation, contraction scour, and lateral migration.

For Deep Foundations (Drilled Shaft and Driven Piling) with Footings or Caps

Placing the top of the footing or pile cap below the streambed a depth equal to the estimated long-term degradation and contraction scour depth will minimize obstruction to flood flows and resulting local scour. Even lower footing elevations may be desirable for pile supported footings when the piles could be damaged by erosion and corrosion from exposure to river or tidal currents. Additional information is given in U. S. Department of Transportation manuals titled Driven Pile Foundations (1966) and Drilled Shafts (1988).

Stub Abutments on Piling

Stub abutments positioned in the embankment should be founded on piling driven below the elevation of the thalweg including-long term degradation and contraction scour in the bridge waterway to assure structural integrity in the event the thalweg shifts and the bed material around the piling scours to the thalweg elevation.

2.4 Piers

1. Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is likelihood that the channel will shift its location over the life of the bridge.
2. Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.
3. Streamline piers to decrease scour and minimize potential for buildup of ice and debris. Use ice and debris deflectors where appropriate.
4. Evaluate the hazards of ice and debris buildup when considering use of multiple pile bents in stream channels. Where ice and debris buildup is a problem, consider that the bent is a solid pier for purposes of estimating scour. Consider the use of other pier types where clogging of the waterway area could be a major problem.
5. Scour analyses of piers near abutments need to consider the potential of larger velocities and skew angles from the flow coming around the abutment.

2.5 Abutments

1. The equations used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and for the most part lack field verification. Because conditions in the field are different from those in the laboratory, these equations tend to over predict the magnitude of scour that may be expected to develop. Recognizing this, it is recommended that the abutment scour equations be used to develop insight as to the scour potential at an abutment. Engineering judgment must be used to determine if the abutment foundation should be designed to resist the computed local scour. As an alternate, abutment foundations should be designed for the estimated long-term degradation and contraction scour. Riprap and/or guide banks should be used to protect the abutment for this alternative. In summary, riprap or some other protection should always be used to protect the abutment from erosion. Proper design techniques and placement procedures for rock riprap and guide banks are discussed in HEC-23.

2. Relief bridges, guide banks, and river training works should be used, where needed, to minimize the effects of adverse flow conditions at abutments.
3. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate passage of the ice.
4. Wherever possible, use spill-through (sloping) abutments. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments.
5. Riprap or a guide bank 15 m (50 ft) or longer, or other bank protection methods should be used on the downstream side of an abutment and approach embankment to protect them from erosion by the wake vortex.

2.6 Superstructures

The design of the superstructure has a significant impact on the scour of the foundations. Hydraulic forces that should be considered in the design of a bridge superstructure include buoyancy, drag and impact from ice and floating debris. The configuration of the superstructure should be influenced by the highway profile, the probability of submergence, expected problems with ice and debris and flow velocities, as well as the usual economic, structural and geometric considerations. Superstructures over waterways should provide structural redundancy, such as continuous spans (rather than simple spans).

2.7 Other Considerations

In addition to the above the following guidance is given:

1. An interdisciplinary team of engineers should design the foundation with expertise in hydraulic, geotechnical and structural design.
2. Hydraulic studies of bridge sites are a necessary part of a bridge design. These studies should address both, the sizing of the bridge waterway opening and the design of the foundations, to be safe from scour. The scope of the analysis should be commensurate with the importance of the highway and consequences of failure.
3. Consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a reasonable and prudent design. Such data should include:
   a. Performance of existing structures during past floods
   b. Effects of regulation and control of flood discharges
   c. Hydrologic characteristics and flood history of the stream and similar streams
   d. Whether the bridge is structurally continuous
4. The principles of economic analysis and experience with actual flood damage indicate that it is almost always cost-effective to provide a foundation that will not fail, even from a very large flood event or superflood. Generally, occasional damage to highway approaches from rare floods can be repaired quickly to restore traffic service. On the other hand, a bridge, which collapses or suffers major structural damage from scour, can create safety hazards to motorists as well as significant social impacts and economic losses over a long period of time. Aside from the costs to the DOTs of replacing or repairing the bridge and constructing and maintaining detours, there can be significant costs to communities or entire regions due to additional detour travel time, inconvenience and lost business opportunities. Therefore, a higher hydraulic standard is warranted for the design of bridge foundations to resist scour than is usually required for sizing of the bridge waterway.
5. Raise the bridge superstructure elevation above the general elevation of the approach roadways wherever practicable. This provides for overtopping of approach embankments and relief from the hydraulic forces acting at the bridge. This is particularly important for streams carrying large amounts of debris, which could clog the waterway at the bridge.
6. The elevation of the lower cord of the bridge should be increased a minimum of 0.9 m (3 ft) above the normal freeboard for the 100-year flood for streams that carry a large amount of debris.
7. Superstructures should be securely anchored to the substructure if buoyant or if debris and ice forces are probable. Further, the superstructure should be shallow and open to minimize resistance to the flow where overtopping is likely.
8. Continuous span bridges withstand forces due to scour and resultant foundation movement better than simple span bridges. Continuous spans provide alternate load paths (redundancy) for unbalanced forces caused by settlement and/or rotation of the foundations. This type of structural design is recommended for bridges where there is a significant scour potential.
9. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour is indeterminate and may be deeper. The topwidth of a local scour hole on each side of the pier ranges from 1.0 to 2.8 times the depth of local scour. A topwidth value of 2.0 times the depth of local scour on each side of a pier is suggested for practical applications.
10. For pile and drilled shaft supported substructures subjected to scour, a re-evaluation of the foundation design may require a change in the pile or shaft

length, number, cross-sectional dimension and type based on the loading and performance requirements and site-specific conditions.

11. At some bridge sites, hydraulics and traffic conditions may necessitate consideration of a bridge that will be partially or even totally inundated during high flows. This consideration results in pressure flow through the bridge waterway. Section 6.6 is a discussion on pressure flow scour for these cases.

3 Basic Concepts and Definitions

3.1 General

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour-resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams (Briaud et al. 1999 a, b). Under constant flow conditions, scour will reach maximum depth in sand- and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstone and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour.

Determining the magnitude of scour is complicated by the cyclic nature of the scour process. Scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes refill with sediment.

All of the equations for estimating contraction and local scour are based on laboratory experiments with limited field verification. However, contraction and local scour depths at piers as deep as computed by these equations have been observed in the field. The equations recommended in this document are considered to be the most applicable for estimating scour depths.

A factor in scour at highway crossings and encroachments is whether it is clear-water or live-bed scour.
Clear-water scour occurs where there is no transport of bed material upstream of the crossing or encroachment or the material being transported from the upstream reach is transported through the downstream reach at less than the capacity of the flow.
Live-bed scour occurs where there is transport of bed material from the upstream reach into the crossing or encroachment.

The methods and equations for determining stream instability, scour and associated countermeasures can be applied to both riverine and coastal waterways (Richardson and Richardson 1993, Richardson et al. p 748 (Richardson and Lagasse Editors 1999). There are many papers discussing scour in tidal waterways in the ASCE's Compendium of "Stream stability and Scour at Highway Bridges" (Richardson and Lagasse editors 1999).

The major difference between scour analysis at highway structures over a riverine waterway and for a structure over a tidal waterway is the magnitude of the design discharge. The design discharge (50-year, 100-year or 500-year) for a riverine waterway is fixed from statistical analysis of peak discharge frequency. The design discharge for a riverine waterway is determined by statistical analysis of peak storm surge frequency. The design discharge in the tidal waterway depends on the elevation of the design storm surge, area and hydraulics of the waterway. If the area of the waterway increases the discharge may also increase. Thus, the design discharge in a tidal waterway may change.

Determination of hydraulic variables to be used in scour calculations for a tidal affected streams given by Richardson and Davis (2001) and by Zevenbergen et al. (1997).

3.2 Total Scour

Total scour at a highway crossing is comprised of three components:

1. Long-term aggradation and degradation of the river bed
2. General scour at the bridge
   a. Contraction scour
   b. Other general scour
3. Local scour at the piers or abutments

These three scour components are added to obtain the total scour at a pier or abutment. This assumes that each component occurs independent of the other. Considering the components additive adds some conservatism to the design.

In addition, lateral migration of the stream must be assessed when evaluating total scour at bridge piers and abutments.

3.2.1 Long-Term Streambed Elevation Changes (Aggradation or Degradation)

Long-term bed elevation changes may be the natural trend of the stream or the result of some modification to the stream or watershed. The streambed may be aggrading, degrading or in relative equilibrium in the vicinity of the bridge crossing. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge; whereas, degradation involves the lowering or scouring of the streambed due to a deficit in sediment supply from upstream. Long-term aggradation and degradation do not include the cutting and filling of the streambed in the vicinity of the bridge that might occur during a runoff event (general and local scour). A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the long-term streambed changes must be estimated. Engineering judgment and consideration of what is the cause of the long-term change in elevation must be used in determining long-term bed elevation changes. If the stream is aggrading, the increase in streambed elevation is not considered in the total scour. But if the stream is degrading, the estimated decrease in elevation of the streambed is included in the total scour.

3.2.2 General Scour

General scour is a lowering of the streambed across the stream or waterway bed at the bridge. This lowering may be uniform across the bed or the depth of scour may be deeper in some parts of the cross-section. General scour may result from contraction of the flow, which results in removal of material from the bed across all or most of the channel width or from other general scour conditions such as flow around a bend where the scour may be concentrated near the outside of the bend. General scour is different from long-term degradation in that general scour may be cyclic and/or related to the passing of a flood.

3.2.3 Local Scour

Local scour involves removal of material from around piers, abutments, spurs and embankments. It is caused by an acceleration of flow and resulting vortices induced by obstructions to the flow. Local scour can be either clear-water or live-bed scour (see Figure 2).

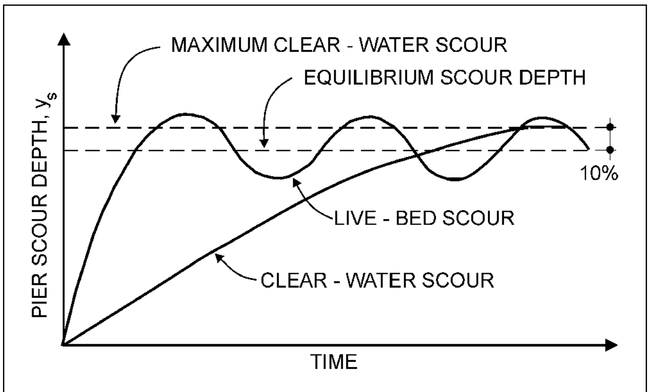

Figure 2: Pier scour depth in a sand bed stream as a function of time and clear-water or live-bed scour

3.2.4 Lateral Stream Migration

In addition to the types of scour mentioned above, lateral migration of the main channel of a stream within a floodplain may affect the stability of piers in a floodplain, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers and abutments. Factors that affect lateral stream migration are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials (see Hydraulic Engineering Circular No. 20 (Lagasse et al. 2001) and "Highways in the River Environment" (Richardson et al. 2001).

4 Long-Term Bed Elevation Changes

4.1 General

Factors that affect long-term bed elevation changes are dams and reservoirs (up- or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs of meander bends (natural or man-made), changes in the downstream channel base level (control), gravel mining from the streambed, diversion of water into or out of the stream, natural lowering of the fluvial system, movement of a bend and bridge location with respect to stream planform and stream movement in relation to the crossing. Tidal ebb and flood may degrade a coastal stream; whereas littoral drift may result in aggradation. The elevation of the bed under bridges which cross streams tributary to a larger stream will follow the trend of the larger stream unless there are controls. Controls could be bedrock, dams, culverts or other structures. The changes in bed elevation decrease when the bridge is further upstream from the confluence with another stream or from other bed elevation controls.

Federal and State agencies should be contacted concerning documented long-term streambed variations. If no data exist or if such data require further evaluation, an assessment of long-term streambed elevation changes for riverine streams should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the bridge crossing, i.e. runoff from the watershed to a stream (hydrology), sediment delivery to the channel (watershed erosion), sediment transport capacity of a stream (hydraulics), and response of a stream to these factors (geomorphology and river mechanics).

With coastal streams, the principles of both river and coastal engineering mechanics are needed. In coastal streams, estuaries or inlets, in addition to the above, consideration must be given to tidal conditions, i.e. the magnitude and period of the storm surge, sediment delivery to the channel by the ebb and flow of the tide, littoral drift, sediment transport capacity of the tidal flows and response of the stream, estuary or inlet to these tidal and coastal engineering factors.

4.2 Estimating Long-Term Bed Elevation (Aggradation or Degradation)

To organize an assessment of long-term aggradation and degradation, a three-level fluvial system approach can be used for either the riverine or tidal environment. The three level approach consists of (1) a qualitative determination based on general geomorphic and river mechanics relationships, (2) an engineering geomorphic analysis using established qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios or future conditions, and (3) physical models or physical process computer modeling using mathematical models such as BRI-STARS (Molinas 1990) and HEC-6 U. S. Army Corps of Engineers 1993) to make predictions of quantitative changes in streambed elevation due to changes in the stream and watershed. Methods to be used in Levels (1) and (2) are presented in HEC-20 and Highways in the River Environment. Sources of information are bridge inspection and maintenance records, stream gaging records, historical mapping and aerial photographs, and field inspection of the site.

5 General Scour

5.1 Introduction

General scour is the general decrease in the elevation of the bed across the bridge opening. It does not include localized scour at the foundations (local scour) or the long-term changes in the streambed elevation (aggradation or degradation). General scour may not have a uniform depth across the bridge opening. General scour can be cyclic, that is, there can be an increase and decrease of the stream bed elevation (cutting and filling) during the passage of a flood.

The most common general scour is contraction scour. There are several cases and flow conditions for contraction scour. Typically, contraction scour occurs where the bridge opening is smaller than the flow area of the upstream channel and/or floodplain. Other general scour conditions can result from erosion related to planform characteristics of the stream, flow around a bend, variable downstream control, or other changes that decrease the bed elevation at the bridge. In this section, methods and equations will be presented to estimate general scour.

5.2 Contraction Scour

5.2.1 Contraction Scour Conditions

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). It may be live-bed or clear-water scour.

Live-bed contraction scour occurs at a bridge, when there is transport of bed material in the upstream reach into the bridge cross- section. With live-bed contraction scour the area of the contracted section which is scoured increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in.

Clear-water contraction scour occurs, when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. 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_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material. Normally, for both live-bed and clear-water scour the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

5.2.2 Critical Velocity for Beginning of Sediment Motion

To determine, if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity the following equation derived in HEC-18 can be used. This equation is **(equation 5.1)**:

$$V_c = K_u\, y^{1/6}\, D^{1/3}$$

where:

- V_c = Critical velocity above which bed material of size D and smaller will be transported, m/s (ft/s)
- y = Average depth of flow upstream of the bridge, m (ft)
- D = Particle size for V_c, m (ft)
- D_{50} = Particle size in a mixture of which 50 percent are smaller, m (ft)
- K_u = 6.19 SI units
- K_u = 11.17 English units

The variable D is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material size in the upper 0.3 m (1 ft) of the stream bed.

5.2.3 Contraction Scour Cases

There are four conditions (cases) of contraction scour at bridge sites depending on the type of contraction and whether there is overbank flow or relief bridges. Regardless of the case, contraction scour can be evaluated using two basic equations:
(1) live-bed scour equation and
(2) clear-water scour equation.

The four conditions (cases) of contraction scour are:

Case 1: Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river;
b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment; or
c. Abutments are set back from the stream channel.

Case 2: Flow is confined to the main channel (i.e. there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at 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 (i.e. clear-water scour).

Case 4: A relief bridge over a secondary stream in the overbank area with bed material transport (similar to Case 1).

5.2.4 Live-Bed Contraction Scour

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section. The modification is to eliminate the ratio of Manning's n **(equation 5.2 and 5.3)**:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k1}$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)}$$

where:
y_1 = Average depth in the upstream main channel, m (ft)
y_2 = Average depth in the contracted section, m (ft)
y_o = Existing depth of flow in the contracted section before scour, m (ft)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2 = Flow in the contracted channel, m^3/s (ft^3/s)
W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)
k_1 = Exponent determined below (Table 5.1):

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

V_* = $(\tau_o/\rho)^{1/2} = (gy_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s (ft/s)
ω = Fall velocity of bed material based on the D_{50}, m/s (Figure 3)
For fall velocity in English units (ft/s) multiply ω in m/s by 3.28
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)
τ_o = Shear stress on the bed, Pa (N/m^2) (lb/ft^2)
ρ = Density of water (1000 kg/m^3) (1.94 $slugs/ft^3$)

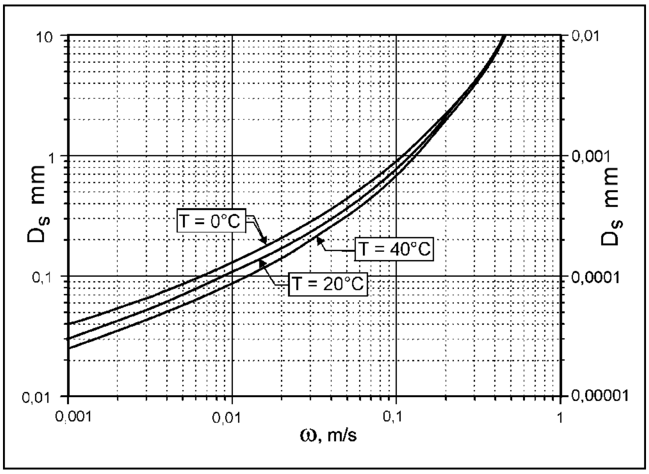

Figure 3: Fall velocity of sand-sized particles with specific gravity of 2.65

Notes:

1. Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b. It is not the total flow for Case 1c. For Case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.
2. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.
3. The Manning's n ratio is eliminated in Laursen live-bed equation to obtain equation 2 for the following reasons. The ratio can be significant for a condition of dune bed in the upstream 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, 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, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning's n will be equal.
4. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the topwidth of the main channel to define these widths. Whether topwidth or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.
5. The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.
6. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.
7. In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_0 depth may be approximated by y_1. Sketches or surveys through the bridge can help in determining the existing bed elevation.
8. Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation and that the smaller calculated scour depth be used.

Live-bed contraction scour depths may be limited by armoring of the bed by large sediment particles in the bed material or by sediment transport of the bed material into the bridge cross-section. Under these conditions, live-bed contraction scour at a bridge can be determined by calculating the scour depths using both the clear-water and live-bed contraction scour equations and using the smaller of the two depths.

5.2.5 Clear-Water Contraction Scour

The recommended clear-water contraction scour equation is based on a development suggested by Laursen. The equation is **(equations 5.4 and 5.5)**:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7}$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)}$$

where:

y_2 = Average equilibrium depth in the contracted section after contraction scour, m (ft)
Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s (ft^3/s)
D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m (ft)
D_{50} = Median diameter of bed material, m (ft)
W = Bottom width of the contracted section less pier widths, m (ft)
y_o = Average existing depth in the contracted section, m (ft)
K_u = 0.025 SI units
K_u = 0.0077 English units

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50}. For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

5.2.6 Contraction Scour with Backwater

The live-bed contraction scour equation is derived assuming a uniform reach upstream and a long contraction into a uniform reach downstream of the bridge. With live-bed scour the equation computes a depth after the long contraction, where the sediment transport into the downstream reach is equal to the sediment transport out.

The clear-water contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that flow goes from one uniform flow condition to another. Both equations calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_o$). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for clear-water scour it would be the energy at the same section before (1) and after (2) the contraction scour.

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

5.3 Contraction Scour Example Problems

5.3.1 Example Problem 1 - Live-Bed Contraction Scour

Given:
The upstream channel width = 98.2 m; depth = 2.62 m. The discharge is 773 m^3/s and is all contained within the channel.
Channel slope = 0.004 m/m.
The bridge abutments consist of vertical walls with wing walls. Bridge width = 37.2 m; with 3 sets of piers consisting of 3 columns, 0.38 m in diameter.
The bed material size: from 0 to 0.9 m, the D_{50} is 0.31 mm and below 0.9 m the D_{50} is 0.70 mm with a fall velocity of 0.10 m/s.
Original depth at bridge is estimated as 2.16 m.

Determine:
The magnitude of the contraction scour depth.

Solution:

1. Determine if it is live-bed or clear-water scour:

   Average velocity in the upstream reach

   $V = 773 / (2.62 \times 98.2) = 3.0$ m/s.

   For velocities this large and bed material this fine live-bed scour will occur. Check by calculating V_c for 0.7 mm bed material size. If live-bed scour occurs for 0.7 mm it would also be live-bed for $D_{50} = 0.3$ mm.

   $V_c = 6.19\,(2.62)^{1/6}\ (0.0007)^{1/3} = 0.65$ m/s

   Live-bed contraction scour is verified.

2. Calculate contraction scour:

   a. Determine K_1 for mode of bed material transport
      $V_* = (9.81 \times 2.62 \times 0.004)^{0.5} = 0.32$ m/s
      $\omega = 0.10$; $V_* / \omega = 3.2$; $K_1 = 0.69$

   b. Live-bed contraction scour
      $y_2 / 2.62 = (98.2 / 36.06)^{0.69} = 2.00$
      $Q_1 = Q_2$
      $y_2 = 2.62 \times 2.00 = 5.24$ m from water surface
      $y_s = 5.24 - 2.16 = 3.08$ m from original bed surface

5.3.2 Example Problem 2 - Alternate Method

An alternative approach to calculating y_s in Problem 1 is to calculate the scour depth using both the clear-water and the live-bed equation and take the smaller scour depth.

a. Live bed-bed scour depth is 3.08 m from Problem 1.
b. Clear-water scour depth (Equation 5.4)
   $D_m = 1.25\ D_{50} = 1.25\ (0.0007) = 0.0009$ m
   Equation 5.6:

$$y_2 = \left[\frac{0.025\,(773)^2}{0.0009^{2/3}\,(36.06)^2}\right]^{3/7} = 21.12 \text{ m}$$

   $y_s = 21.12 - 2.16 = 18.96$ m from original bed surface

c. Live-bed scour (3.08 m < 18.96 m). The sediment transport limits the contraction scour depth rather than the size of the bed material.

5.3.3 Example Problem 3 - Relief Bridge Contraction Scour

The 1952 flood on the Missouri River destroyed several relief bridges on Highway 2 in Iowa near Nebraska City, Nebraska. The USGS made continuous measurements during the period April 2 through April 29, 1952. This data set is from the April 21, 1952 measurement (measurement # 1013). The discharge in the relief bridge was 368 m^3/s.

Q = 368 m^3/s;
Bridge Width (minus piers) = 91.4 m;
Bridge Flow Area = 706.43 m^2;
$V_{average} = 0.52$ m/s;
$y_0 = 1.28$ to 1.62 m;
D_{50} = (estimated between 0.2 and 0.3 mm) use 0.3 mm as D_m;
Clear-water scour because of low velocity flow on the floodplain (equation 5.4).

Calculate y_2: **(Equation 5.7)**

$$y_2 = \left[\frac{0.025\,(368)^2}{0.0003^{2/3}\,(91.4)^2}\right]^{3/7} = 6.89 \text{ m}$$

y_2 = 6.89 m from the water surface, this compares to 7.71 m measured at the site

5.4 Other General Scour Conditions

In a natural channel, the depth of flow is usually greater on the outside of a bend. In fact, there may well be deposition on the inner portion of the bend at a point bar. If a bridge is located on or close to a bend, the general scour will be concentrated on the outer portion 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 inside of the bend as the flow increases. This can increase scour and nonuniform distribution of scour in the bridge opening. In some cases during high flow the point bar may have a channel (chute channel) eroded across it. This can further skew the distribution of scour in the bridge reach.

The relatively shallow straight reaches between bendway pools are called crossings. With changes in discharge and stage the patterns of scour and fill can also change in the crossing and pool sequence. These geomorphic processes are discussed in more detail in HEC-20 and Highways in the River Environment (HIRE). These processes are considered part of general scour. They are cyclic and may be in equilibrium around some general bed elevation. There are no equations for predicting these changes in elevation. Generally, a study of the stream using aerial photographs and/or successive cross section surveys can determine trends. In this case, the long-term safety of the bridge depends, primarily, on inspection.

Some general scour conditions are associated with a particular channel morphology. Braided channels will have deep scour holes when two channels come together downstream from a bar or island (confluence scour). At other times a bar or island will move into the bridge opening concentrating the flow onto a pier or abutment or changing the angle of attack. In anabranching flow, where flow is in two or more channels around semi-permanent islands, there is a problem of determining the distribution of flow between the channels, and over time the distribution may change. The bridge could be designed for the anticipated worst-case flow distribution or designed using the present distribution. In either case, inspection and maintenance personnel should be informed of the potential for the flow distribution and scour conditions to change.

Other general scour can be caused by short-term (daily, weekly, yearly or seasonal) changes in the downstream water surface elevation that control backwater and hence, the velocity through the bridge opening. Similarly, a bridge located upstream or downstream of a confluence can experience general scour caused by variable flow conditions on the main river and tributary. Because this scour is reversible, it is considered other general scour rather than long-term aggradation or degradation. These channel changes and other general scour conditions are also discussed in HEC-20 and Highways in the River Environment.

5.4.1 Determining Other General Scour

Scour at a bridge cross-section resulting from variable water surface elevation downstream of the bridge (e.g. tributary or downstream control) is analyzed by determining the lowest potential water-surface elevation downstream of the bridge insofar as scour processes are concerned. Then one can determine contraction and local scour depths using these worst-case conditions.

General scour in a channel bendway resulting from the flow through the bridge being concentrated toward the outside of the bend 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 general scour across the bridge cross-section for unusual situations involves particular skills in the application of principles of river mechanics to the site-specific conditions. To determine the scour across the bridge opening in many bridge crossings will require 2-dimensional (2-D) computer programs (for example FESWMS (Froelich 1996, or U. S. Army corps of Engineers 1997) or a physical model (HIRE (Richardson et al. 2001 or HEC-23 (Lagasse 20001). Brigham Young University's (2000) SMS system is an important aid in 2-dimensional modeling. Such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

6 Determination of Local Pier Scour

6.1 Introduction

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties and the geometry of the pier and footing. The bed material characteristics are granular or non-granular, cohesive or non-cohesive, erodible or non erodible rock. Granular bed material ranges in size from silt to large boulders and is characterized by the D_{50} and a coarse size such as the D_{84} or D_{90} size. Cohesive bed material is composed of silt and clay, possibly with some sand, which bonded chemically. Rock may be solid, massive or fractured. It may be sedimentary or igneous and erodible or non-erodible.

Flow characteristics of interest for local pier scour are the velocity and depth just upstream of the pier, the angle the velocity vector makes to the pier (angle of attack) and free surface or pressure flow. Fluid properties are viscosity and surface tension, which for the field case can be ignored.

Pier geometry characteristics are its type, dimensions and shape. Types of piers include single column, multiple columns or rectangular; with or without friction or tip bearing piles; with or without a footing or pile cap; footing or pile cap in the bed, on the surface of the bed, in the flow or under the deck out of the flow. Important dimensions are the diameter for circular piers or columns, spacing for multiple columns, and width and length for solid piers. Shapes include round, square or sharp nose, circular cylinder, group of cylinders or rectangular. In addition, piers may be simple or complex. A simple pier is a single shaft, column or multiple columns exposed to the flow. Whereas, a complex pier may have the pier, footing or pile cap and piles exposed to the flow.

There are many pier scour equations in the literature, as a result of the many laboratory studies. To determine which equation to recommend to the states to use in their scour evaluations an extensive review of the literature was made. The criteria used were to select the equation that gave the minimum depth of scour but encompassed all available scour data. A study by Jones (1983) of the more common equations showed that the Colorado State University (CSU) (Richardson et al. 2001) equation enveloped all the data, but gave lower values of scour than any of the equations studies. On the basis of Jones' studies the CSU equation was recommended in the Interim Procedures that accompanied FHWA's Technical Advisory (U. S. Department of Transportation 1988). With modifications, the CSU equation was recommended in all editions of HEC-18. In 1996 Mueller compared 22 scour equations using field data collected by the USGS (Landers et al. 1999). He concluded that the HEC-18 equation was good for design, because it rarely under predicted measured scour depth. However, it frequently over-predicted the observed scour. The data contained 384 field measurements of scour at 56 bridges.

6.2 Local Pier Scour Equation

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water pier scour (Richardson and Davis, 2001). The equation predicts maximum pier scour depths. The equation is **(equation 6.1)**:

$$\frac{y_S}{y_1} = 2.0\, K_1 K_2 K_3 K_4 K_w \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

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, Figure 4 and Table 6.1
K_2 = Correction factor for angle of attack of flow, Equation 6.4
K_3 = Correction factor for bed condition, Table 6.2
K_4 = Correction factor for armoring by bed material size, Equation 6.5 and Table 6.3
a = Pier width, m (ft)
L = Length of pier, m (ft)
Fr_1 = Froude Number directly upstream of the pier = $V_1 / (gy_1)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the pier, m/s (ft/s)
g = Acceleration of gravity (9.81 m/s²) (32.2 ft/s²)

The correction factor, K_2, for angle of attack of the flow, θ, is calculated using the following equation **(equation 6.2)**:

$$K_2 = (\cos\ \theta + L/a \sin\ \theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12. Table 6.2 illustrates the magnitude of the effect of the angle of attack on local pier scour.

Based on studies by Chang (1987) and Melville and Sutherland (1988) the maximum scour depth for round nose piers aligned with the flow is **(equation 6.3)**:

$$y_s \leq 2.4 \text{ times the pier width (a) for } Fr \leq 0.8$$
$$y_s \leq 3.0 \text{ times the pier width (a) for } Fr > 0.8$$

Scour depths larger than given by equation 6.3 for round nose piers aligned with the flow should be questioned and carefully evaluated.

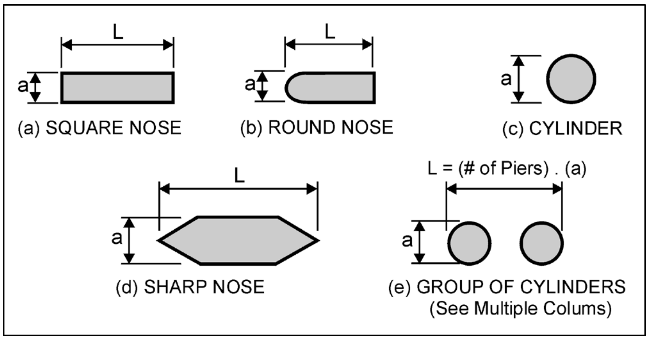

Figure 4: Common pier shapes

Notes:

1. The correction factor K_1 for pier nose shape should be determined using Table 6.1 for angles of attack up to 5 degrees. 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 for L/a = 12 as a maximum in Table 6.2 and Equation 6.2.

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

Table 6.1: Correction factor, K_1, for pier nose shape

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
Angle = skew angle of flow L = length of pier, m			

Table 6.2: Correction Factor, K_2, for angle of attack, θ, of the flow

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 $\geq$ 0.6	1.1
Medium Dunes	9> H $\geq$ 3	1.2 to 1.1
Large Dunes	H $\geq$ 9	1.3

Table 6.3: Increase in equilibrium pier scour depths, K_3, for bed condition (N/A = not applicable)

2. The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow.
3. The correction factor K_3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with Equation 6.1. 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 percent greater than the predicted equation 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 percent larger.
4. Piers set close to abutments (for example at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.

K_4 decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} equal to or larger than 2.0 mm and D_{95} equal to or larger than 20 mm (Mueller and Jones 1999).

If D_{50} < 2 mm or D_{95} < 20 mm, then $K_4 = 1$
If $D_{50} \geq 2$ mm and $D_{95} \geq 20$ mm, then **(equation 6.4)**:

$$K_4 = 0.4\,(V_R)^{0.15}$$

where **(equation 6.5)**:

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0$$

and:

V_{icDx} = the approach velocity (m/s or ft/sec) required to initiate scour at the pier for the grain size D_x (m or ft) **(equation 6.6)**

$$V_{icD_x} = 0.645 \left(\frac{D_x}{a}\right)^{0.053} V_{cD_x}$$

V_{cDx} = the critical velocity (m/s or ft/s) for incipient motion for the grain size D_x (m or ft) Equation 5.1.

where:
- a = pier width
- 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 percent of the bed material is finer, m (ft)

While K_4 provides a good fit with 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 with an increase in D_{50}.

The minimum value of K_4 is 0.4 and it should only be used when $V_1 < V_{icD50}$.

6.3 Pier Scour Correction Factor for Wide Piers K_w

Flume studies on scour depths at wide piers in shallow flows and field observations of scour depths at bascule piers in shallow flows indicate that existing equations, including the CSU equation, overestimate scour depths. Johnson and Torrico (1994) suggest the following equations for a K_w factor to be used to correct Equation 6.1 for wide piers in shallow flow.

The correction factor should be applied when the ratio of depth of flow (y) to pier width (a) is less than 0.8 (y/a < 0.8); the ratio of pier width (a) to the median diameter of the bed material (D_{50}) is greater than 50 (a/D_{50} > 50); and the Froude Number of the flow is subcritical.

Equations 6.7 and 6.8

$$K_w = 2.58 \left(\frac{y}{a}\right)^{0.34} Fr^{0.65} \quad \text{for} \quad V/V_c < 1$$

$$K_w = 1.0 \left(\frac{y}{a}\right)^{0.13} Fr^{0.25} \quad \text{for} \quad V/V_c \geq 1$$

Engineering judgment should be used in applying K_w because it is based on limited data from flume experiments. Engineering judgment should take into consideration the volume of traffic, the importance of the highway, cost of a failure (potential loss of lives and dollars) and the change in cost that would occur if the K_w factor is used.

6.4 Scour Depths for Complex Pier Foundations

6.4.1 Introduction

As Salim and Jones (1995, 1996, and 1999) point out, most pier scour research has focused on solid piers with limited attention to determining the scour depths for (1) pile groups, (2) pile groups and pile caps or (3) pile groups, pile caps and solid piers exposed to the flow. 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). In the general case, the flow could be obstructed by three substructure elements, herein referred to as the scour-producing components, which include the pier stem, the pile cap or footing and the pile group. Nevertheless, ongoing research has determined methods and equations to determine scour depths for complex pier foundations. The results of this research are recommended for use and are given in HEC-18 (Richardson and Davis 2001).

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. However, the methods presented in this section provide a good estimate of scour for a variety of complex pier situations.

The procedure listed below are recommended for determining the depth of scour for any combination of the three substructure elements exposed to the flow (Jones and Sheppard 2000).

1. The scour depths should be determined 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 paper.
2. If needed use computer programs (HEC-RAS (U. S. Army Corps of Engineers 2001), WSPRO (Arneson and Sherman 1998), FESWMS (Froelich 1996) to compute the hydraulic variables.
3. Total scour depth is determined 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.”
4. 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.
5. Determine the scour depths for each component exposed to the flow using the equations and methods presented in the following sections.
6. Add the components to determine the total scour depths.
7. 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, safety and costs.
8. Conduct a physical model study, if engineering judgment determines it will reduce uncertainty, increase the safety of the design and/or reduce cost.

6.4.2 Superposition of Scour Components Method of Analysis

The components of a complex pier are illustrated in Figure 5 (Jones and Sheppard 2000).

The variables illustrated in Figure 5 and others used in computations are as follows:

f = Distance between front edge of pile cap or footing and pier, m (ft)

h_o = Height of the pile cap above bed at beginning of computation, m (ft)

h_1 = $h_o + T$ = height of the pier stem above the bed before scour, m (ft)

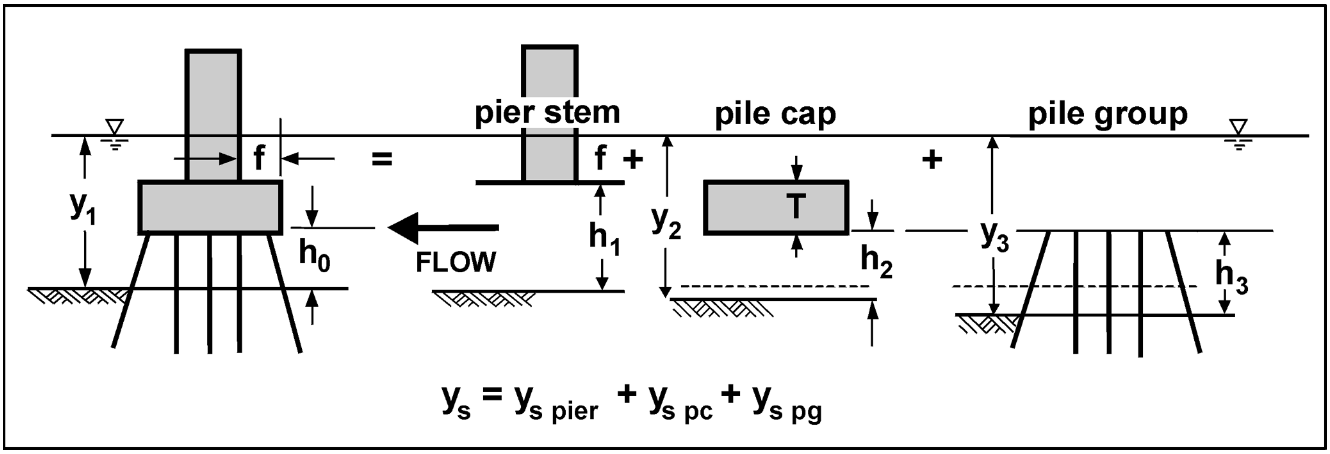

Figure 5: Definition sketch for scour components for a complex pier (Jones and Sheppard 2000)

h_2 = $h_o + y_{s\ pier}/2$ = height of pile cap after pier stem scour component has been computed, m (ft)
h_3 = $h_o + y_{s\ pier}/2 + y_{s\ pc}/2$ = 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)
y_1 = Approach flow depth at the beginning of computations, m (ft)
y_2 = $y_1 + y_{s\ pier}/2$ = adjusted flow depth for pile cap computations m (ft)
y_3 = $y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group computations, m (ft)
V_1 = Approach velocity used at the beginning of computations, m/sec (ft/sec)
V_2 = $V_1(y_1/y_2)$ = adjusted velocity for pile cap computations, m/sec (ft/sec)
V_3 = $V_1(y_1/y_3)$ = adjusted velocity for pile group computations, m/sec (ft/sec)

Total scour from superposition of components is given by **(equation 6.9)**:

$$y_s = y_{s\ pier} + y_{s\ pc} + y_{s\ pg}$$

where:
y_s = Total scour depth, m (ft)
$y_{s\ pier}$ = Scour component for the pier stem in the flow, m (ft)
$y_{s\ pc}$ = Scour component for the pier cap or footing in the flow, m (ft)
$y_{s\ pg}$ = Scour component for the piles exposed to the flow, m (ft)

Each of the scour components is computed from the basic pier scour Equation 6.1 using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in the list of variables for Figure 7, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap.

The procedure, along with example problems, is given in HEC-18 (Richardson and Davis 2001).

6.5 Multiple Columns Skewed to the Flow

For multiple columns (illustrated as a group of cylinders in Figure 6) skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor for angle of attack would be smaller than for a solid pier. Raudkivi (1986) 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."

In the application of Equation 6.1 with multiple columns spaced less than 5 pier diameters apart, the pier width 'a' is the total projected width of all the columns in a single bent, normal to the flow angle of attack (Figure 6). For example, three 2.0 m (6.6 ft) cylindrical columns spaced at 10.0 m (33 ft) would have an 'a' value ranging between 2.0 and 6.0 m (6.6 and 33 ft), depending upon the flow angle of attack.

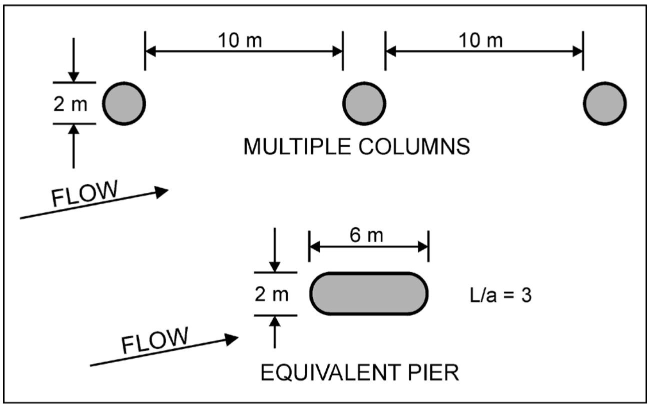

Figure 6: Multiple columns skewed to the flow

The scour depth for multiple columns skewed to the flow can also be determined by determining the K_2 factor using Equation 6.4 and using it in Equation 6.1. The width "a' in Equation 6.1 would be the width of a single column. An example problem illustrates all three methods of obtaining the scour depth for multiple columns.

If the multiple columns are spaced 5 diameter or greater apart and debris is not a problem, limit the scour depths to a maximum of 1.2 times the local scour of a single column.

With debris in the flow, consider the multiple columns and debris as a solid elongated pier in Equation 6.1.

6.6 Pressure Flow Scour

Pressure flow, which is also denoted as orifice flow, occurs, when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure (Figure 7). The resulting flow under the bridge being a complex combination of the plunging flow and orifice flow.

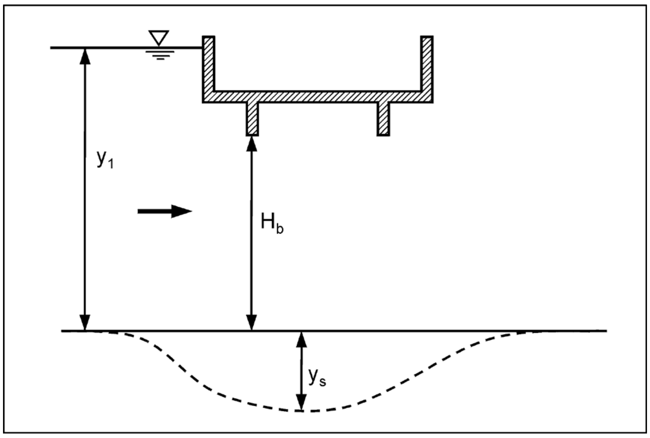

Figure 7: Definition sketch of vertical contraction scour resulting from pressure flow

The hydraulic bridge computer models WSPRO or HEC-RAS are suitable for determination of the amount of flow which will flow over the roadway embankment, over the bridge as weir flow and through the bridge opening as orifice flow, provided that the top of the highway is properly included in the input data. These models can be used to determine average flow depths and velocities over the road and bridge, as well as average velocities under the bridge. It is recommended that one of these models be used to analyze the scour problem when the bridge is overtopped with or without overtopping of the approach roadway.

With pressure flow, the local scour depths at a pier or abutment can be much larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subjected to pressure flow results from the flow being directed downward towards the bed by the superstructure (vertical contraction of the flow) and by increasing the intensity of the horseshoe vortex. The vertical contraction of the flow can be a more significant cause of the increased scour depth. However, in many cases, 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 which must pass under the bridge due to weir flow over the bridge and/or approach embankments.

As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lower velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping of the bridge and approach embankments.

Studies of pressure flow scour have been made in flumes at Colorado State University and FHWA's Turner Fairbank Highway Research Center which indicate that pier scour can be increased 200 to 300 percent by pressure flow (Abed 1991, Abed et al. 1991 and Jones et al. 1993). Arneson (1999) conducted a more extensive study of pressure flow scour under live bed conditions. FHWA's Turner Fairbank Laboratory and Arneson's study concluded that (1) pressure flow scour is a combination of vertical contraction scour and local pier scour, (2) the local pier scour component was approximately the same as the free-surface local pier scour measurements for the same approach flow condition and (3) the two components were additive. Arneson's equation, derived from multiple linear regression of his data, for bed vertical contraction scour is **(equation 6.10)**:

$$\frac{y_{vcs}}{y_1} = -5.08 + 1.27\left(\frac{y_1}{H_b}\right) + 4.44\left(\frac{H_b}{y_1}\right) + 0.19\left(\frac{V_a}{V_c}\right)$$

where:

- y_{vcs} = Depth of vertical contraction scour relative to mean bed elevation, m (ft)
- y_1 = depth of flow immediately upstream of the bridge, m (ft)
- H_b = Distance from the low chord of the bridge to the average elevation of the stream bed before scour, m (ft)
- V_a = Average velocity of the flow through the bridge opening before scour occurs, m/s (ft/s)
- V_c = Critical velocity of the D_{50} of the bed material in the bridge opening, m/s (ft/s)

The procedure for calculating pier scour for pressure flow is as follows:

1. Determine the flow variables using a 1-dimensional or 2-dimensional computer model such as WSPRO, HEC-RAS, FESWMS or RMA-2.
2. Calculate the critical velocity V_c of the D_{50} of the bed material in the bridge opening.
3. Compute the vertical contraction scour (Equation 6.10).
4. Compute the local pier scour using Equations 6.1 and the other procedures presented in previous sections.
5. Add the two scour components to obtain the local scour for pressure flow.

6.7 Scour from Debris on Piers

Debris lodged on a pier can increase local scour at a pier by increasing pier width and deflect a component of flow downward. When floating debris is lodged on the pier, the scour depth can be estimated by assuming that the pier width is larger than the actual width. Melville and Dongol (1992) have conducted a limited quantitative study of the effect of debris on local pier scour and have made some recommendations which support this approach. However, additional studies are needed.

6.8 Topwidth of Scour Holes

The topwidth of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation (Richardson and Abed 1993) **(equation 6.11)**.

$$W = y_s \ (K + \text{Cot}\ \theta)$$

where:

- W = Topwidth of the scour hole from each side of the pier or footing, m
- y_s = Scour depth, m (ft)
- K = Bottom width of the scour hole, related to the depth of scour
- θ = Angle of repose of the bed material ranging from about 30° to 44°

The angle of response of cohesionless material in air ranges from about 30° to 44°. Therefore, if the bottom width of the scour hole is equal to the depth of scour y_s (K = 1), the topwidth in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme, if K = 0, the topwidth would vary from 1.07 to 1.8 y_s. Thus, the topwidth could range from 1.0 to 2.8 y_s. A topwidth of 2.0 y_s is suggested for practical applications.

6.9 Physical Model Studies

For unusual or complex pier foundation configurations a physical model study should be made. The scale between model and prototype is based on the Froude criteria; that is, the Froude number for the model should be the same as for the prototype. In general it is not possible to scale the bed material size. Also, at flood flows in sand bed streams the sediment transport conditions will be live-bed and the bed configuration will be plane bed. However, in the model live-bed transport conditions could be ripples or dunes. These are incomparable pier scour conditions. Therefore, it is recommended that a bed material be used that has a critical velocity just below the model velocity (i.e. clear-water scour conditions). This will usually give the maximum scour depth; but a careful study of the results needs to be made by persons with field and model scour experience. For additional discussion of the use of physical modeling in hydraulic design, see HIRE and HEC-23.

6.10 Pier Scour Example Problems (SI units)

6.10.1 Example Problem 1 - Scour at a Simple Solid Pier

Given:
Pier geometry: a = 1.22 m, L = 18 m, round nose
Flow variables: y_1 = 3.12 m, V_1 = 3.36 m/s
Angle of attack = 0°, g = 9.81 m/s^2
Froude No. = 3.36/(9.81 x 3.12)$^{0.5}$ = 0.61
Bed material: D_{50} = 0.32 mm, D_{95} = 7.3 mm
Bed Configuration: Plane bed

Determine:
The magnitude of pier scour depth.

Solution:
Use Equation 6.1.

$$y_s / 3.12 = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \times 1.0 \times (1.22 / 3.12)^{0.65} \times 0.61^{0.43} = 0.97$$

$$y_s = 0.97 \times 3.12 = 3.03 \text{ m}$$

6.10.2 Example Problem 2 - Angle of Attack

Given:
Same as Problem 1 but angle of attack is 20°.

Solution:
Use Equation 6.4 to compute K_2.

If L/a is larger than 12, use L/a = 12 as a maximum in equation 6.4.

L/a = 18 /1.22 = 14.8 >12 use 12

K_2 = (Cos 20 + 12 Sin 20)$^{0.65}$ = 2.86

y_s = 3.03 x 2.86 = 8.7 m

6.10.3 Example Problem 3 - Coarse Bed Material

Given:
Same as Problem 1 but the bed material is coarser.
Bed material: D_{50} = 17.8 mm, D_{95} = 96.3 mm
Bed configuration: Plane Bed

Determine:
Will the coarse bed material decrease local scour depth.

Solution:
Use Equations 6.5, 6.6, 6.7 and 6.8

K_4 = 1, if D_{50} < 2 mm or D_{95} < 20 mm

If $D_{50} \geq$ 2 mm and $D_{95} \geq$ 20 mm, then:

K_4 = 0.4 $(V_R)^{0.15}$

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}}$$

where:
V_{icDx} = approach velocity required to initiate scour at the pier for the grain size D_x, m/s

$$V_{icD_x} = 0.645 \left(\frac{D_x}{a}\right)^{0.053} V_{cD_x}$$

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

$$V_{cD_x} = 6.19\, y_1^{1/6} D_x^{1/3}$$

V_{cD50} = 6.19 (3.12)$^{1/6}$ (0.0178)$^{1/3}$ = 1.95 m/s
V_{cD95} = 6.19 (3.12)$^{1/6}$ (0.0963)$^{1/3}$ = 3.43 m/s
V_{icD50} = 0.645 (0.0178 / 1.22)$^{0.053}$ (1.95) = 1.01 m/s
V_{icD95} = 0.645 (0.0963 / 1.22)$^{0.053}$ (3.43) = 1.93 m/s

$$V_R = \frac{3.36 - 1.01}{1.95 - 1.93} = 117.5$$

K_4 = 0.4 (117.5)$^{0.15}$ = 0.82
y_s = 0.82 x 3.03 = 2.48 m

6.10.4 Example Problem 4 - Scour at Multiple Columns

Calculate the scour depth for a pier that consists of six 0.406 m columns spaced at 2.29 m with a flow angle of attack of 26°. Debris is not a problem and there is no armoring at this site.

Data:
Columns:
6 columns 0.406 m, spaced 2.29 m
Velocity: V_1 = 3.4 m/s
Depth: y_1 = 6.1 m
Angle of attack: 26°
Spacing coefficient = S/a = 2.29/0.406 = 5.6; S/a > 5.0
Assume K_3 = 1.1 for plane bed condition

Determine:
The depth of local scour:

Solution:
Three methods of calculating the scourdepth will be illustrated:

a. Scour depth according to Raudkivi (1986) is 1.2 times the local scour of a single column

$$\frac{y_s}{6.1} = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \left(\frac{0.406}{6.1}\right)^{0.65} \times \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = .0.266$$

y_s = 6.1 x 0.266 x 1.2 = 1.95 m

b. Compare this value with that computed by collapsing the columns.

Collapsed pier width = 6 x 0.406 = 2.44 m
Projected pier width = L Sin 26^0 + a Cos 26^0 = 2.44 Sin 26^0 + 0.406 Cos 26^0 = 1.44 m

$$\frac{y_s}{6.1} = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \left(\frac{1.44}{6.1}\right)^{0.65} \times \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.604$$

y_s = 6.1 x 0.604 = 3.68 m

c. The scour depth can be calculated for multiple columns by calculating the depth for a single column and multiplying it by the K_2 factor given in Equation 6.4. For example:

K_2 = (Cos 26^0 + 2.44/0.406 Sin 26^0)$^{0.65}$ = 2.27

$$\frac{y_s}{6.1} = 2.0 \times 1.0 \times 2.27 \times 1.1 \times 1.0 \left(\frac{0.406}{6.1}\right)^{0.65} \times \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.603$$

$y_s = 6.1 \times 0.603 = 3.68$ m

Spacing between columns for this pier is greater than 5 times column diameter so method (a) applies. Also, a model study of the pier gave a scour depth of 1.95 m. Therefore:

$y_s = 6.1 \times 0.266 \times 1.2 = 1.95$ m

6.10.5 Example Problem 5 - Pier Scour with Pressure Flow

An existing bridge is subjected to pressure flow to the top of a solid guard rail at the 100-year return period flow. There is only a small increase in flow depth at the bridge for the 500-year return period flow due to the large overbank area. A HEC-RAS model of the flow gives the following data:

Data:
y_1 = 9.75 m, V_1 = 2.93 m/s, q_1 = 28.56 cm/m
Pier width a = 0.914 m, is round nose, solid, aligned with the flow
Sand bed with D_{50} = 0.4 mm and D_{84} = 0.9 mm
Distance from stream bed to lower chord (H_b) = 7.93 m before scour

Calculate the local pier scour:

Vertical Contraction Scour Depth

$y_s/y_1 = -5.08 + 1.27\, y_1/H_b + 4.44\, H_b/y_1 + 0.19\, V_a/V_c$

$V_c = 6.19\,(y_1)^{1/6}(D_{50})^{1/3} = 6.19\,(9.75)^{1/6}(0.0004)^{1/3} = 0.669$ m/s

$V_a = q_1/H_b = 28.56/7.93 = 3.60$ m/s

$y_s/9.75 = -5.08 + 1.27\,(9.75/7.93) + 4.44\,(7.93/9.75) + 0.19\,(3.60/0.669) = 1.12$

$y_s = 9.75 \times 1.12 = 10.9$ m

Local Pier Scour

$y_2 = H_b + y_s = 7.93 + 10.92 = 18.85$ m

$V_2 = V_a\,(H_b/y_2) = 3.60\,(7.93/18.85) = 1.51$ m/s

$y_s/y_1 = 2.0\, K_1\, K_2\, K_3\, K_4\, (a/y_1)^{0.65}\, (Fr)^{0.43}$

$K_1 = K_2 = K_4 = 1.0$; $K_3 = 1.1$;

$Fr = 1.52 / (9.81 \times 18.85)^{0.5} = 0.11$

$y_s/18.85 = 2.0 \times 1.1 \times (0.914/18.85)^{0.65}\,(0.11)^{0.43} = 0.12$

$y_s = 18.85 \times 0.12 = 2.26$ m

Total Scour

$y_s = 10.92 + 2.26 = 13.2$ m

7 Evaluating Local Abutment Scour

7.1 Introduction

The flow obstructed by the abutment and approach highway embankment forms two erosion potentials. One is a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment, and the other is a vertical wake vortex at the downstream end of the abutment (Figure 8).

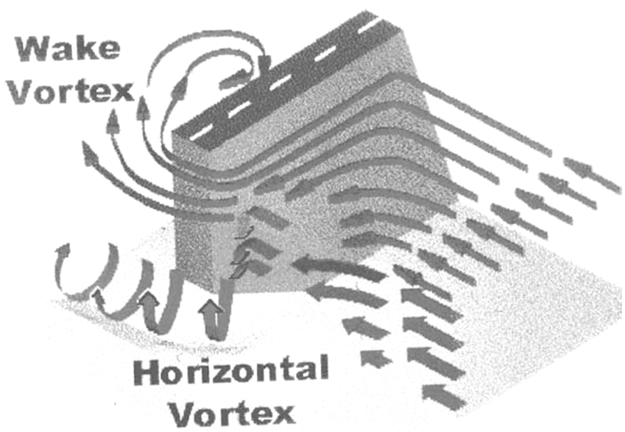

Figure 8: Schematic representation of abutment scour

The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers and the vortex that forms at the downstream end is similar to the wake vortex that forms downstream of a pier. Research has only been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called horseshoe) vortex and numerous abutment scour equations have been developed to predict this scour depth.

Abutment failures and erosion of the fill also occur from the action of the downstream wake vortex. However, research and the development of methods to determine the erosion from the wake vortex has not been conducted. In general, if the downstream area of the abutment and approach embankment is protected with adequately sized riprap, no erosion will occur.

7.1.1 Abutment Site Conditions

Abutments can be set back from the natural stream bank, placed at the bankline or, in some cases, actually set into the channel itself. Common designs include stub abutments placed on spill-through slopes and vertical wall abutments, with or without wingwalls. Scour at abutments can be live-bed or clear-water scour. The bridge and approach road can cross the stream and floodplain at a skew angle and this will have an effect on flow conditions at the abutment. 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.

7.1.2 Abutment Skew

The skew angle for an abutment (embankment) is depicted in Figure 9. For an abutment angled downstream, the scour depth is decreased, whereas the scour depth is increased for an abutment angled upstream.

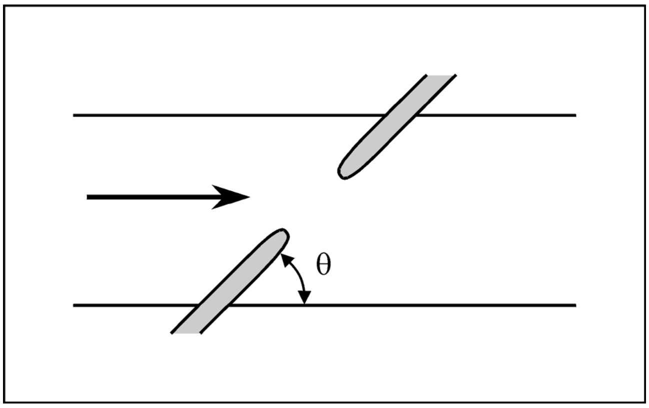

Figure 9: Orientation of embankment angle, q, to the flow

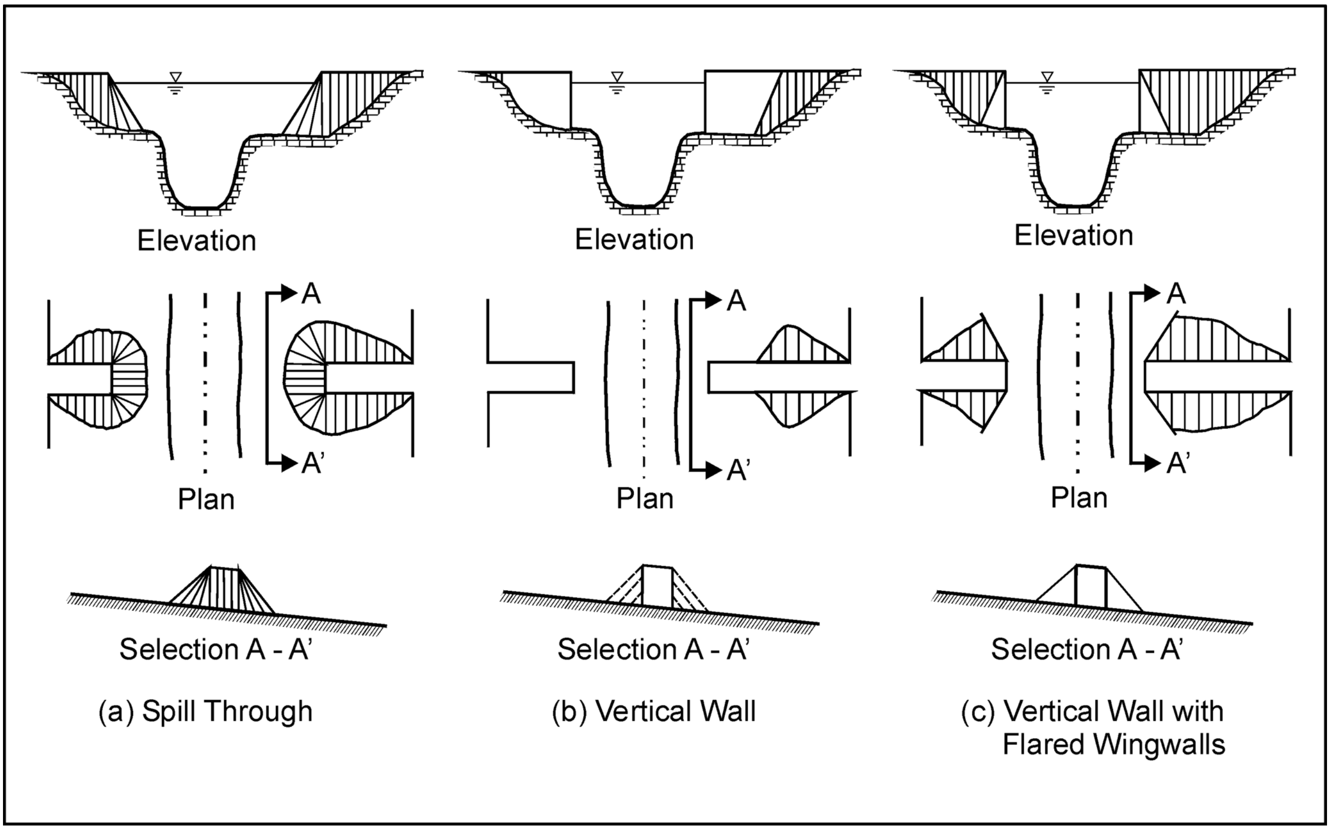

Figure10: Abutment shape

7.1.3 Abutment Shape

There are three general shapes of abutments: (a) spill-through abutments, (b) vertical walls without wing walls, and (c) vertical-wall abutments with wing walls (Figure 10). These shapes have varying angles to the flow. As shown in Table 7.1, depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments. Similarly, scour at vertical wall abutments with wingwalls is reduced to 82 % of the scour of vertical wall abutments without wingwalls.

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

Table 7.1: Abutment shape coefficients

7.2 Abutment Scour Equations

7.2.1 Overview

Equations for predicting abutment scour depths such as Liu et al. (1961), Laursen (1980), Froehlich (1989) and Melville (1992) are based entirely on laboratory data. The problem is that little field data on abutment scour exist (Table 7.1).

Until recently, the equations in the literature were developed using the abutment and roadway approach length as one of the variables. This approach results in excessively conservative estimates of scour depth. Richardson and Richardson (1993) pointed this out in a discussion of Melville's (1992) 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 11 illustrates the difference. Thus, as Strum (1999) Richardson and Richardson (1993, 1999) and Kouchakzadeh and Thompsend (1999) pointed out, equations for predicting abutment scour would be more applicable to field conditions, if they included the discharge intercepted by the embankment rather than embankment length.

7.2.2 Designing for Scour at Abutments

Unfortunately, well documented and field tested equations to determine scour depths caused by the horizontal vortex, are not available at this time. Therefore, HEC-18 states:

The preferred design approach is to place the abutment foundation on scour resistant rock or on deep foundations. Available technology has not developed sufficiently to provide reliable abutment scour estimates for all hydraulic flow conditions that might be reasonably expected to occur at an 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 they are protected with rock riprap and/or with a guide bank placed upstream of the abutment designed in accordance with guidelines in HEC-23 (Lagasse 2001). Cost will be the deciding factor.

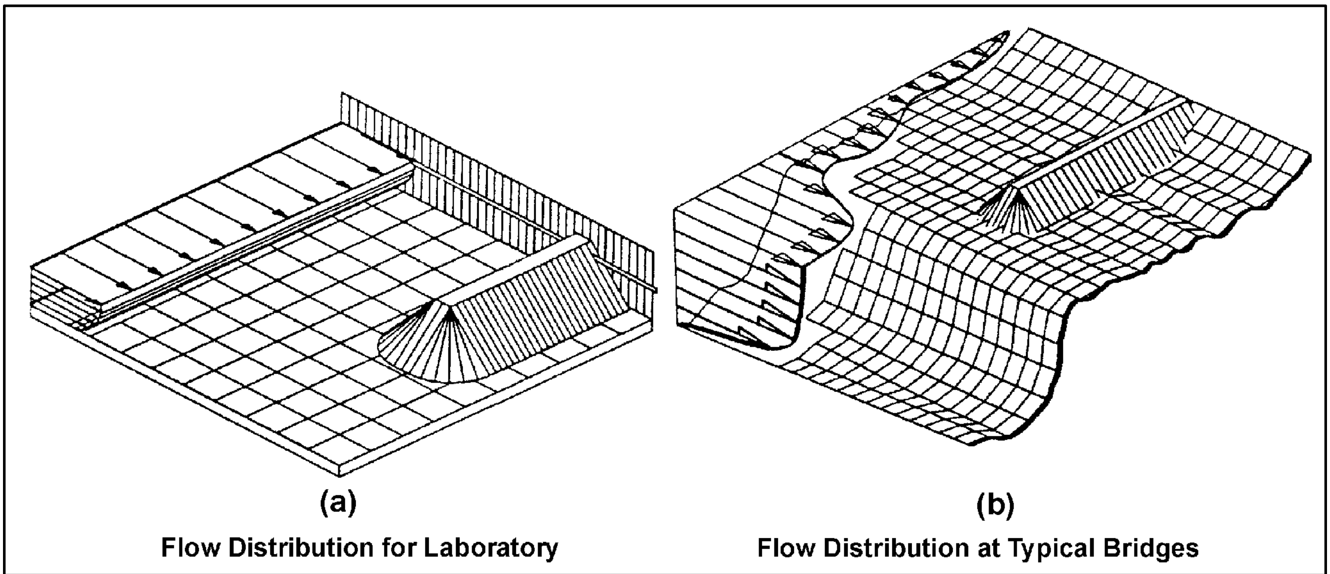

Figure11: Comparison of (a) laboratory flow characteristics to (b) field flow conditions

As a minimum, abutment foundations should be designed assuming no ground support (lateral or vertical) as a result of soil loss from long-term degradation, stream instability and contraction scour. The abutment should be protected from local scour using riprap and/or guide banks. Guidelines for the design of riprap and guide banks are given in HEC-23. To protect the abutment and approach roadway from scour by the wake vortex several DOTs (Departments of Transportation) use a 15-meter (50-ft) guide bank extending from the downstream corner of the abutment. Otherwise, the downstream abutment and approach should be protected with riprap or other countermeasures.

In the following sections, to aid in the design of the foundation and placement of rock riprap and/or guide banks, Froehlich's (1989) live-bed scour equation and HIRE (Richardson et al. 2001) equations are presented. The equations can be used for either clear-water or live-bed scour.

7.2.3 Froehlich's Live-Bed Abutment Scour Equation

Froehlich (1989) analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following **equation 7.1**:

$$\frac{y_s}{y_a} = 2.27\, K_1 K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

where:

K_1 = Coefficient for abutment shape (Table 7.1)
K_2 = Coefficient for angle of embankment to flow
K_2 = $(\theta/90)^{0.13}$ (see Figure 9 for definition of θ)
$\theta < 90^0$ if embankment points downstream
$\theta > 90^0$ if embankment points upstream
L' = Length of active flow obstructed by the embankment, m (ft)
A_e = Flow area of the approach cross section obstructed by the embankment, m^2 (ft^2)
Fr = Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e = Q_e/A_e, m/s (ft/s)
Q_e = Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
y_a = Average depth of flow on the floodplain (A_e/L), m (ft)
L = Length of embankment projected normal to the flow, m (ft)
y_s = Scour depth, m (ft)

It should be noted that Equation 7.1 is not consistent with the fact that as L' tends to 0, y_s also tends to 0. The 1 was added to the equation so as to envelope 98 percent of the data. Guidance is given in HEC-18 for estimating L'. Use Froehlich equation when $L'/y < 25$.

7.2.4 HIRE Live-Bed Abutment Scour Equation

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the U. S. Army Corps of Engineers) was developed in HIRE (Richardson et al. 2001). The modified equation, referred to herein as the HIRE equation, is applicable, when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which was derived **equation 7.2** :

$$\frac{y_s}{y_1} = 4\, Fr^{0.33} \frac{K_1}{0.55} K_2$$

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)
Fr = Froude Number based on the velocity and depth adjacent to and upstream of the abutment
K_1 = Abutment shape coefficient (from Table 7.1)
K_2 = Coefficient for skew angle of abutment to flow calculated as for Froehlich's equation

7.3 Abutment Scour Example Problems (SI units)

7.3.1 Example Problem 1 - Scour at a Spill Through Abutment (Froehlich Equation)

Left Abutment
Determine the scour depth for the left abutment for a bridge over a stream with the following data:

Abutment
Angle across floodplain and stream of 10° in the downstream direction on the left side,
spill through,
set back from main channel 20 m,
abutment and approach length 25 m.

Flow data:
Effective length of flow intercepted by approach and abutment L' = 19.6 m
Average depth on floodplain y_a = 1.24 m,
Discharge intercepted by approach and abutment Q_e = 17.51 cm,
Velocity V_e = 0.72 m/s,
L/y = 19.6 / 1.24 = 15.8 < 25.

Therefore use the Froehlich Equation.

$$\frac{y_s}{y_a} = 2.27\, K_1 K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

Fr = 0.72/(9.81 x 1.24) = 0.21

$K_1 = 0.55$, $K_2 = (10/90)^{0.13} = 0.75$

$$\frac{y_s}{1.24} = 2.27 \times 0.55 \times 0.75 \left(\frac{19.6}{1.24}\right)^{0.43} \times$$

$$0.21^{0.61} + 1 = 2.18$$

y_s = 2.18 x 1.24 = 2.7 m

7.3.2 Example Problem 2 - Scour at a Spill Through Abutment (HIRE Equation)

Right Abutment
Determine the scour depth for the right abutment for a bridge over a stream with the following data:

Abutment
Angle across floodplain and stream of 10° in the upstream direction on the right side,
spill through,
set back from main channel 13.5 m,
abutment and approach length 68 m.

Flow data:
Effective length of flow intercepted by aproch and abutment L´ = 42.8 m
Average depth on floodplain y_a = 1.31 m,
Discharge intercepted by approach and abutment Q_e = 44.29 cm,
Velocity V_e = 0.79 m/s,
L´/y = 42.8 / 1.31 = 32.7 > 25.
Therefore use the HIRE Equation.

$$\frac{y_s}{y_1} = 4\, Fr^{0.33} \frac{K_1}{0.55} K_2$$

Fr = 0.79/(9.81 X 1.31) = 0.22

$K_1 = 0.55$, $K_2 = (100/90)^{0.13} = 1.01$

$$\frac{y_s}{1.31} = 4\, 0.22^{0.33} \frac{0.55}{0.55}\ 1.01 = 2.45$$

y_s = 2.45 x 1.31 = 3.2 m

8 Summary

In addition to the material presented in this paper, HEC-18 (Richardson et al. 2001) provides the following information for guidance for the analysis and evaluation of stream stability and scour for bridges.

- Glossary of Terms
- Scour analysis for tidal waterways
- Worked example problems for scour at bridges over tidal and riverine waterways
- Description of the U. S. national scour evaluation program
- Description of the national bridge scour inspection program as-well-as suggested methods
- Case histories of several bridge failures from scour
- Description of the development of a plan of action to protect a scour critical bridge

The plan of action to include:

- Schedule for timely design and construction of a replacement bridge or scour countermeasures
- Development of a scour monitoring and inspection program
- Design and installation of scour measurement instruments to monitor for scour
- Instruction and appointment of personnel to close a bridge if needed
- Discussions of scour in cohesive soils
- Description of equipment and equations to determine the scour rate in cohesive soils (Briaud et al. 1999 a, b)
- Discussion of the consideration of the probability of extreme events such as scour, earthquake, vessel collision etc.
- Additional procedures for determining abutment scour depths (Strum 1999 and ABSCOUR (Chang and Davis 1999 a, b))
- Copy of the U. S. Department of Transportation Technical Advisory
- FHWA 1991 memorandum “Scourability of Rock Formations”

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