Manual

Bridge Scour Manual

March 2013

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Amendment Register

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Foreword

This manual is intended to be a concise summary of the state of knowledge and practice for the design and evaluation of bridges for scour. This manual does not provide a minimum standard nor is it prescriptive. It seeks only to provide guidance to aid with designing adequate structures and avoid expensive and urgent rehabilitation of existing structures. In many cases guidance from qualified and experienced engineers will still be required and site specific requirements should be incorporated. It is recognised that following a flood event, repair works happen rapidly and with limited access to resources. Therefore, chapter three of this manual should be consulted to identify the best available approach with the limited resources to hand.

The work presented herein is based around published guidance adapted for use as an internal resource for the exclusive usage of Departmental employees. It does not constitute engineering advice and further input by experienced RPEQ engineers is still required in interpreting design requirements. Information contained herein relies heavily on the work presented within the fifth edition (2012) of Hydraulic Engineering Circular HEC-18 “Evaluating Scour at Bridges” and companion documents, fourth edition (2012) HEC-20 "Stream Stability at Highway Structures," and third edition (2009) HEC-23 "Bridge Scour and Stream Instability Countermeasures." These three comprehensive documents are the repositories of the latest available procedures, advice notes and guidance on bridge scour. In addition, this manual draws from previous editions and continued research by various agencies (US and UK): NCHRP, CIRIA, DfT, HA, FHWA, various U.S. DOTs (particularly Maryland, Texas and New Jersey), technical associations (Austroads) and universities (particularly University of Auckland). As such, the work published by these institutions is gratefully acknowledged and used herein. Where available, weblinks to the original documents are provided in the reference section. In addition, any figures, tables or substantial text have been specifically referenced.

This manual will be accompanied by advice on scour for culverts, floodways, embankments and minor drainage channels. The reader is directed to Section 2.6 for advice on floodways and chapters nine and ten of TMR’s *Road Drainage Manual 2010* provides further guidance for these structures.

In time, this manual will be revised to provide a concise field guide to address critical aspects within the construction, maintenance, management and remediation phases of structures. This will be harmonised with TMR Standard Drawings, Standard Specifications and other Departmental publications.

Contents

Figures

Tables

Notation

The following symbols are used throughout this manual and are consistent with the nomenclature in the FHWA HEC publication series. Note that Melville and Coleman and CIRIA have adopted different conventions.

A	Flow Area, m^2
A_e	Flow area of the approach cross-section obstructed by the embankment, m^2
a	Pier width, m
D	Characteristic bed particle size
d_{50}, D_{50}	Sediment size for which 50% of the sediment is finer, median sediment size by weight, m
D_{90}	size of bed material such that 90% of the material is finer, m
f	Lacey's Silt Factor
Fr_1	Froude Number directly upstream of the pier, or abutment = $V_1/(gy_1)^{0.5}$
g	Acceleration of gravity (9.81 m/s^2)
K	Coefficient, Holmes
k_1	Coefficient for Richardson and Davis (1995) live-bed general scour equation; $0.59 < k1 > 0.69$ (from mostly contact-bed transport to mostly suspended-bed material transport)
K_1	Correction for pier and abutment shape, FHWA
K_2	Correction for angle of attack of flow, or angle of embankment to flow FHWA
K_3	Correction factor for bed condition, FHWA
K_4	Correction factor for armouring by bed material size, FHWA
K_5	Correction factor for pier width; FHWA
K_{yb}	Depth - pier width factor; Melville and Coleman
K_{yL}	Depth - abutment length factor; Melville and Coleman
K_I	Flow Intensity factor; Melville and Coleman
K_d	Sediment size factor; Melville and Coleman
K_s	Shape factor; Melville and Coleman
K_s*	Adjusted shape factor; Melville and Coleman
K_θ	Flow alignment factor; Melville and Coleman
K_θ*	adjusted flow alignment factor; Melville and Coleman
K_G	Approach channel geometry factor; Melville and Coleman
K_t	Time factor; Melville and Coleman
L	Pier length, m FHWA; abutment length, m Melville and Coleman
L'	Length of abutment projected normal to flow, m

n^*	Manning's roughness coefficient
Q	Discharge, m^3/s; Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s
q	Flow per unit width, Q/W, m^2/s
q_1	Flow per unit width upstream of the pier or abutment, m^2/s
$Q1_m$	flow rate in the approach main channel transporting sediment, m^3/s
Q_2	total flow rate through the contracted section, m^3/s
Q_e	Flow obstructed by the abutment and approach embankment, m^3/s
s	specific gravity of stream bed material
S	hydraulic gradient
s_s	specific weight of sediment (Pa/m^3) density of sediment (kg/m^3) x g (m/s^2)
s_w	specific weight of water (Pa/m^3) density of water (kg/m^3) x g (m/s^2)
V	mean, average or design flow velocity, m/s
V_{max}	max flow velocity, m/s typically $1.25V_{des}$
V	mean, average or design flow velocity, m/s
V_1	approach channel velocity, m/s
V_c	critical mean velocity of flow at the threshold condition for sediment movement, m/s
V_e	Q_e/A_e; m/s
V_{icDx}	Approach velocity, m/s, corresponding to critical velocity for incipient scour in the accelerated flow region at the pier for grain size D_x, (m)
V_{cDx}	Critical velocity, m/s, for incipient motion for the grain size D_x (m)
W	Bottom width of the contracted channel less pier width(s), m Richardson and Davis (1995) clear-water scour
W_1	bottom width of the approach channel, m
W_2	bottom width of the contracted channel, less the pier widths, m
y	mean depth of flow
y_1	flow depth just upstream of the pier or at the abutment, excludes local scour, m
y_a	thickness of the armour layer (m)=2 D_c
y_{bs}	maximum scoured flow depth in a bend, m
y_{cs}	maximum scoured flow depth at a confluence scour hole, m
y_s	Depth of scour measured below upstream bed level
y_{ms}	flow depth from water surface to mean scour depth, m
$\bar{y}_{ms}$	average flow depth in the degraded anabranches approaching a confluence, m

$(y_{ms})c$	flow depth from water surface to mean scour depth in a constricted channel, m
y_r	water level rise from low water to flood stage, m
α	angle of channel confluence, degrees
τc	critical boundary shear stress (Pa)
ϕ	Pier Shape Factor in Froehlich's Equation based on the shape of the pier nose

1 OVERVIEW OF BRIDGE SCOUR

Although much laboratory research has been carried out on scour at particular types of structures, such as bridge piers, there are still significant gaps in knowledge and general understanding. The difficulty of making field measurements at structures during high flows tends to hide the potential seriousness of the problem, because scour holes can refill after the flood peak has passed. The consequent lack of reliable field data has made it difficult to verify predictions of potential scour depths obtained from small-scale laboratory tests. It is likely that many laboratory tests have over-simplified the complex nature of scour.

The principal objective of the Bridge Scour manual is to provide Departmental staff with a concise and practical summary on the causes of scour, how to quantify its effects and how to protect against it. This manual is not a prescriptive guide and available budget, time and resources will also influence the adopted solution. This manual serves as a guide to inform decision-making and avoid inappropriate design. This document is neither a policy nor a minimum design standard.

The manual consists of the following three sections:

Section 1 – An Introduction to Scour

- catchment characteristics
- scour processes
- the scour characteristics at different types of structures

Section 2 – Designing for Scour

- designing new structures to reduce scour
- assessment of existing structures
- estimation of scour risks

Section 3 – Countermeasures for Existing Scour

- scour protection systems
- methods of installation

This manual is concerned with the fluvial environment and scour at bridge structures. It does not cover marine scour or scour predominantly caused by wave action. It should be noted that current knowledge about the facets of scour varies greatly. While scour at bridge piers has been the subject of considerable study, only limited published information is available about scour at revetments and scour depth at abutments in complex floodplains. Therefore, this document is not intended to be a definitive and comprehensive guide. Rather it is intended to concisely summarise the available information and provide a path to more comprehensive and contemporary sources of information.

At present the objective of the initial version is to identify the key issues that need to be considered and suggest general ways in which these factors should be assessed. This document will be expanded to provide chapters on culvert and embankment scour. In addition, a concise field guide is under development, though much of this material is already contained in Section 3. This manual is intended to be a living document updated with experiences from practitioners in Queensland tailored to meet their needs. As such, feedback and suggestions for topics are welcomed.

1.1 An Introduction to Scour

Scour in watercourses and drainage paths causes significant damage to the environment and engineering infrastructure, refer **Figure 1.1**. In order to minimise the long term costs, infrastructure must be designed and protected from scour.

Figure 1.1: Derailment and railway embankment failure during Cyclone Grant on Adelaide to Darwin railway (picture: Michael Franchi)

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. While different materials scour at different rates, the **ultimate scour depth in cohesive or cemented soils can still be as deep as scour in sand-bed streams**. Under flow conditions typical of actual bridge crossings, several flood events are needed to reach ultimate scour. Determining the magnitude of scour is complicated by the cyclic nature of some scour processes. Scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes are refilled with sediment. This is known as live bed scour.

The equations for estimating scour are based on laboratory experiments with limited field verification. While uncertainty in predicting scour still remains, the equations recommended in this document are considered to be the best available methods for estimating scour depths at time of writing.

1.2 Total Scour

Total scour at a bridge crossing considers three primary components:

- long-term degradation of the river bed;
- contraction scour at the bridge; and,
- local scour at the piers and abutments.

These three scour components are shown in **Figure 1.2** below

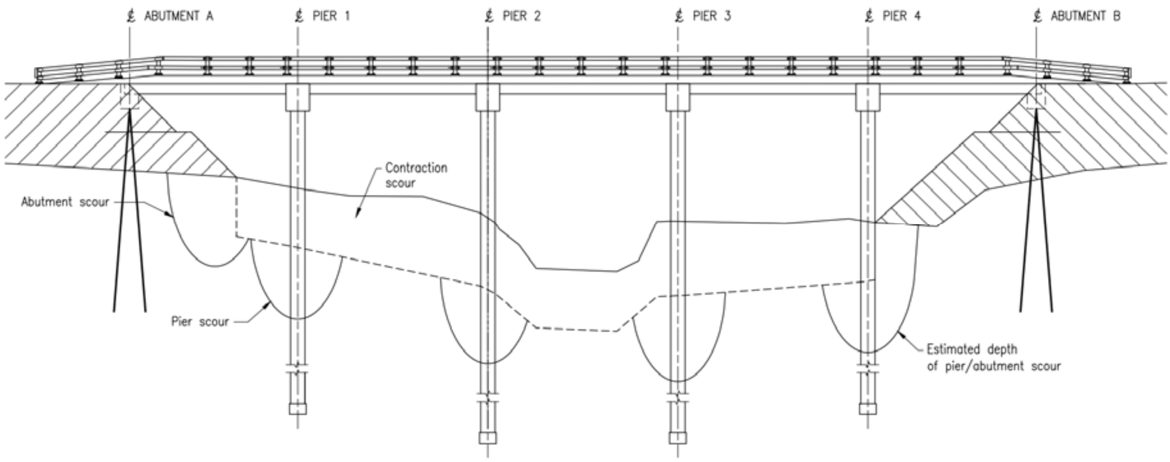


Figure 1.2: Components of scour at a bridge

Adding these scour components together provides the total scour value. This design approach assumes that each component is independent and provides a level of conservatism. However, there are also **other types of scour** that occur in specific situations relating to stream instability. This includes the potential for **lateral migration** of a stream. These types will be dealt with briefly to provide a wider understanding of a scour problem's complexity.

1.3 Catchment Characteristics

Scour results from the natural or man-made contraction of a channel, the migration of a channel, general degradation of the watercourse and the local scour caused by piers and abutments. Studying the land use, hydrology and geomorphology of the catchment provides an understanding of the extent and type of scour that will occur.

The geology, hydrologic variability and anthropomorphic land use changes make Queensland streams susceptible to scour. The natural variation in flows cause the channel to move and change shape. The movement of the channel is caused by continuing removal and deposition of sediment. A channel will migrate and oscillate between aggradation and degradation until a (brief) equilibrium is reached. Disturbances to a catchment and channel, for example the construction of a bridge over a watercourse, have the potential to alter flow conditions during a flood event. The altered flow introduces instability in the channel until a new equilibrium is found.

Queensland streams also have a higher hydrologic variability in Australia or around the world (Weeks, 2010). Arid streams have higher variability of flow and may be more susceptible to scour. Queensland's high variability of flows is a reflection of the high intensity rainfall and less frequent nature of storms. Many Queensland catchments have dispersive soils which are more susceptible to degradation and stream instability. Land use changes will also contribute to scour. Understanding the factors leading to scour will provide better scour protection measures.

1.3.1 Long-term degradation of the river bed

Bridge failure due to scour can be caused by aggradation as well as degradation. Changes to a stream's characteristics, for example dredging, near a bridge can lead to the failure of the bridge. The removal or change in vegetation cover along a river floodplain and within the catchment can also affect bridge scour potential.

Scour in Queensland is a factor of land use changes, flow properties of the streams and the soil type. Much of Queensland has experienced agricultural land clearing. Additionally, highly erosive soils are located across the state. The flood flows in Queensland catchments are typically of a higher intensity with respect to global rivers, and many of the streams are ephemeral. As such, the majority of Queensland streams are scour susceptible.

1.3.2 Indicators of scour

As outlined, the process of scour is a complex relationship between many characteristics. There are a number of indicators that drive the complex process of scour. **Table 1.3.2** includes the stream and catchment characteristics that are indicators of scour. The characteristics may contribute to aggradation (A), degradation (D) or lateral instability (L) in the channel.

Table 1.3.2: Stream Characteristics and indicators of potential general scour, Melville and Coleman Bridge Scour (2000)

Category	Characteristic	Indicator of potential for scour	Potential
Geomorphic	Stream size	Larger size	DL
	Flow habit	Climate change increasing flows	DL
		Climate change decreasing flows	AL
		Ephemeral stream in an arid region	DL
	Valley setting	Natural lowering of the fluvial system	DL
		Lower relief	ADL
		Bridge at downstream section of an alluvial delta	ADL
		Bridge at upstream section of an alluvial delta	AL
	Floodplains	Contraction of the floodplain width	ADL
	Sinuosity	Straight reach of length greater than 10 channel widths	ADL
		Bend located upstream of the bridge	ADL
		Meander growth and shift – possibly evidenced by wide (un-vegetated) point bars opposite cut or slumped banks	ADL
		Recently formed ox-bow lake (billabong)	ADL
	Braided or anabranched streams	Island formation and shift	ADL
		Confluence formation and shift	ADL
	Width variability and bars	Constriction of the channel width	ADL
		Constriction of the bridge waterway	ADL
		Wide un-vegetated zones on point bars	ADL
		Equi-width stream on narrow point bars	DL
		Random width stream with wide, irregular point bars	ADL
		Channel bar formation and shift	ADL

Category	Characteristic	Indicator of potential for scour	Potential
	Slope control point	Bedrock, boulder or weir control removal	DL
		Nickpoint (headcut) erosion and migration	DL
	Channel boundaries	Channel boundaries composed of alluvium	ADL
	Bed material	Erodible material	ADL
	Banks	Erodible bank material	ADL
		Indicators of active bank erosion	ADL
		Bank slopes greater than 30% or little woody vegetation cover on the banks	ADL
		Wide un-vegetated zones on point bars	ADL
	Bed levels	Decreasing	DL
		Increasing	AL
	Channel slope	Decreasing	ADL
		Increasing	ADL
Hydraulic	Flood flows	Flows of large magnitudes	DL
	Flood stages	Increasing for same frequency flows	AL
		Decreasing for same frequency flows	ADL
	Flood frequencies	Increasing	DL
	Water surface profile (tidal influence)	Lowered downstream control level (river, lake or	DL
		Raised downstream control level (river, lake or sea)	AL
Land use changes	Land movement in catchment	Deposition of landslide and bank material in the stream system	ADL
	Deforestation	Exposed land surface and loosened sediment	ADL
		Loose debris (not sediment)	DL
	Agricultural activity	Exposed and loosened ground surface	ADL
	Land clearing	Exposed and loosened ground surface	ADL
	Fire	Exposed and loosened ground surface	ADL
	Urbanisation	Covering and sealing of ground surface	DL
	Catchment vegetal cover	Increasing or decreasing	ADL

Category	Characteristic	Indicator of potential for scour	Potential
	Riparian vegetation	Removal or vegetation	ADL
	Channel straightening	Upstream channel straightening	DL
		Downstream of local channel straightening	AL
	Channelisation	Constraining flows and sediments	ADL
	Artificial cutoff formation	Cut-off formation to shorten flow path	ADL
	Flow diversion or confluence upstream or downstream of the bridge site	Flow with low sediment load leaving the stream	AL
		Flow with high sediment load leaving the stream	DL
		Flow with low sediment load entering the stream	DL
		Flow with high sediment load entering the stream	AL
	Dam construction	Construction upstream of the bridge site	DL
		Construction downstream of the bridge site	AL
	Dam removal	Removal upstream of bridge site Removal downstream of bridge site	AL DL
	Sediment dumping	Dumping of waste sediment into the stream system	AL
	Dredging and streambed mining	Removal of sediment from the stream system	DL
	Channel clearing	Removal of debris from the stream system	DL

A = Aggradation, D= Degradation, L = Lateral Instability

It is difficult to produce an accurate prediction of scour due to the inter-related nature of many of the characteristics listed above. Numerous scour estimation methods exist for predicting general and local scour, however they are based around laboratory studies that simplify actual characteristics of catchments.

1.3.3 Stream stability and migration

Scour is a natural process that drives the evolution of the land. Over time, erosion causes changes to the shape and size of rivers. As the channel tries to reach an equilibrium state, the river will evolve from a straight channel to a meandering channel and then to a braided channel. **Figure 1.3.3A** shows the evolution of an incised channel. Eventually, the channel reaches a point of relative stability. The watercourse will continue to respond to changes as it moves towards a state of stability.

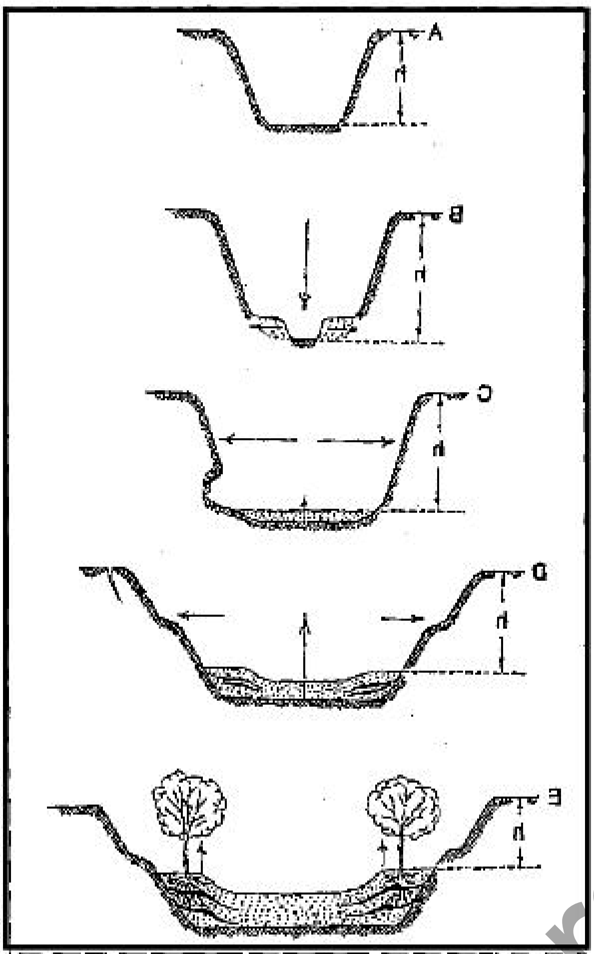


Figure 1.3.3A: Evolution of incised channel from initial incision (A, B) and widening (C,D) to aggradation (D,E) and eventual relative stability (Schumm et al 1984)

Streams are dynamic and the channel moves both laterally and deeper. The most dynamic form is a braided stream. Scour frequently occurs at the confluence of the two channels. This scour depth can be 1 to 2 times the average flow depth.

Figure 1.3.3B compares natural channel classifications with their relative stability. Stability is influenced by channel type, size and shape, as well as the sediment load in the channel. The stability of the channel is related to how easily its shape can change.

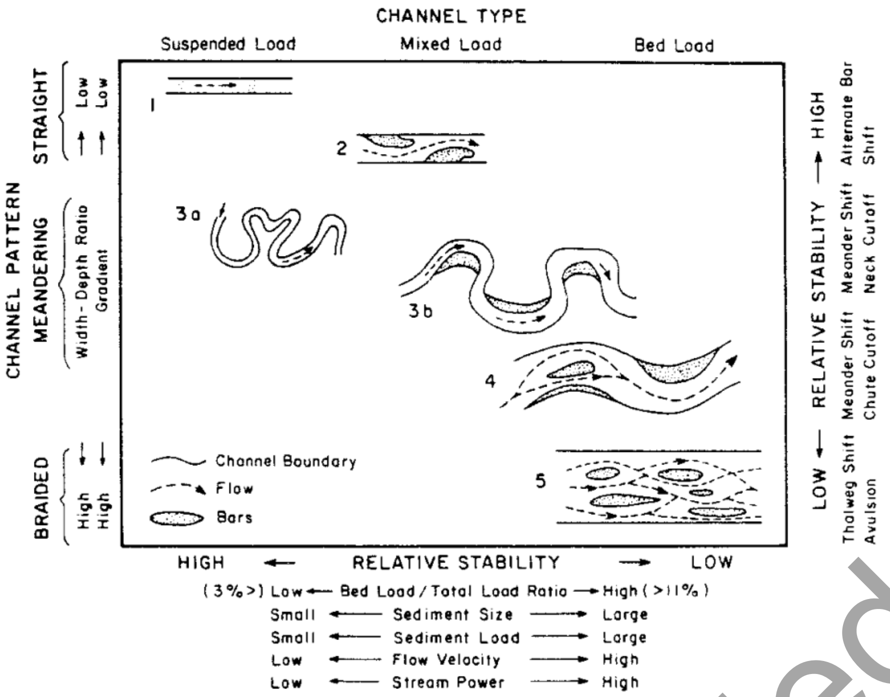


Figure 1.3.3B: Channel pattern and relative stability (Shen et al, 1981)

Conversely, a bridge crossing is static. Once built, it fixes the watercourse at one place in time and space. A meandering stream moving laterally will erode a bridge approach. This will affect contraction and local scour because of changes in flow direction. Scour at the bridge may be gradual, or the result of a single major flood event. Also, the direction and magnitude of the stream's movement is not easily predicted. These factors are discussed below and comprehensive analysis techniques are presented in HEC-20.

Riverbank Failure

The material in the river bank will determine the stability of the bank. Non-cohesive soils are likely to be washed away particle by particle. Cohesive soils are less affected by surface velocities and are more likely to fail due to mass wasting.

Erosion of non-cohesive banks is related to: particle size, bank slope, the direction and magnitude of the velocity adjacent to the bank, turbulent flow, shear stress exerted on the banks, seepage forces, piping, and wave forces.

The failure of river banks can be determined through a geotechnical analysis. **Figure 1.3.3C** shows the typical failure surfaces for cohesive and non-cohesive soils. To determine the mode of failure of stratified banks a geotechnical analysis may be required.

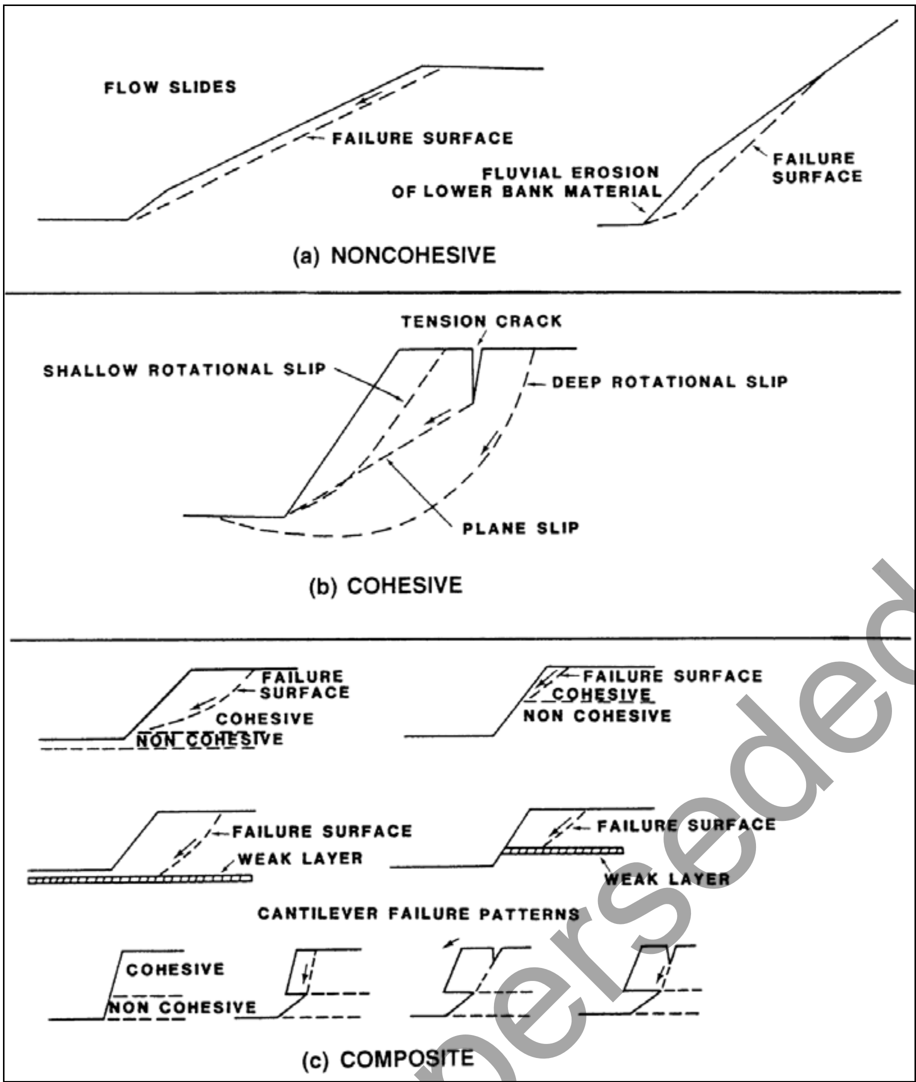


Figure 1.3.3C: Typical bank failure surfaces: (a) non-cohesive, (b) cohesive, and (c) composite (Brown, 1985)

Bend Scour

Straight channels are highly unstable, so natural streams have bends or meanders. As the flow direction changes at a bend, the force of the water can cause scour. Scouring of the bend causes the channel to move. Migration of meandering channels can be quite significant. The movement of a meander may lead to the failure of nearby infrastructure, such as a bridge.

Figure 1.3.3D is a plan view of a typical meandering stream and the hydraulic features associated with bends in channels.

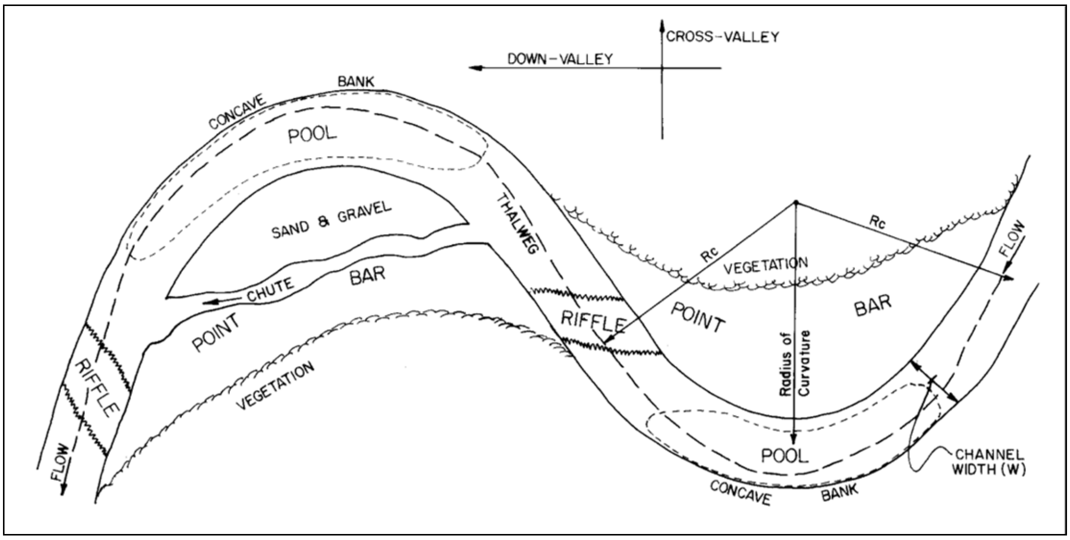


Figure 1.3.3D: Plan view of a typical meandering stream (HEC20, 2012)

Cutback Scour

Cut-back scour occurs when the stream bed is lowered at one location by scour or dredging of the bed is carried out. This situation results in a step in the stream bed gradient. To return to a more uniform gradient and energy head the stream will cut back the bed upstream and in so doing will lower the bed level from a maximum at the scour hole or dredging location. If a bridge is located upstream of a scour hole or dredging location there is a risk that the bed levels at the bridge will be lowered. The implications for a bridge will depend on the depth of scour and the foundation levels or the pile embedment length.

Overview of stream stability measures

Measures to address lateral shifting and stream instability may include realignment of the road, changes in bridge design, construction of river control works, protection of abutments with riprap, or careful monitoring of the river in a bridge inspection program. To accommodate future channel migration, consider placing footings/foundations at the same elevation as those located in the main channel. Lateral shifting will require river training works, bank stabilisation with riprap, and/or guide banks. The design of these works is beyond the scope of this document. Design methods are given by FHWA in HEC-23 (FHWA 2009), HDS 6 (FHWA 2001), and similar publications.

The geology and geomorphology of a bridge crossing needs to be studied to determine the potential for long-term bed elevation changes at a bridge site. Quantitative techniques for streambed aggradation and degradation analysis are covered in detail in HEC-20 (FHWA 2012). These techniques include:

- incipient motion analysis;
- analysis of armouring potential;
- equilibrium slope analysis; and,
- sediment continuity analysis.

Additionally, sediment transport concepts and equations are discussed in detail in HDS 6 (FHWA 2001), and HDS 7 (FHWA 2012).

1.4 Contraction Scour

Contraction scour is caused by a constriction in the floodplain, this can occur naturally between rock outcrops preventing the stream from migrating. Likewise, contraction scour at infrastructure occurs when the flow area of a stream is reduced, either by a bridge or when overbank flow is confined by roadway embankments. From the continuity principle, a decrease in flow area results in an increase in average velocity and bed shear stress. This increases erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. The quantity of transported bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until equilibrium is reached; i.e. the bed shear stress decreases such that no sediment is transported out of the reach. Contraction scour is different from long-term degradation in that contraction scour occurs in the vicinity of the constriction (bridge), it may be intermittent, and/or related to the passing of a particular flood event.

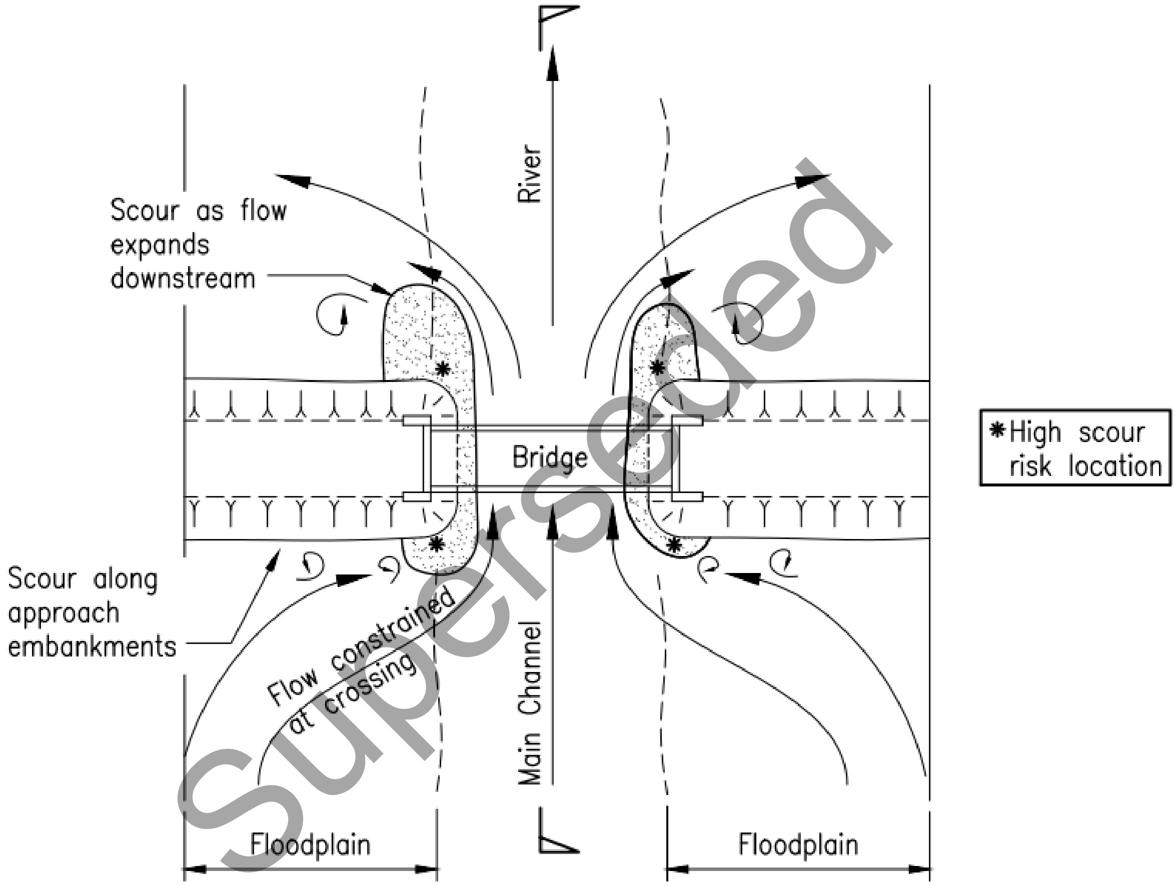


Figure 1.4: Contraction scour and high risk locations

1.4.1 Clearwater scour

Clear-water scour occurs when either there is no bed material transported from the upstream reach into the downstream reach. In other words the velocity of the river is less than the critical velocity of the bed material in the river (i.e. $v/_{vc}$ <1). The maximum local scour depth is reached when the flow can no longer remove bed material from the scour area. An example of this is an upstream weir allowing sediment to settle out behind a weir before it spills over. The flow then allows additional bed material to be taken into suspension.

With clear-water contraction scour, the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress ($_{\tau o}$) on the bed is equal to the critical velocity (V_c) or the critical shear stress ($_{\tau c}$) of a certain particle size (D) in the bed material.

Critical Velocity $V_c = 6.19 {y_{ms}}^{1/6} {D_{50}}^{1/3}$

The recommended clear-water contraction scour equation is:

Clear Water Equation HEC-18

V/V_c < 1,

$$\left(y_{ms}\right)_c = \left[\frac{0.025Q^2}{D_m^{2/3}W^2}\right]^{3/7}$$

$$y_s = \left(y_{ms}\right)_c - y_0$$

D_m = 1.25 D_{50}.

1.4.2 Live bed 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. Therefore the stream velocity is greater than the critical velocity of the bed material (v/$_{vc}$ > 1). With live-bed contraction scour the area of the contracted section increases until sediment transport out of the contracted section equals the sediment transported in.

Live-bed contraction scour depths may be limited by armouring of the bed by large sediment particles in the bed material. 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 then using the smaller of the two depths.

Live-bed contraction scour is typically cyclical and due to the high suspended sediment load, more abrasive. For example, the bed scours away during the rising stage of a runoff event and fills on the falling stage. The cyclic nature of contraction scour causes difficulties in determining contraction scour depths after a flood. As such, this is why scour depths need to be calculated and why post flood inspections are necessary.

Figure 1.4 indicates the relative development of scour at a pier over three flood events. Clear water contraction scour can be caused by the approaches to a bridge cutting off floodplain flow. Material is progressively lost from the abutments and not replaced. This can cause clear-water scour at a setback portion of a bridge section or a relief bridge/culvert because the out of bank flow does not normally transport significant concentrations of sediment bed material. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment.

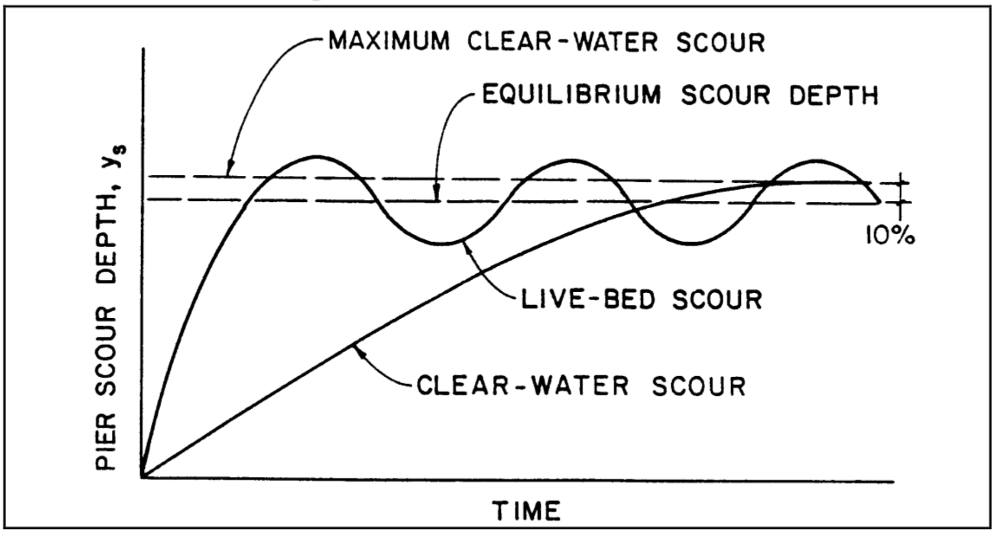


Figure 1.4.2: Pier scour depth as a function of time

To determine if transport of bed material is likely, compute the critical velocity at the approach section for the D50 of the bed material and compare to the mean velocity at the approach section. To determine if the bed material will be washed through the contraction determine the ratio of the shear velocity (V*) in the contracted section to the fall velocity (ω) of the D_{50} of the bed material being transported from the upstream reach. If the ratio is much larger than 2, then the bed material from the upstream reach will be mostly suspended bed material discharge and may wash through the contracted reach (clear-water scour).

Other factors that can cause contraction scour are:

- natural stream constrictions,
- long highway approaches to the bridge over the floodplain,
- debris accumulation,
- natural berms along the banks due to sediment deposits,
- vegetative growth in the channel or floodplain, and
- pressure flow.

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.

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 (Laursen 1960). The equation assumes that bed material is being transported from the upstream section.

Live-Bed Conditions $V/V_c \geq 1$, Richardson and Davis 1995 modified from Laursen 1960	$0.59 < k_1 > 0.69$ mostly contact-bed transport to mostly suspended-bed material transport	$\frac{(y_{ms})_c}{y_{ms}} = \left(\frac{Q_2}{Q_{1m}}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1}$ $d_s = y_o - (y_{ms})_c$

where:

y_{ms}	= Average depth in the upstream main channel, m
V	= mean (upstream) flow velocity, m/s
Q_{1m}	= flow rate in the approach main channel transporting sediment, m3/s
Q_2	= total flow rate through the contracted section, m^3/s, $Q_2 = Q1_m$ x % area open through bridge
W_1	= bottom width of the approach channel, m
W_2	= bottom width of the contracted channel, m
W	= Bottom width of the contracted channel less pier width(s), m
k_1	Exponent = 0.59 if V*/w < 0.5 (mostly contact-bed transport to mostly suspended-bed material transport) Exponent = 0.64 if V*/w = 0.5 – 2.0 (some suspended-bed material transport) Exponent = 0.69 if V*/w > 2.0 (mostly suspended-bed material transport)

$(y_{ms})c$ = Average equilibrium depth in the contracted section after contraction scour, (m)

y_0 = Average existing depth in contracted section, (m)

y_s = Depth of scour, m

1.4.3 Pressure flow scour (Vertical contraction scour)

Figure 1.4.2 and **1.4.3A** illustrate the flow characteristics at a fully submerged bridge superstructure. Note that the bridge superstructure mentioned in this section refers to a continuous cross section of the structural (i.e. deck) and non-structural (i.e. guardrail) elements that span the waterway. These elements can produce significant blockage when partially or fully inundated.

Figure 1.4.3A: Overtopping of the Burke and Wills bridge is an example of vertical contraction scour

Discharge under the superstructure can be conservatively assumed to be all approach flow below the top of the superstructure at height (h_b + T), where h_b is the vertical size of the bridge opening prior to scour and T is the height of the obstruction, i.e. girders, deck, and parapet.

For non-overtopping flood events, all discharge upstream goes into the bridge opening. The depth at the location of maximum scour is comprised of three components: h_c, the vertically contracted flow height from the streamline bounding the separation zone under the superstructure at the maximum scour depth, y_s, the scour depth, and t, the maximum thickness of the flow separation zone.

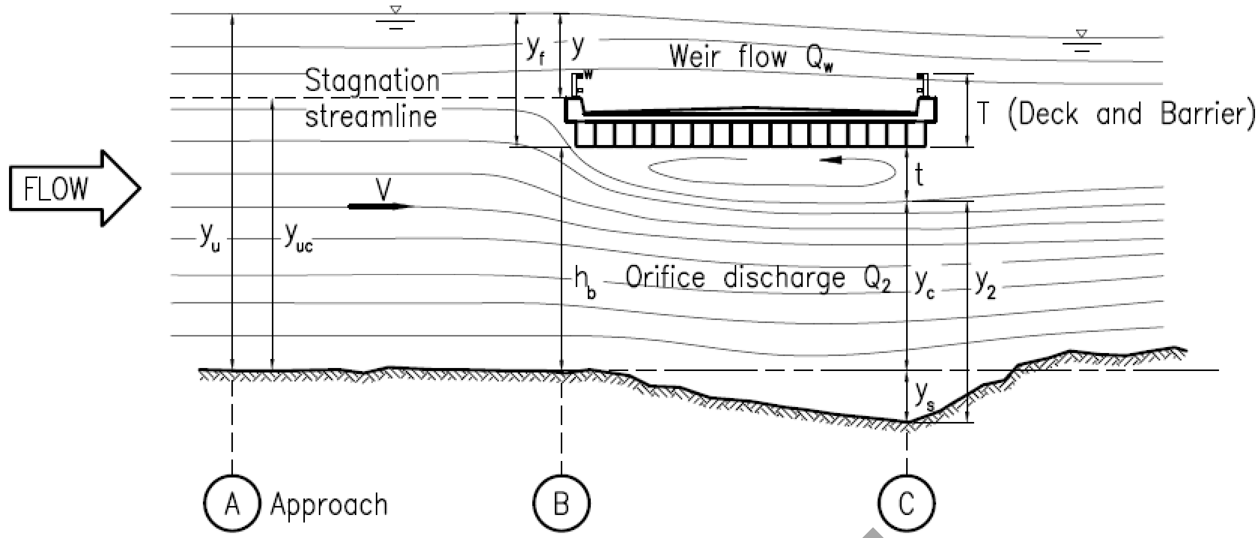


Figure 1.4.3B: Vertical contraction and definition for geometric parameters (HEC-18, 2012)

The pressure scour depth y_s is determined by using the horizontal contraction scour equations to calculate the height, y_s + y_c, required to convey flow through the bridge opening at the critical velocity.

1.5 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. The processes driving local scour are complex. Interruptions to fluid flows will alter the velocity and pressure distributions around and downstream of the obstruction. Vortices will form within the separated layer. **Figure 1.4.3B** and **Figure 1.5A** show the processes behind localised scour at piers and abutments. Vortices form upstream and downstream of pier and abutment

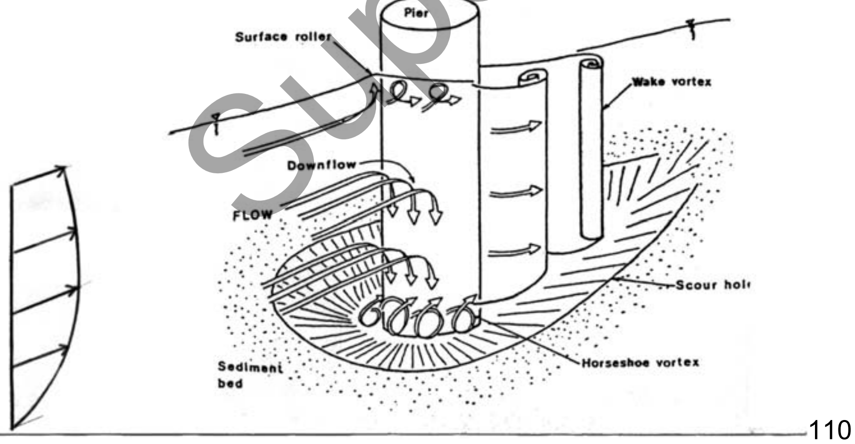


Figure 1.5A: Scour at a bridge pier (HEC-18, 2012)

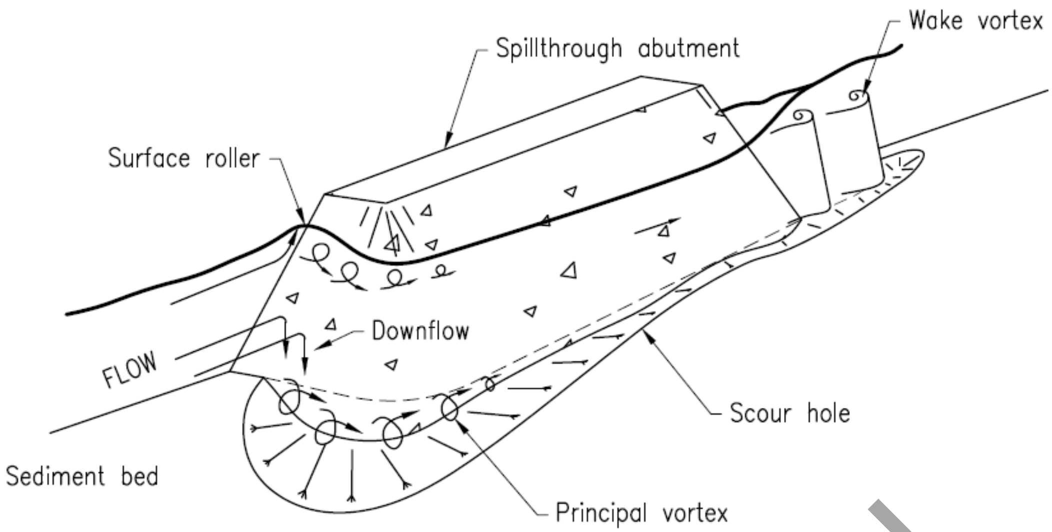


Figure 1.5B: Scour at a bridge abutment (HEC-18, 2012)

Flow around vertical cylinders, such as bridge piers will be turbulent. The resulting vortex system will consist of flows moving in a downward direction in front of the pier. The velocity of the flow will push the vortex system around the pier. When observed in plan view the vortex system resembles a horseshoe. Horseshoe vortices will become stable only after an equilibrium scour depth has formed.

1.5.1 Local scour - bridge piers

The design and configuration of a bridge substructure will impact on scour development at the bridge piers and abutments. Local scour at piers can lead to severe damage to footings as shown in **Figure 1.5B**. The shape of the piers and the footing type alter the flow pattern around the pier. While pier design is dependant on site specific factors such as the superstructure, soil conditions and construction procedures, the pier's influence on the flow should also be considered.

Figure 1.5.1A: Local scour damage at piles on the Logan River

Hydrodynamically shaped piers help reduce the generation of turbulent flow. Flow alignment will contribute to increased erosion. A river will respond to alterations to flow conditions through erosion until an equilibrium state is reached.

To understand pier scour, it is necessary to understand the flow field at a pier, and how the flow field varies with pier width and shape. Flow depth and foundation material are also important measures.

The erosive forces exerted on the foundation material are generated by flow contraction around the pier, namely by a pronounced down-flow along the pier's leading edge. Variations of pier width and shape, and flow depth, alter this flow field and will either enhance or weaken these flow features.

Figure 1.5.1A depicts the arrangement of flow at a pier. Flow approaching the pier decelerates, impinges against the pier's centerline, and then strongly deflects both down and up the pier's face. These two vertical flows act almost as wall-attached jet-like flows along the pier's centerline, one directed up toward the free surface, and the other down toward the bed. The down-flow is driven by the resulting downward gradient (below the still water level) of stagnation pressure along the pier's leading face. As the scour hole develops, the down-flow is augmented by the approach flow diverging into the scour hole (NCHRP 2011a).

In addition to the vertical component of flow at the pier's leading face, flow contracts as it passes around the sides of the pier and local values of flow velocity and bed shear stress increase. For many piers, the increases are such that scour begins at the sides of a pier. Once the scour region develops as a hole fully around the pier, the down-flow and the horseshoe vortices strengthen. Scour-hole formation draws flow into the hole.

The flow field, during all stages of scour development, is marked by the presence of organised, coherent turbulence structures, notably:

- A horseshoe vortex system forms around the leading perimeter of the pier. These vortices wrap around the base of the pier such that the legs are oriented approximately parallel to the approaching flow. The legs break up and are shed intermittently;
- Small but very energetic elongated eddies (vortex tubes whose main axis is approximately vertical relative to the bed) in the detached shear layers;
- Large-scale rollers or wake vortices, which form behind the two flanks of the pier, and are shed into its wake. As they convect away from the pier, the wake vortices expand in diameter, then dissipate and break up;
- A horizontal vortex formed by flow passing over the stationary, depositional mound formed at the exit slope from the scour hole. The location and size of the mound depend on the power of the wake vortices shed from the pier (the weaker the vortices, the closer the mound to the pier); and,
- A surface roller situated close to the junction between the free surface and the upstream face of the pier. The roller is akin to a bow wave of a boat moving through water.

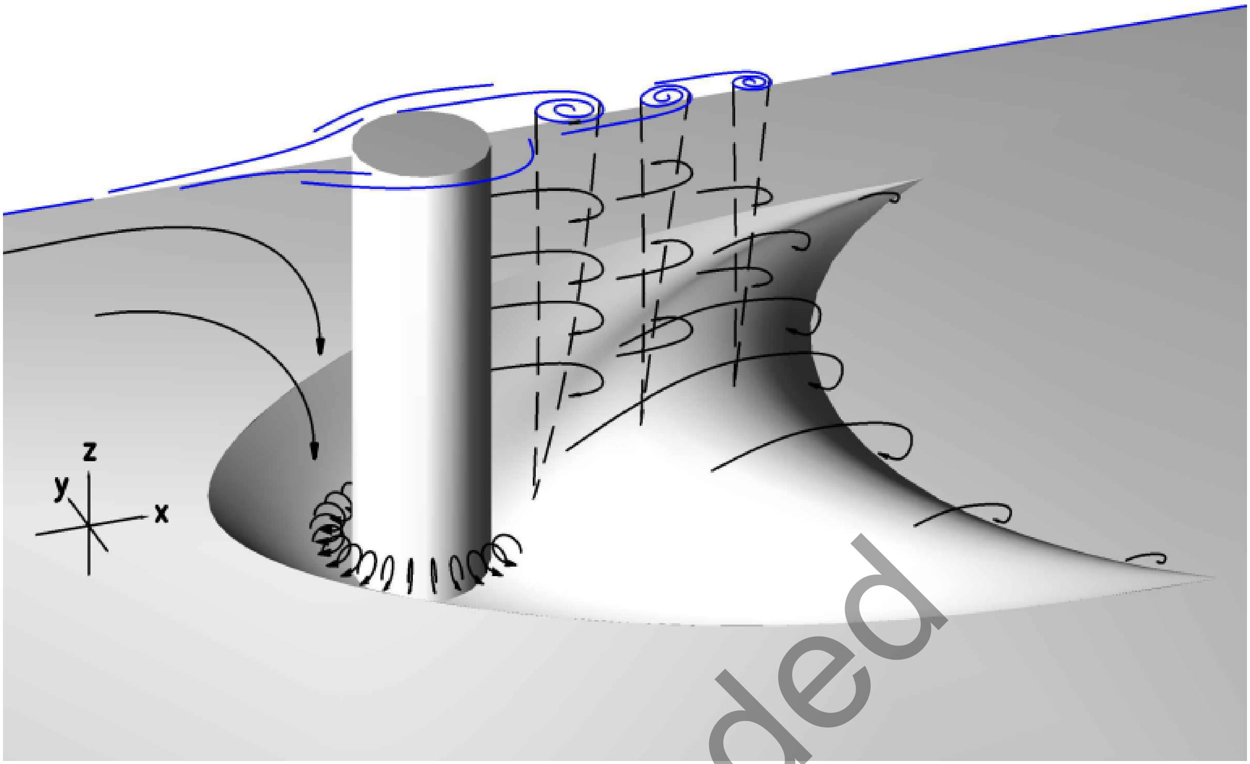


Figure 1.5.1B: Flow profile around a circular bridge pier. Vortices form downstream. (HEC18, 2012)

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics and the geometry of the pier and footing.

Granular bed material ranges in size from sand to large boulders and is characterized by the D50 and a coarse size such as the D90 size. Flow characteristics for local pier scour are the velocity and depth just upstream of the pier, the angle of the velocity vector at the pier (angle of attack) and water surface level, and if applicable, pressure flow conditions.

Pier geometry characteristics include type, dimension and shape. Dimensions are the diameter for circular piers, spacing for multiple piles, and width and length for 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 pier exposed to the flow. A complex pier may have the pier, footing or pile cap, and piles exposed to the flow.

Local scour at piers has been studied extensively in the laboratory; however, there is limited field data to confirm the results of laboratory work. The laboratory studies have been mostly of simple piers, but there have been some laboratory studies of complex piers. Often the studies of complex piers are model studies of actual or proposed pier configurations. As a result of the many laboratory studies, there are numerous pier scour equations. In general, the laboratory derived equations are for live-bed scour in (cohesionless) sand-bed streams.

In summary, the down-flow impingement on the bed, along with the wide range of turbulence structures present in the flow field, entrain and transport material from the scour hole. The details and interaction of the flow field vary with pier shape, angle of attack, and the stage of scour development between initiation and equilibrium, but the essential consideration is that these flow features are responsible for scour.

1.5.2 Bridge pier size

Piers can be categorised in terms of the relationship between flow depth, “y” and pier width, “a”. These three categories produce significantly different pier scour morphologies:

- narrow piers ($y/a > 1.4$), for which scour typically is deepest at the pier face;
- transitional piers ($0.2 < y/a < 1.4$); and,
- wide piers ($y/a < 0.2$), for which scour typically is deepest at the pier flank

The pier flow field may become more complicated if the pier has a complex shape, such as a pier supported on a pile cap underpinned by a pile cluster. Additionally, the close proximity of an abutment and/or a channel bank further complicates the flow field.

Narrow piers ($y/a > 1.4$)

The main features of the flow field at narrow piers can be explained by viewing the flow field and scour at an isolated cylindrical pier in a relatively deep, wide channel. An interacting and unsteady set of flow features entrains and transports sediment from the pier foundation. The following features evolve as scour develops, namely:

- flow impact against the pier face, producing a down-flow and an up-flow with roller;
- flow converging, contracting, then diverging;
- the generation, transport and dissipation of large-scale turbulence structures (macro-turbulence) at the base of the pier-foundation junction (commonly termed the horseshoe vortex); and
- detaching shear layer at each pier flank; with wake vortices convected through the pier's wake.

Transition piers ($0.2 < y/a < 1.4$)

The main flow-field features described for narrow piers exist in the flow field of piers within the transition range of $0.2 < y/a < 1.4$, but the features now begin to alter in response to reduction of depth and/or increase in pier width. As this ratio decreases, it partially disrupts the formation of the features, and thereby reduces erosive strength. The down-flow at the pier face is retarded as it has a shortened length over which to develop, whereas the up-flow “bow wave” remains essentially unchanged. The circulation of the large-scale turbulence structures (Horseshoe vortex) weakens as the down-flow weakens, and the vertically aligned turbulence structures (wake vortices) also weaken due to the increased importance of bed friction in a shallow flow.

Wide piers ($y/a < 0.2$)

For wide piers, the flow approaching the pier decelerates, turns, and flows laterally along the pier face before contracting and passing around the sides of the pier. The down-flow at the pier face is weakly developed, and only slightly erodes the foundation at the pier’s centre. Erosive turbulence structures now principally comprise wake vortices. Scour is deepest along the pier flanks. (NCHRP 2011a and c).

For a given flow depth, greater pier width increases flow blockage and therefore causes more of the approach flow to be swept laterally along the pier face than around the pier's flanks. Increased blockage modifies the lateral distribution of approach flow over a longer distance upstream of a pier. The flow field around each side of a wide pier is essentially the same as those at some types of abutment.

1.5.3 Local scour - bridge abutments

Scour occurs at abutments when the abutment and roadway embankment obstructs flow. Several causes of abutment failures during post-flood field inspections of bridge sites have been documented. **Figure 1.5.1B** shows the scour damage at an abutment.

Figure 1.5.3A: Scour of abutment protection

These failures were due to:

- overtopping of abutments or approach embankments;
- lateral channel migration or stream widening processes;
- contraction scour; and/or,
- local scour at one or both abutments

Note that failure of piers is the least common failure mechanism. Abutment damage is often caused by a combination of these failure modes. As a general rule, the abutments that are most vulnerable to damage are those located at or near the channel banks. Where abutments are set-back from the channel banks, especially on wide floodplains, large local scour holes have been observed with scour depths as much as four times the approach flow depth.

Flow through a bridge waterway narrowed by a bridge abutment is essentially flow around a short streamwise contraction. **Figure 1.5.3A** illustrates the characteristic flow features and the link between the contraction and the formation of a complex flow field around the abutments.

The flow width narrows and the flow accelerates through the contraction, generating macro-turbulence structures (eddies and various vortices spun from the contraction boundary) that shed and disperse within the flow. Flow contraction and turbulence at many bridge waterways, is complicated by the shape of the channel.

The flow obstructed by the abutment then accelerates and often forms a vortex starting at the upstream end of the abutment and running along the toe of the abutment. Generally a wake vortex forms at the downstream end of the abutment.

Alluvial non-cohesive sediment (sands and gravels) most frequently form the bed of a main channel. Whereas the channel's floodplain may be formed from considerably finer sediments (silts and clays), typically causing the floodplain soil to be more cohesive in character than the bed sediment of the main channel. The banks of the main channel usually are formed from the floodplain soils. This may allow them to behave cohesively and stand at a fairly steep slope.

Note that most abutments have an earthfill approach embankment formed of compacted soils. The soils may have been excavated from the floodplain or have been brought to the bridge site from elsewhere. The earthfill embankment is placed and compacted to a specific value of shear strength to support the traffic load.

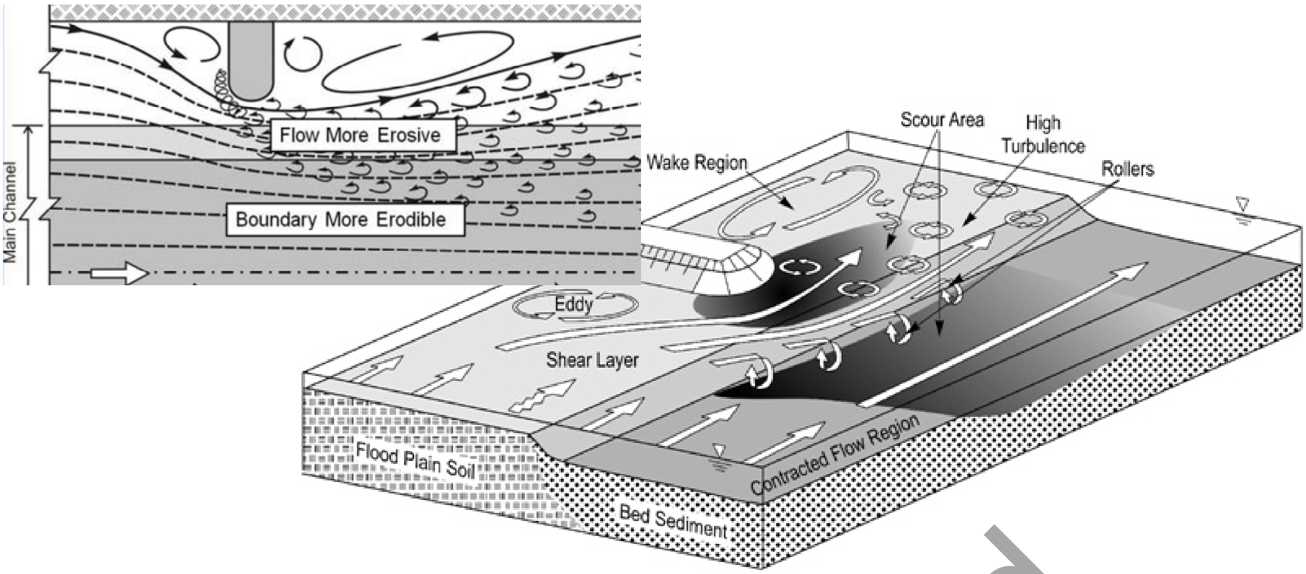


Figure 1.5.3B: Flow structure generated by floodplain/main channel flow interaction (NCHRP 2011b)

Abutment scour depends on the interaction of the flow obstructed by the approach and the flow in the main channel. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length. Abutment scour depth depends on abutment shape, flow in the main channel, flow intercepted by the abutment and directed to the main channel, sediment characteristics, cross-sectional shape of the main channel (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment. In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. Most of the early laboratory research failed to replicate these field conditions.

2 DESIGNING FOR SCOUR AT HIGHWAY STRUCTURES

2.1 Reducing Scour by Design

The aim of this section is to provide a method for estimating scour depths and outline the approaches for minimising the risk of scour during the design stage. The third section of the manual provides guidance on scour protection measures. However, the best scour countermeasure is considered design. **Figure 2.1A** is the Burke and Wills Bridge after the 2010 flood event on the Cooper Creek. Many of the problems experienced were due to inadequate allowance for flooding during the design stage. Only typically experienced scenarios are covered here and consultation with other reference material is still advised. Equations for more complicated pier arrangements including wide piers, debris blockage, pressure flow and the NHRCP 24-20 abutment scour approach are also available but are beyond the scope of this document.

Figure 2.1A: Pier and abutment scour due to inadequate design

The first section introduced stream stability and the inter-related factors that contribute to scour. To estimate scour depth it is necessary to understand the flow field around individual structural components. The estimation and prevention of scour at a site requires the main factors contributing to scour to be identified and understood.

Hydraulic modelling of a bridge site is an integral part of any bridge design. These studies should address both the sizing of the bridge waterway and to ensure that the foundations can be designed to minimise scour. The scope of the hydraulic analysis should be commensurate with the complexity of the situation, the importance of the road crossing and consequences of failure.

Consideration must be given to the limitations and gaps in existing knowledge when using currently available formulae for estimating scour. The interdisciplinary team needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data and conditions at the site to achieve a reasonable design. Such data should include:

- performance of existing structures during past floods,
- effects of regulation and control of flood discharges, and
- hydrologic characteristics and flood history of the stream.

It must be recognized that damage to bridge approaches from rare floods can be repaired relatively quickly to restore traffic service. On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards as well as significant social impacts and economic losses for a prolonged period of time. Therefore, scour resistant bridge foundations should be designed to a higher hydraulic standard. These concepts are reflected in the following general design procedure.

There are many methods, and these include equations by Holmes, Neill, Faraday and Charlton, Melville and Coleman, the CSU equation, FHWA HEC-18 equation, Froehlich equations and HIRE equations. Much of the available research still does not completely explain the complex phenomenon of scour nor will it ever provide results able to be calibrated. Technological advances have allowed for more detailed measurements of the factors relating to fluid and sediment movement. The process is very complicated making it difficult to develop a simple and accurate method to estimate scour for all field scenarios. All methods were derived from laboratory studies and many over-simplify catchment characteristics and are derived from flume models using non-cohesive particles. Furthermore, scour develops and occurs irregularly depending on a range of factors including the duration of flood events and antecedent conditions. The development of scour depth is dependant on certain conditions being reached during a particular flood event, the uniformity of particle size may also effect the development of scour, refer **Figure 2.1B**.

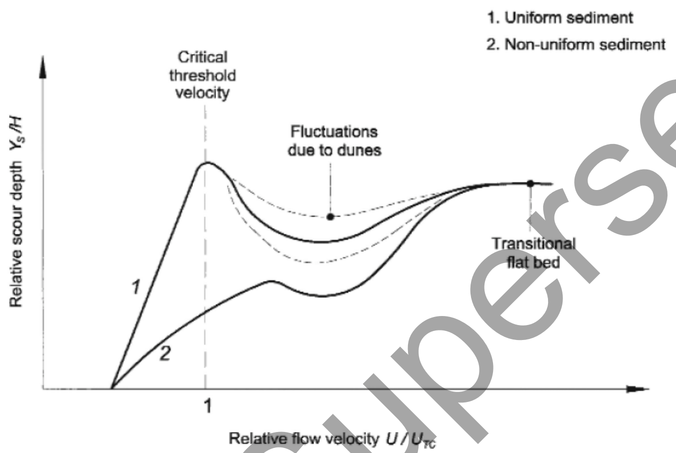


Figure 2.1B: Variation of scour depth dependant on flow velocity and particle uniformity (CIRIA, 2005)

2.2 Summary of Scour Reduction

During a major flood, higher-than-average flow velocities may cause a short-term lowering of bed levels. There may also be a tendency for the flow to attack the banks and thereby widen the channel. When designing structures to withstand possible scour, it is assumed that any erosive action is primarily concentrated towards the bed. The amount of short-term scour that occurs within a channel during a single flood event is difficult to predict because information on rates of natural scour is very limited.

A key factor in the general lowering of the bed level is that it will only occur if the rate at which sediment is transported downstream from the reach exceeds the rate at which sediment arrives from upstream. An overall increase in higher flow velocity and transport rate does not cause scour. Scour is caused by an imbalance between the amounts of sediment in transport at the upstream and downstream ends of the channel reach. Principally, any existing transport imbalance is accentuated during a flood. Particular consideration should be given to cases where upstream features (natural or another structure), act to limit the amount of sediment entering a reach that contains a structure susceptible to scour. Similarly, the removal of a downstream flow control structure could increase downstream flow velocities and sediment transport rates, leading to progression of erosion upstream.

One possible method of predicting short-term changes in bed level is to use a numerical model such as the two-dimensional sediment transport (morphological) and hydrodynamic models. Models can provide useful information about the behaviour of large, complex rivers and their use should be considered for major projects in which the effects of scour could be significant. However, the limitations of this type of model must be recognised. The data requirements, available time and costs involved in this type of numerical modelling may not be appropriate for smaller projects for more localised impacts. Wherever possible, the ability of a two dimensional morphological model to reproduce past changes in channel geometry should be calibrated before it is used to predict short-term or long-term changes in the future.

2.3 Scour Design Exceedance Probability

As noted in Section 2.1, bridge foundations should be designed to withstand the effects of scour caused by hydraulic conditions from floods much larger than the design flood. Economic analysis and experience with actual flood damage indicates that it is almost always cost-effective to provide a foundation that will not fail, even from very large events. Using the Risk of Failure Equation 11.3 from *Australian Rainfall and Runoff* 1987 it can be seen that during a 50 year design life there is a 39.5 percent chance that a bridge designed to pass the 1% AEP flood (100y ARI) will experience that scale flood or larger, refer **Figure 2-3**. [NOTE: 50 year design life and 50 year ARI flood event are not related concepts].

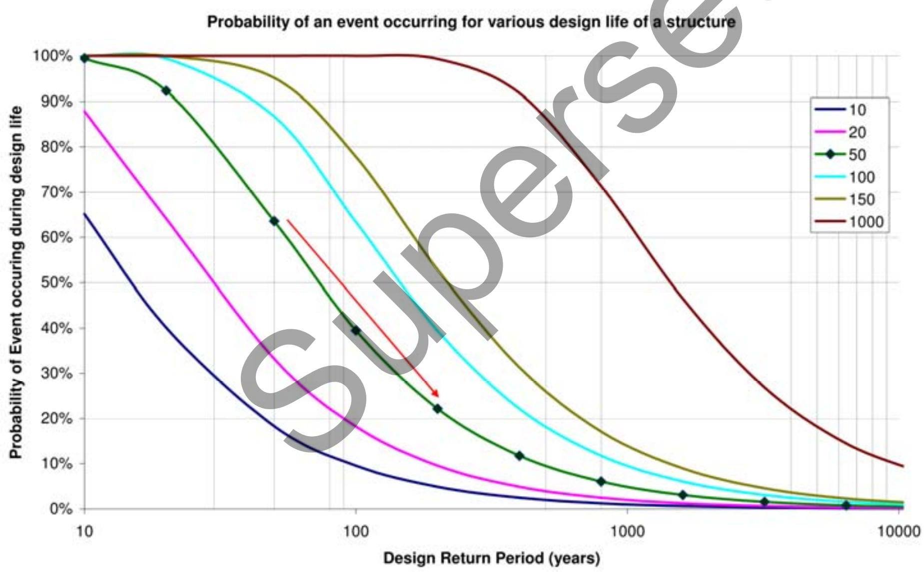


Figure 2-3: Probability of one or more occurrence during design life

Similarly, there is a 63 percent chance that a bridge that is designed to pass the 2% AEP flood event (50y ARI) will experience that or a larger flood during a 50 year design life. Using the larger values for the scour design flood frequency for the 0.5% AEP flood and a 50 year design life reduces the exceedance value down to 22 percent. This is a substantial level of risk reduction. Designing for a higher level of scour than the hydraulic design flood ensures a level of redundancy after the hydraulic design event occurs.

$P = 1 - (1-1/Y)^L$ Equation 11.3 AR&R

Where L is design life in years, P is probability of one or more exceedance during a design life, Y is average recurrence interval.

2.4 Bridge Design Consideration

2.4.1 Introduction

The following section complements the requirements of Section 3.4 and 3.12 of TMR's *Design Criteria for Bridges and Other Structures 2012*. The location and size of bridge openings influence stream stability. The design of bridge components must consider the effects of bridge encroachments on the local stability of a stream. Where possible, it is prudent to utilise designs which promotes stability and minimises undesirable stream responses. This applies to individual component design as well as to the design of the total crossing system, including any stream instability countermeasures.

Encroachment in the stream channel by abutments and piers reduces the channel section and may cause significant contraction scour. Severe constriction of floodplain flow may cause approach embankment failures and serious contraction scour in the bridge waterway. Auxiliary (relief) openings should be carefully designed. On wide floodplains the design should seek to avoid excessive diversion of floodplain flows towards the main bridge opening. Skewed crossings of floodplains should also be minimised as much as possible.

The increase in the velocity through the bridge's waterway opening occurs as a result of the increase in the energy head. The restriction in the waterway results in water banking upstream to a level sufficient to develop the additional head to increase the velocity to maintain equilibrium flow. The increase in height of the water upstream is termed afflux.

The amount of afflux will determine the extent of flooding in adjacent land. The acceptable level of afflux for design depends on the upstream land use, i.e. housing, agriculture or undeveloped natural land. The implications for any possible change in the future land use will need to be considered. The need to pass flood flows of a certain flood event will determine the following details of the bridge:

Length

In most cases it is not economical to bridge the full width of flood flow and the problem reduces to what is an acceptable length of bridge. As a consequence, the road embankment in the approaches to the bridge causes a restriction on the flow occurring under natural conditions. Consideration of the increase in velocity and hence scour potential and afflux would be the main determining factors for the length of a bridge.

Height of abutments

The height of abutments should be considered in determining the length of a bridge. High abutments result in large retaining structures and embankments with inherent stability issues both in terms of the surcharge load to underlying material and the long term structural issues including rotations and horizontal deflections. Instances have occurred where vertical and horizontal displacements at high abutments in soft soils has resulted in structural distress to the abutment and jamming of expansion joints.

Bridge height

The bridge height will be influenced by a number of factors being flood height, navigation clearance and span lengths.

Flood height

For high level bridges the deck level adopted will be above the design flood level. The clearance from the underside of the superstructure to the flood level (including freeboard) should be a minimum of 0.6 – 1.00 m. However the type, amount and size of debris likely may require an increased freeboard depending on local conditions.

Navigation clearance

The height of a bridge may be fixed by the local waterway authority for navigational clearance requirements. The requirements of the local waterway authority need to be established at the design concept stage.

Span lengths

In some cases the minimum span lengths may be determined by the size of the debris carried by the stream. The potential exists for a debris dam to be built up by log lengths greater than the spans.

2.4.2 Scour Design Event

The hydraulic capacity of bridges varies for each project. However, design of scour protection should consider the flood event that produces the highest velocity and greatest bed shear. In most cases this is the flood event that overtops the bridge, and it is usually during the rising limb of the flood event. In other words, the greatest velocity does not correspond with peak flood level.

As alluded to in Section 2.3, the economics of repairing post scour damage will always favour preventative design. The final bridge design should balance all of the competing interests, remembering that if the 1% AEP design flood (100y ARI) is used for a collection of bridges, then over a 50 year lifespan the design flood will be exceeded for 4 out of every 10 bridges (39.5%).

Therefore, the aim of bridge design should identify the flood event that produces the highest velocities and worst case. As a simple rule, the scour design event should be considered as the design flood event that produces an overtopping event plus an additional 300mm in water surface. This additional overtopping amount and increased blockage factors can be tailored to the site characteristics for factors such as debris loading. Specifying a particular design event will ignore the subtle differences between designs and is being deliberately avoided. Furthermore, an extreme flood event (2000) may not be the design flood that produces the greatest turbulence or highest velocity in a reach. The structural design of the bridge still requires the design to be tested against the extreme flood event.

The size of a bridge will have an impact on scour. The length and height of deck and the channel geometry determine the waterway area of the bridge. The waterway area of the bridge and flow conditions of the channel will determine the velocities through the bridge. High velocities will result in scour.

As flood levels are increased, the flow begins to be built up behind the bridge deck. Up to the point of overtopping, the flow through the bridge becomes pressure flow as flow is driven through the bridge opening by the additional hydraulic head (thickness of the superstructure). This often also includes debris trapped on guardrail increasing the hydraulic head by the additional blockage. Therefore, in terms of scour reduction it is preferable to increase the waterway area of the bridge by widening the bridge rather than increasing the deck level and associated bridge approach embankments. The flow velocities through wider bridges will be lower as there will be less constriction (horizontal and vertical) in the channel, minimising pressure flow and scour of the bed material. Unfortunately, there are conflicting budget constraints that restrict an increase in bridge width more than increasing the height of a bridge.

Abutment scour usually occurs within several zones of sediment and soil, leading to different rates of erosion. The bed of the main channel is more erodible than the floodplain, because the bed is formed of loose sediment while the floodplain is formed from more cohesive soil often protected by a cover of vegetation. Abutments are essentially short, erodible (in the direction of flow) contractions. As the flow width narrows, the stream velocity increases as does the associated turbulence. Higher flow velocities and large-scale turbulence around an abutment may erode the abutment. Field observations indicate that, two prime scour regions develop:

- The first region is where the channel or overbank bed is least resistant to hydraulic erosion. This could be the main bed if flow velocities are sufficiently large.
- The second region is where the flow velocities and turbulence are greatest. This usually is near the abutment.

For an abutment set back on a floodplain, laboratory experiments indicate that deepest scour usually coincides with the region where flow contraction is greatest (NCHRP 2011).

When high flood immunity is not feasible an optimum bridge design should consider the accompanying bridge approach levels. Building the bridge approach embankment across the floodplain at, or above, the level of the bridge deck will exacerbate contraction scour. In some situations, typically when crossing floodplains on inland rivers, it may be preferable to deliberately reduce the level of the approach embankment to below the deck soffit. **Figure 2.4.2** illustrates the difference between a Sag and Crest Vertical curves in a bridge design. A crest vertical curve (Approach B) will lower the approach to below the bridge deck. As water levels rise, the bridge approach acts as a weir and flow overtops the bridge approach, protecting the bridge. In Approach A, severe vertical contraction scour results from flow being dammed being the road embankment and being forced through, and over, the bridge. In most cases a shallow flow over a wide bridge approach may exceed the flow through the bridge's waterway opening. This minimises pressure flow and reduces the hydraulic forces acting at the bridge and scour of the bed. The bridge experiences lower velocity and less pressure flow. In some cases it is better, in terms of reducing scour and time of submergence, for the bridge approaches to be overtopped, and the flood to be allowed to travel unhindered downriver. This is especially true in locations where bridge flood immunity is very low or in some cases where afflux is a concern due to nearby houses.

Referring to **Table 2-2** velocities through bridge approaches should be kept below 2.5 m/s or lower. This requires finding a careful balance between time of submergence and bridge scour concerns.

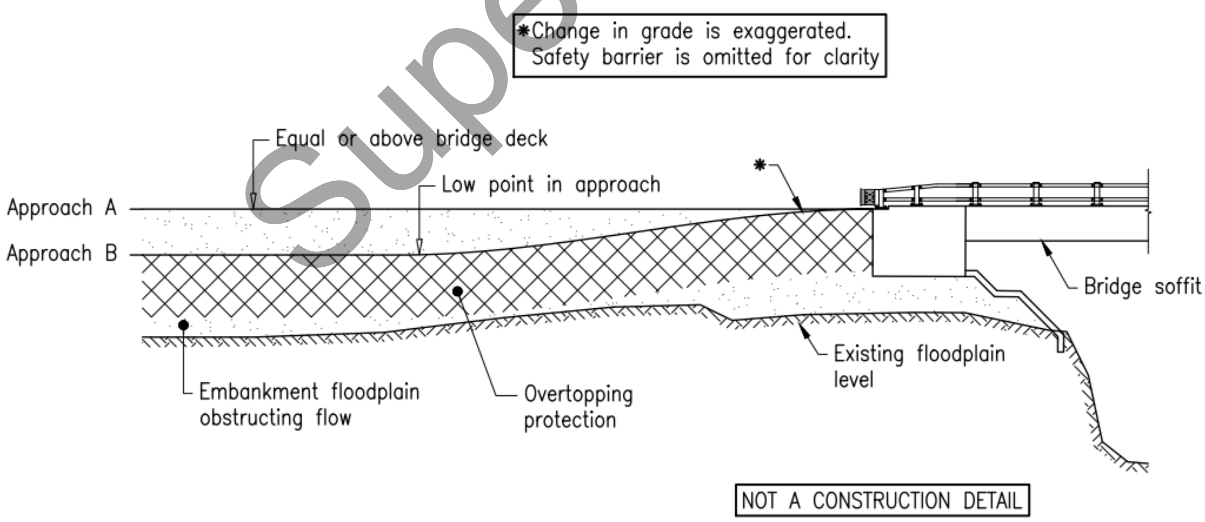


Figure 2.4.2: Crest vertical curve (Approach B) will minimise vertical contraction scour (Approach A – blocks more floodplain flow) Refer 2-14for cross section.

2.4.3 General design considerations

The following sections on bridge design are intended to complement detail and criteria outlined in TMR's *Design Criteria for Bridges and Other Structures*.

Scour protection maybe needed at the approach embankment for the overtopping flows. This is particularly important for streams carrying large amounts of debris which could clog the waterway and guardrail at the bridge. For streams that carry a large amount of debris it is recommended that the elevation of the bridge soffit be increased a minimum one metre above the design water surface (Austroads, 1994).

- Superstructures should be anchored if debris loadings are high and excessive forces are probable. Further, the superstructure should be as shallow as possible with wide pier spacing to minimise resistance to the flow where overtopping and debris are likely. Note debris loadings (branches and long grasses and so on) can be high due to prolonged periods between floods. This and other land use factors lead to a build up of debris within a catchment. During large floods a solitary vehicle or caravan can become lodged as debris and create contraction scour.
- Continuous span bridges withstand forces due to scour and any resulting 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.
- 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 maybe deeper than independent estimates at one or the other. The top width of a local scour hole on each side of a pier ranges from one to three times the depth of local pier scour. A top width value of two times the depth of local scour on each side of a pier is suggested for practical applications.

- For driven pile and cast in place pile foundations subjected to scour, a re-evaluation of the foundation design may require a change in the pile length, number, cross-sectional dimension and type based on the loading and performance requirements and site-specific conditions.

2.4.4 Skew Angle

The adverse effects of skew need to be considered in the design particularly for skew angles in excess of 20° (Guide to Bridge Technology, Austroads 2009). TMR's *Design Criteria for Bridges and Other Structures* (Clauses 1.2.1.2 and 4.11.5.3) outline the limitations of high skew bridges. TMR prefers that the maximum skew angle is less than 45°. Aside from increasing turbulence and scour potential, as shown in **Figure 2.4.4**, the effects of large skew angles include:

- Non-uniform distribution of loads to bearings, particularly those at the acute corners of a deck. Instances have occurred where the deck at the acute corner has lifted off the adjacent bearing under dead load only.
- This has implications for the deck in terms of flexural behaviour. In regard to bearings this situation results in increased loads to adjacent bearings that will overload them. In the case of elastomeric bearings the reduced loads at bearings may lead to bearings 'walking out' under shear displacements because of the lack of friction at the rubber/concrete interface.
- On large skew bridges there is a tendency for decks to rotate due to the fact that any longitudinal deck movement will cause the piers to deflect normal to their transverse axis.

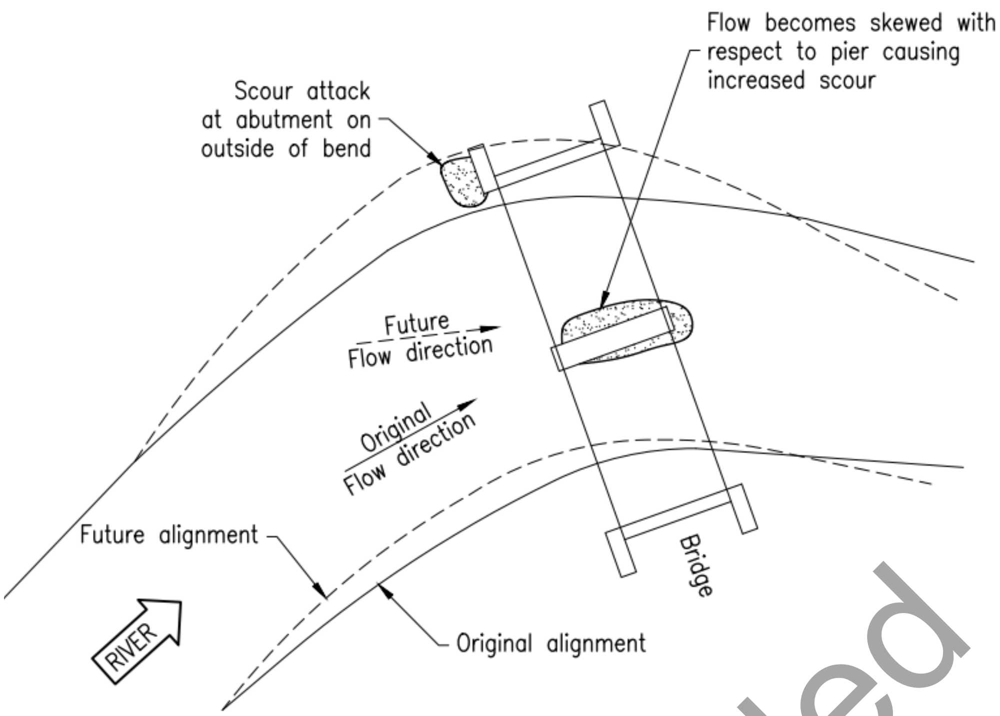


Figure 2.4.4: Change in channel alignment causing skewed flow and scour potential (CIRIA 2005)

2.4.5 Tolerable velocities

Tolerable velocities within a channel depend on the stream bed material. Particles will begin to move once the critical shear stress is reached. The flow velocity ('v' in m/s) required to move and suspend a particle increases with particle size. The method for calculating sediment criteria is based on bed shear stress, τ (Nm^{-2}), it is calculated using:

$\tau_o = \rho g n*^2 \, v^2 / y^{1/3}$ where ρ is the density of water, n* Manning's roughness

g = gravity, y = depth

Scour will occur once the bed shear stress exceeds the critical shear stress. The critical stress can be calculated using:

$\tau_c = K_S g y(\rho_S - \rho)$ where K_s is the Shields coefficient, ρ_s is the density of sediment.

As mentioned in Section 1.4.1, an alternative method involves calculation of critical velocity for comparison with stream velocity. The critical velocity can be calculated by:

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

where:

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

Table 2.4.5 contains typical velocities at which erosion in natural streams will start to occur.

Table 2.4.5: Critical velocities to initiate erosion of river materials (Hoffmans and Verheii 1997)

Type of material	Critical velocity, V_c (m/s)
Cohesive materials	
Loamy sand, light loamy clay with low compaction	0.4
Heavy loamy clay with low density	0.5
Low density clay	0.6
Light loamy clay with medium compaction	0.8
Heavy loamy clay with medium density	1.0
Light loamy clay (dense)	1.2
Clay of medium density	1.3
Heavy loamy clay (dense)	1.5
Hard clay (<0.002mm)	1.9
Non-cohesive materials, various sizes	
Sand (0.063 – 2mm)	0.4 – 0.6
Gravel (2 – 63mm)	0.9 – 1.5
Rocks (>63mm)	2.0 – 3.5

The Road Drainage Manual contains suggested allowable flow velocities in streams and through bridges in order to reduce the risk of erosion. A designer is responsible for ensuring that velocities are limited as much as possible. However, the velocities that occur within a channel prior to the inclusion man-made structures should be used as a design guideline. The calculated natural velocity should not be increased by a new bridge or structure by more than 30%. Previous guidance had preferred to limit the velocities to a value of below 2.5 m/s, as this is a reasonable value for many natural streams that comprise a mixture of cohesive and non-cohesive bed materials. However, this may not be appropriate in all cases.

2.4.6 Piers

The use of pier footings will significantly affect the local scour as seen in **Figure 2.4.6**. There will be a lower erosion risk by utilising buried footings (**Figure** (b)), footings with a conical transition or collar footings. Erosion will be minimised as the footings will act as physical barriers against scour. There will be a greater potential for erosion when a projecting footing is used as shown in **Figure** (c). The projecting footing will act as a barrier to the flows, trapping the horseshoe vortices. The trapped vortices will cause the scour hole to deepen. Where no footings are used, an equilibrium scour hole will form.

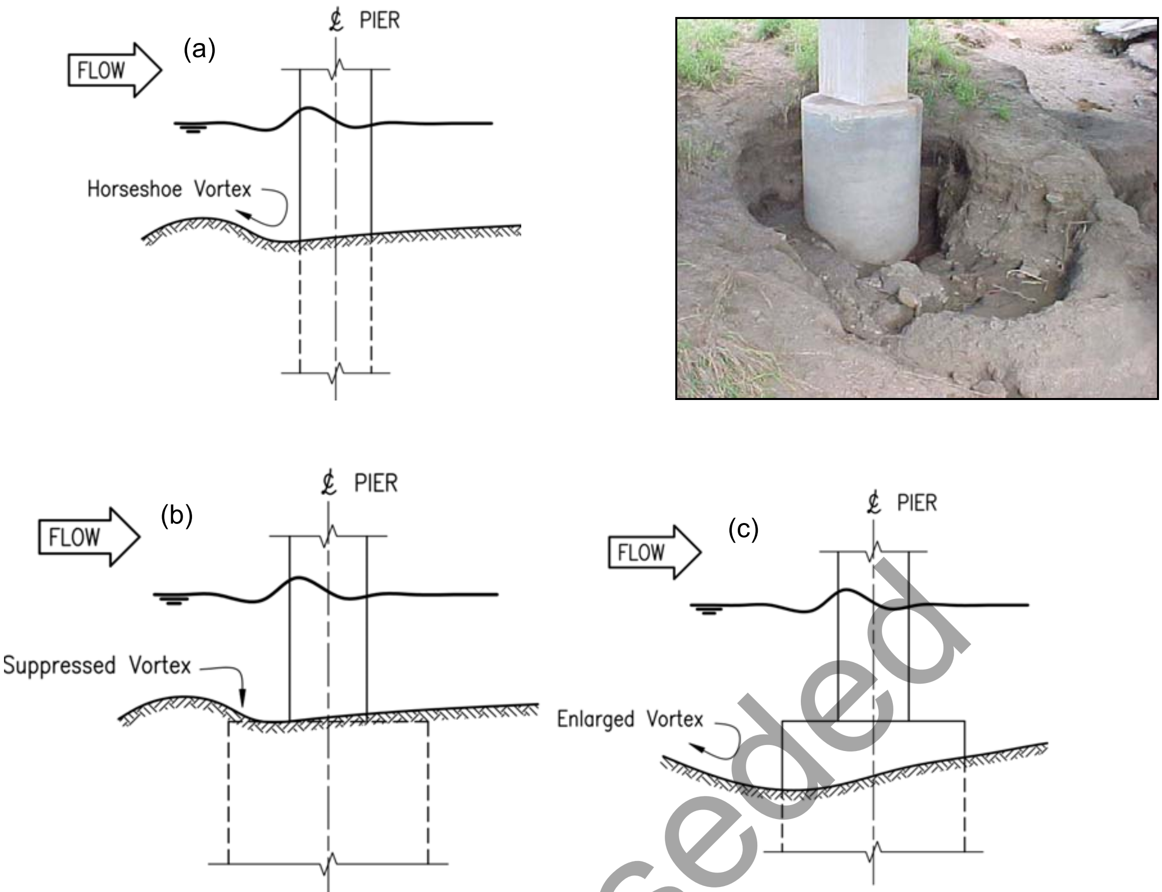


Figure 2.4.6: Flow profile around a projecting bridge footing (Neill, 1975)

- Since the thalweg (refer **Figure 1.3.3B**) of channels can migrate within a bridge opening, all piers in the main channel should be designed to the same elevation. Pier foundations on the overbank should be designed to the same elevation as pier foundations in the stream channel unless it can be determined with a reasonable degree of certainty over the life of the bridge that the overbanks are stable and the main channel will not migrate toward the overbank areas.
- Align piers with the direction of flood flows. Assess the hydraulic advantages of circular piers, particularly where flood patterns are complex and change with flood stage.
- Streamline piers to decrease scour and minimize potential for build-up of debris. Use debris deflectors where appropriate.
- Evaluate the hazards of debris build-up when considering use of multiple pile bents in stream channels. Where debris build-up is a problem, consider the bent a solid pier for purposes of estimating scour. Consider use of other pier types where clogging of the waterway area could be a major problem.
- Scour analyses of piers near abutments need to consider the potential of larger velocities and higher skew angles from the flow coming around the abutment.

2.4.7 Abutments

Because conditions in the field are different from those in the laboratory, the abutment scour calculations can over-predict the magnitude of scour. It is recommended that one of several approaches for accommodating abutment scour be used to ensure that abutments or the fill material placed around them does not fail.

- The most widely used approach relies on the use of a **designed scour countermeasure** to keep scour from developing at the base of the abutment or adjacent embankments. This approach provides an advantage in that a reasonable and cost effective approach for determining abutment foundation depth can be used. However, it also relies on a properly designed and regularly inspected scour countermeasure. Section 3 provides detail on procedures for designing and configuring scour countermeasures. Greater detail can be found in HEC-23, *Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volumes 1 and 2* (FHWA 2009).
- The second approach assumes all embankment fill material has washed away and that the abutment essentially behaves as a pier. This approach provides an advantage in that the failed embankment can be more easily repaired than a failed abutment, but provides a disadvantage due to the adverse flow conditions in the floodplain and channel near the abutment. Consequently, treating the abutment as a pier and estimating scour accordingly could lead to deep foundation depths. Information on computing scour for pier foundations is found in Chapter 7 of HEC-18.
- The recommended approach (Froehlich's abutment scour equation) relies on using procedures specifically developed for estimating abutment scour. If these methods are used it is imperative that the hydraulic variables used by the empirical methods be accurately and realistically determined. Information on Froehlich's abutment scour equation is presented in Section 2.5. Further detail on computing abutment scour is presented in Chapter 8 of HEC-18.

Engineering judgment must be used to determine which approach provides a reasonable and cost effective result.

To minimize the effects of adverse flow conditions at abutments, relief bridges, guide banks, and river training works can be used. Use riprap or other bank protection methods on the upstream side to protect against accelerating flows. On the downstream side, use riprap to protect abutments from flow expansions and wake vortices. A guide bank (upstream and/or downstream) is also a useful countermeasure for moving the scour location from the toe of the abutment to the toe of the (sacrificial) guide bank.

2.4.8 Bridge foundations

Spread footings on rock highly resistant to scour

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

Deep foundations (drilled shaft and driven piling) with footings

To minimise obstruction to flood flows, place the top of the footing or pile cap below the streambed at a depth equal to the estimated contraction 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 riverine flows or tidal currents.

Where either piled or spread footings are selected the site needs to be assessed for scour susceptibility. Factors to consider include:

- Evidence of local scour around the foundations of an existing bridge
- Evidence of bank scour as a result of turbulent flow adjacent to abutments

- Examination of bore logs to assess the alluvium particle sizes in terms of the anticipated restricted velocity through the bridge
- Examination of bore logs to determine changes in the alluvium size that may indicate scour depths
- Examination of bore logs to determine marked change in SPT values that may indicate scour depths
- Assessing the situation downstream of the bridge for cutback scour
- Depth to rock and potential scour depth in alluvium if driven piles are to be used
- Historic changes in position of channels. Instances have occurred where piles away from an existing channel have been driven to a higher level than those in the channel. Subsequent channel movements have compromised the integrity of those piles necessitating major additional underpinning works.

2.4.9 Superstructure

The design of the superstructure can have a significant impact on scour at the foundation. Hydraulic forces that should be considered include drag and impact from floating debris. The configuration of the superstructure will be influenced by the road geometry, the annual average time of submergence, expected issues with debris and flow velocities. In addition, the usual economic, structural and geometric considerations will also apply.

Drag forces

Drag forces on a submerged or partially submerged superstructure can be calculated by

$$F_d = C_d \rho H V^2 /2$$

where: F_d = Drag force per unit of length of bridge, (N/m)

C_d = Coefficient of drag (2.0 to 2.2)

ρ = Density of water, (1000 kg/m^3)

H = Depth of submergence, (m)

V = Velocity of flow, (m/s)

Most debris is derived locally along the banks upstream from the bridge. After being mobilised, debris typically moves as individual items which tend to concentrate in the thalweg of the stream. To minimise potential problems during a major flood *Hydraulic Engineering Circular 20* provides guidance for evaluating the abundance of debris upstream of a bridge crossing and then to implement mitigation measures, such as removal and or containment.

Debris forces

Like scour, detailed procedures for computing forces imposed on bridge superstructures by floating debris is also lacking, despite the fact that debris contributes to many failures. Floating debris may consist of logs, trees, caravans, automobiles, storage containers, tanks, timber, houses, and many other typical items representative of a floodplain's usage. This complicates the task of computing impact forces since the mass and the resistance to crushing of the debris contributes to the impact force.

A general equation for computing impact forces is:

$$F = M\, dv/dt = MV^2 /2S$$

where:

F	=	Impact imparted by the debris, (N)
M	=	Mass of the debris, (kg)
S	=	Stopping distance, (m)
V	=	Velocity of the floating debris prior to impact, (m/s)

2.5 Calculating Scour Depth

2.5.1 Introduction

The need to minimise future flood damage to Queensland's bridges requires that additional attention should be devoted to developing and implementing improved procedures for designing, protecting and inspecting bridges for scour. In short, the purpose of calculating scour is to:

- design new bridges to resist damage resulting from scour;
- evaluate existing bridges for their vulnerability to scour; and,
- correctly size scour countermeasures.

This section summarises the current information on provided in the Federal Highway Administration (FHWA) design publication Hydraulic Engineering Circular (HEC) 18, *Evaluating Scour at Bridges and CIRIA's C551 Manual on Scour*

2.5.2 Suggested contraction scour procedure

The following procedure is suggested, but alternative methods may be used. To aid understanding of equations a Glossary and Notation pages are provided at the end of this manual. The practical procedure to implement the following calculation method in a project should utilise the HEC-RAS scour module and informed from, where available, two-dimensional flood models. Application of this modelling software is described in Section 2.6. The procedure concentrates on determining the likely depth of scour in the main incised channel of a fluvial river. Significant natural scour does not normally occur on a floodplain during out-of-bank flows, because of the lower velocities and the protection provided by vegetation. However, separate scour estimates should be made for flood-relief culverts and access tunnels through embankments. Estimation of the maximum natural scour that is likely to occur in a tidal estuary due to combinations of high tides and fluvial flows is outside the scope of this manual and usually requires a combination of historical data and specialist study.

- Prior to this procedure, it is assumed that the design flood discharge, Q (m^3/s), for the river has been determined as part of a flood study, together with the corresponding water level. Key levels and flows just at the structure(s) are required, refer to **Figure 2.5.2** for equation terms.

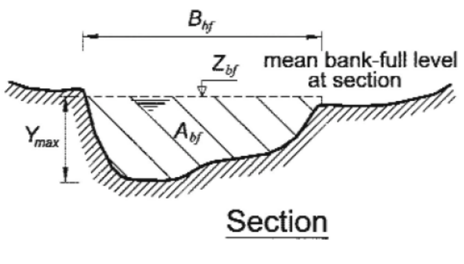


Figure 2.5.2: Section defining equation terms (CIRIA, 2005)

- If the floodplain is blocked or partially blocked by embankments, estimate the flow rate that will be passed by any flood-relief culverts through the embankments and subtract them from the flow rate. The remaining flow is the design flow rate passing through the main opening in the embankment at the contracted section.
- The main opening usually consists of the incised channel, plus one or more higher- level floodplain channels if the main opening is wider than the width, B_{bf} (m), of the incised channel. Determining the rate of flow, Q_1 (m^3/s), within the main incised channel is an iterative hand calculation and therefore should be calculated using modelling software (such as a 1D model like HEC-RAS 4.1, MIKE11 or 2D models like TUFLOW or MIKE21. Coupled 1D/2D models are recommended but should always be checked against HEC-RAS).
- The following explanation outlines the calculations and required parameters that are easily performed in modelling software.
- Note when calculating values of flow area, A_i (m^2), and surface width, B_i (m), for each sub-channel, allowance should be made for any structures such as piers or abutments that restrict or partially block the flow. Various software packages allow for these blockages in different manners.
- Use the appropriate scour regime equations for contraction scour, local pier and abutment scour. This can be selected from within the HEC-RAS bridge scour module or consulting the equations listed later to calculate predicted dimensions at the flow restriction, $_R$ (Width, B_R, Wetted Perimeter P_R, Area A_R and Depth, Y_R) of the incised channel if it were able to reach an equilibrium condition corresponding to the flow rate, Q_1.
- In most cases, the predicted channel width, B_R (m), will not be equal to the net width, B_1 (m), of the incised channel (where B_1 is equal to the bankfull width, B_{bf} less the width, measured transverse to the flow, of any obstructions in the incised channel). It is unlikely that the channel width would be able to adjust to the appropriate value during the limited time of a typical flood. However, if $B_R > B_1$ significantly, some bank erosion is likely to occur and protection works may be required in the vicinity of the structure.
- It is assumed that the flood flow will scour the bed of the incised channel in the contracted section, so as to provide the cross-sectional area, A_R (m^2) giving the same predicted mean velocity as the scour regime. Part of this required area will be provided by the flow occurring in the incised channel above the level, Z_{bf} (m), of the top of the banks.

It should be assumed that the lowest bed level, Z_{min}, could occur at any transverse position across the width of the incised channel, unless its position is constrained by external factors (such as the existence of locally inerodible material in the channel, bed protection works, or location on the outside of a bend).

2.5.3 Suggested local scour procedure

Use the procedure above to estimate contraction scour. Noting the earlier comment, the following procedure can be implemented using HEC-RAS and is outlined in Section 2.6. Estimate the bed level, Z_o , that can be expected to occur just upstream of the structure during the design flood. For design purposes, it should normally be assumed that $Z_o = Z_{min}$, the lowest predicted level of natural or contraction scour.

If the water level in the design flood is Z_D (m), calculate the local flow depth, y_o (m), upstream of the structure from $y_o = Z_D - Z_o$ and also the corresponding value of the local depth-averaged velocity, V, for that flow depth.

Determine the value of the depth-averaged critical threshold velocity, V_C (m/s), using the guidance in Section 1.2.3 and Ackers and White (1973) formula. Local scour can be determined as either clear-water type or live bed if the general bedload movement of sediment in the vicinity of the structure.

Calculate the maximum depth of local scour, Y_S (m), at the structure using the information for the particular bridge design (piers and abutments) configuration as appropriate. This detail is outlined separately in later sections.

The lowest level of local scour, Z_{LS} (m above datum), predicted to occur at the structure during the design flood is then given by: $Z_{LS} = Z_o - Y_S$.

The deepest point of scour normally occurs near the upstream side of the structure, but its position may be affected by the geometry of the structure and by the angle of incidence between the approaching flow and the longitudinal axis of the structure. Estimates of the dimensions of the scour hole can be obtained by assuming that the slopes of the upstream faces of the scour hole are equal to the natural angle of repose of the sediment forming the bed. On the downstream sides of the scour hole, the slopes are likely to be flatter and equal to about half the angle of repose.

In the case of a structure with spread footings, there will be a significant threat to its stability if the predicted scour level, Z_{LS}, at the lowest point is near to or below the level of the underside of the footings.

In the case of a structure supported by deeper piled foundations, the stability will be determined by what lengths of pile need to remain buried below ground level to provide the required bearing loads and resistance to bending. In this case, the more relevant criterion is likely to be the average bed level occurring in the scour hole around the perimeter of the structure during the design flood. If the length of the structure parallel to the flow is more than about twice its width, the average bed level is likely to be higher than the level, Z_{LS}, at the lowest point. An estimate of the average bed level for a structure such as a bridge pier can be made by assuming that the deepest scour occurs at the upstream end and that the bed slopes upwards at an angle equal to half the natural angle of repose of the bed sediment.

2.5.4 Limitations of the scour estimation procedure

Scour at bridge crossings in a riverine environment is a result of a complex interaction between:

- river flow;
- channel bed and bank materials which can be highly variable;
- channel shape (bends, floodplain widths); and,
- bridge structure configuration.

Scour types include bend scour, cut-back scour, contraction scour, pressure scour and local scour. These different types of scour can occur simultaneously. However, for design purposes each type of scour is dealt with separately and the effects of each scour are assumed to be cumulative to provide a total scour value. The uncertainty of this complex interaction is further compounded by issues relating to:

- river hydrology, where the accuracy of peak flow estimates may be typically of the order of +/- 50%;
- sediment transport estimates where the accuracy of the predictive methods are frequently of the order of +/- 100%; and
- hydraulic analysis is typically quantified by mean velocity and depth of flow in contrast to actual stream flow, which is highly three-dimensional and very difficult to numerically model.

For each of these reasons it is very important to appreciate that there is a great deal of uncertainty not only in the estimation of bridge scour, but also in the various parameters used to derive bridge scour.

Most predictive formulae for the estimation of bridge scour have been derived from laboratory experimentation that tends to provide worst-case conditions. They are typically conducted in loose, cohesionless soils and in many instances do not represent actual field conditions. The outcome of this is believed to be a conservative estimation of scour at bridge sites.

Equations for predicting abutment scour depths such as Liu et al. (1961), Laursen (1980), Froehlich (TRB 1989), and Melville (1992) are based entirely on laboratory data. There is very little field data on abutment scour. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line drawn through the laboratory data. Laursen's equations are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment. Froehlich's equations were derived from dimensional analysis and regression analysis of the available laboratory data. Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.

Until recently, the equations in the literature were developed using the abutment and bridge approach length as one of the variables. This approach may result in excessively conservative estimates of scour depth since the discharge in the laboratory flume intercepted by the abutment is directly related to the abutment length. However, in the field, this is rarely the case due to floodplain storage, attenuation and other factors.

Typically, the hydraulics engineer does not receive feedback on scour depths attained in the field or on the performance of the abutment protection. Regular bridge soundings and flow monitoring is recommended for sites where scour persists. To provide a greater understanding an attempt at calibrating observed scour with the suggested equations and recorded flows should be coordinated. Estimated scour depths can be compared with the bridge soundings at each pier and abutment and any available bore log data.

Recognising the uncertainty and limitations of the available studies, the scour design equations provided in this manual should be used as guides in the design.

2.5.5 Contraction scour

Contraction scour equations are based on sediment transport. In the case of live-bed scour, the fully developed scour in the bridge cross section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. 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.

Four conditions (cases) of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches, refer **Figure 2.5.5A**. Case 1 conditions include:

a) The river channel width becoming 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, refer **Figure 2.5.5B**.

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).

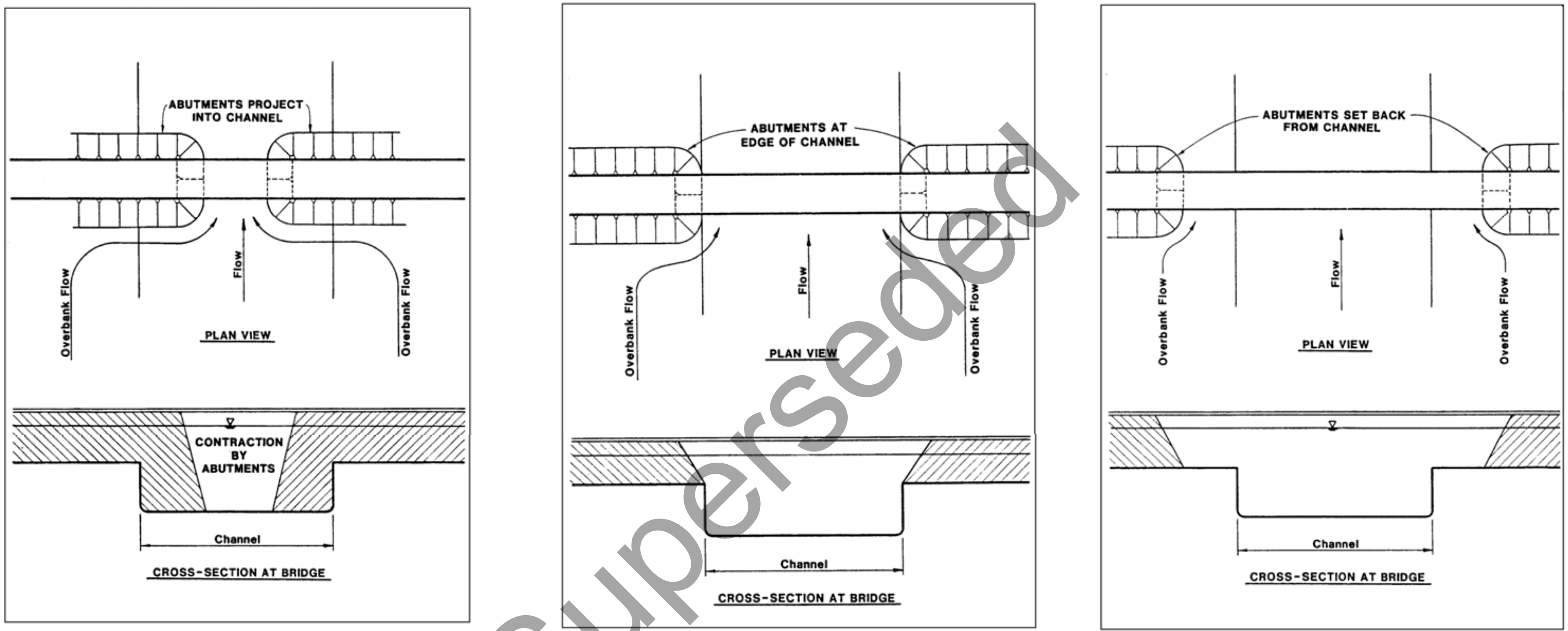
Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to Case 1).

Cases 1, 2 and 4 may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows.

Case 1c is very complex. The depth of contraction scour depends on factors among others, as:

- how far back from the bankline the abutment is set,
- the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.),
- whether the stream is narrower or wider at the bridge than at the upstream section,
- the magnitude of the overbank flow that is returned to the bridge opening, and
- the distribution of the flow in the bridge section.

Case 3 may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.

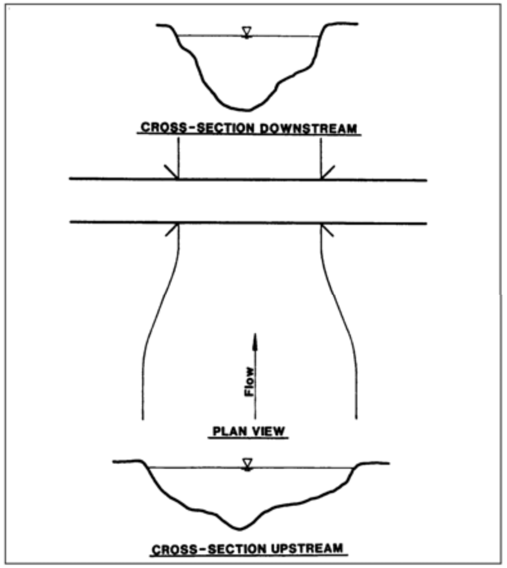


Case 1a: Abutments project into channel

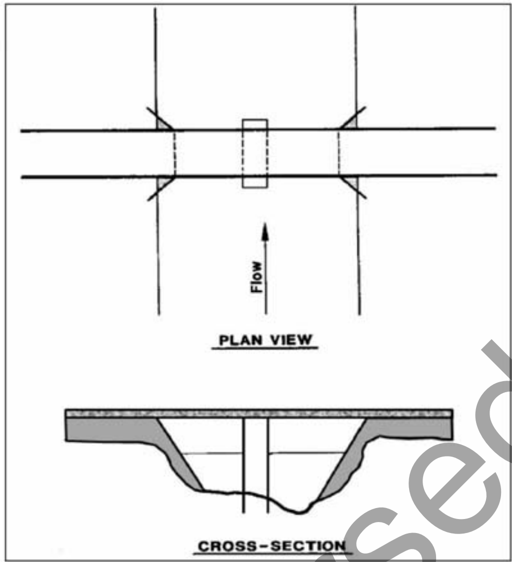
Case 1b: Abutments at edge of channel

Case 1c: Abutments set back from channel

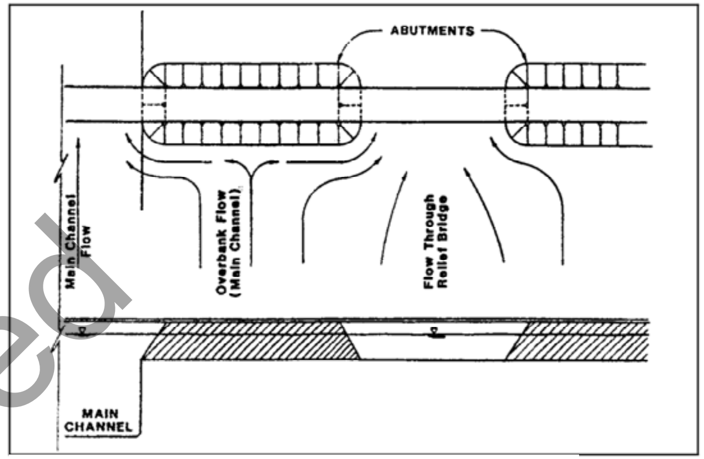
Figure 2.5.5A: Contraction Scour Definitions Case 1 (HEC 18, 2012)



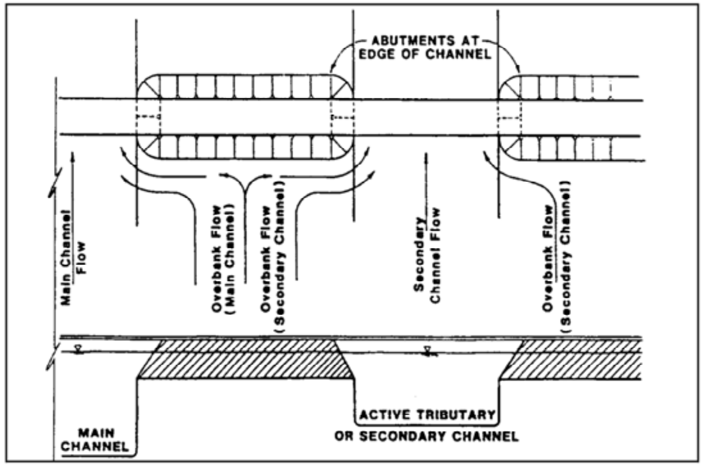
Case 2a: River Narrows



Case 2b: Bridge abutments and/or piers constrict flow



Case 3: Relief bridge over floodplain



Case 4: Relief bridge over secondary stream

Figure 2.5.5B: Contraction Scour Definitions Case 2, 3 and 4 (HEC 18, 2012)

2.5.6 Pier scour

The HEC-18 pier scour equation (based on the CSU equation) is recommended for both live-bed and clear-water pier scour. The CSU equation is the default in HEC-RAS and requires only the pier nose shape (K1), the angle of attack (K2), condition of the bed (K3) and the 95^{th} percentile size of bed material (D95). The equation predicts maximum pier scour depths. Basic applications include simple pier substructure configurations and riverine flow situations in alluvial sand-bed channels. The equation can be adapted for wide pier applications (y/a < 0.2), more complex (3-element) substructure configurations, multiple columns skewed to the flow, estimating scour from debris on piers, and scour in tidal waterways. HEC-18 should be consulted for more comprehensive explanation of these sophisticated configurations (refer HEC-18, Sections 7.3 to 7.6 inclusive).

The alternative Florida Department of Transport approach represents the complexity of the bridge pier scour flow field and the full range of pier geometries (narrow, transition and wide as described in Section 1.5.2)

FHWA HEC-18 Equation (based on the Colorado State University 'CSU' Equation)

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left[\frac{a}{y_1}\right]^{0.65} Fr_1^{0.43}$$

L	= Pier length, m
a	= Pier width, m
Fr_1	= Froude Number directly upstream of the pier = $V_1/(gy_1)^{0.5}$
V_1	= Mean velocity of flow directly upstream of the pier, m/s
g	= Acceleration of gravity (9.81 m/s^2)
K_1	= Correction for pier nose shape
	= Correction for angle of attack of flow from Table or the equation below.
K_2	$K_2 = \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65}$, if L/a >12 use L/a = 12 as a maximum
K_3	= Correction factor for bed condition, Table

Figures 2.5.6A and 2.5.6B provide a definition for solving the equation.

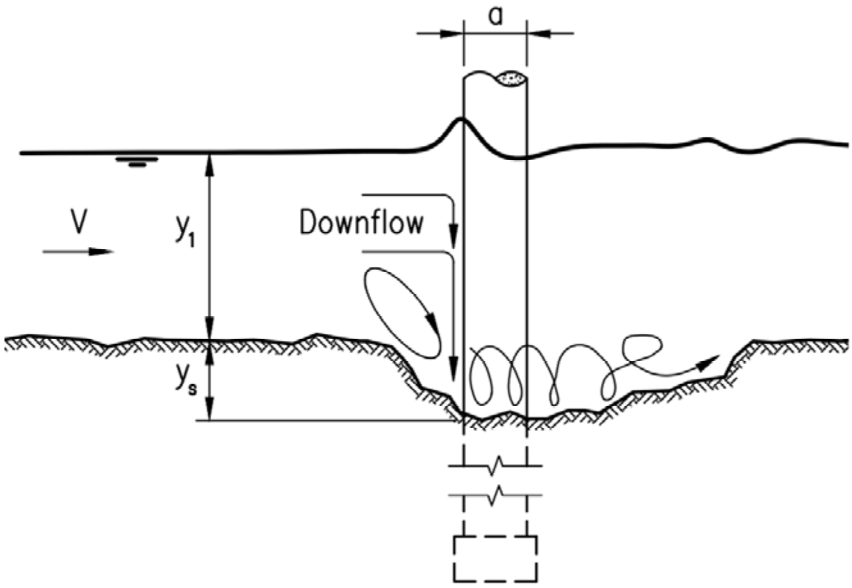


Figure 2.5.6A: Definition sketch for circular pier scour

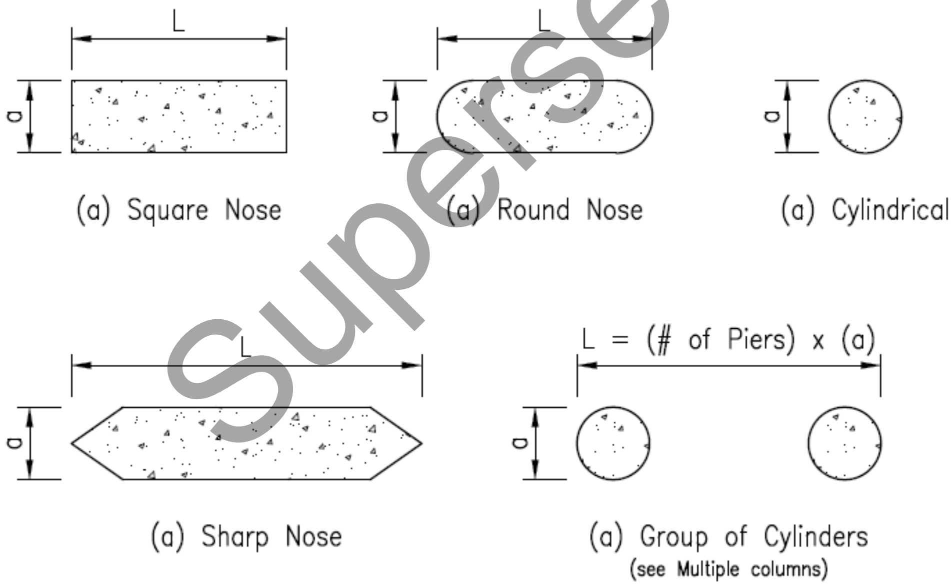


Figure 2.5.6B: Common pier shapes and correction factors

For round nose piers aligned with the flow the maximum depth of scour, ys, is:

$y_s = 2.4b$ for $Fr \leq 0.8$ $\qquad$ $y_s = 3.0b$ for $Fr > 0.8$

The correction factor K1 for pier nose shape should be determined using **Table 2.5.6A** for angles of attack up to five 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 2.5.6B**. 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.

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

- a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or
- 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.

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 the HEC-18 Equation. Refer to **Table 2.5.6C** for the K3 factor. 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.

Table 2.5.6A: FHWA K_1 factors

Shape of Pier nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinders	1.0
Group of cylinders	1.0
Sharp nose	0.9

Table 2.5.6B: FHWA K_2 factors

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

Table 2.5.6C: FHWA K_3 factors

Bed Condition	Dune Height	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
small dunes	3 >H>0.6	1.1
medium dunes	9>H>3	1.2 to 1.1
large dunes	H>9	1.3

The factor K_4 is used to decrease scour depths in order to account for armouring of the scour hole. The factor is only applied when the D50 of the bed material is greater than 0.2 mm and the D95 is greater than 2 mm. This factor is automatically calculated by HEC-RAS and is function of the water depth just upstream of the pier.

The projected pier width with respect to the direction of flow is represented by the factor 'a' and is part of the Froehlich equation. The factor is calculated based on the pier width, shape, angle and length. A further correction for the Froehlich equation involves adjusting for pier nose shape.

Debris accumulation on piers

Floating woody debris (drift) that lodges and accumulates at bridge piers creates additional obstruction to flow and transforms the pier geometry into one that is effectively wider than if debris were not present, refer **Figure 2.5.6C**. Equations have been developed to estimate the effective width based on the original (debris-free) pier geometry, size and shape of the debris accumulation.

Bridge design and scour assessment requires the estimation of the size and shape of debris on bridge piers. This is largely a matter of experience and judgment. Most woody debris is derived from bank failures on the main channel and major tributaries upstream of the bridge. Land management practices can also influence the production of debris. This includes the maintenance practices which can vary. Debris accumulation can grow to very large dimensions before removal. It is preferred that regions aggressively remove debris from bridges even when only a few logs are present to prevent the potential snagging of additional material.

The best approach to estimating the size, shape, and dimensions of debris is to review the history and maintenance practices in the region. Experience with debris on similar river systems will provide insight on potential problems in the future. NCHRP Report 653, *Effects of Debris on Pier Scour* provide guidance on estimating the delivery of floating debris at a bridge, and the processes by which it accumulates on the structure.

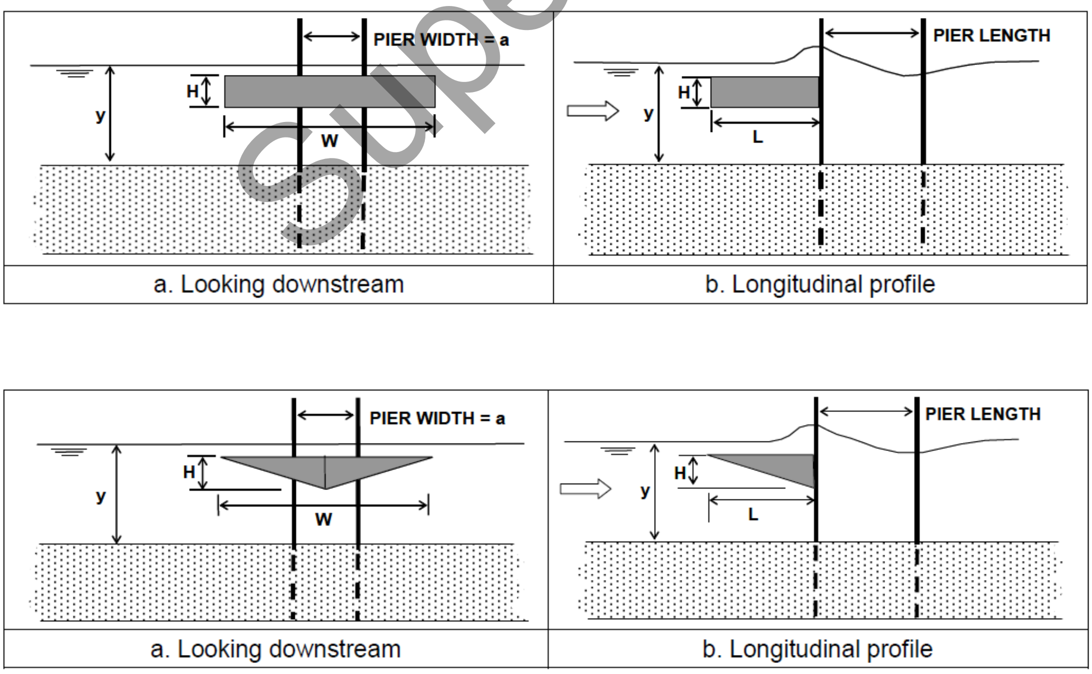


Figure 2.5.6C: Idealised dimensions for debris accumulation from NCHRP Report 653

Perhaps the most important result from the NCHRP research is the fact that the greatest amount of debris-induced scour at the pier occurs when the length of the debris in the upstream direction (dimension "L" in the above figures) is equal to the approach flow depth ("y"). When the debris accumulation has grown to this dimension, the plunging flow created by the debris is focused at the base of the pier, reinforcing the horseshoe vortex.

During the NCHRP study, it was also found that when debris floats at the water surface during a flood event, all of the approach flow is forced to plunge beneath the debris, with no flow going over the top. Therefore, the guidance developed for estimating pier scour with debris assumes that the debris is floating at the surface during a flood, which is a very likely condition during the peak of a flood event.

Effective pier width with debris

Based on the results of the NCHRP study, a relatively simple equation was developed that can be used to estimate the equivalent pier width, denoted a*d, when debris is present. The equation considers the shape, width and height of the debris in addition to the unobstructed pier width and the depth of the approach flow. The equation yields an equivalent pier width that can be used in the HEC-18 pier scour equation to estimate the local scour depth at the pier. As previously noted, the most severe scour at the pier occurs when the length of the debris (in the upstream direction) is equal to the flow depth.

Based on the shape of the debris, the effective width of a pier's a* with debris loading is calculated by:

$$a^*_d = \frac{K_1(HW) + (y - K_1 H)a}{y}$$

where:

a^*_d	=	Effective width of pier when debris is present. (m)
a	=	Width of pier perpendicular to flow, (m)
K_1	=	0.79 for rectangular debris, 0.21 for triangular debris
H	=	Height (thickness) of the debris, (m)
W	=	Width of debris perpendicular to the flow direction, (m)
y	=	Depth of approach flow, (m)

2.5.7 Scour at abutments

Most abutment scour prediction equations use the length of an abutment's embankment projected normal to flow as an independent variable. Conveyance and associated velocity and flow depth at the outer extremes of a floodplain are much less, particularly in wide and heavily vegetated floodplains. This flow is typically referred to as "ineffective" flow. When applying abutment scour equations that use the length of embankment projected normal to flow, it is imperative that the length used is the length of embankment that is blocking "live" flow.

The length of embankment blocking "live" flow (L') can be illustrated by a graph of velocity versus distance (L) across a representative cross-section upstream of the bridge, refer **Figure 2.5.7**. Where a wide portion of a cross-section is required to convey discharge in the floodplain, then the total length (L) of embankment blocking this flow should not be included when determining the length of embankment used in the abutment scour prediction relationship. Alternately, if the flow in a significant portion of the cross-section has low velocity and/or is shallow, then the length of embankment blocking this flow should probably not be used either. HEC-RAS (USACE 2010a) computes conveyance versus distance across a cross section.

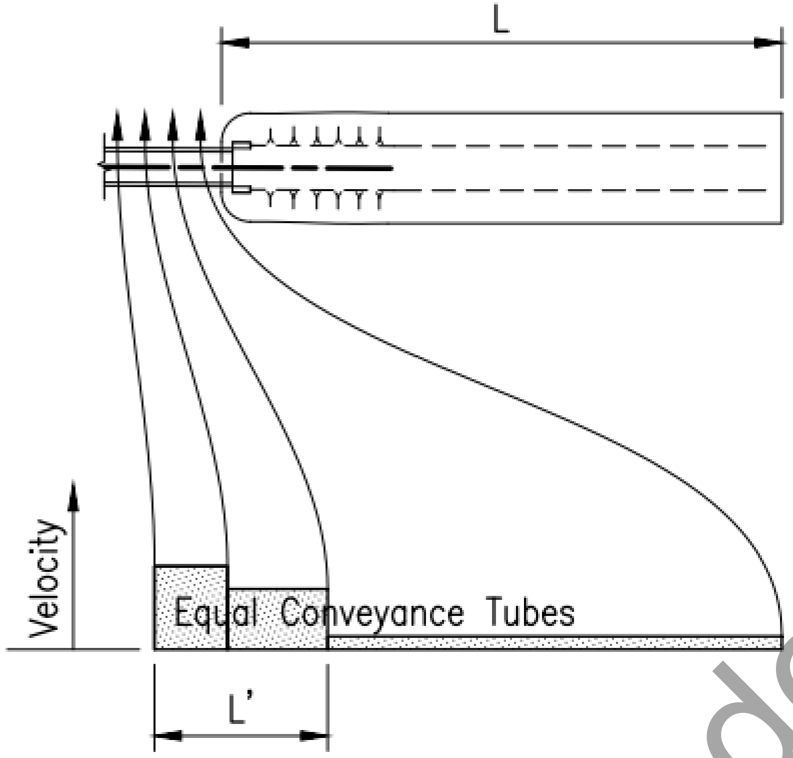


Figure 2.5.7A: Determination of embankment length blocking live flow for abutment scour

Further, the figure shows the plan view of an embankment blocking three equal conveyance tubes on the right floodplain at a bridge. Since the right conveyance tube occupies the majority of floodplain but conveys only one-third of the floodplain flow, it should not be included in the "live" flow area for determining L′. In this case the length of embankment, L′, blocking the "live" flow is approximately the length of the two inner conveyance tubes. In the event that the conveyance versus distance graph does not show a conclusive break point between "live" flow and ineffective flow, an alternative procedure is to estimate L′ as the width of the conveyance tube directly upstream of the abutment times the total number of conveyance tubes (including fractional portions) obstructed by the embankment. This length is more representative of the uniform flow conditions in the laboratory experiments used to develop abutment scour equations.

Abutment scour can be computed using the modified HIRE (Richardson, 1990/FHWA 2001) or Froehlich (1989) equations. The input and data required to undertake the calculation is as follows:

HIRE abutment scour equation (FHWA 2001)

Modified HIRE Equation

$$\frac{d_s}{y_1} = 4Fr_1^{0.33}\frac{K_1}{0.55}\text{K2}$$

The modified HIRE equation, is applicable when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25. Table 2-6 provides the values for K_1. **Figure 2.5.7B** defines the calculation of the K_2 from the skew angle. This is similar to the field observations on the Mississippi River from which the equation was derived. HEC-RAS will default to the Froehlich equation when this ratio is less than 25. Also note that the equation listed above supersedes the original HIRE equation (1990) built into HEC-RAS.

Froehlich's (TRB 1989) abutment scour equation

Froehlich's live-bed, Abutment Scour Equation

$$\frac{d_s}{y_1} = 2.27 K_1 K_2 \left[\frac{L'}{y_a}\right]^{0.43} Fr_1^{\,0.61} + 1$$

L' = Length of abutment projected normal to flow, m

A_e = Flow area of the approach cross-section obstructed by the embankment, m^2

Fr_1 = Froude Number of approach flow adjacent to and upstream of the abutment
= $V_e / (gy_a)^{1/2}$; (Froehlich 1989, HIRE 1990)

V_e =Velocity of approach flow, = Q_e/A_e; m/s

Q_e = Flow obstructed by the abutment and approach embankment, m^3/s

y_1 = Average depth of flow on the flood plain or at the abutment on the overbank or in the main channel, m

d_s = Scour depth, m

Table 2.5.7: Coefficient (K_1) for abutment shape

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

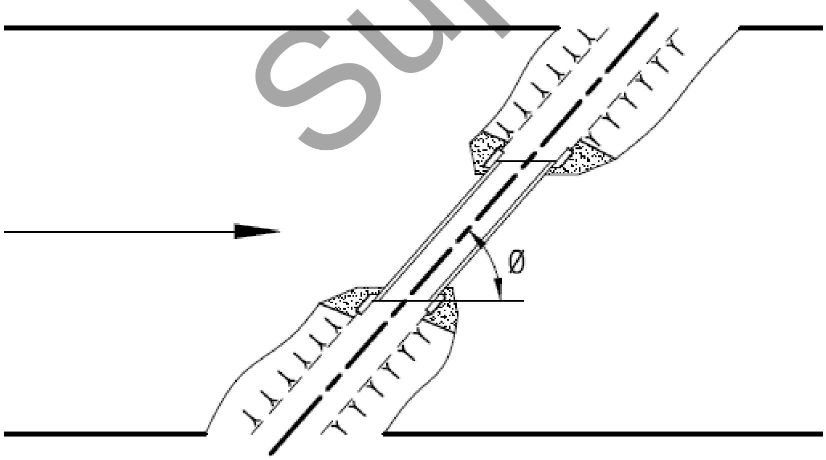


Figure 2.5.7B: Embankment Angle (theta K_2), Abutment Shape – K_1 Coefficients.

Coefficient for skew angle of embankment to flow $K_2 = (\theta/90)_{0.13}$;

$\theta < 90^o$ if embankment points downstream

$\theta > 90^o$ if embankment points upstream

2.6 Scour at bridge approaches and floodways

Overtopping of bridges and their approaches should always be considered as part of the design process. Typically scour of bridge approach embankments is more disruptive than scour at the bridge itself. Therefore, it is cost effective to provide an appropriate level of protection at bridge approaches, particularly where the embankment projects well into the floodplain, refer Case 1b in **Figure 2.5.5A**. This will minimise the repair time for post flood closure works. It may not be practical or cost-effective to protect a floodway for extreme flood events, so a cost-benefit analysis should be carried out to compare the cost of additional scour protection to the cost of on-going maintenance and repair works, to determine the appropriate extent of protection (that is, to compare the cost of regular minor damage and ongoing maintenance works, to the cost of catastrophic failure, road closure and total reconstruction).

Scour damage will occur first on the downstream face of the embankment before advancing through the road pavement. In severe cases, the scour will continue advancing until the embankment is breached. The causes of scour at these positions are due to:

- impinging super-critical velocity at the toe of the batter slope,
- the drag/shear resistance on the batter slope,
- an uplift force caused by the embankment geometry,
- shear/drag resistance on the running surface, and also,
- approach velocity.

2.6.1 Limiting Floodway Scour Potential

It should be noted that during the early stages of overtopping relatively high velocities may be present and thus slope stability should be a design consideration. Also note that the maximum flow rarely corresponds to the peak velocity. The risk of damage to the downstream shoulder can be reduced by rounding the shoulder as much as possible, to avoid the generation of negative pressures at the change of flow direction. A radius of approximately 3.3 m is recommended.

Flow through the embankment can lead to high uplift pressures under impervious types of batter slope protection such as concrete slabs and pump-up revetment mattresses. Relief holes are required to allow drainage through the protection system and avoid pressure build-up. Dumped graded rock and gabion mattresses are not impervious and pressure build-up is unlikely to be a problem. Leakage at the upstream side of a concrete cut-off wall can lead to significant pressures acting on the upstream face of the wall. Destabilising negative pressures can also result at the downstream shoulder due to abrupt changes in grade. If these pressures exceed the passive resistance of the soil wedge at the shoulder, failure of this wedge may occur. Significant forces can act on upstands near the downstream shoulder (such as kerbs and guardrail posts). These high forces promote localised scour damage, which can act as a starting point for progressive scour damage by other means. Upstands should be avoided wherever possible.

2.6.2 Embankment Batter Protection

The need for upstream protection will depend upon the velocity of flow, the time it is submerged, and the skew to the direction of flow. TMR's *Road Drainage Manual 2010* should also be referred to when undertaking design. When upstream protection is provided to protect against high approach velocities, it is not generally necessary to protect the full height of the batter, but only the road shoulder and the top of the batter. However, for floodways that are submerged for long periods, it is usual to provide similar protection on the upstream batter to that provided on the downstream batter.

Downstream protection of floodway embankment batter slopes may be either flexible or rigid. All protection should sit flush with the road pavement at the shoulder to avoid high pressures resulting at any sharp steps or grade changes. Examples of flexible and rigid protection are listed below.

Flexible Protection

Dumped graded rock (riprap), defined as graded stone dumped upon a prepared slope. In most areas dumped rock is the least costly type of protection. A suitable length toe (typically 1 - 1.5 times the embankment height or three metres) should be provided at the base of the rock to protect the embankment against the high velocities at the change of grade.

Wire enclosed rock is generally used in locations where the sourcing of large graded, loose dumped rock is not readily available or is uneconomic. The size of rock should be larger than the openings in the wire enclosure, and a suitable length toe is required as above. The wire used should be PVC coated to avoid corrosion.

Flexible mats comprise individual small high-density concrete blocks cast onto a geotextile loop matting. Each mat is generally about 5 m by 2.5 m and protection is provided by laying the mats side by side with an overlap. These are proprietary products and the designer should refer to the manufacturer's technical literature for advice on their application and installation.

Flexible pump-up revetment mattresses are concrete filled nylon mattresses where the concrete flows into discrete segments that are largely independent once the concrete has set, providing a degree of flexibility. These are proprietary products and the designer should refer to the manufacturer's technical literature for advice on their application and installation.

Rigid Protection

Grouted rock is dumped or hand placed rock with the voids filled with mass concrete. The concrete should be sufficiently fluid to fill all voids over the full depth of the rock layer. It is generally used in locations where stone of a size suitable for other forms of protection is not economically available. It is also useful where only a small depth is available for construction of rock protection (such as over culvert pipes) or where access to construct larger rock is difficult.

Rigid pump-up revetment mattresses are nylon mattresses into which a small aggregate concrete is pumped. These are proprietary products and the designer should refer to the manufacturer's technical literature for advice on their application and installation.

Concrete slab protection is plain or reinforced concrete slabs poured or placed on the surface to be protected. This type is not often used due to its high cost, but may be warranted at crossings subject to extended periods of inundation. It may also be warranted in exceptionally high velocity situations, where other types of protection are inadequate.

Rigid protection is susceptible to undermining by scour, especially at the toes of batters, and should not be used unless the design engineer is confident that scour will not occur. Combinations of flexible and rigid systems may also be considered.

The use of a concrete cut-off wall at the downstream shoulder is recommended when high velocities are expected at the shoulder. The purpose of this wall is to prevent scour damage at the shoulder from progressing into the road pavement. These walls are typically 0.50-0.75 m deep and 0.20-0.30 m wide and are generally constructed of low strength mass concrete.

Where necessary a permeable geotextile filter should be placed between the embankment fill and the flexible scour protection. A graded sand/gravel filter may also be used.

Typical Detail

The typical details in this section are intended as a guide only, the *Road Drainage Manual, 2010* provides further explanation. **Figure 2.6.2** provides typical detail for use on a bridge approach and floodway that requires protection from overtopping flows. This detail will require adjustment to suit the abutment protection detail. The type, extent and thickness of rock protection; the use of geofabrics; the depth of cut off wall; the use of concrete slab batter protection or rock mattresses; and other issues should be considered on a site-specific basis on the advice and the guidance of relevant literature.

The protection consists of a cement-stabilised pavement with a two-coat seal, and rock protection to the downstream batter slope with a geofabric underlay. The geofabric provides some resistance to scouring of the pavement due to the high pressure at the road shoulder, but is suitable for low velocities only. The increased scour protection at the shoulder. The concrete cut-off wall provides improved resistance to scour damage at the shoulder, and the toe of the rock protection improves the stability of the batter slope rock protection and decreases the risk of scour downstream of the floodway. The two-coat seal should overlap the concrete cut off wall as shown.

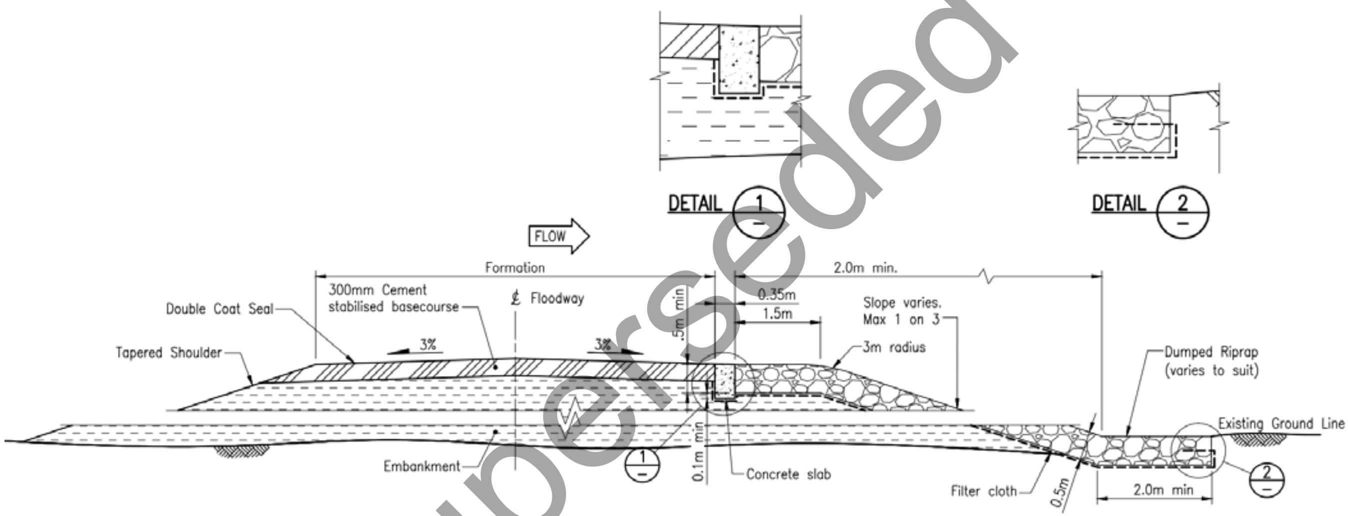


Figure 2.6.2: Typical detail for overtopping at bridge approach / floodway

2.7 Modelling with HEC-RAS Scour Module

Hydrologic Engineering Center within the US Army Corps of Engineers developed a River Analysis System, commonly known as HEC-RAS. The modelling program is freely available and is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The program is capable of calculating steady flow water surface profiles; simulating unsteady flow and completing sediment transport/ moveable boundary computations. It is widely used to calculate bridge hydraulics and the program is available for download for free. The following section is intended to introduce scour modelling concepts.

This section is written for the reader who is conversant in hydraulic modelling with HEC-RAS and other modelling software. More detailed information can be found in the HEC-RAS Users Manual and HEC-RAS Applications Guide available on the internet. More general readers may find this section useful when evaluating proposals or understanding submissions from consultants. Additional information is also available in:

http://www.ncwe.org.au/arr/Website_links/ARR_Project15_TwoDimensional_Modelling_DraftReport.pdf

The calculations presented in this manual can be undertaken using HEC-RAS. It is recommended that any assessment of scour analysis should involve a HEC-RAS model, preferably calibrated. However, HEC-RAS alone may not be appropriate in all scour design circumstances. The calculation of complex floodplain flow distribution is better suited to two dimensional software packages such as MIKE Flood or TUFLOW. Velocity and depth outputs can be extracted from the 2D domain of these modelling packages and imported into HEC-RAS for further analysis of particular bridge locations. Adjustments of the flow distribution parameters will still be required.

The scour module requires an underlying HEC-RAS hydraulic model to be set up and run as per normal. Scour analysis requires water surface elevations at a bridge site along with flow velocities to determine the depth of total scour at the bridge. Three types of scour are calculated; contraction scour, pier scour and abutment scour.

It must be noted that HEC-RAS' scour module does not calculate general scour, which occurs as the result of natural morphological processes irrespective of whether a structure is there. General scour may include long term degradation or aggradation, lateral stream bed migration or scour due to a natural constriction or bend. This effect should be considered separately.

The HEC-RAS scour analysis is based on the waterway design procedures and standards detailed in the FHA scour evaluation publication, with the scour extent established using one of the Hydraulic Design options of the HEC-RAS model. Scour estimates use the hydraulic parameters together with bed material data and the following less than realistic assumptions, namely:

- cohesionless material; and,
- a homogeneous soil profile.

The output from the underlying hydraulic model is automatically incorporated into the bridge scour computation window. Site specific variables (D50, D95 and K factors) must be entered or confirmed by the user. The data may be edited if required. **Figure 2.7A** shows the *Hydraulic Design – Bridge Scour* main screen. The following section explains each of the input parameters.

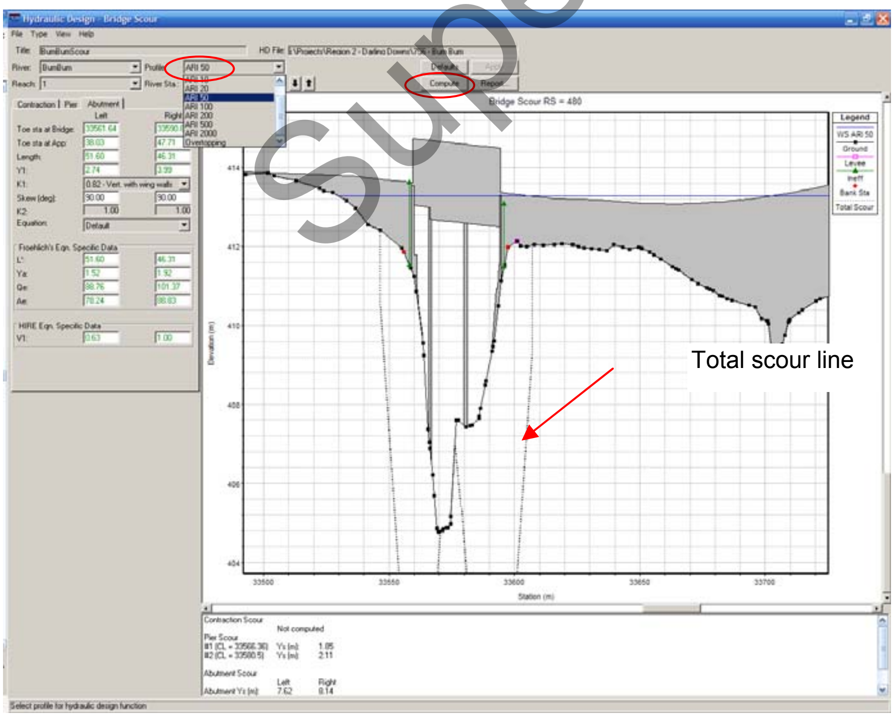


Figure 2.7A: Bridge Scour module in HEC-RAS

Firstly, the user must select the bridge and flow profile (red oval:ARI50) generated in earlier hydraulic results. These values will be used for the scour analysis.

For contraction scour, the equations for live bed or clear water scour are used, refer **Figure 2.7B**. The variables (shown in blue ovals) for median grain size, D50 and abutment coefficient shape K1 must be entered by the user. Note that in the K1 tab, the water temperature value defaults to 15C which is low for most Queensland sites. The contraction scour tab is divided into three columns – left overbank LOB, main channel and right overbank ROB. This allows the program to calculate contraction scour for the three areas of the cross section.

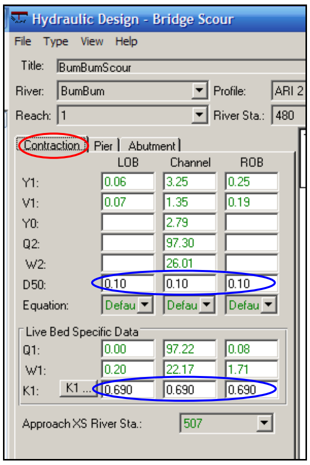


Figure 2.7B: Bridge Scour module – Contraction Scour

The 'compute' button is selected to run the scour calculation model. The results are displayed in tabular and graphical form. The program automatically considers the critical velocity for the approach velocity for each bank and the channel to determine if live bed or clear water scour is possible. The appropriate scour calculation is automatically selected when Equation is left as 'Default'.

For pier scour the modified CSU (Richardson and Davis, 1995) equation or the Froehlich Pier Scour Equation may be selected at the yellow circle, refer **Figure 2.7C**. The values in green are sourced from the earlier hydraulic analysis. The K1, K2, K3 and D95 can be edited by the user. Refer **Tables 2-2** to **2-4** for the FHWA's recommended K factors. For pier scour calculations the user has the option of selecting the maximum velocity and depth or the local velocity and depth. In general, the maximum values should be used to account for thalweg migration within the cross-section.

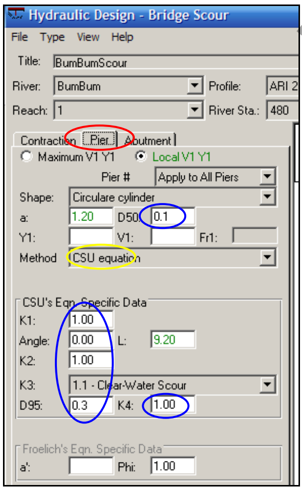


Figure 2.7C: Bridge Scour module – Pier Scour

Abutment scour is computed by either the HIRE equation (Richardson 1990) or Froehlich's Abutment Scour Equation, refer **Figure 2.7D** and to Section 2.5.7 generally. The user only needs to select the abutment type and skew as the remaining variables will be automatically computed. The scour at the left and right abutments are calculated separately. The program calculates the L / y1 ratio to determine the appropriate equation to use.

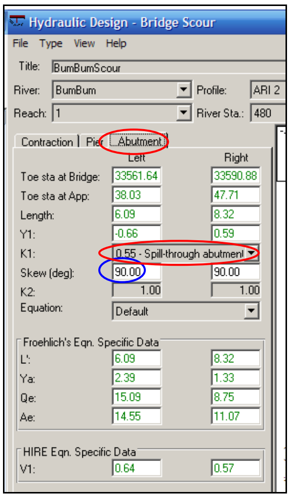


Figure 2.7D: Bridge Scour module – Abutment Scour

The total scour is a combination of the contraction scour and the individual pier and abutment scour at each location and the results are output in both graphical and tabular format. The total bridge scour is the summation of the contraction, pier and abutment scour values. This information can be output as a table by selecting ‘Report…’ and the Graphic can be saved using the File >> Copy Plot to Clipboard.

2.7.1 Worked example of HEC-RAS

More comprehensive example and worked Tutorial model is available from HEC-RAS website http://www.hec.usace.army.mil/software/hec-ras. However, the following emphasises the steps that should be undertaken when performing a scour analysis with HEC-RAS.

Initial hydraulic analysis

Undertake a hydrologic analysis and background study of the bridge site. Minimum detail required is drainage area, mean slope, rainfall intensity and frequency and duration (IFD) information from the Bureau of Meteorology and Engineers Australia’s *Australian Rainfall & Runoff (1987).* Detailed steps are not covered here as they should be familiar to the practitioner. They are presented in the Department’s *Road Drainage Manual*. Other required information is:

- Topographic map of the stream and its floodplain, the location of the bridge crossing and stream channel cross-sections;
- Geomorphology information regarding estimated channel degradation, the channel lateral movement zone, D50 soil particle sizes in the channel/flood plain and whether the type of scour to be expected is clear-water or live-bed;
- Surface and subsurface information on channel bed load, floodplain soils, borings, etc;
- Geometric information about the bridge and approach roads;
- HEC-RAS runs for the given hydraulic conditions including:
  - stream channel cross-sections, hydraulic data tables,
  - reliable bridge tailwater elevations,
  - selection of appropriate approach section and flow distribution, and
  - appropriate flow distribution at bridge with regard to channel, floodplain and overtopping flows.

Develop water surface profile(s) using HEC-RAS or other modelling program (typically a 2D unsteady model such as TUFLOW or MIKE Flood). This model will determine flow upstream of, through and downstream of the bridge. The design discharges should include the overtopping flow, to develop the anticipated worst case scour conditions at the bridge. It is important that the underlying hydraulic model be calibrated to ensure that Manning's "n" values provide a proper flow distribution between the channel and floodplains. The HEC-RAS scour module is based around one-dimensional flow modelling and therefore requires accurate flow distribution to evaluate scour.

Equally important is sufficient downstream representation to establish reliable bridge tailwater elevations. Inaccurate tailwater elevations can have a significant effect on scour results. Tailwater investigations typically do not extend far enough downstream, specifically on low-gradient streams. Normal depth assumptions for downstream boundary conditions should include a tailwater sensitivity analysis. Downstream control structures such as bridges, culverts and dams should be assessed for their effect on tailwater. Complex hydraulic conditions such as a downstream confluence or tidal flow may necessitate investigating multiple tailwater scenarios.

Flow distribution adjustments

It should be noted that the HEC-RAS flow distribution option does not perform any adjustments to the flow; rather it simply divides the initial flow distribution (based on conveyance) into the number of flow subsections specified by the user:

Steady Flow Analysis window>> Options menu>> Flow Distribution Locations.

Therefore, flow adjustments are necessary to provide for a reasonable progression of flow. It is recommended that for complex projects the time-varying results are extracted from two dimensional floodplain models. This will help with modifications of the HEC-RAS flow distributions. It is emphasized that the adjustment process should be carried out by experienced HEC-RAS modellers who understand the significance and validity of required adjustments.

The goal of the flow distribution adjustments is to provide a reasonable progression of channel and overbank flows from upstream of the approach section to downstream of the structure. There are three typical flow distribution cases:

a) Bridge abutments located at or near the channel banks, no overtopping of structure;

b) Abutments set back from channel banks, no overtopping of structure; and

c) Abutments set back from channel banks, with overtopping of the structure.

Where the abutments are at or near the channel banks, assume all of the flow is in the channel. For abutments that are set back from the channel, determine target flow distribution values through the structure using the:

Steady Flow Simulation Analysis window>>Options menu>>Flow Distribution Locations.

For overtopping flow, the target values should be adjusted to account for any weir flow that is on the left overbank, channel and right overbank at the structure, dividing the total weir discharge provided by HEC-RAS based on proportions of the weir length. The HEC-RAS percentage flows in the left overbank, channel and right overbank for the third case (with overtopping) are for flow through the bridge only and they must be recomputed based on total discharge.

Look for trends in the flow distribution that HEC-RAS computes prior to any adjustments by reviewing flow (Q percent left, Q percent channel and Q percent right) in a user-defined HEC-RAS table. Look for reasonably consistent flow in the overbanks and the channel for sections upstream of the influence of the structure or a consistent flow contraction that shows flow moving into the channel as it approaches the structure. The latter scenario may require only minor adjustments in the flow distribution. Start flow distribution adjustments several sections upstream of the approach section selected for the scour evaluation. To redistribute flow start with the overbanks areas by adjusting Manning's roughness up or down and/or make the edges of the floodplain ineffective. Flow prior to the contraction should stay fairly consistent, with percent flow changes between successive sections within an overbank or in the channel that do not exceed 15%. For larger streams and rivers, a maximum 20% change may be more appropriate.

For a typical flow contraction (Cases 1 and 2), the main channel discharge should steadily increase in the direction of flow as flow is pushed into the channel from the overbanks. Changes to roughness and/or ineffective area limits can be used to achieve this pattern. Overtopping conditions (Case 3) need to be carefully considered in terms of the downstream flow distribution since tailwater elevation and the hydraulics of the bridge can be affected. Immediately downstream of the bridge, overbank flow should be limited to the flow overtopping the road and/or bridge. In typical situations, the flow through the bridge cannot expand quickly enough to be effective on the overbanks just below the structure. A blocked obstruction may be used to reflect this condition; that is, reduce the amount of flow in the section immediately downstream of the bridge. To add flow to an overbank area, the elevation of the floodplain can be lowered. This may be necessary in a situation where HEC-RAS places all the flow in an incised channel, but overtopping flow on a roadway approach is known to exist.

If the bridge hydraulics changes due to the downstream flow distribution adjustments (revised tailwater elevation or flow through the bridge, etc.), a second iteration in the adjustments may be needed to establish new target values. If the percent of the total flow that overtops the road is 15% or less, there probably will not be much of a change in the target values and no changes to the flow distribution would likely be required.

Any changes to the HEC-RAS geometry to adjust the flow distribution must be reasonable. For instance, adjustments to Manning's roughness values in the channel or overbank areas must be within the bounds of what could reasonably be expected based on site conditions and engineering judgement. The adjustments should result in relatively minor changes in water-surface elevations as compared with the initial condition.

Example of flow distribution adjustment

For this example stream, the flow distribution at the river stations for the initial HEC-RAS run is presented in **Table 2.7.1A**, (from Kester, 2010). Target values of 48% of the flow in the main channel (MC), 34% on the left overbank (LOB) and 18% on the right overbank (ROB) at the bridge were selected as the basis for the flow adjustments in the upstream river stations. Note that this example assumes that the flow distribution upstream of RS 6000 as computed initially by HEC-RAS is reasonable.

Table 2.7.1A: Initial flow distribution from HEC-RAS model

River Station (RS)	Percent LOB	Percent MC	Percent ROB	Comments
7000	14	50	36	Reasonable distribution
6000	14	50	36	Begin adjustments
5000	16	31	53	Too little flow in MC, too much flow on ROB
4500 Approach XS	**25**	**25**	**50**	Too little flow in MC, too much flow on ROB
3500	47	25	28	Too little flow in MC, too much flow on ROB and LOB
2000	57	32	11	Too little flow in MC, too much flow on LOB
1500 Bridge XS	***34***	***48***	***18***	***Bridge Target Values***
1000	50	36	14	Too little flow in MC, too much flow on LOB
100	17	79	5	Too much flow in MC, too litte flow on LOB

The simplest approach to redistributing the flow is to make adjustments to Manning's roughness values within HEC-RAS using the:

Geometric Data window >> Tables menu >> Manning's roughness n or k values.

The initial roughness values in the channel or overbank can either be raised to reduce the flow or lowered to increase the flow, resulting in flow being shifted from one portion of the cross section to another. The adjustments were initiated at RS 6000, working in the downstream direction. **Table 2.7.1B** shows the initial and adjusted flow distributions. Notice that Table 1 shows too little flow in the channel from RS 5000 to RS 2000. Therefore, channel roughness values were decreased for these river stations to shift flow to the channel.

Table 2.7.1B: Initial Flow Distribution and Adjusted (Adj.) Flow Distribution

River Station (RS)	Percent LOB		Percent MC		Percent ROB	
	Initial	Adj.	Initial	Adj.	Initial	Adj.
7000	14	-	50	-	36	-
6000	14	17	50	39	36	44
5000	16	20	31	42	53	38
4500 Approach XS	25	23	25	43	50	35
3500	47	27	25	49	28	24
2000	57	30	32	50	11	20
1500 BR (Targets)[1]	***30***		***51***		***19***	
1000	50	29	36	55	14	16
100	17	28	79	64	5	8

[1] Note that the target values changed slightly due to decreased bridge tailwater.

There is too much flow is on the right overbank from RS 5000 to RS 3500 and roughness's were raised to shift flow. The end result is that flow was shifted from the right overbank to the channel in order to produce the pattern of the contraction of the flow that is expected to occur. **Table 2.7.1C** highlights the roughness changes that were made to redistribute the flow in this example.

Table 2.7.1C: Manning's Roughness Adjustments

River Station (RS)	ROB		MC		LOB	
	Initial n	Adj. n	Initial n	Adj. n	Initial n	Adj. n
6000	0.1	0.08	0.04	0.05	0.1	0.08
5000	0.1	0.09	0.04	0.033	0.1	0.15
4500 Approach XS	0.1	0.18	0.04	0.03	0.1	0.16
3500	0.1	0.18	0.04	0.03	0.1	0.16
2000	0.1	0.18	0.04	0.031	0.1	0.08
1500 BR	-	-	-	-	-	-
1000	0.12	0.12	0.04	0.035	0.12	0.14
100	0.12	0.08	0.04	0.055	0.12	0.08

The flow distribution based on the revised (lower) tailwater elevation is still appropriate, since the target values changed only slightly. This is due to the fact that the amount of overtopping flow is fairly low (less than 15%). Notice that the channel portion of the flow distribution at the approach section has changed dramatically from the initial condition to the adjusted condition. Table 2.7.1C indicates that at the approach section (RS 4500), the channel flow increased significantly from 25% to 43%. This higher approach channel discharge results in a lower scour depth at the bridge. The calculated scour depth for the initial flow is 3.4m. The adjustments to flow distribution reduce the scour depth to 0.8 m.

HEC-RAS bridge scour module output

The total depth of scour is a combination of long-term bed elevation changes, contraction scour, and local scour at each individual pier and abutment. Once the scour is computed, HEC-RAS automatically plots the scour at the upstream bridge cross section. An example of the output plot is shown in **Figure 2.7.1A**. HEC-RAS plots both contraction scour and total local scour. The contraction scour is plotted as a separate line below the existing condition cross section data. The local pier and abutment scour are added to the contraction scour, and then plotted as total scour depths.

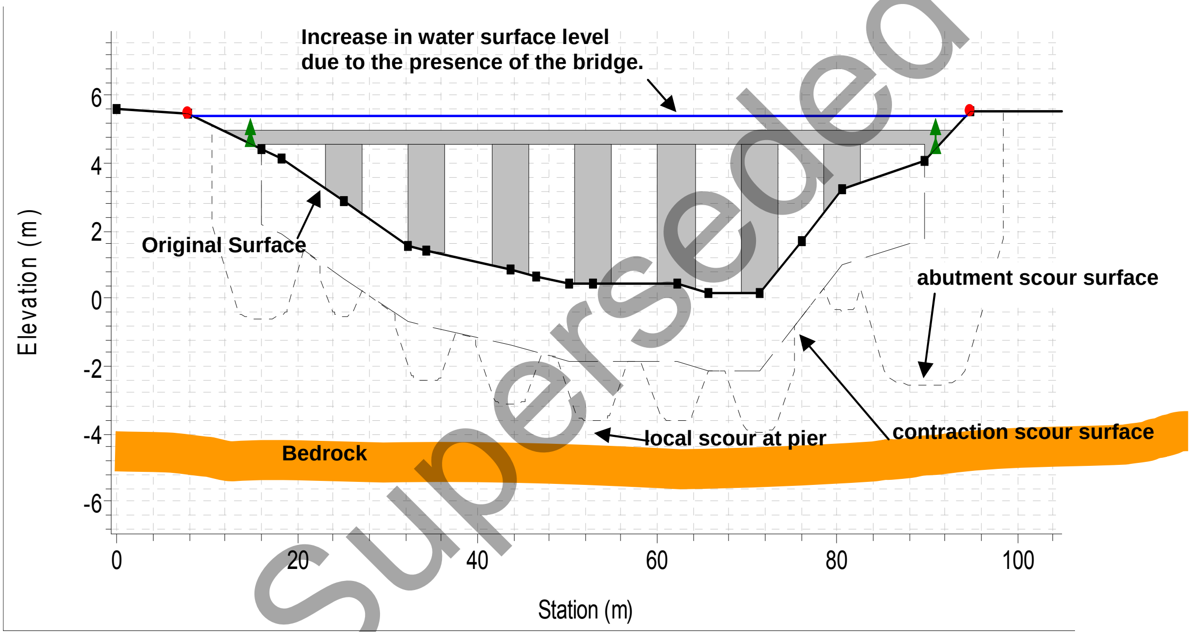


Figure 2.7.1A: HEC-RAS Total scour profile at bridge (annotated) Pier width exaggerated for clarity

The figure also indicates total scour profiles at the upstream face of the bridge for the corresponding flood event. Also, shown are the deck, abutments and piers. The figure was annotated with the approximate location of bedrock taken from the geotechnical report.

Additionally, HEC-RAS outputs results in tabular format, this is shown in **Figure 2.7.1B**. This provides the starting bed elevation, contraction scour depth, pier scour depth, total depth, elevation of total scour depth, and base elevation of piers for various flood events.

A thorough scour analysis will indicates whether the pier may fail, but there could be limiting factors of subsurface strata of clay, shale, sandstone, or limestone that may prevent formation of such a deep scour hole. It is necessary that a sub-surface geotechnical investigation be performed for the safety of the piers.

Interpretation of HEC-RAS output

HEC-RAS will output design information that will require analysis and interpretation. The interpretation of the outputs requires the user to be very familiar with adjusting the model to achieve reasonable and representative values. A calibrated model is the preferred starting point for scour design but in many cases not always achievable. Therefore, some level of judgement and understanding is required. It is likely that there will be some uncertainty about key information. Therefore the user should undertake sensitivity testing to determine the bounds of scour. As noted throughout this document, scour is based around laboratory models and assumptions drawn from the behaviour of cohesive material. HEC-RAS implements these equations and requires considered input to generate reasonable estimates of scour in sites where the material type varies and bedrock may be present. It is recommended that outputs are compared against historical information, geotechnical borelogs and check calculations. For example, scour will cease once it reaches bedrock, if deep scour depths are calculated where shallow rock outcrops are present this will relocate the area of scour formation or increase velocities. This alternative scour location may require more scour protection than is suggested by HEC-RAS output. Conversely, scour may have also occurred in the past due to debris blockage or breaching of upstream dams. HEC-RAS may not provide a scour depth due to this scenario being outside the bounds of the calculation.

In most events, it is recommended that pile depths incorporate the loss of scoured material in the structural design calculation. This adopts a degree of conservatism in the structural design case.

Pier Scour

Pier: #1 (CL = 33566.36)

Input Data

Pier Shape:	Circular cylinder
Pier Width (m):	1.20
Grain Size D50 (mm):	0.00010
Depth Upstream (m):	6.39
Velocity Upstream (m/s):	0.89
K1 Nose Shape:	1.00
Pier Angle:	0.00
Pier Length (m):	9.20
K2 Angle Coef:	1.00
K3 Bed Cond Coef:	1.10
Grain Size D90 (mm):	0.00010
K4 Armouring Coef:	1.00

Results

Scour Depth Ys (m):	1.85
Froude #:	0.11
Equation:	CSU equation

Pier: #2 (CL = 33580.5)

Input Data

Pier Shape:	Circular cylinder
Pier Width (m):	1.20
Grain Size D50 (mm):	0.10
Depth Upstream (m):	5.84
Velocity Upstream (m/s):	1.24
K1 Nose Shape:	1.00
Pier Angle:	0.00
Pier Length (m):	9.20
K2 Angle Coef:	1.00
K3 Bed Cond Coef:	1.10
Grain Size D90 (mm):	0.20
K4 Armouring Coef:	1.00

Results

Scour Depth Ys (m):	2.11
Froude #:	0.16
Equation:	CSU equation

Abutment Scour

	Left	Right
Input Data		
Station at Toe (m):	33561.64	33590.88
Toe Sta at appr (m):	38.03	47.71
Abutment Length (m):	51.60	46.31
Depth at Toe (m):	2.74	3.99
K1 Shape Coef:	0.82 - Vert. with wing walls	
Degree of Skew (degrees):	90.00	90.00
K2 Skew Coef:	1.00	1.00
Projected Length L' (m):	51.60	46.31
Avg Depth Obstructed Ya (m):	1.52	1.92
Flow Obstructed Qe (m3/s):	88.76	101.37
Area Obstructed Ae (m2):	78.24	88.83
Results		
Scour Depth Ys (m):	7.62	8.14
Qe/Ae = Ve:	1.13	1.14
Froude #:	0.29	0.26
Equation:	Froehlich	Froehlich

Figure 2.7.1B: Example HEC-RAS Tabular Result for Scour Design

Summary

Developing a HEC-RAS model with a reasonably consistent flow distribution pattern is a good example of what can be done to improve the accuracy of scour estimates and avoid over-prediction. Experienced HEC-RAS users should be able to make flow distribution adjustments in a relatively short time frame, say two to three hours. Other reasons for high estimates of scour may include:

- Over-estimating the design discharge. This may occur in the use of hydrologic models, if the models are not calibrated properly;
- Selection of overly-conservative calibration factors for modelling computations,
- Inaccurate measurements/estimates of soil properties,
- Addition of all the various elements of scour (contraction scour, pressure scour, pier scour, channel movement, bend scour, degradation, etc.) to compute total scour when it may not be reasonable to assume that all possible types of scour will occur at the same time. These combinations should be evaluated on a case by case basis.

Using modelling software like HEC-RAS allows the user to quickly conduct sensitivity tests on the input parameters. The user can test the effect of various factors (such as soil particle size) on scour depths and can print out a complete report for each factor in a matter of a few minutes. This approach is best left to engineers with a practical understanding of the inter-relationships of the various factors affecting the computation of scour. Design considerations for scour should include all factors affecting the bridge foundations as discussed within earlier sections of this manual.

2.8 Conclusion

This section has covered how to calculate scour depth and concepts relating to scour at bridges. The first step in good design is to minimise scour by appropriate design techniques that appreciate stream stability. This involves selecting bridge sites and avoiding designs that worsen scour, such as minimising skew and under-sizing waterway area. The inter-disciplinary team should undertake an investigation that is warranted by the complexity of the site and significance of the crossing. This will require collection of background and historical information about the site and proposed designs. In most cases this should involve a calibrated two dimensional floodplain model and complementary HEC-RAS calculations. Two dimensional hydraulic models will create a velocity vector map and flow distribution for the natural, existing and proposed bridges cases. This will provide an understanding of the bridge's impact on afflux and the velocities that are tolerable in the channel. Further, this model will help identify any long-term and general scour problems within the floodplain. The bridge should be sited well away from any problematic locations. This section has outlined the calculations required to determine contraction and local scour depths. It is recommended that the practitioner has an understanding of the theory and limitations of scour calculations as outlined in this section. Day to day detailed analysis of bridge options can then be completed using outputs from the hydraulic model input to the HEC-RAS model or via hand calculations. It is critical that the scour design is completed by a practitioner conversant in the limitations and able to interpret the results in the context of the design. As noted throughout this report, scour is not very well understood and there are limitations in the understanding of the processes. Careful application of the equations is recommended. The following section outlines how to design countermeasures based on scour depths.

3 COUNTERMEASURES FOR EXISTING SCOUR SUSCEPTIBLE BRIDGES

3.1 Introduction

FHWA's HEC-23, "*Bridge Scour and Stream Instability Countermeasures*" consists of a two volume manual devoted to scour countermeasures. It serves as the best available reference for scour counter measures. While this wealth of information is a useful reference to the engineer, many of the measures are not likely to be implemented in Queensland.

The following section is a concise summary of information found within HEC-23 (http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf). This chapter should be read in conjunction with MRTS03 and Standard Drawings 1540-1555. Any contradiction between these documents is unintended and those standards take precedence.

Recent experience has shown that overtopping of bridges leads to the majority of bridge closures. Bridges can overtop due to inadequate hydraulic capacity or sometimes debris blockage. The greatest damage has frequently been observed between the bridge approach and the abutment. Historically, this is the intersection between the road and bridge designer's responsibility. Protection of the bridge should consider the impacts of overtopping flows at the roadway. Further information regarding repair of washed out bridge approaches is detailed in **Section 2.6.2 Embankment Batter Protection**. Other related information can be found in **Sections 2.4.2 Scour Design Event** and **2.5.6** regarding debris blockage.

Countermeasures are defined by HEC-23 as a measure incorporated at a bridge site to monitor, control, inhibit or minimise stream stability problems and bridge scour. In many cases, the best countermeasure is appropriate design that avoids causing stream instability. This chapter is written to assist in protecting existing bridges that have experienced scour problems. It is noted that re-opening a bridge occurs when time is of the essence and resources are limited. While re-opening the bridge is urgent initially, it is important that more comprehensive repairs are implemented before the next flood event. This may mean re-visiting a bridge site to undertake planned and corrective maintenance in accordance with the following chapter.

Further specialist advice should still be sought as this chapter cannot cover all of the likely scenarios encountered throughout Queensland. Tidal conditions are not considered. Chapter 9.13 of the *Road Drainage Manual* contains detail on culvert outlet protection that can be read in conjunction with Sections 3.3 and 3.4.

Over the last several decades, a wide variety of countermeasure structures, armouring materials and monitoring devices have been used at existing bridges to mitigate scour and stream stability problems. While most bridge inspectors/engineers are familiar with standard countermeasures such as riprap, it is unlikely that they are knowledgeable of the full spectrum of countermeasures currently available and in use. A reference table is provided at the end of this chapter.

3.1.1 Countermeasures for scour susceptible bridges

Since scour susceptible bridges are already in place, options for structural or physical modifications such as replacement or foundation strengthening are limited and expensive. Unless these bridges are programmed for replacement, their continued operation will ultimately require the design and installation of a scour countermeasure. **Figure 3.1.1** is an example of bridge scour countermeasures having been installed as part of emergency repair works.

Figure 3.1.1: Various protection measures used in repair works

Riprap is, and will remain, one of the primary scour countermeasures to resist local scour forces at abutments of typical bridges. Riprap is generally abundant, inexpensive and requires no special equipment. However, proper design and placement is essential. Section 3.3 and 3.4 provide guidelines for proper grading and placement methods. When designing riprap countermeasures, maintaining an adequate hydraulic opening through the bridge must be considered. Improperly placed riprap may reduce the hydraulic opening significantly and create contraction scour problems. If placed improperly, riprap can increase local scour forces. Although riprap is widely used, the following countermeasures can be considered as alternatives to riprap, but are not all covered here:

Armouring countermeasures

- Rock riprap
- Gabion boxes/ rock mattresses
- Sack gabions
- Grouted riprap
- Grout-filled mats
- Articulating concrete blocks

River training countermeasures

River training structures alter stream hydraulics to mitigate undesirable erosional and/or depositional conditions. They are commonly used on unstable stream channels to redirect stream flows to a more desirable location through the bridge. The following options are not discussed here due to the specialist nature of this work.

- Spurs (both permeable and impermeable)
- Bendway weirs
- Guide banks
- Drop structures and check dams

3.1.2 Scour protection design for bridges

Quantitative scour estimates provide an indication of the site's susceptibility to scour. Armouring the areas affected by scour with a layer of non-erosive material will protect the scour affected areas. Typical scour repair methods at bridges include:

- Dumped rock over a geofabric layer at piers, abutments and channel banks
- Gabion mattresses over a geofabric layer at piers, abutments and channel banks
- Concrete (shotcrete) at bridge abutments

Normally the scour protection is used to fill any scour holes that have formed to the original bed levels. Rigid measures (concrete slabs etc) are not as desirable due to potential for catastrophic failure. Flexible scour protections have an ability to self heal once a failure mode commences.

If shotcrete (concrete) is used at the bridge abutments for scour repair it must be tied into the abutment slope. If it is not properly tied into the slope it can be undermined and result in further damage to the abutment. This method is not recommended, particularly where the scour is being caused by a geotechnical failure of the embankment slopes.

The following is provided as outline information only. It does not constitute or replace specialist engineering assistance. Design advice must always be sought from an RPEQ engineer with relevant experience.

3.2 Components of Scour Countermeasures

Many scour countermeasures consist of a filter layer (geotextile or granular) overlain by a heavy-duty armour (usually rock riprap) and maybe a form of containment (basket or cables) holding it together. Correct design of the filter layer is essential and often overlooked. Filters limit the loss of fines, while providing a free-flowing interface. The permeability of the geotextile should be ten times that of the underlying soil. Too broad a filter will enable the rock riprap to roll off the filter and compromise the countermeasure. This and other factors are discussed in greater details below.

TMR provides a set of standard drawings that detail abutment protection. Further detail on scour protection can be found within **Standard Drawings 1540** to **1554**.

3.3 Filter Design

3.3.1 Introduction

NCHRP Reports 568 and 593 (refer Section 3.4 for URL) describe the importance of filters to the successful long-term performance of armouring-type countermeasures. Based on a survey of the existing state of practice, these reports indicate that filter design criteria have typically been the most overlooked aspect of revetment riprap design. It is recommended that more emphasis be given to ensuring compatibility between the filter and the underlying soil.

Correct filter design reduces the effects of piping by limiting the loss of fines, while simultaneously maintaining a permeable, free-flowing interface. Seepage flow and turbulence at the water-filter interface induces the migration of soil particles. The particle size distribution of the base soil underlying an armour layer must be determined to properly design a filter for particle retention. For example, when a filter with relatively large pores overlies a uniform fine-grained soil, piping of the fine particles may continue unabated, since there are no particles of large and intermediate sizes to prevent their migration through the filter. Conversely, the presence of large and intermediate sized particles in the soil matrix prevents clogging from occurring at the soil-filter interface when filters with relatively small pores are used. In addition to particle retention, filters must have sufficient hydraulic conductivity (sometimes referred to as "permeability") to allow unimpeded flow of water from the base soil through the filter material. This is necessary for regulating the particle migration process at the soil-filter interface. Secondly, it helps minimise hydrostatic pressure from seepage out of the channel bed and banks, typically caused by seasonal groundwater fluctuations or flood events. The hydraulic conductivity of the filter should never be less than the material below it (whether base soil or another filter layer). **Figure 3.3.1** illustrates the typical process that occurs during and after a flood event. Seepage forces can result in piping of the base soil through the armour layer. If a filter is less permeable than the base soil, an increase of hydrostatic pressure can build beneath the armour layer. A properly designed, permeable filter material will alleviate problems associated with fluctuating water levels.

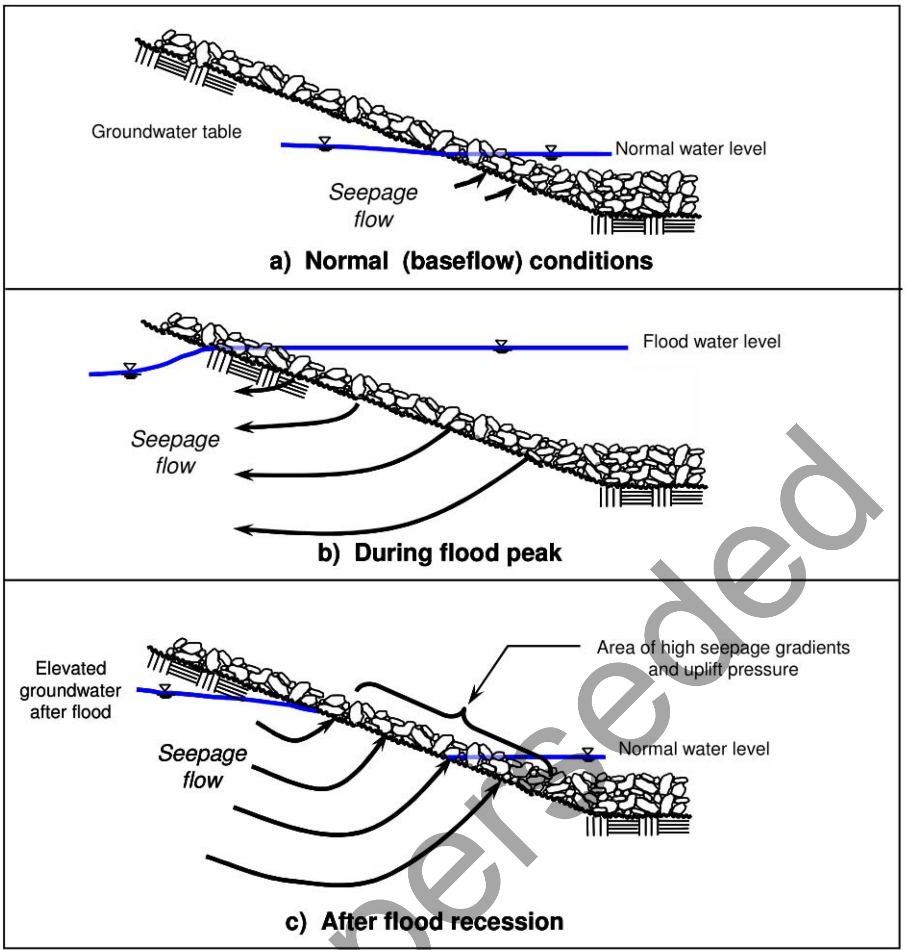


Figure 3.3.1: Effect of a flood on water levels and seepage patterns (HEC-23, 2009)

Base soil properties

Base soil is defined here as the subgrade material upon which the filter and armour layer (riprap, for example) will be placed. Base soil can be native in-place material, or imported and re-compacted fill. The following properties of the base soil should be obtained for proper design of the filter, whether using a geotextile or a granular filter.

Soils are classified based on laboratory determinations of particle size characteristics and the physical effects of varying water content on soil consistency. Typically, soils are described as coarse-grained if more than 50% by weight of the particles is larger than a 0.075 mm mesh, and fine-grained if more than 50% by weight is smaller than this size. Sands and gravels are examples of coarse-grained soils, while silts and clays are examples of fine-grained soils. The fine-grained fraction of a soil is further described by changes in its consistency caused by varying water content and by the percentage of organic matter present.

Particle size distribution

The most important soil property for filter design is the range of particle sizes in the soil. Particle size is a simple and convenient way to assess soil properties. Also, particle size tends to be an indication of other properties such as hydraulic conductivity. Characterising soil particle size involves determining the relative proportions of gravel, sand, silt, and clay in the soil. This characterization is usually done by sieve analysis for coarse-grained soils or sedimentation (hydrometer) analysis for fine-grained soils.

Plasticity

Plasticity is defined as the property of a material that allows it to be deformed rapidly, without rupture, without elastic rebound, and without volume change. A standard measure of the plasticity of soil is the Plasticity Index (PI), which should be determined for soils with a significant percentage of clay. The results associated with plasticity testing are referred to as the Atterberg Limits.

Porosity

Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.

Hydraulic conductivity

Hydraulic conductivity, sometimes referred to as permeability, is a measure of the ability of soil to transmit water. These test the amount of water passing through a saturated soil sample over a specified time interval, along with the sample's cross-sectional area and the hydraulic head at specific locations. The soil's hydraulic conductivity is then calculated from these measured values. Hydraulic conductivity is related more to particle size distribution than to porosity, as water moves through large and interconnected voids more easily than small or isolated voids. Various equations are available to estimate hydraulic conductivity based on the grain size distribution. However, further consultation with geotechnical and materials engineers are required for estimating this property. Table 3-1 lists typical values of porosity and hydraulic conductivity for alluvial soils.

Table 3.3.1: Typical values for porosity and hydraulic conductivity of Alluvial Soils (McWhorter and Sunada 1977)

Material	Porosity	Hydraulic Conductivity (cm/s)
Gravel, coarse	0.28	4×10^{-1}
Gravel, fine	0.34	3×10^{-2}
Sand, coarse	0.39	5×10^{-2}
Sand, fine	0.43	3×10^{-3}
Silt	0.46	3×10^{-5}
Clay	0.42	9×10^{-8}

3.3.2 Granular filter properties

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters may be used alone or as a transitional layer between a predominantly fine-grained base soil and a geotextile.

Particle size distribution

In general, the gradation curve of the granular filter material should be approximately parallel to that of the underlying base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material that overtops it. Note that this procedure can be used to determine rock riprap and filter sizing providing the ratio (max A) of coarse D_{50} / finer D_{50}. Heibaum (2004) presents a summary of a procedure originally developed by Cistin and Ziems whereby the D_{50} size of the filter (coarser layer) is selected based on the coefficients of uniformity (D_{60} /D_{10} = U) of both the finer base soil layer (U_I) and the filter material (U_{II}). With this method, the grain size distribution curves do not necessarily need to be parallel, refer **Figure 3.3.2A** and **Figure 3.3.2B**.

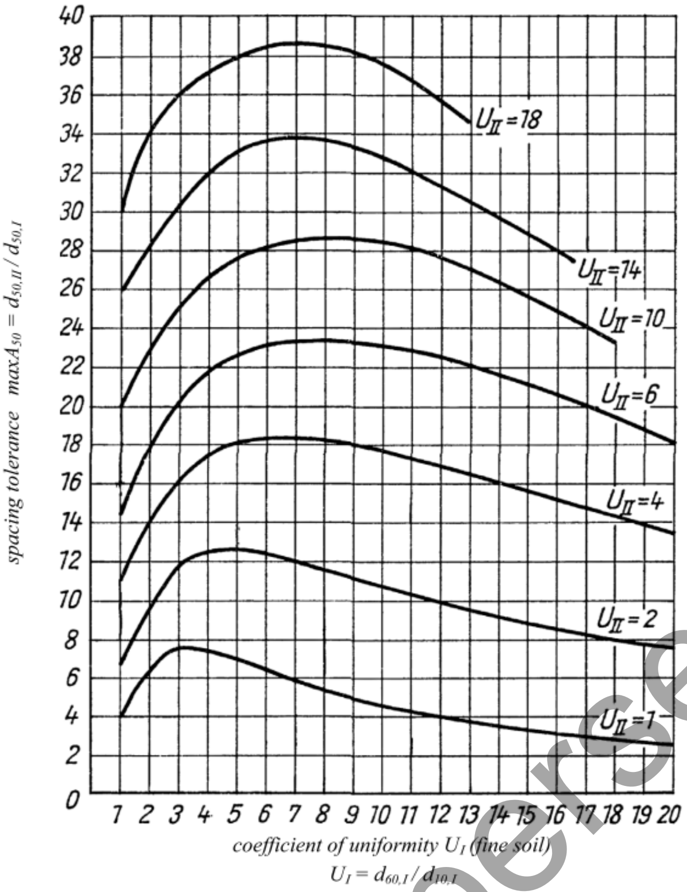


Figure 3.3.2A: Selection of D_{50} size for overtopping granular layer

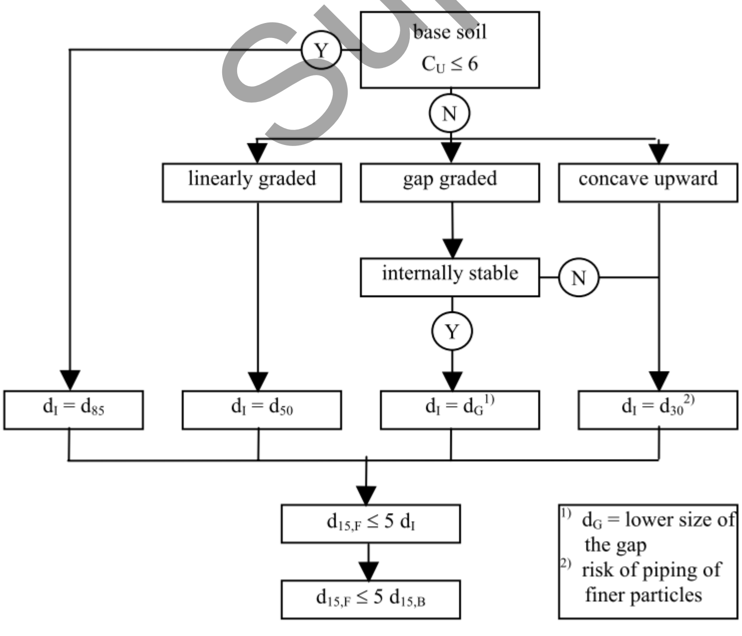


Figure 3.3.2B: Selection of D_{15} filter layer

Hydraulic conductivity

Hydraulic conductivity of a granular filter material is determined by laboratory test, or estimated using relationships relating hydraulic conductivity to the particle size distribution. The hydraulic conductivity of a granular layer is used to select a geotextile when designing a composite filter. For countermeasure installations, the hydraulic conductivity of the filter should be at least 10 times the hydraulic conductivity of the underlying material.

Porosity

Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.

Thickness

Practical issues of placing a granular filter indicate that a typical minimum thickness of 150 to 200 mm should be specified. For placement under water, thickness should be increased by 50%.

Quality and durability

Aggregate used for a granular filter should be hard, dense, and durable.

3.3.3 Geotextile filter properties

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of hydraulic conductivity, pore size (otherwise known as Apparent Opening Size, or AOS) and porosity (or percent open area). In addition, geotextiles must be sufficiently strong to withstand the stresses during installation. Values of these properties are available from manufacturers.

Only woven monofilament or nonwoven needle-punched geotextiles should be considered for filter applications. Slit-film, spun-bonded, or other types of geotextiles are not suitable as filters. If a woven monofilament fabric is chosen, it should have a Percent Open Area (POA) greater than 4%. If a nonwoven needle-punched fabric is chosen, it should have a porosity greater than 30%, and a mass per unit area of at least 400 grams per square meter. The following list briefly describes the most relevant properties of geotextiles for filter applications.

Hydraulic conductivity

The hydraulic conductivity of a geotextile is a tested property of geotextiles that is reported by manufacturers for their products. The hydraulic conductivity is a measure of the ability of a geotextile to transmit water across its thickness. It is typically reported in units of centimetres per second (cm/s). This property is directly related to the filtration function that a geotextile must perform, where water flows perpendicularly through the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ, is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ, is defined as K divided by the geotextile thickness, t, in centimetres. Hydraulic conductivity (and permittivity) are extremely important in filter design.

Porosity

Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to non-woven geotextiles only. Porosity is used to estimate the potential for long term clogging, and is typically reported as a percentage.

Percent open area (POA)

POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles only. POA is used to estimate the potential for long term clogging, and is typically reported as a percentage.

Thickness

As mentioned above, thickness is used to calculate hydraulic conductivity. It is typically reported in millimetre.

Grab strength and elongation

These relate to the force required to initiate a tear in the fabric when pulled in tension. They are reported in Newtons as measured in a testing apparatus having standardised dimensions. The elongation measures the amount the material stretches before it tears, and is reported as a percent of its original (unstretched) length.

Tear strength

Force required to propagate a tear once initiated.

Puncture strength

Force required to puncture a geotextile using a standard penetration apparatus. There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving countermeasures have been discussed here. As previously mentioned, geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and covered by the armour layer, these stresses do not represent the long-term environment that the geotextile will experience.

3.3.4 Installing geotextiles under water

Placing geotextiles under water is problematic for a number of reasons. Most geotextiles that are used as filters beneath riprap are made of polyethylene or polypropylene. These materials have specific gravities ranging from 0.90 to 0.96, meaning that they will float unless weighted down or otherwise anchored to the subgrade prior to placement of the riprap (Koerner 1998). In addition, unless the work area is isolated from river currents by a cofferdam, flow velocities greater than about 0.3 m/s create large forces on the geotextile. These forces cause the geotextile to act like a sail; and will often resulting in wavelike undulations of the fabric that are extremely difficult to control. In mild currents, geotextiles (pre-cut to length) have been placed using a roller assembly, with sandbags to hold the fabric temporarily.

To overcome these problems, a product known as SandMat™ was developed. This blanket-like product consists of two non-woven geotextiles (or a woven and a non-woven) with sand in between. The layers are stitch-bonded or sewn together to form a heavy, filtering geocomposite. The composite blanket exhibits an overall specific gravity ranging from approximately 1.5 to 2.0, so it sinks readily. According to Heibaum (2002), this composite geotextile has sufficient stability to be handled even when loaded by currents up to approximately 1 m/s. At the geotextile - subsoil interface, a non-woven fabric should be used because of the higher angle of friction compared to woven geotextiles.

Placing a sand filled filter underwater

Sand-filled geotextile containers made of non-woven needle punched fabric are particularly effective for placement under water as shown in **Figure 3.3.4**. The fabric for the geotextile containers should be selected in accordance with the filter design criteria presented above, and placed such that they overlap to cover the required area. Geotextile containers can be fabricated in a variety of dimensions and weights. Each geotextile container should be filled with sand only to about 50 to 65% of the container's total volume so that it remains flexible and "floppy." The geotextile containers can also serve to fill a pre-existing scour hole around a pier prior to placing the gabion mattresses. For more detail, see HEC-23.

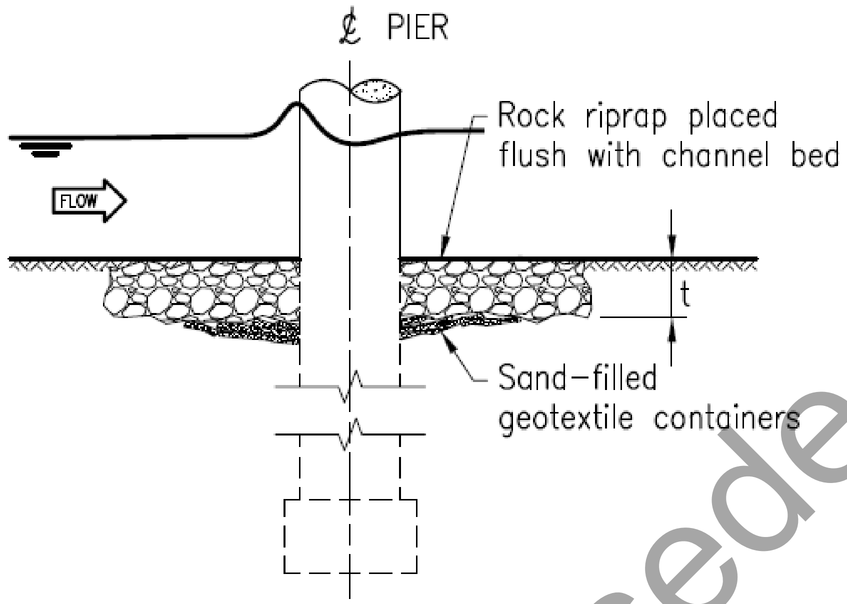


Figure 3.3.4: Schematic diagram showing the use of sand filled geotextile as a filter

3.4 Rock Riprap

3.4.1 Introduction

When properly designed and used for erosion protection, riprap has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost, and it can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. This design guideline considers the application of riprap as a pier scour countermeasure.

Design of a pier scour countermeasure system using riprap requires knowledge of the:

- river bed and foundation material;
- flow conditions including velocity, depth and orientation;
- riprap characteristics of size, density, durability, and availability;
- pier size, shape, and skew with respect to flow direction; and
- type of interface material between the riprap and underlying foundation.

The system typically includes a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and ex-filtration to occur while providing particle retention.

Bridge pier riprap design is based, primarily, on research conducted under laboratory conditions with little field verification. Flow turbulence and velocities around a pier are of sufficient magnitude that large rocks move over time. Bridges have been lost due to the removal of riprap at piers by turbulent high velocity flow. Usually this does not happen during one storm, but is the result of the cumulative effect of a sequence of high flows. Therefore, if rock riprap is placed as scour protection around a pier, the bridge should be monitored and inspected during and after each high flow event to ensure that the riprap is stable. **Figure 3.4.1** is plot that helps demonstrate the change between the guidance provided in Austroads (1994) guidance and the latest guidance in HEC-23.

The guidance provided in this document for pier protection applications of riprap has been developed primarily from the results of NCHRP Project 24-07(2) (Lagasse et al. 2007) and NCHRP Project 24-23 (Lagasse et al. 2006). Found at the following address:
http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_593_RefDoc.pdf

http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_568.pdf

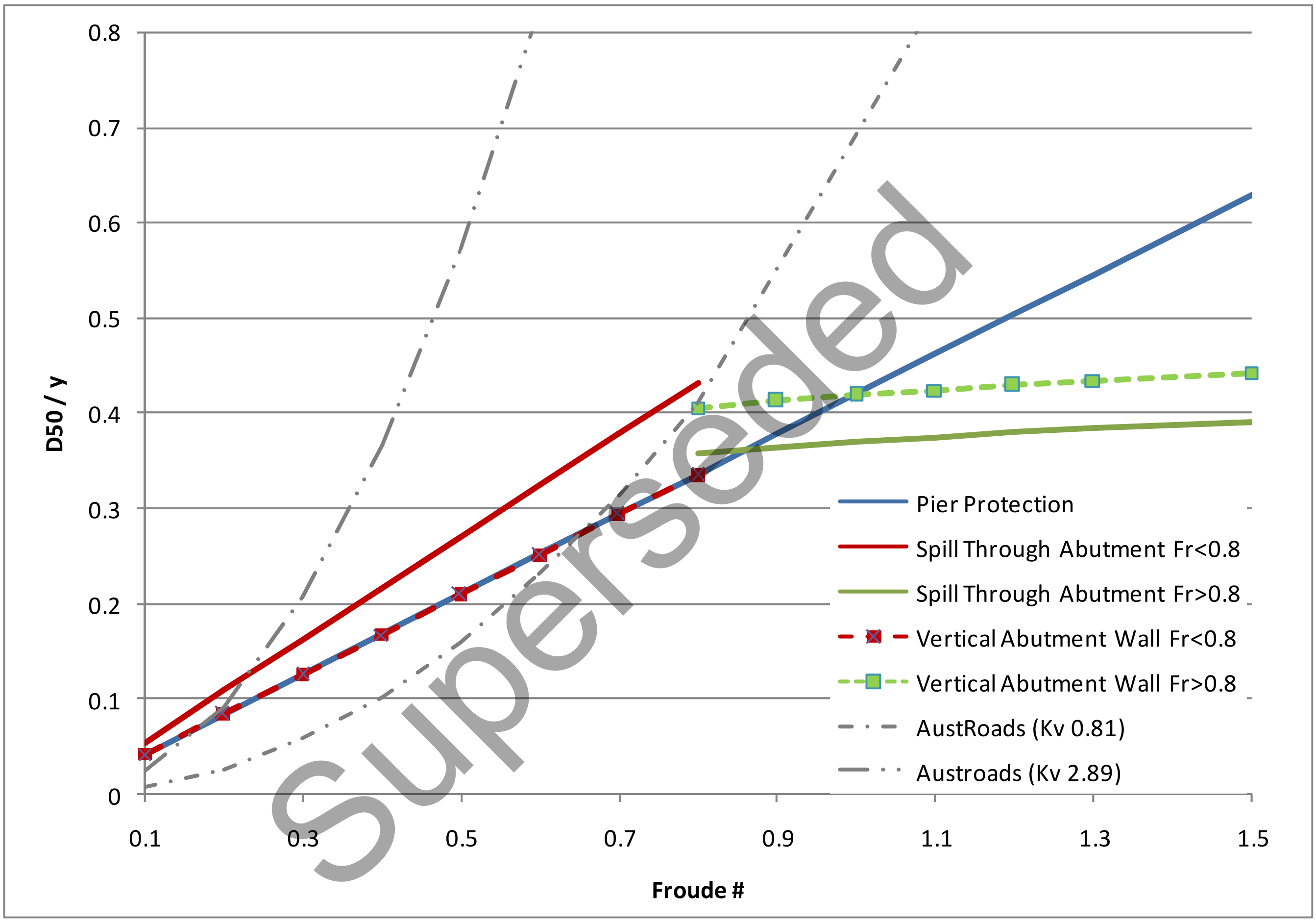
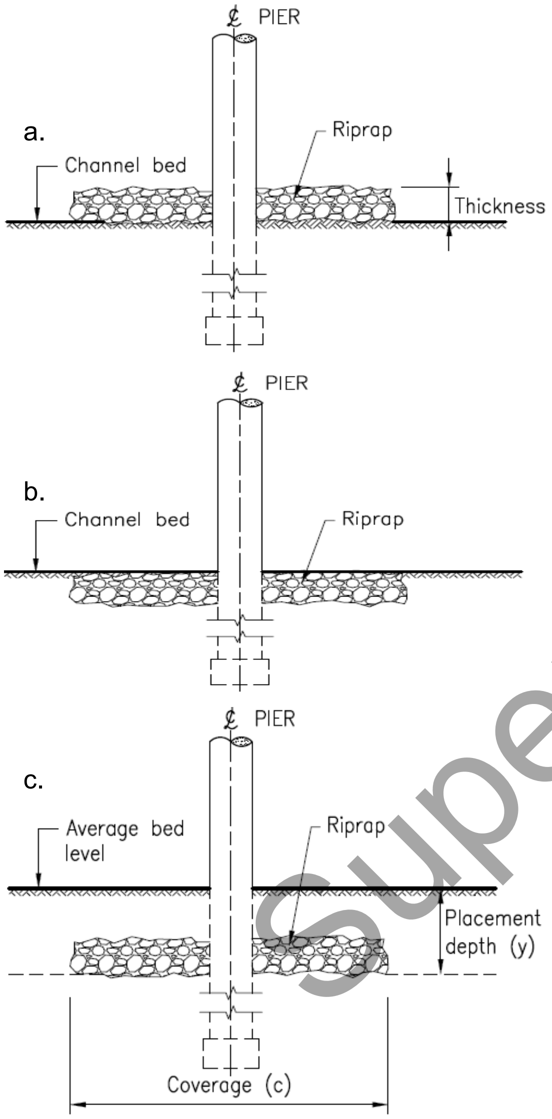


Figure 3.4.1: Summary of Rock sizing and Froude number for various depths

3.4.2 Bridge pier riprap

Most of the early work on the stability of pier riprap considers the size of the riprap stones and their ability to withstand high approach velocities and buoyant forces. Secondary currents induced by bridge piers cause high local boundary shear stresses, high local seepage gradients and sediment erosion from the streambed surrounding the pier. The addition of riprap also changes the boundary stresses.

There are at least a dozen equations for sizing bridge pier riprap that can be considered for design (Lagasse et al. 2007, Melville and Coleman 2000). Typically, the stability of riprap is expressed in terms of the Stability Number, N_{sc} which is used in numerous equations to size riprap. This approach, which derives from the work of Isbash (1936) considers turbulence intensity to determine rock size. Riprap stone size is designed using the critical velocity near the boundary where the riprap is placed. However, many of the pier riprap sizing equations are modified versions of bank or channel protection equations and so, the use of this approach has limitations when applied at bridge piers because of the strongly turbulent flows near the base of a pier. Most of the remaining equations are based on threshold of motion criteria or empirical results of small-scale laboratory studies conducted under clear-water conditions with steady uniform flow. Options for the sizing and locating of pier riprap protection are shown in **Figure 3.4.2A**. **Figure 3.4.2B** provides further guidance on sizing to avoid common failure modes in riprap design.

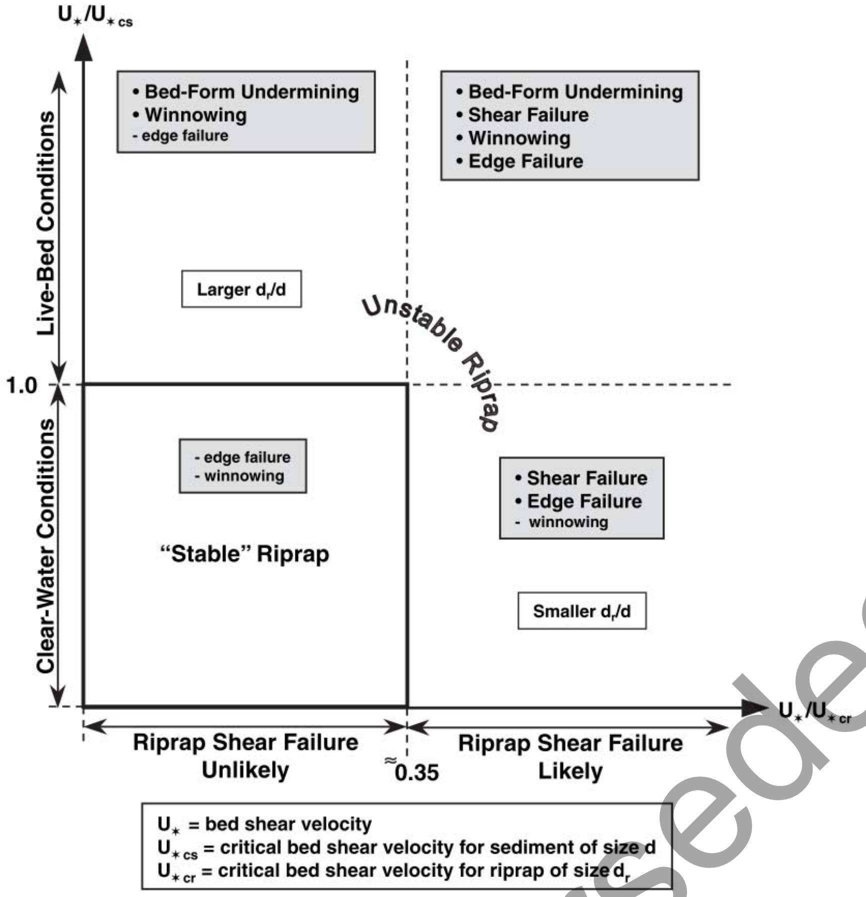


This option interferes with available waterway area and can worsen contraction scour

Ideal option but can be difficult to implement for construction constraints

Coverage is at least 2 pier widths. Filter omitted for clarity

Figure 3.4.2A: Typical pier riprap configurations (filter omitted for clarity)



Source: modified from Lauchlan (1999)

Figure 3.4.2B: Summary of pier riprap failure conditions for bed regimes

Sizing rock riprap at bridge piers

To determine the required size of stone for riprap at bridge piers, NCHRP Project 24-23 recommends using the rearranged Isbash equation to solve for the median stone diameter:

$$d_{50} = \frac{0.692(V_{des})^2}{(S_g - 1)2g}$$

where:

d_{50} Particle size for which 50% is finer by weight, (m)

V_{des} Design velocity for local conditions at the pier, (m/s)

S_g Specific gravity of riprap (usually taken as 2.65)

g Acceleration due to gravity, (9.81 m/s^2)

It is important that the velocity used is representative of conditions in the immediate vicinity of the bridge pier including the constriction caused by the bridge. If the cross-section or channel average velocity, V_{avg} is used, then it must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 K_2 V_{avg}$$

If a velocity distribution is available from the flow distribution output of a 1D model or directly from a 2D model, then only the pier shape coefficient (K1) should be used. The maximum velocity in the active channel V_{max} is often used since the channel could shift and the highest velocity could impact any pier.

$$V_{des} = K_1 V_{max}$$

K1 Shape factor equal to 1.5 for round-nose piers or 1.7 for square-faced piers

K2 Velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach, to 1.7 for a pier located in the main current of flow around a sharp bend)

V_{avg} Channel average velocity at the bridge, m/s

V_{max} Maximum velocity in the active channel, m/s

Once a design size is established, a standard gradation class can be selected, if design criteria and economic considerations permit. Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed riprap installation, but economically a less expensive one.

Layout dimensions

Based on NCHRP Project 24-07(2) the optimum performance of riprap as a pier scour countermeasure was obtained when the riprap extended a distance of two times the pier width in all directions around the pier (Lagasse et al. 2007).

In the case of piers where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential caused by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armour layer should be multiplied by a factor K_α , which is a function of the width (a) and length (L) of the pier (or pile bents) and the skew angle α as given below (Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a\cos\alpha + L\sin\alpha}{a} \right)^{0.65}$$

Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation, refer **Figure 3.4.2Aa.**. Placing the top of the riprap flush with the bed is ideal for inspection purposes, and does not create any added obstruction to the flow. Mounding riprap around a pier is <u>not acceptable</u> for design, because it obstructs flow, captures debris, and increases scour at the periphery of the installation.

The riprap layer should have a minimum thickness of three times the D50 size of the rock. However, when the depth of contraction scour through the bridge opening exceeds 3 times the D50, the thickness of the riprap must be increased to the full depth of the contraction scour plus any long-term degradation. In river systems where dune bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of Karim (1999) and/or van Rijn (1984). In general, an upper limit on the crest-to-trough height (Δ) < 0.4y where 'y' is the depth of flow. This suggests that the maximum depth of the bed form trough below ambient bed elevation will not exceed 0.2 times the depth of flow. Additional riprap thickness due to any of these conditions may warrant an increase in the extent of riprap away from the pier faces, such that riprap launching at a 1V:2H slope underwater can be accommodated. When placement of the riprap must occur under water, the thickness should be increased by 50%.

A filter layer is typically required for riprap at bridge piers, refer **Figure 3.4.2C**. The filter should not be extended fully beneath the riprap; instead, it should be terminated 2/3 of the distance from the pier to the edge of the riprap. When using a granular stone filter, the layer should have a minimum thickness of four times the d50 of the filter stone or 150mm, whichever is greater. As with riprap, the layer thickness should be increased by 50% when placing under water.

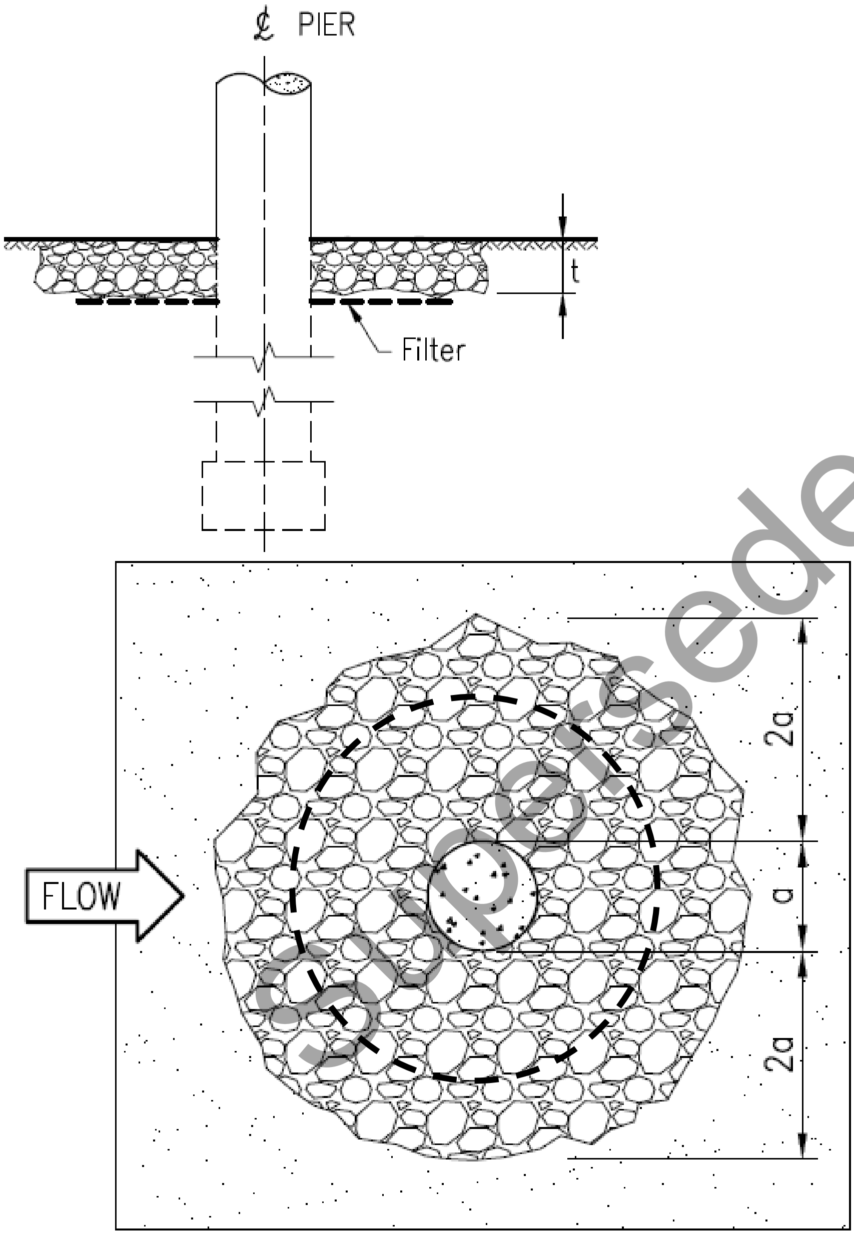


Figure 3.4.2C: Riprap layout diagram for pier scour protection

The importance of the filter component of any riprap installation should not be underestimated. There are two kinds of filters used in conjunction with riprap; granular filters and geotextile filters. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations of the filter layer. In cases where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered. Guidance on the design of granular and geotextile filters is provided in Section 3-3.

Sand-filled geotextile containers made of properly-selected materials provide a convenient method for controlled placement of a filter in flowing water. This method can also be used to partially fill an existing scour hole when placement must occur under water, as illustrated in **Figure 3.4.2D**.

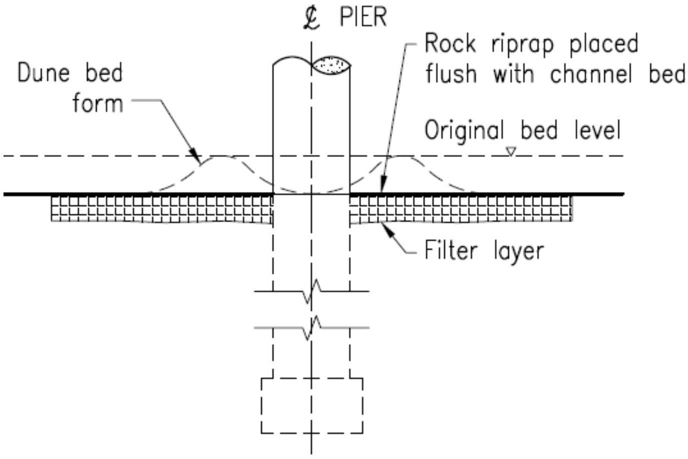


Figure 3.4.2D: Schematic diagram showing sand filled geotextile container beneath pier riprap

Where a flexible protection is not laid at the level of natural plus contraction scour, a greater extent may be required to provide a launching apron to protect against degradation of the adjacent bed. **Figure 3.4.2E** provides a guide on correct and incorrect installation of flexible protection at a bridge pier.

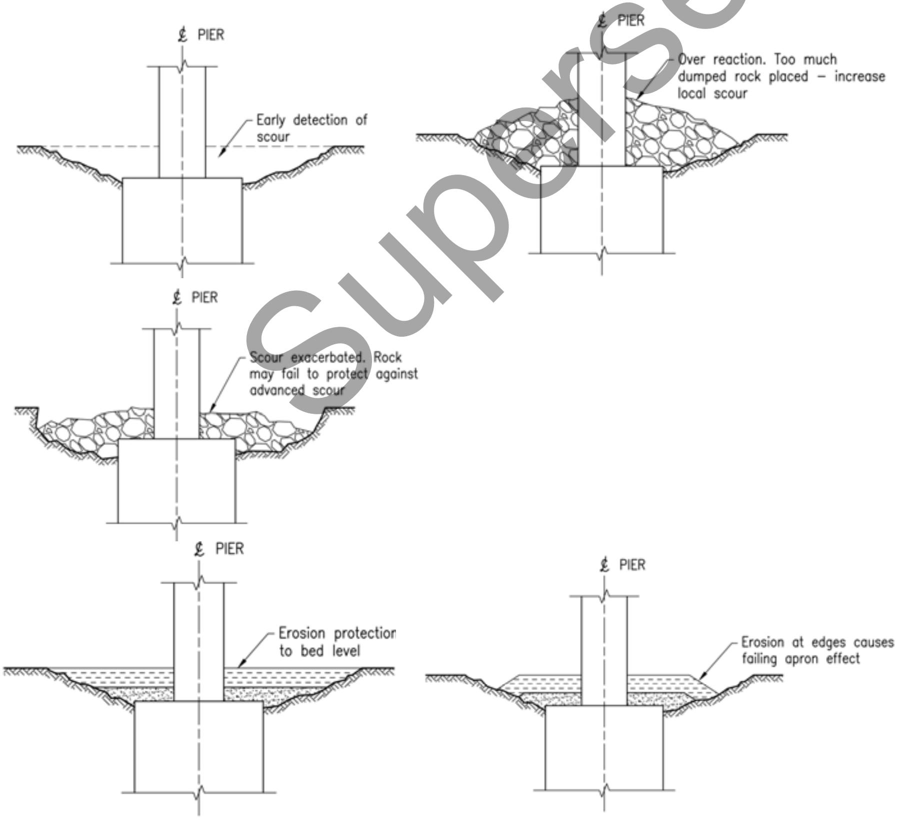


Figure 3.4.2E: Incorrect and correct methods for arresting pier scour using rock riprap.

Specifications for bridge pier riprap

Riprap design methods typically yield a required size of stone that will result in stable performance under the design loadings. Because stone is produced and delivered in a range of sizes and shapes, the required size of stone is often stated in terms of a minimum allowable representative size. For pier scour protection, the designer specifies a minimum allowable d50 for the rock comprising the riprap, thus indicating the size for which 50% (by weight) of the particles are smaller. Stone sizes can also be specified in terms of weight (e.g. W50) using an accepted relationship between size and volume, and the known (or assumed) density of the particle.

For the shape, weight, density, and gradation of bridge pier riprap, specifications developed for revetment riprap are applicable (Lagasse et al. 2006). HEC-23 Design Guideline 4 recommends gradations for ten standard classes of riprap based on the median particle diameter d50 as determined by the dimension of the intermediate ("B") axis. These gradations were developed under NCHRP Project 24-23 and Report 568, *Riprap Design Criteria, Recommended Specifications, and Quality Control*. The proposed gradation criteria are based on a nominal or "target" d 50 and a uniformity ratio D85 / D15 that results in riprap that is well graded. The target uniformity ratio is 2.0 and the allowable range is from 1.5 to 2.5 (Lagasse et al. 2006).

Recommended tests for rock quality

Standard test methods relating to material type, characteristics, and testing of rock and aggregates recommended for revetment riprap are applicable to bridge pier riprap (see Design Guideline 4 in HEC-23). In general, the test methods recommended are intended to ensure that the stone is dense and durable, and will not degrade significantly over time.

Rocks used for riprap should only break with difficulty, have no earthy odour, no closely spaced discontinuities (joints or bedding planes), and should not absorb water easily. Rocks comprised of appreciable amounts of clay, such as shales, mudstones, and claystones, are never acceptable for use as riprap.

Rock riprap at abutments

A stable riprap toe is the most important feature in the design of riprap abutment protection. The toe depth should be below the depth of calculated contraction scour. **Standard Drawing 1544** provides a typical detail for protecting abutment toes from scour forming. Modifications to this design will be required to accommodate site specific conditions. It is recognised that it is difficult to obtain correctly graded stone between a flood event and re-opening the crossing.

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. The types of failures described above are initiated as a result of the obstruction to the flow caused by the abutment and highway embankment and subsequent contraction and turbulence of the flow at the abutments.

Design approach

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. Cost will be the deciding factor (Richardson and Davis 2001).

In summary, 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. Fifteen metre guide banks extending from the downstream corner of the abutment can protect the abutment and approach roadway from scour by the wake. Otherwise, the downstream abutment and approach should be protected with riprap or other countermeasures.

Sizing rock riprap at abutments

The FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour (Pagán-Ortiz 1991, Atayee 1993). The first study investigated vertical wall and spill-through abutments which encroached 28 and 56% on the floodplain, respectively. The second study investigated spill-through abutments which encroached on a floodplain with an adjacent main channel. Encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock riprap consistently failed at the toe downstream of the abutment centreline. For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centerline of the abutment.

For Froude Numbers $(V/(gy)^{1/2}) \leq 0.80$, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]$$

where:

D_{50} = median stone diameter, ft (m)
V = characteristic average velocity in the contracted section (explained below), ft/s (m/s)
S_s = specific gravity of rock riprap
g = gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)
y = depth of flow in the contracted bridge opening, ft (m)
K = 0.89 for a spill-through abutment
1.02 for a vertical wall abutment

For Froude Numbers >0.80, Equation 14.2 is recommended:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]^{0.14}$$

where:

K = 0.61 for spill-through abutments
K = 0.69 for vertical wall abutments

In both equations, the coefficient K, is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure. Both of these equations are envelope relationships that were forced to over predict 90% of the laboratory data.

Specifications for bridge abutment riprap

Refer Section 3.4.2 for specifications.

Recommended tests for rock quality

Refer Section 3.4.2 for tests.

Installation and constructability

Where possible, riprap should be laid so that the stones pack into a close interlocking layer with the size of voids minimised. Where laid on geotextile, great care is needed not to damage the geotextile. Ideally, the first layer of stone should be placed to give as much contact with the surface of the geotextile as possible. A common problem encountered during the construction of riprap is checking the size and gradation of stone used. Various methods have been developed for quality control of stone for riprap. At its most basic, a stockpile should be visually examined to check minimum, maximum and average stone sizes. In addition, it is often useful to weigh stones to obtain an example of each of the three sizes (minimum, maximum and average), which can be set aside for comparison against stockpiles. Where large quantities of stone are used, inspection can involve sorting several truckloads of stone into piles of three or four different stone sizes. Each pile is then weighed and compared with the total sample weight, thus giving the proportion of the total stone in each size category. A representative stone in each pile can then be weighed to define the typical weight that each pile represents.

An alternative method is to monitor as-placed riprap gradings using surface sampling techniques. This involves measuring the size of stones exposed on a constructed section of riprap to give a representation of the plan area occupied by different stone sizes. A sample set of stones should then be weighed to convert the sizes to weights and to develop a grading. Surface sampling can consist of measuring either all the stones within a defined area or all the stones along a defined line (for example, along the line of a tape). Measurement can be carried out on the ground or photography can be used. Computerised measurement techniques are available but not widespread. Another problem is that the stone can segregate during loading at the quarry, or during handling and placing, giving a different as-laid grading to that at the quarry. Careful quarrying, loading and placing practices are needed to avoid this. The stone may need to be remixed before placing to reduce segregation.

3.5 Steel-wire Gabion and Mattresses

3.5.1 Introduction

Gabion and mattresses are containers constructed of steel-wire mesh and filled with rocks. The length of a gabion mattress is greater than the width, and the width is greater than the thickness, refer **Figure 3.5.1** for typical dimensions. Diaphragms are inserted width wise into the mattress to create compartments. Wire is typically galvanized or coated with polyvinyl chloride to resist corrosion, and either welded or twisted into a lattice. Stones used to fill the containers can be either angular rock or rounded cobbles; however, angular rock is preferred due to the higher degree of natural interlocking of the stone fill. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous structure.

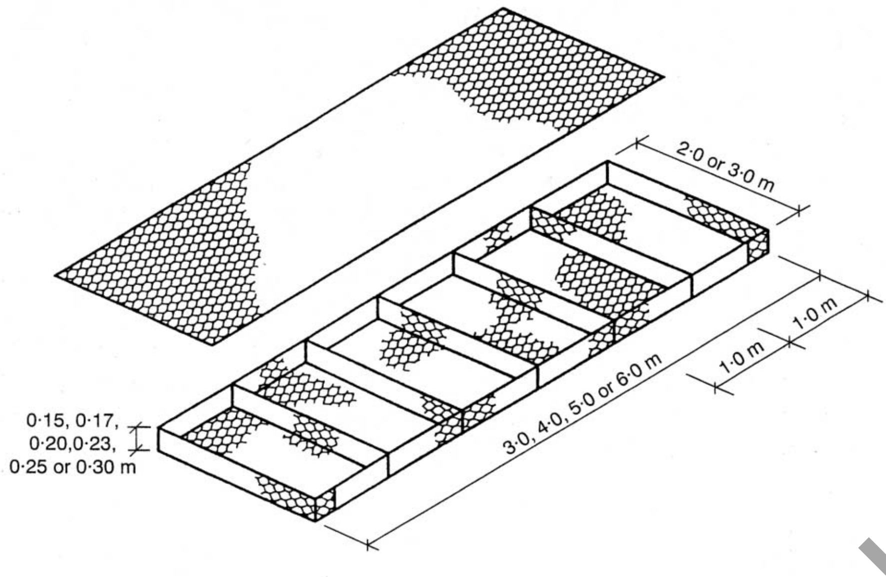


Figure 3.5.1: Typical gabion mattress dimensions

The wire mesh allows the gabions to deform and adapt to changes in the subgrade while maintaining stability. Additionally, when compared to riprap, less excavation of the bed is required and smaller, more economical stone can be used. The obvious benefit of gabion mattresses is that the size of the individual stones used to fill the mattress can be smaller than stone that would individually be too small to withstand the hydraulic forces of a stream (Freeman and Fischenich 2000).

Placement of Gabions at Embankments and Abutments

The layout of gabions shall be to that shown in the applicable **Standard Drawing (1552 - 1554)**. The width of the abutment toe protection should be at least three metres or a multiple of flow depth. This may need to be integrated with pier scour protection measures. For small bridge openings with high scour potential, the gabion nets should extend the full width between the two abutments.

3.5.2 Types of gabions

The following types of gabions are commonly used as armouring countermeasures:

Gabion sacks

They are used when construction in “the dry” is not possible. In the absence of cofferdams, gabion sacks are placed directly in water. The size of a gabion sack range between 500 mm to 900 mm.

Gabion boxes or baskets

Gabion boxes are larger in size than sacks. The minimum dimension of a gabion box ranges between 600mm to 1.2 m. They are more suitable for higher velocities.

Rock filled mattresses

Mattresses are thinner than sacks or boxes and have less weight per unit area. Minimum thickness varies between 200 mm to 450 mm. The mattress is manufactured in greater lengths and tied together. For higher scour depths, two mattresses can be placed on top of each other. They are the most commonly used form.

Wire enclosed riprap

It differs from the Mattress in that it is larger in size and is a continuous framework rather than individual interconnected boxes or baskets. They can be used for slope protection at riverbanks or as guidebanks. Riprap sizes that are used are less uniform when compared to other three types discussed above.

Durability of the wire mesh under long term exposure to the flow conditions at bridge piers has not been demonstrated; therefore, the use of gabion mattresses as a bridge pier scour countermeasure has an element of uncertainty (Parker et al. 1998). **Figure 3.5.2** provides the typical construction layout for using gabion mattresses at bridge piers. Note the two third extent of the underlying filter layer.

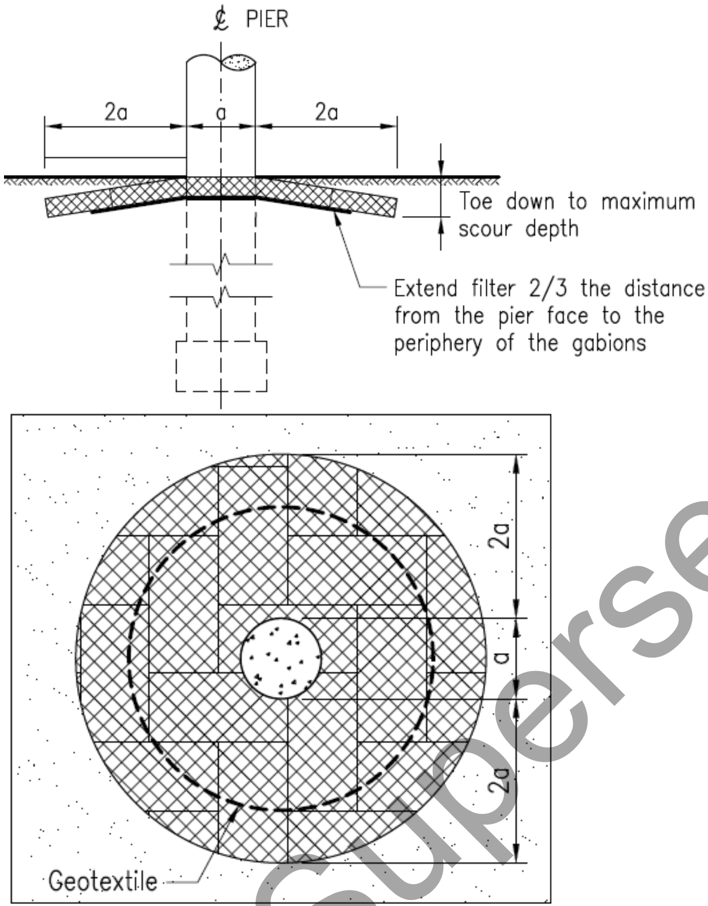


Figure 3.5.2: Gabion mattress layout for pier scour countermeasure

Successful long-term performance of gabion mattresses depends largely on the integrity of the wire. Due to the potential for abrasion by coarse bed load, gabion mattresses are not appropriate for gravel bed streams and should only be considered for use in sand or fine streams. Additionally, water quality of the stream must be noncorrosive (i.e., nonsaline and nonacidic). A polyvinyl chloride (PVC) coating should be used for applications where the potential for corrosion exists.

The Mesh Pattern can be hexagonal, rectangular or V-shaped. For rivers with gravel and cobble beds, the abrasion of wires is greater. In such cases, use double layer of mesh or increase the mesh diameter to minimum 3 mm.

Sizing of gabions and critical velocity

By enclosing the stones within the wire mesh, smaller size stone can be used when compared to the conventional riprap. Typically, thickness of gabions varies between a third to two-thirds of thickness of riprap. Sizing of gabions should be based on technical advice from manufacturers. The thickness of gabions should be determined on the basis of critical velocity of flow. The critical velocity is the velocity where the mattress reaches the limit of deformation. **Table 3.5.2** provides the lower and upper bounds and the design should be selected on the lower bound (critical velocity).

Table 3.5.2: Gabion sizing (Agostoni, 1988)

Type	Thickness (mm)	Rock fill size (mm)	d50 (mm)	Critical velocity (m/s)	Limiting velocity (m/s)
Mattress	150	70-100	85	3.5	4.2
	180	70-150	110	4.2	4.5
	230	70-100	85	3.6	5.5
	250	70-150	120	4.5	6.1
	300	70-120	100	4.2	5.5
		100-150	125	5.0	6.4
Gabions	500	100-200	150	5.8	7.6
		120-250	190	6.4	8.0

3.5.3 Materials

Rock fill

Standard test methods relating to material type, characteristics and testing of rock and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) and are recommended for specifying the rock fill used in gabion mattresses. In general, the test methods recommended in this section are intended to ensure that the stone is dense and durable, and will not degrade significantly over time.

Rocks used for gabion mattresses should only break with difficulty, have no earthy odour, no closely spaced discontinuities (joints or bedding planes) and should not absorb water easily. Rocks comprised of appreciable amounts of clay, such as shales, mudstones, and claystones, are never acceptable for use as fill for gabion mattresses.

Gabion mattresses and components

Successful gabion performance depends not only on properly sizing and filling the baskets, but also on the quality and integrity of the wire comprising the basket compartments, diaphragms, lids, and lacing wire. Investigations conducted under NCHRP Project 24-07(1) (Parker et al. 1998) concluded that the lacing wire in particular proved to be the weakest link of gabion mattress systems. Wire should be single strand galvanized steel; a PVC coating may be added to protect against corrosion where required.

The wire mesh may be formed with a double twist hexagonal pattern or can be made of welded wire fabric. Fasteners, such as ring binders or spiral binders, must be of the same quality and strength as that specified for the gabion mattresses.

Flexibility of the gabion mattress units is a major factor in the successful performance of these systems. The ability to adjust to differential settlement or other changes in the subgrade is desirable. For example, settlement around the perimeter of a gabion mattress installation at a bridge pier is beneficial if scour at the edges of the mattresses occurs. Rigid systems are more prone to undermining and subsequent damage to the mesh, and are therefore less suitable for use at bridge piers. Designers are encouraged to further familiarise themselves with the flexibility and performance of various gabion mattress materials and proprietary products for use in riverine environments.

Longitudinal extent

The revetment armour should be continuous for a distance which extends both upstream and downstream of the region that experiences hydraulic forces severe enough to dislodge and/or transport bed or bank material. The minimum distances recommended are an upstream distance of one channel width and a downstream distance of 1.5 channel widths. The channel reach which experiences severe hydraulic forces is usually identified by site inspection, examination of aerial photography, hydraulic modelling or a combination of these methods.

Many site-specific factors have an influence on the actual length of channel that should be protected. Factors that control local channel width (such as bridge abutments) may produce local areas of relatively high velocity and shear stress due to channel constriction, but may also create areas of ineffective flow further upstream and downstream in "shadow zone" areas of slack water. In straight reaches, field reconnaissance may reveal erosion scars on the channel banks that will assist in determining the protection length required.

In meandering reaches, since the natural progression of bank erosion is in the downstream direction, the present limit of erosion may not necessarily define the ultimate downstream limit. HEC-20 provides guidance for the assessment of lateral migration. The design engineer is encouraged to review this reference for proper implementation.

Vertical extent

The vertical extent of the revetment should provide freeboard above the design water surface. A minimum freeboard of 300 to 600 mm should be used for unconstricted reaches and 600 to 1200 mm for constricted reaches. If the flow is supercritical, the freeboard should be based on height above the energy grade line rather than the water surface. The revetment system should either cover the entire channel bottom or, in the case of unlined channel beds, extend below the bed far enough so that the revetment is not undermined by the maximum scour which for this application is considered to be toe scour, contraction scour, and long-term degradation.

3.5.4 Installing the gabion mattress system

Manufacturer's assembly instructions should be followed. Mattresses should be placed on the filter layer and assembled so that the wire does not kink or bend. Mattresses should be oriented so that the long dimension is parallel to the flow and internal diaphragms are perpendicular to the flow. Prior to filling, adjacent mattresses should be connected along the vertical edges and the top edges by lacing, fasteners, or spirally binding. Custom fitting of mattresses around corners or curves should be done according to manufacturer's recommendations.

Care should be taken during installation so as to avoid damage to the geotextile or subgrade during the installation process. Mattresses should not be pushed or pulled laterally once they are on the geotextile. Preferably, the mattress placement and filling should begin at the upstream section and proceed downstream. If a mattress system is to be installed starting downstream and proceeding in the upstream direction, another option involves constructing a temporary toe trench at the front edge of the mattress system to protect against flow which could otherwise undermine the system during flow events that may occur during construction. On sloped sections, placement and filling shall begin at the toe of the slope and precede upslope, where practical.

Installation and constructability

Delivery from long distances will be cost prohibitive. The type and size of gabions should be selected from locally available sizes. In all cases, gabion designs must be based on hydraulic conditions, long-term durability and ease of maintenance. Excavation machines and small cranes may be used for pre-excavation and for lifting and placing of sacks, boxes and mats in position. The crane can be located on bridge approaches (usually shoulder) or adjacent to riverbed, if access is possible.

Successfully gabion protection can be achieved by constructing them according to the following principles:

- the formation should be well prepared to give a firm foundation
- filter layers or geotextile should be carefully laid to ensure there are no gaps or tears
- mesh and wiring should be tight, with compartment diaphragms tensioned before and during filling with tight lacing of wire
- where slopes are built up with boxes of 0.5 m height or more, they should be internally braced with horizontal cross ties to prevent bulging
- where the thickness of mattresses is 0.5 m or greater, they should be vertically braced to reduce stone movement and hence bulging at the surface
- panels should be laced together in a continuous operation, not using separate twists of wire
- stone should be packed tightly when filling
- the flat parts of stones should be laid against mesh to maximise the contact area and minimise the area of unsupported mesh between stones
- gabion boxes are sometimes filled with large stones on the outside and small stones packed inside – this should be avoided as the small stones tend to be lost through the voids between the larger stones, leading to collapse
- compartments should be slightly overfilled with stone to allow for minor settling of stones and so that the mesh lid is tightly stretched over the top of the stone – the top layer of mesh can be tied down at mid-span to help minimise movement
- the lid should be well laced down and adjacent units should be fully laced together
- where gabions need to be placed against the side of a structure, the required edge shape should be obtained by folding corners of the cells and not by cutting the mesh – for structures such as piers, gabions can be kept in position by tightly lacing them together around the perimeter of the structure if it is necessary to tie gabions directly to a structure (such as a cofferdam or sheet-piled wall), attachments should not be made to the wire mesh because these would be likely to distort and weaken the cells - instead, bearing plates should be placed within the gabions so that the attachment forces are transmitted and spread directly to the stone within the cells
- compartments that are cut to fit awkward shapes present potential weak spots – care is needed to ensure that they are laced to adjacent compartments adequately.

Rock filled mattress placement under water

Mattresses placed in water require close observation and increased quality control to ensure a continuous countermeasure system. A systematic process for placing and continuous monitoring is required to verify the quantity and layer thickness is important.

Excavation, grading, and placement of mattresses and filter under water require additional measures. For installations of a relatively small scale, diversion of the stream around the work area can be accomplished during the low flow season. For installations on larger rivers or in deeper water, the area will require an approach that is safe and environmentally responsible. A site specific approach with appropriate work method statements will be required.

For rivers with less than a metre depth of water, cofferdams may not be required and sand bags could be used. However, cofferdams and silt curtains would probably be required for greater depths. Watertight timber or steel sheeting could be driven into riverbed. The excavated soil should be placed on the banks for reuse. After placing the gabions, 150mm to 300mm layer of excavated soil should be placed on top and compacted. Temporary sheeting should be withdrawn and any voids filled up. A layer of grass or thin vegetation may be grown to stabilise the topsoil. In locations where riverbed has eroded due to recent floods, excavation may not be required and gabions may be deposited directly under water by a barge or within reach by an excavator. This is more economical since a cofferdam will markedly increase costs.

Depending on the depth and velocity of the water, sounding surveys using a sounding pole or sounding basket on a lead line, divers, sonar bottom profiles, and remotely operated vehicles (ROVs) can provide some information about the mat placement and toe down.

Filter requirements

The importance of the filter component of mattress installation should not be underestimated. Geotextile filters are most commonly used with mattresses, although coarse granular filters may be used where native soils are coarse and the particle size of the filter is large enough to prevent winnowing through the rock fill of the gabion mattresses. When using a granular stone filter, the layer should have a minimum thickness of four times the d50 of the filter stone or 150mm, whichever is greater. The D50 size of the granular filter should be determined by using the procedure presented in Design Guideline 16 of this document. When placing a granular filter under water, its thickness should be increased by 50%.

Guidelines for seal around the pier

An observed key point of failure for gabion mattress systems at bridge piers during laboratory studies occurs at the joint where the mat meets the bridge pier. During NCHRP Report 593, securing the geotextile to the pier prevented the leaching of the bed material from around the pier, refer **Figure 3.5.4** for a collar arrangement around a pier. This procedure worked successfully in the laboratory, but there are constructability implications that must be considered when using this technique in the field, particularly when placing the mattress under water. A grout seal is not intended to provide a structural attachment between the mattress and the pier, but instead is a simple method for plugging gaps to prevent bed sediments from winnowing out between the mattress and the structure. In fact, structural attachment of the mattress to the pier is strongly discouraged. The transfer of moments from the mat to the pier may affect the structural stability of the pier, and the potential for increased loadings on the pier must be considered. When placing a grout seal under water, an anti-washout additive is required.

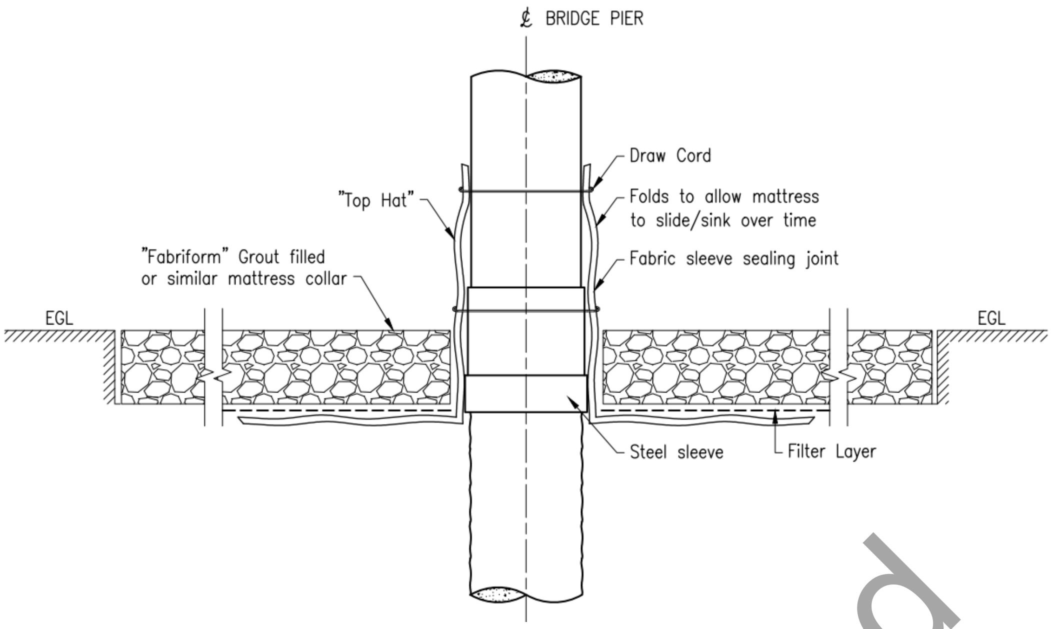


Figure 3.5.4: A possible flexible collar arrangement at a pile to seal joint with a mattress

Anchors

Anchors are not typically used with gabion mattress systems; however, the system should be toed down to a termination depth at least as deep as any expected contraction scour and long-term degradation, or bed form troughs, whichever is greater. Where such toe down depth cannot be achieved, for example where bedrock is encountered at shallow depth, a gabion mattress system with anchors along the front (upstream) and sides of the installation are recommended. The spacing of the anchors should be determined based on a factor of safety of at least 5.0 for pullout resistance based on calculated drag on the exposed leading edge. Spacing between anchors of no more than 1.2m is recommended.

Durability and Maintenance

The following types of failures may occur and may be avoided by good construction practice:

- Failure of meshes and stones fallout due to corrosion, abrasion and damage during construction.
- Winnowing failure due to erosion of underlying bed material through the gabions due to failure of filter layers and inadequate gabion thickness during floods.
- Excessive movement of stone within the baskets may occur at high currents due to poor packing.

3.6 Grout-filled mattresses

Grout-filled mattresses (mats) are comprised of a double layer of strong synthetic fabric, typically woven nylon or polyester, sewn into a series of pillow-shaped compartments that are connected internally by ducts. An example is shown in **Figure 3.5.1**. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Mats are typically sewn together or otherwise connected prior to filling.

Figure 3.6: A possible flexible collar arrangement at a pile to seal joint with a mattress

When set, the grout forms a mat made up of a grid of interconnected blocks. Grout-filled mats are reinforced by cables laced through the mat before the concrete is pumped into the fabric form, creating what is often called an articulating block mat (ABM). Flexibility and permeability are important functions for stream instability and bridge scour countermeasures. Therefore, systems that incorporate filter points or weep holes (allowing for pressure relief across the mat) combined with relatively small-diameter ducts (to allow breakage and articulation between the grout blocks) are the preferred products.

Grout-filled mat systems can range from very smooth, uniform surface conditions that approach cast-in-place concrete in terms of surface roughness, to extremely irregular surfaces exhibiting the roughness of moderate size rock riprap. Because this type of revetment is fairly specialized, comprehensive technical information on specific mat types and configurations is available from a number of manufacturers of this type of revetment. Mats are typically available in standard nominal thicknesses of 100, 150 and 200 mm. A few manufacturers produce mats up to 300 mm thick.

There is limited field experience with the use of grout-filled mat systems as a scour counter-measure for bridge piers. More frequently, these systems have been used for shoreline protection, protective covers for underwater pipelines, bridge abutment spill slopes, and channel armouring where the mat is placed across the entire channel width and keyed into bridge abutments or stream banks. The guidance for pier scour applications provided in this document has been developed primarily from NCHRP Report 593 (Lagasse et al. 2007).

The benefits of grout-filled mats are that the fabric installation can be completed quickly, without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the forms and pumping those with concrete grout can be performed in areas where room for construction equipment is limited.

3.6.1 Materials

Geotextile form

The geotextile comprising the fabric form must exhibit sufficient strength to resist the pressure of the grout during filling. Cords connect the upper layer of fabric to the lower layer at the centre of each compartment. The cords are interwoven with the fabric in two sets of four cords each, one set for the upper layer and one set for the lower layer.

The grout-filled ducts should be no more than 10% of the maximum thickness of the block compartment so that flexibility and articulation can be achieved in the finished installation. Cables enter and exit each compartment through opposing grout ducts; alternatively, cable ducts may be provided for insertion of cables through each compartment. When cable ducts are used, the maximum allowable diameter should be limited to 25 mm.

Cables

Cables are installed between the two layers of fabric prior to filling with grout. The cables run through the individual compartments in a manner that provides for both lateral and longitudinal connection. The cables enter and exit the compartments through opposing grout ducts. Cables should be high tenacity, low elongation continuous filament polyester fibres, with a core contained within an outer jacket. The core should be between 65 to 75% of the total weight of the cable.

Cable splices are made with aluminium compression fittings such that a single fitting results in a splice strength of 80% of the breaking strength of the cable. Two fittings separated by a minimum of 6 in. (150 mm) should be used per splice. When the installation is completed, the cables and splices are completely encased by the concrete grout.

3.6.2 Grout

The concrete grout consists of a mixture of Portland cement, fine aggregate, water, admixtures, and fly ash (optional) to provide a pumpable slurry. The grout should have an air content of not less than 5% nor more than 8% of the volume of the grout, and should obtain a minimum 28-day compressive strength of 13,750kPa. The mix should result in a dry unit weight of the cured concrete of no less than 2,080 kg/m^3. Prior to installation, the grout should be tested for flowability using the flow cone method of ASTM D 6449, with an efflux time not less than 9 seconds or more than 12 seconds using this method. The Engineer may require adjustment of the mix proportions to achieve proper solids suspension and optimum flowability.

3.6.3 Layout details for grout-filled mat

Flexibility of the grout-filled mats is a major factor in the successful performance of these systems. The ability to adjust to differential settlement, frost heave, or other changes in the subgrade is desirable. For example, settlement around the perimeter of a grout-filled mat at a bridge pier is beneficial if scour occurs around the periphery of the mat. Some mat products are more rigid than others, and are therefore more prone to undermining and subsequent damage. Rigid systems are less suitable, in general, for use as bank protection or as a bridge scour countermeasure. Designers are encouraged to familiarize themselves with the flexibility and performance of various grout-filled mat materials and products for use in riverine environments.

3.7 Summary

Scour protection measures are designed to protect the channel bed and banks from the erosive forces causing scour. As shown above they can be categorised as: flexible and rigid systems. Flexible systems can cope with some movement without losing their armouring capability and so can adjust to settlement or movement of the underlying and adjacent surface or bed. Such systems are susceptible to failure from movement of the armour material, either because it is undersized or because of loss of material at its edges. Rigid systems cannot adjust to changes in the underlying surface and are often impermeable. While normally more resistant to erosion, they are susceptible to failure by undermining and uplift (seepage pressure). Factors influencing materials choice are outlined in **Table 3-3**.

The cost of the system is dependent on various factors, including availability of materials, such as rock, the length of haulage routes to the site, and the type of access available for construction. In general, the systems incorporating concrete are more expensive, unless there are long haul routes for rock. The cost of construction underwater tends to be considerably higher than construction in the dry.

Stream encroachment and other applicable permits will be required in accordance with the existing environment protection laws.

Table 3.7: Selection of scour countermeasure

H – High M – Moderate L – Low ✓ Appropriate ▯ Maybe appropriate × Inappropriate	Underwater construction	Repairs	Construction cost	Maintenance cost	Restricted access	Environmental suitability	High velocity flow	Vertical stream instability	Lateral instability
Riprap	×	✓	L	M	▯	✓	✓	✓	▯
Mattresses	✓	✓	M	M	▯	✓	▯	✓	▯
Gabions	×	✓	M	M	▯	✓	▯	▯	▯
Grout filled mattress	✓	✓	H	M	✓	▯	▯	✓	▯
Rigid grout filled bags	✓	✓	M	L	✓	▯	✓	▯	▯
Concrete aprons	×	✓	H	L	✓	▯	✓	×	▯
Stone pitching	×	×	M	M	▯	▯	✓	▯	▯
Protective collars	×	×	L	L	▯	✓	✓	✓	✓
Sheet piling	▯	✓	M	L	▯	✓	✓	✓	✓

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5 GLOSSARY

abutment	End support of a bridge; walls flanking the water channel through a hydraulic structure
accretion	Process by which particles carried by the flow of water are deposited and accumulate (the opposite of **erosion**)
afflux	Difference between water levels upstream and downstream of a structure due to the energy losses caused by the structure
aggradation	General or progressive rise in the bed of a channel by the accumulation of sediment (the opposite of **degradation**)
apron	A layer of stone, concrete or other scour protection placed on a channel bed in the vicinity of a bridge or other hydraulic structure, or at the toe of bank protection
armour layer	Outer layer of larger and/or more durable material used in bank protection or wave protection
armouring	(1) Natural process whereby an erosion-resistant layer of relatively large particles is formed on a stream bed due to the removal of finer particles by erosion and/or the rearrangement of irregularly shaped particles (2) Placement of an armour layer
backwater effects	The effect, in subcritical flow, which flow conditions in one locality have on the flow conditions upstream
bank	Strip of land forming the edge of any channel or body of water; in this manual, left bank and right bank are as viewed looking in a downstream direction
bank protection	Works to protect a bank from erosion
bankfull discharge	Discharge which just fills a channel without overtopping its banks, generally considered to dominate natural channel-forming processes
bar deposit	Layer of sediment deposited in an area of relatively slack water, such as on the inside of a river bend
bathymetry	Topography of the bed of the sea, estuary or other water body
bedform	A recognisable flow-related relief feature on the bed of a channel
bend scour	See **scour**
berm	Horizontal ledge formed in the side slope of an embankment or cutting
braided river	An alluvial river having two or more channels that form a braided pattern and whose size, length and transverse position tend to vary considerably in successive floods

bridge opening	The cross-sectional area or width beneath a bridge that is available for the conveyance of water
caisson	Hollow structure with substantial impermeable walls sunk through ground or water to form a permanent shell of a deep foundation
canal	Channel constructed to convey or contain water, usually for navigation or irrigation
canalised river	River, modified for navigation or irrigation
catchment	The land which drains (normally naturally) to a given point on a river or drainage system
catchment area	The area of a catchment
channel	(1) Natural or man-made open passage designed to contain and convey water (2) The part of a body of water deep enough to be used for navigation through an area otherwise too shallow for navigation (3) The deepest part of a body of water through which the main volume of current passes
clear-water scour	See **scour**
cofferdam	Sheet piling or embankment, normally temporary, constructed to afford access to an area that is normally submerged
competent velocity	Velocity at the threshold of motion of bed material
confluence scour	See **scour**
constriction scour	See **scour**
contraction scour	See **scour**
control structure	Device constructed across a channel or between water bodies or water passages, used to control the discharge passing the device and/or the water level on either side of the device
conveyance (of channel)	Function of the flow area, shape and roughness of a channel, which can be used as a constant in a formula relating discharge capacity to channel gradient
critical flow	Water flow at which the specific energy is a minimum for a given discharge (and Froude number is unity)
critical velocity	Velocity at critical flow
culvert	Covered channel or large pipe to convey water below ground level, for instance under a road, railway or urban area, or beneath a building or other structure
current meter	Instrument for measuring water velocity
degradation	General or progressive drop in the bed of a channel by erosion (the opposite of **aggradation**)
design flood	Flood parameters adopted for the design of a hydraulic structure in a fluvial, estuarine or coastal environment

dike (or dyke)	(1) Embankment, usually earthen, built to prevent or control the extent of flooding (same as flood bank) (2) Embankment, usually earthen and protected with revetment or riprap, built to guide flows within a channel, encourage accretion and/or inhibit erosion (see also **spur dike** and **groyne**) (3) Small artificial watercourse (same as ditch)
discharge	Flow rate expressed in volume per unit time
discharge intensity	Discharge per unit width of a channel (may be averaged over a cross-section or local to a point of interest)
distributary	Channel or smaller river that divides from the principal river and takes part of the flow
dredging	Underwater excavation
ebb (or ebb tide)	Tidal flow associated with falling tide; flow from estuary to sea
eddy	Single vertical vortex
embankment	An artificial, usually earthen, structure, constructed to prevent or control flooding, or for various other purposes including carrying roads and railways
energy dissipator	Device to contain and concentrate the degradation of surplus energy of fast flowing water and protect the downstream bed and banks against scour
entrainment	Process by which sediment particles are dislodged from the bed by hydrodynamic lift and drag forces
erosion	Process by which particles are removed by the action of wind, flowing water or waves (the opposite of accretion)
estuary	The mouth of a river connected to the sea, where both fluvial and tidal effects occur and interact
estuarine	Relating to an estuary
faggot	Bundle of cut branches, used as revetment
fall (or settling) velocity	Terminal velocity at which a particle falls in a still liquid
falling (or launching) apron	Apron of riprap or similar material that subsides as scour occurs, designed to protect the side slopes of a scour hole and prevent scour undermining a structure or the toe of bank protection
fascine mattress	Bundles of brushwood bound together to make a foundation mat to protect the bed of a channel against erosion
filter layer	A layer of granular material or geotextile laid beneath an armour layer, revetment or underlayer, to prevent the passage of fine material
flexible revetment	A revetment with sufficient flexibility to accommodate moderate deformation while maintaining contact with the underlying formation

flood defence works	Works to prevent or alleviate flooding, including works designed to convey and contain water and to resist erosion due to the action of waves and currents
flood tide	Tidal flow associated with rising tide; flow from sea to estuary
flood relief channel	Channel designed to carry excess water during flood conditions
floodplain	Area of nearly flat land bordering a river that is partly or wholly covered with water during floods
fluvial	Relating to a river
foundation	Construction to transmit forces to the supporting ground
freeboard	The height of the crest of a bank, flood bank, bridge or structure above the water level
Froude number	Dimensionless parameter representing the ratio between the inertia and gravity forces in a fluid, taking the value of unity for critical flow
gabion	Cuboid or tubular container made of wire or plastic mesh and filled with stones, used to form a retaining wall or provide protection against scour
gauge board	Graduated vertical scale, fixed to a structure or post, against which may be read the water surface level relative to a datum
general scour	See **scour**
geomorphology	See **morphology**
geotextile	Permeable synthetic fabric used in conjunction with soil for the function of filtration, separation, drainage, reinforcement or erosion protection
groyne	Wall or embankment built out from the coast or a river bank to inhibit erosion of the shore or river bank and/or encourage accretion (see also **spur dike**)
headwater level	Water level upstream of a bridge or other hydraulic structure
hydraulic grade line	Profile of the free surface of flowing water along a channel, or of an imaginary line representing the pressure head in a pipe or culvert flowing full
hydraulic jump	Abrupt rise in water level when flow changes from supercritical to subcritical, with associated dissipation of energy
hydraulic structure	Any structure used to control flows or any structure built in a position where it may affect, or be affected by, flows
hydraulics	Applied science concerned with the behaviour and flow of liquids
hydrograph	Graph that shows the variation with time of level or discharge of water in a river, channel or other water body
hydrology	Science of the occurrence and movement of water over and below the surface of the earth, from the moment of precipitation to the moment of entry into the sea or of evaporation into the atmosphere

incised channel	Main channel in which smaller flows are normally contained; the banks of a natural incised channel are often well defined and formed by processes of lateral erosion or deposition
invert level	Level of the lowest internal part of the cross-section of a bridge or closed conduit
launching apron	See **falling apron**
littoral drift (or transport)	Migration of material in a direction parallel to the sea shore by the actions of waves and current
live-bed scour	See **scour**
local scour	See **scour**
main river	That part of a river (which may include related flood relief channels) which is so designated on maps approved by the Ministry of Agriculture, Fisheries & Foods (MAFF) (under the Water Resources Act 1991; includes certain structures "for controlling or regulating the flow of water into, in or out of the channel")
meandering	Natural process of deviation of a river from a straight course, in which erosion occurs on the outside of bends and accretion occurs on the inside of bends
migration	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank
morphology	The science of landform, concerned in the case of rivers with processes such as meandering, bed material mobility and the geometry of the channel cross-section
natural scour	See **scour**
nickpoint recession	A sudden drop in bed level that propagates upstream in a river or watercourse by erosion of the bed
non-main river	Any natural watercourse not designated as main river
normal depth	Depth of water in conditions of uniform flow
overtopping	The passage of water over a feature such as a flood defence bank or bridge, due to high water levels
pile	Slender structural member substantially underground intended to transmit forces into loadbearing strata below the surface of the land
pile cap	Construction at the head of one or more piles that transmits forces from a structure to one or several piles
piled foundation	Foundation that incorporates piles
pitching	Regularly sized and shaped stones or concrete blocks placed in an ordered fashion as protection against erosion
pitot tube	Device used to determine velocity of fluid flow, by measuring the pressure in an open tube facing into the flow
reach	A length of channel between defined boundaries

regime channel	A channel formed in erodible material which has reached a state of virtual equilibrium, with no long-term aggradation or degradation, but which may be subject to meandering
reinforced grass	Grass surface that has been artificially augmented (for example using a geotextile) to increase its resistance to erosion
retaining wall	A wall designed to support the bank of a river or other channel and form an impermeable boundary to the water passage
return period	Average interval of time between years in which events occur that equal or exceed a given magnitude
revetment	Bank protection system consisting of an outer armour layer to resist erosion by flow or waves and an underlayer to drain and retain the underlying material
riprap	Quarried stone placed in a random fashion as protection against erosion
river	Any natural watercourse (including modified watercourses) carrying perennial flow
rubble	Rocks of irregular shape and size
saltation	Sediment transport in which the particles remain close to the bed and are bounced along
scour	Erosion resulting from the shear forces associated with flowing water and wave action – in this manual normally used to represent erosion associated with currents and the presence of bridges and other hydraulic structures
bend scour	Natural scour due to the presence of a bend
clear-water scour	Scour (normally contraction or local scour) where the bed material in the flow upstream of the scour hole is at rest (compare **live-bed scour**)
confluence scour	Natural scour due to a confluence
constriction scour	Used by some authors to mean the same as contraction scour, but not used in this manual
contraction scour	Scour that affects all or most of the channel bed in the vicinity of a bridge or other hydraulic structure, associated with higher velocities and shear stresses caused by narrowing of the channel
general scour	Used with a range of different meanings by various authors; not used in this manual to avoid confusion
live-bed scour	Scour (normally contraction or local scour) where there is a general movement of bed material, ultimately with a balance between the sediment entering and leaving the scour hole (compare **clear-water scour**)
local scour	Scour that results directly from the impact of individual structural elements (eg piers and abutments) on the flow and occurs only in the immediate vicinity of those elements

scour protection works	Works to prevent or mitigate scour
sediment	Fine material transported in a liquid that settles or tends to settle
sediment concentration	The concentration of sediment at a point in a flow, expressed as a ratio of the sediment per unit mass or volume of the liquid or the liquid–sediment mixture
sediment transport	Movement of sediment under the action of waves and currents
sediment transport capacity	The sediment concentration that given flow conditions are capable of transporting
settling velocity	See **fall velocity**
shear stress (due to fluid flow)	Force per unit area exerted by fluid on boundary of channel and acting tangential to its surface in the direction of flow
shear velocity	A measure of the shear stress, having the dimensions of velocity
Shields criterion	Threshold of movement of particles, expressed in terms of two dimensionless numbers, the entrainment function and the particle Reynolds number, based on experimental work by Shields (and others) on granular sediment
sluice (or sluice gate)	Rectangular gate that moves vertically between guides
soffit level	Level of the highest internal part of the cross-section of a bridge or closed conduit
specific energy	Sum of the depth of water and the velocity head
spring tides	Tides on the two occasions per lunar month when the predicted range between successive high water and low water is greatest
springing (level)	The point on an arched bridge where the curve of the arch intersects the face of the pier or abutment (the tangent point where the arch represents a semicircle)
spur dike (or spur)	Embankment built out from the coast or a river bank to inhibit erosion of the shore or river bank and/or encourage accretion (see also **groyne**)
stage	Elevation of water surface relative to an established datum
stilling basin	Energy dissipator comprising a basin in which a hydraulic jump occurs
subcritical flow	Flow in a channel at less than critical velocity, at which the Froude number is less than unity
supercritical flow	Flow in a channel at greater than critical velocity, at which the Froude number is greater than unity
superflood	A flood larger than the design flood which key structures are required to survive but which may cause an acceptable degree of damage
suspended sediment	Sediment that travels at almost the same velocity as the surrounding liquid and is prevented from settling by the effects of flow turbulence
tailwater level	Water level downstream of a bridge or other hydraulic structure

thalweg	Line of maximum water depth along river or channel
threshold of movement	The condition when particles begin to be entrained into the flow from the river bed or banks, as the velocity is progressively increased
tidal range	Vertical distance between low and high tide levels
tides	Periodic rising and falling of water resulting from the gravitational attraction of the moon, sun and other astronomical bodies, together with the effects of coastal aspect and bathymetry
training works	Works designed to influence the location, flow, scouring or silting characteristics, or aid navigation in a river or estuary
underlayer	Part of scour protection lying beneath the armour layer (see also **filter layer**)
uniform flow	Flow of water in a channel in which the depth and velocity remain constant along the channel
unit discharge	Discharge per unit width of a channel (may be averaged over a cross-section or local to a point of interest)
velocity head	Measure of the kinetic energy of flowing water, represented as the vertical height to which water would rise in a pitot tube
vena contracta	Cross-section of minimum area in a jet of water downstream of a gate or other flow aperture
vortex	A mass of rotating or swirling liquid, in the present context generally caused by an obstruction such as a pier or abutment
wake	Waves downstream of an obstacle or caused by a moving vessel
watercourse	Route along which water flows by gravity
waterway	Channel used, previously used, or intended for the passage of vessels
wave	Short period oscillation of a water surface caused by the action of wind or by the passage of vessels (see **wake**)
wave height	Vertical distance between the crest and trough of a wave
weir	Structure over which water may flow, used to control the upstream water in a channel or other body of water, and/or to measure the discharge