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Manual on scour at bridges and other hydraulic structures

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Summary

Structures built in or near rivers and other channels can be vulnerable to scour around their foundations. If the depth of scour becomes significant, the stability of the foundations may be endangered, with a consequent risk to the structure of damage or failure. The factors influencing the development of scour are complex and vary according to the type of structure. Protection works for preventing scour need to be designed to withstand the flow forces imposed on them and have to be practicable to build and install, while minimising adverse environmental effects.

The objective of this CIRIA project has been to produce a manual for engineers engaged in the design, construction, operation and maintenance of structures in the water environment that may be subject to scour of erodible beds or banks. This manual is relevant to scour problems in the UK and worldwide, affecting both new and existing structures.

Manual on scour at bridges and other hydraulic structures

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This document was written by Mr Richard May of HR Wallingford Ltd, Mr John Ackers of Binnie Black & Veatch and Mr Andrew Kirby of Mott MacDonald, with contributions from Mr Charles Rickard of Mott MacDonald.

Following CIRIA's established practice, the research project was guided by a steering group, which comprised:

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Note

Government reorganisation has moved DETR responsibilities variously to the Department of Trade and Industry (DTI), the Department for the Environment, Food and Rural Affairs (DEFRA), and the Department for Transport, Local Government and the Regions (DTLR). References made to the DETR in this publication should be read in this context.

For clarification, readers should contact the Department of Trade and Industry.

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Glossary

The definitions used in the glossary come from various sources, including BS 6100, *Guide to bridge hydraulics* (Neill, 1973) and *HEC18* (Richardson and Davis, 1995), but many have been revised or written by the authors to suit the context of this manual.

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 bank	Embankment, usually earthen, built to prevent or control the extent of flooding

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	State in which a river, canal or estuary has adjusted its gradient and cross-section to an equilibrium condition; dynamic equilibrium between accretion and erosion

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

natural scour Used by some authors with various different meanings; in this manual includes scour at confluences and bends, long-term degradation, channel migration and the regime conditions for the design flood.

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

Notation

A	Cross-sectional area of flow	m^2
A_{bf}	Value of A for bank-full flow	m^2
$\bar{A}_{bf}$	Average value of A_{bf}	m^2
A_c	Catchment area	km^2
A_s	Cross-sectional area below bank level for regime channel	m^2
a	Coefficient in Equation (4.46)	–
B	Surface width of flow	m
b	Length of spur dike or groyne measured normal to bank	m
B_{bf}	Value of B for bank-full flow	m
$\bar{B}_{bf}$	Average value of B_{bf}	m
B_i	Surface width for each sub-channel	m
C	Conveyance parameter for channel or sub-channel	–
C_I	Factor given by Equations (5.3), (5.7) and (5.9) for effect of turbulence intensity on stability of protection	–
C_p	Factor in Equation (5.11) for stability of stone pitching	–
C_s	Factor in Equation (5.5) for effect of rock shape on stability of riprap	–
C_t	Factor in Equation (5.5) for effect of blanket thickness on stability of riprap	–
C_v	Factor in Equation (5.5) for effect of velocity distribution on stability of riprap	–
c_o	Coefficient in Equation (4.34)	–
D	Diameter of circular pier; diameter of circular culvert	m
d	Representative particle size	m
d_{mm}	Mean size of sediment in mm	mm
d_n	Size of equivalent cube having same volume as particle or rock	m
d_{n50}	Size of equivalent cube having same volume as median particle size	m
d_s	Size of equivalent sphere having same volume as particle or rock	m
d_{65}	Particle size for which 65 per cent by weight of the sediment is smaller (similarly for other size fractions such as d_{50} and d_{90}); for gravels, size is measured on intermediate axis of particles	m
d_{50Z}	Size of gravel particle, measured on minor axis, for which 50 per cent by weight of the sediment is the sediment is smaller	m
d_*	Non-dimensional particle size defined by Equation (4.43)	–
E	Energy head	m
E_R	Average rate of bank retreat	m/year
F_c	Froude number for culvert defined by Equation (5.15)	–
F_d	Percentage by weight of sediment finer than particle size, d	%
F_r	Froude number	–
F_s	Shields entrainment function	–
f	Silt factor defined by Equation (4.6)	–
G	Horizontal length of structure measured along longitudinal axis	m
G_a	Horizontal length of abutment in direction of flow	m
g	Acceleration due to gravity	m/s^2
H	Horizontal width of structure measured normal to longitudinal axis	m

H_a	Horizontal width of abutment projecting into flow	m
I	Turbulence intensity	–
I_b	Turbulence intensity near the bed (at $y/y_o = 0.1$)	–
K_G	Factor in Equations (4.21) to (4.25) for effect of bank type on regime geometry of channel	–
K_I	Factor in Equation (5.5) for effect of bank slope on stability of riprap	–
K_S	Factor in Equations (5.4), (5.8) and (5.10) for effect of overall slope on stability of protection	–
K_T	Factor in Equations (5.4), (5.8) and (5.10) for effect of turbulence on stability of protection	–
K_Y	Factor in Equations (5.4), (5.8) and (5.10) for effect of flow depth on stability of protection	–
k_d	Factor in Equations (5.4), (5.8) and (5.10) for effect of longitudinal slope on stability of protection	–
k_l	Factor in Equations (5.4), (5.8) and (5.10) for effect of side slope on stability of protection	–
k_s	Effective roughness of sediment bed	m
L	Displacement distance of stone placed through flowing water	m
L_p	Minimum length of scour protection required downstream of culvert outlet	m
L_y	Design life of structure	years
L_{12}	Distance between section 1 (upstream) and section 2 (downstream) (measured along the direction of flow)	m
M	Coefficient determining required permeability of geotextile	–
N	Return period of a flood	years
n	Manning resistance coefficient of channel	–
O_{90}	Characteristic opening size in geotextile for which 90 per cent of openings are finer	m
P	Length of the wetted perimeter of channel	m
P_r	Probability of design event being exceeded	–
p	Porosity of stone filling within gabion	–
Q	Discharge in channel or from culvert	m^3/s
Q_{bf}	Value of Q for bank-full conditions	m^3/s
Q_D	Value of Q for design flood	m^3/s
Q_{DC}	Design flow rate through contracted section of river or channel	m^3/s
q	Flow rate per unit width of channel or structure	m^2/s
q_1	Value of q in section of channel contracted by spur dike	m^2/s
R	Hydraulic radius of flow (= A/P)	m
R_{e*}	Particle Reynolds number	–
r_c	Radius of curvature at the apex of channel bend	m
S	Average longitudinal energy gradient of the bed of the channel	–
S_e	Energy gradient along channel	–
S_F	Factor of safety for local scour	–
S_f	Factor of safety in Equation (5.5) for stability of riprap	–
s	Relative density of sediment (= ρ_s/ρ)	–
T	Thickness of concrete block	m
t	Thickness of layer of stone pitching	m
T_I	Turbulence intensity (= u_{rms}/u, at height $y = 0.1\ y_o$ above bed)	–
U	Local value of depth-averaged velocity	m/s
U_*	Shear velocity of flow	m/s

U_b	Bed velocity (at height $y = 0.1\ y_o$ above bed)	m/s
U_s	Local depth-averaged velocity at upstream end of structure	m/s
U_{TC}	Value of U at the threshold of movement for sediment at the bed	m/s
U_t	Value of U at the toe of bank	m/s
U_1	Depth-averaged flow velocity within horizontal two-dimensional jet	m/s
U_2	Depth-averaged velocity in flow downstream of scour hole	m/s
u	Local point velocity	m/s
u_{rms}	Root-mean-square fluctuation in local velocity	m/s
V	Mean flow velocity in channel (= Q/A)	m/s
W	Weight of stone (strictly, should be termed "mass" but "weight" is more common usage)	kg
W_{15}	Weight of stone such that 15 per cent by weight of sample is lighter (similarly for other weight fractions such as W_{50} and W_{100})	kg
W_{n50}	Characteristic median weight of stone	kg
Y_{bf}	Mean depth of channel	m
Y_{dike}	Depth from water surface to the point of lowest scour adjacent to spur dike	m
Y_{max}	Depth of flow at lowest point in invert of channel	m
Y_{rev}	Depth from water surface to the point of lowest scour adjacent to guide bank or revetment	m
Y_S	Maximum depth of local scour measured below the bed-level just upstream of the structure or below the invert level of apron or outlet channel	m
y	Height above the bed	m
y_o	Local water depth	m
y_p	Vertical distance from bed to underside of pile cap	m
y_T	Tailwater depth downstream of structure measured from unscoured bed level	m
y_1	Vertical thickness of two-dimensional jet	m
Z	Level (above datum)	m
Z_{bf}	Level of top of banks	m
Z_b	Average bed level	m
Z_{bend}	Lowest bed level in bend of channel	m
Z_{con}	Lowest bed level in zone immediately downstream of confluence of two rivers or channels	m
Z_D	Water level in design flood	m
Z_{inv}	Invert level of apron, culvert, outlet structure or downstream channel	m
Z_{LS}	Lowest level of local scour	m
Z_{min}	Lowest bed level in cross-section of channel	m
Z_o	Local bed level	m
α	Angle between the longitudinal axis of structure and the direction of the approaching flow	degrees
β	Width of meander belt	m
γ_1	Factor for effect of flow alignment on local scour at spur dikes	–
γ_2	Factor for effect of structure shape on local scour at spur dikes	–
γ_3	Factor for effect of channel location on local scour at spur dikes	–
δ	Angle between plunging jet and water surface	degrees
ε	Angle of bank to horizontal	degrees
ζ	Factor for effect of sediment size on depth of local scour	–
η	Confluence scour factor	–
θ	Angle formed by the junction of two rivers	degrees

κ	Factor for effect of sediment size on depth of local scour produced by 2-D horizontal jets and plunging jets	–
κ_g	Permeability of geotextile	m/s
κ_s	Permeability of soil	m/s
λ	Darcy-Weisbach friction factor of flow	–
λ_a	Axial wavelength of meander pattern	m
μ	Factor in Equations (5.4), (5.8) and (5.10) for effect of location on stability of protection	–
ν	Kinematic velocity of water	m^2/s
ξ	Bend shape factor	–
ρ	Density of water	kg/m^3
ρ_s	Dry density of sediment particles	kg/m^3
σ_1	Factor applied to regime depth for local scour at guide banks and revetments	–
σ_2	Factor applied to flow depth for local scour at guide banks and revetments	–
σ_3	Factor applied to regime depth for local scour at spur dikes	–
τ_c	Critical shear stress at threshold of sediment movement	N/m^2
Φ_{abut}	Factor for effect of abutment shape on depth of local scour	–
Φ_{angle}	Factor for effect of flow angle on depth of local scour at structure	–
Φ_{depth}	Factor for effect of water depth on depth of local scour at structure	–
Φ_{shape}	Factor for effect of structure shape on depth of local scour	–
$\Phi_{velocity}$	Factor for effect of flow velocity on depth of local scour	–
Φ_{abut}	Factor for effect of abutment shape on depth of local scour	–
ϕ	Angle of internal friction of sediment; angle of repose	degrees
χ	Angle of channel invert to horizontal	degrees
Ψ_{CR}	Stability parameter for movement of riprap or stone within gabions	–
ψ	Channel shape factor	–
Ω_{depth}	Factor for effect of water depth on depth of local scour at abutment	–

1 Introduction

Structures built in rivers and estuaries are prone to scour around their foundations. If the depth of scour becomes significant, the stability of the foundations may be endangered, with a consequent risk of the structure suffering damage or failure. There have been several cases of bridge failures – some causing loss of life and most resulting in significant transport disruption and economic loss – as a result of scour. Many of the well-known cases are mentioned in Section 2.4, together with some statistical data. Further information on some selected failures is included among the case studies in Appendix 2.

The factors influencing the development of scour are complex and differ according to the type of structure. Protection works for preventing scour need to be designed to withstand the flow forces imposed on them, and have to be practicable to build and install while minimising environmental effects.

1.1 DEFINITION OF SCOUR

Scour is the removal of material from the bed and banks of a channel by the action of water. Although it may be greatly affected by the presence of structures encroaching on the channel, scour is a natural phenomenon caused by the flow of water over an erodible boundary. In a river, scour is normally most pronounced when the bed and river banks consist of granular alluvial materials. It also occurs in cohesive materials, such as clay, and even deeply weathered rock can be vulnerable in some circumstances.

Scour may occur as a result of natural changes of flow in the channel, as part of longer-term morphological changes to the river, or as a result of man's activities, such as the building of structures in the channel or the dredging of material from the bed.

It is useful to classify scour into various types, but definitions in the literature vary widely, particularly in defining "general scour". For the present manual, the following broad definitions have been adopted and the term "general scour" is not used:

local scour — scour that results directly from the impact of individual structural elements (for example, piers and abutments) on the flow and occurs only in the immediate vicinity of those elements

contraction scour — scour affecting all or most of the channel bed in the vicinity of a bridge or other hydraulic structure, associated with higher velocities caused by narrowing of the channel

natural scour — includes all scour processes that are not covered by "local scour" or "contraction scour", typically including bed degradation and lateral channel movement.

Further information on these types of scour is given in Chapter 2; additional definitions appear in the Glossary.

1.2 OBJECTIVES AND SCOPE OF MANUAL

Although much laboratory research has been carried out on scour at particular types of structure, 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 often fill in again after the peak of a flood has passed. The consequent lack of reliable field data also makes it difficult to verify predictions of potential scour depths obtained from small-scale laboratory tests.

The principal objectives of this CIRIA manual are therefore to provide engineers with authoritative guidance on the causes of scour, how to quantify its effects and how to protect against it. To achieve this objective, it has been necessary to combine up-to-date technical information with the practical experience that has been gained by engineers in constructing and operating schemes in the field. The manual covers:

- different types of scour (local, contraction and natural)
- the scour characteristics of different types of structures (ranging from bridge piers to linear revetments)
- the effects of different types of flow conditions (such as uni-directional fluvial flows, bi-directional tidal flows, jet flows from sluices)
- the design of new structures, and the assessment and repair of existing structures
- permanent structures and temporary works
- similarities and differences between UK conditions and larger-scale problems overseas
- alternative protection systems and methods of installation
- assessment of scour risks
- costs and benefits of protection works
- environmental issues.

Although estuarine conditions are included, the manual is mainly concerned with the fluvial environment and, in particular, scour induced by currents. It does not cover marine scour nor scour predominantly by wave action, for which reference should be made to *Scour at marine structures* (Whitehouse, 1998).

The manual contains limited coverage of risk assessment and the costs and benefits of scour protection works, with the objective of identifying the key issues that need to be considered and suggesting general ways in which these factors should be assessed. However, it is not possible to give detailed methodologies and quantitative data on risks and costs. Risk assessment is a complex subject and many of the factors that affect scour cannot be determined accurately; risk assessments of different options are therefore likely to be relative. Similarly, estimates of the costs resulting from scour failures or the construction costs of protection schemes vary considerably from case to case, depending on the type, size and location of the structure.

In the UK, specific methodologies have been developed for Railtrack (HR Wallingford, 1993) and the Highways Agency (Binnie Black & Veatch, 1998) for assessing potential scour at existing bridges and other structures crossing rivers and other watercourses. These enable the structures to be screened in terms of whether scour is a significant factor and whether further study or additional protection works may be needed. The assessment procedures are not superseded by the present manual and they should continue to be used for Railtrack and Highways Agency schemes. The manual complements the procedures by providing information on a wider range of structures and topics, such as the design, construction and maintenance of protection schemes.

Scour downstream of dams and high-head energy dissipators is not covered by the manual. The flow conditions are very different in those cases and the scale of the problems usually requires them to be investigated on a case-by-case basis. Three aspects of scour assessment and protection are also outside the scope of this manual:

- the estimation of flood discharges
- details of methods for determining or predicting flow conditions at structures (for example, backwater calculations and flood-routeing analysis)
- the design of river training works.

Although these issues are described in general terms, readers are referred to other documents for detailed guidance on them.

It should be noted that current knowledge about scour is varies greatly. Scour at bridge piers has been the subject of considerable study, for example, whereas little published information is available about scour at pipelines and revetments.

Although the authors have endeavoured to make this manual as comprehensive as possible, some parts of the book inevitably contain more detailed and definitive information than others.

Table 1.1 *Summary structure and contents of manual*

Chapters	Contents
1	**Introduction** – Definition of scour, objectives and scope of manual, approach to studies, reader guide.
2	**Scour processes** – Types of scour, sediment types and behaviour, time effects, failure mechanisms.
3	**Inputs to the design process** – Guidance on the data required for scour assessments, methods of collection.
4	**Estimation of scour** – Methods for estimating amounts of natural scour, contraction scour and local scour at structures, assessment of existing structures.
5	**Scour protection** – Methods for reducing scour, design of structures to withstand scour, installation of materials that resist scour, design details, construction aspects, monitoring for scour.
6	**General issues –** Environmental factors, temporary works, risk assessment, cost and benefit analyses.
Appendices	**Contents**
1	Monitoring equipment
2	Case studies

Finally, it is important to stress that various factors interact to create scour problems at hydraulic structures. Solutions to these problems therefore require a multi-disciplinary approach. For major projects, inputs from hydrologists, geomorphologists, geotechnical engineers, foundation engineers, as well as from hydraulic engineers, need to be carefully integrated at all stages of the work from preliminary planning through to detailed design, construction and operation.

1.3 READER GUIDE

This manual is intended for readers with interests and responsibilities covering the design, construction, ownership, maintenance and assessment of bridges and other hydraulic structures. The document is also relevant to designers of non-wet structures on river floodplains, such as access tunnels through road embankments, that are normally dry but that may be at risk from scour from flows occurring during flood conditions. The manual contains six chapters and two appendices (see Table 1.1), a comprehensive glossary of terms and references.

1.3.1 Further reading

For those readers who wish to gain further knowledge in the subject of scour, the authors recommend the following publications (publication details are given in the References). There are some variations in terminology and definitions of which readers should be aware.

BREUSERS, H N C and RAUDKIVI, A J (1991)
Scouring

In particular Chapter 2, for its coverage of soil erosion and sediment transport.

HAMILL L (1999)
Bridge hydraulics

A recent book giving good coverage of hydraulic aspects of bridge design, including scour and flow forces

HOFFMANS, G J C M and VERHEIJ, H J (1997)
Scour manual

Gives wide coverage of the subject for a range of hydraulic structures, including the results of comprehensive evaluations of the methods against available data sets.

LAGASSE, P F, SCHALL, J D, JOHNSON, F, RICHARDSON, E V and CHANG, F (1995)
Stream stability at highway structures, 2nd edition

A comprehensive reference on stream stability problems and recommended practice in the USA.

LAGASSE, P F, RICHARDSON, E V, SCHALL, J D and PRICE, G R (1997)
Instrumentation for measuring scour at bridge piers and abutments

A detailed description of a recent comprehensive investigation into scour monitoring techniques, including recommendations on practicable systems (summarised in Appendix 1 of this manual).

MELVILLE, B W and COLEMAN, S E (2000)
Bridge scour

A new book giving comprehensive coverage of bridge scour calculation methods and factors affecting scour, together with many case studies drawn from experience in New Zealand.

NEILL, C R (editor) (1973)
Guide to bridge hydraulics

The “original” book on bridge hydraulics, which was seen by its editor as “tentative” and was hoped to “act as a challenge to research and to the development of more positive design methods”. Nevertheless, it remains widely used and respected.

RICHARDSON, E V and DAVIS, S R (1995)
Evaluating scour at bridges, 3rd edition (HEC18)

A comprehensive reference on scour evaluation methods and on recommended practice in the USA. There are several other relevant publications in the same series, as listed in the Bibliography.

VARMA, C V J, RAO, M K, and MATHUR, P (1989, 1994)
River behaviour, management and training, Volume I (1989) and Volume II (1994)

An updated and expanded version of an earlier manual by Joglekar (1971), this contains important field data and guidance on the design and construction of scour protection systems and training works for large rivers, with particular emphasis on Indian practice and experience.

2 Scour processes

The types of scour dealt with in this manual result from erosion of the channel boundaries by flowing water. The amount of scour that can occur at a structure placed in the flow, and the speed at which the scour develops, depend on:

- the position and type of structure
- the flow conditions affecting it
- the characteristics of the channel boundary materials in the vicinity of the structure and in the upstream reach.

These factors, together with further discussions on the classification of scour and the types of scour failure to which various structures are vulnerable, are the subject of this chapter.

Each of the factors that contribute to scour is subject to a significant degree of uncertainty or difficulty in making long-term predictions. Information available on major floods at the design stage may be limited and, during the life of a structure, the flow conditions may be altered by changes in catchment usage or climate. The responses of natural channels to erosion in short-term floods and over longer periods are hard to predict accurately, partly because of an incomplete understanding of the physical processes involved and partly because they interact in a complex way and are affected by random factors. Therefore, the risk of a particular depth of scour occurring and of it causing damage to a structure cannot be determined in the same way and to the same accuracy as structural design parameters, where loadings, material properties and responses are known more precisely. The question of risk assessment is discussed more fully in Section 6.3.

2.1 TYPES OF SCOUR

As noted in Chapter 1, scour may conveniently be classified according to the circumstances and structures that give rise to it. Definitions in the literature vary, so it is important to be clear in the definitions used, avoiding where possible those whose meaning may be ambiguous. This book classifies scour as either "local", "contraction" or "natural". It is more convenient to consider them in the reverse order, however, starting with the most widespread and moving to the most local. Marine scour, boat scour and scour resulting from high-head jets and energy dissipation are also briefly considered.

2.1.1 Natural scour

The types of scour which, in this manual, are grouped under the "natural" class comprise:

- degradation of the channel (and its converse, aggradation)
- lateral channel migration
- regime conditions (representing an equilibrium between degradation and aggradation)
- bend scour
- confluence scour.

These phenomena are associated mainly with the characteristics of the whole catchment and the overall form of the river, rather than the locality of the bridge or hydraulic structure being considered. In that sense they may be considered as "natural", although it is acknowledged that some of the processes can be influenced by man-induced changes affecting the catchment or the river.

In certain textbooks and references, some or all of these phenomena are called "general scour", but in others that term also includes contraction scour. To avoid potential confusion, therefore, it was, decided to avoid use of the term "general scour" in this manual.

Degradation, aggradation and regime conditions

Degradation and aggradation are the processes of long-term erosion and deposition of bed material in a river, perhaps over a decade or a century, that affect its longitudinal profile. They normally occur as a series of progressive steps, predominantly during floods, but exclude the more localised effects of scour during a particular flood event.

Degradation usually appears as a general lowering of bed levels along a reach of river, and is caused by the reach seeking to adjust its longitudinal gradient to match the requirements of the flows and sediment loads that it carries. If the sediment load entering the reach is lower than the actual transport capacity within the reach, degradation starts at the upstream end and works its way downstream, so as to reduce the overall longitudinal gradient. However, if the channel downstream of the reach in question has a greater sediment transport capacity, degradation starts at the downstream end of the reach and works its way upstream, leading to an overall increase in the longitudinal gradient. In the case of aggradation, the above causes and effects are reversed. Clearly, channel degradation is the more critical condition when considering scour at structures. From this description of the processes involved, it can be seen that, to obtain early warning of channel degradation problems at a structure, it is necessary to monitor long-term changes in the river over significant distances upstream and downstream.

In many rivers there is an approximate equilibrium or "regime" (with no continuing degradation or aggradation in the area of interest). However, the stable regime conditions to which the river has become adjusted may be disturbed by changes resulting from natural processes and/or human interference. These changes may include:

Catchment changes

- increased runoff and/or sediment supply from deforestation, increased urbanisation or land drainage
- increased or reduced river flows and/or sediment loads caused by the creation or removal of reservoirs, abstractions or inter-basin transfers

River channel changes

- natural morphological changes, such as meander progression and cut-offs
- flood protection schemes, including relief channels and channel enlargement
- gravel mining from the channel bed
- other channel changes, such as dredging, weed clearance and canalisation

The influence of other structures

- removal of a downstream "control", such as a weir or bridge, that previously inhibited degradation at the site of interest
- creation or removal of an upstream structure that affects the sediment supply.

The assessment of degradation and aggradation can be a specialist matter, but guidance for routine cases, in particular for evaluating the regime conditions for the design flood, is given in Chapter 3.

Further information is available in textbooks such as Pemberton and Lara (1984), Breusers and Raudkivi (1991), HEC18 (Richardson and Davis, 1995), HEC20 (Lagasse *et al*, 1995), and Hoffmans and Verheij (1997).

Channel migration

Channel migration may occur naturally or as a result of human activity, and may be associated with any of the causes that give rise to degradation and aggradation. Migration of the entire river channel as part of the process of meander progression, or movement of the deep-water channel within the same overall channel banks, can affect the scour exposure of a bridge or other structure whose foundations may have been fixed in relation to an earlier channel position. In some cases, migration may occur rapidly in response to a particular flood event, but in other cases it may be gradual.

In a braided channel, the channel positions are continuously changing. The deepest natural scour is likely to be associated with the confluence of two major channels, downstream of a bar or island in the channel.

Taking account of the potential for channel migration is an important part of the design or assessment of fluvial and estuarine structures. As a general rule, if there is potential for channel migration, the foundations should be designed or assessed on the basis of any credible shifts of the deep-water channel or channels. Alternatively, training works may be carried out to limit the possible movement of the deep-water channel.

Bend scour

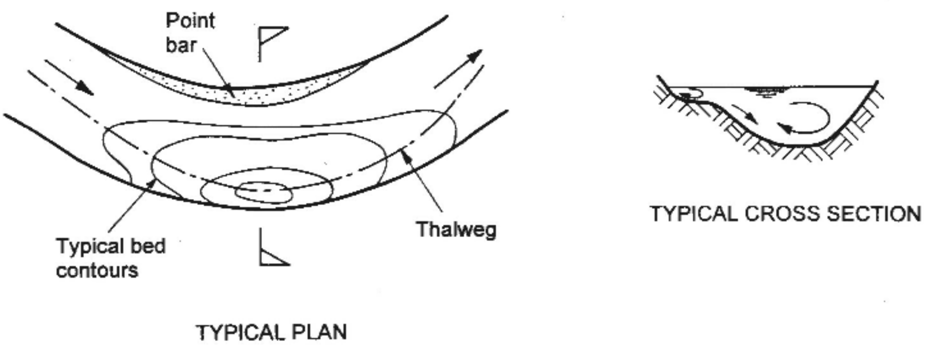


Figure 2.1 *Bend scour*

Scour at bends may be associated with meander progression, but the term "bend scour" is more narrowly defined, covering the additional bed scour due to the flow curvature. A particular feature of the flow curvature is the generation of secondary (spiral) currents, which tend to increase the scour towards the outside of the bend.

Bend scour, which is illustrated in Figure 2.1, is a complex process, depending upon:

- bend curvature
- width-to-depth ratio
- bank erodibility
- bed material grading and strata.

Confluence scour

Additional scour occurs at the confluence between two rivers or between channels in a braided river. Melville and Coleman (2000) describe the process as follows:

> Typically, the two streams of flow from converging channels meet at the centreline of the confluence, plunge to the channel bed, and then return to the water surface along the sides of the confluence.

They go on to point out the resemblance to the case of two bends placed back to back.

2.1.2 Contraction scour

Contraction scour (called "constriction scour" in some textbooks) is normally the result of confining the width of the river channel, for example between bridge abutments and piers. A major part of the contraction is often due to the approach embankments to a bridge, which cause the flows on the floodplain to join the main channel and pass through the bridge opening (Figure 2.2). Contraction scour – or an aggravation of previous contraction scour – may also occur from the removal or lowering of a downstream "control", or from an increase in discharges at the site.

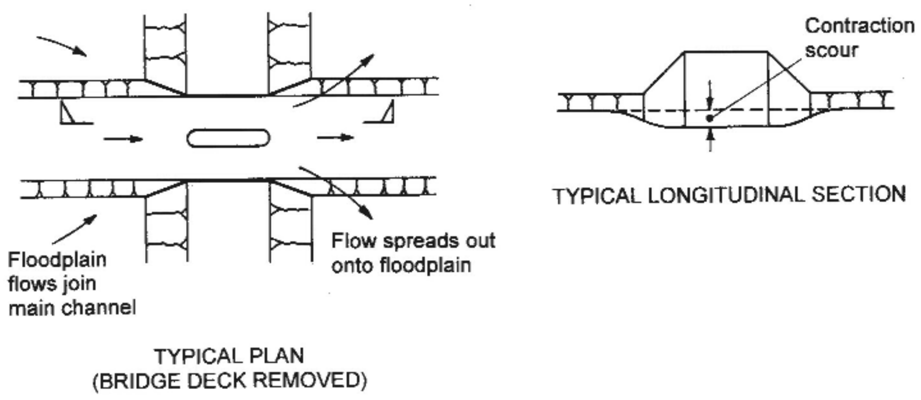


Figure 2.2 *Contraction scour at a bridge*

Contraction scour comprises the removal of material from all or most of the river bed width as a result of the increased velocities and shear stress on the channel bed. It can be estimated from considerations of the stability of the bed material in relation to the flow conditions. It is normal practice to assume that the amount of contraction scour varies across the channel width and distribution factors are available for a variety of situations.

Contraction scour may occur in both clear-water or live-bed conditions, as discussed in Section 2.3.1.

2.1.3 Local scour

Local scour is associated with particular local features that obstruct and deviate the flow, such as bridge piers, abutments and dykes, and occurs in their immediate locality. The structures increase the local flow velocities and turbulence levels and, depending on their shape, can give rise to vortices that exert increased erosive forces on the adjacent bed. As a result, the rates of sediment movement and erosion are locally enhanced around the structures, leading to local lowering of the bed relative to the general level of the channel.

Local scour has received wide attention from researchers, mainly because it is amenable to physical model testing, resulting in the availability of suitable estimation formulae for a variety of situations. Particular or unusual cases can also be model-tested as needed.

Formulae are available for predicting the equilibrium scour depth in clear-water and live-bed conditions and also for predicting the scour depth during the development phase, before equilibrium is reached. Information on local scour is available for the following features:

- bridge piers (of various shapes)
- bridge abutments and other similar structures
- river training works and linear revetments
- spur dikes (groynes) and other banks at various angles to the flow direction
- weirs and sills
- closures of cofferdams and diversion works.

In some cases, the extent, shape and surface gradients within the scour hole can also be predicted.

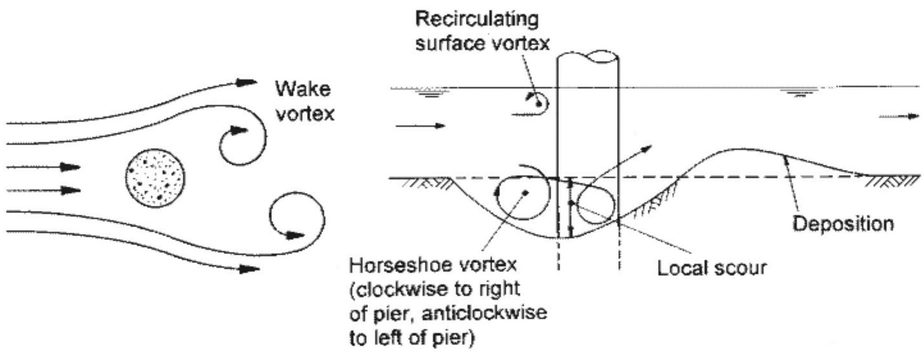


Figure 2.3 *Local scour and vortices at a circular pier*

In the case of structures such as bridge piers and cofferdams, part of the flow approaching the pier is deflected downwards to the bed and rolls up to create what is often described as a "horseshoe vortex" (Figure 2.3) around the front face of the structure; the vortex intensifies the local flow velocities and acts to erode sediment from the scour hole and transport it downstream. Vertical "wake" vortices caused by flow separation from the sides of the pier can also erode the bed, but, in fluvial conditions, the deepest scour tends to occur at the upstream face of the structure, as a result of the action of the horseshoe vortex. Material eroded from this hole is usually deposited towards the downstream end of the structure, to a level above that of the surrounding bed. The wake vortices are transported downstream by the flow and can create twin longitudinal scour holes; this type of scour may need to be considered if there is another structure farther downstream that is located within the wake created by the first structure.

As the scour develops, the increase in local flow depth decreases the strength of the erosive action at the bed; as a result, the rate of scour decreases and eventually reaches an equilibrium. In the case of clear-water scour (see Section 2.3.1), this occurs when the flow is no longer able to transport sediment particles out of the scour hole; for live-bed scour, equilibrium occurs when the rate at which sediment is eroded from the hole matches the rate at which it enters due to bedload transport over the upstream section of channel bed.

2.1.4 Total scour

The total scour depth associated with a particular structure is the sum of:

- any applicable natural scour (such as channel migration scour, degradation, confluence scour or bend scour)
- the contraction scour (if applicable)
- the local scour.

In this manual, each of these components of the total scour is evaluated separately in the sequence given above, with the local bed elevation resulting from each component being taken as the starting condition for the estimation of the next component (Figure 2.4).

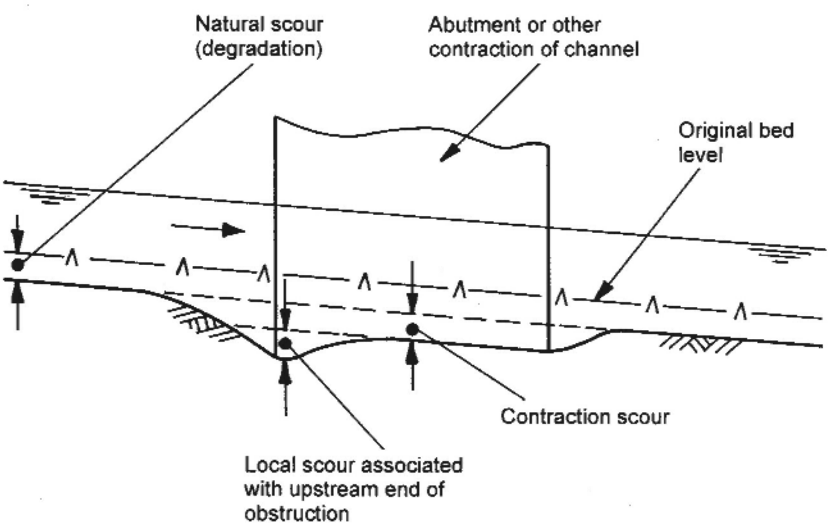


Figure 2.4 *Total scour*

2.1.5 Marine scour

In principle, scour in the marine environment is governed by the same processes that affect fluvial scour, and can be considered in the same categories of "natural", "contraction" and "local". There are, however, additional considerations arising from such issues as tidal flows, littoral drift, the interaction of tidal and fluvial currents in estuaries, and normally a greater exposure to wave action.

There are alternative up-to-date guidance documents on marine scour, in particular *Scour at marine structures* (Whitehouse, 1998) and Chapter 7 of *Scour manual* (Hoffmans and Verheij, 1997). The present manual does not cover marine scour, and is mainly concerned with the fluvial environment, in particular with scour induced predominantly by currents.

2.1.6 Boat scour

Scour may be increased by the effects of navigation, in terms of:

- the "blockage" or displacement effect of the vessel, causing increased local velocities around it
- the currents caused by the vessel's propeller
- the surface waves (wake) caused by the vessel (which usually have most effect on the banks)

The effects of navigation on the amounts of scour may be considered by adding the associated flow velocities to those which occur in the absence of boats. In some cases the effects are likely to be minimal, due to the short durations and small velocities in relation to the natural velocities. In canals, the passage of boats is likely to be the principal cause of any scour. Methods of calculating the currents and wave heights produced by boats moving in restricted waterways are given in PIANC *Supplement to Bulletin No 57* (1987).

2.1.7 High-head scour

Erosion of bedrock occurs in high-head situations, affecting massive hard igneous rocks as well as weaker sedimentary rocks. The primary mechanism for rock detachment in such situations is generally attributed to pressurisation of the joints. This is beyond the scope of the present manual, but further information can be found in various specialist technical papers (for example, Mason, 1993, and Whittaker and Schleiss, 1984).

2.2 SEDIMENT TYPES AND BEHAVIOUR

There is a general presumption for scour estimation that the bed material comprises alluvial deposits (sediments), or some other readily erodible material. The bed materials may be non-cohesive (typically gravels, sands and silts) or cohesive (typically, silts and clays).

Where the bed material is a competent unweathered bedrock, or is only slightly weathered, then it is essentially resistant to the forms of scour (excluding the action of high-head jets) that come within the scope of this manual. Such scour as does occur in these cases would be expected to be governed by the rate of weathering of the bedrock, rather than by the hydraulic conditions, so would not normally be significant within the design life of any bridge or other hydraulic structure.

In the cases covered by this manual, scour is therefore essentially a phenomenon concerned with the interaction between the hydraulic conditions and the mobility of readily erodible bed materials. In some cases, the quantification of the scour depth is virtually independent of the bed material: in other situations, the nature of the bed materials is an important factor and can be affected by issues such as armouring of the scour hole.

2.2.1 Non-cohesive sediments

Non-cohesive sediments are characterised by a granular structure, with individual particles being susceptible to erosion when the applied fluid forces (drag and lift) are sufficient to overcome the stabilising forces due to gravity and contact with adjacent bed particles.

A great deal of research has been carried out into the threshold of movement of uniform sediments in unidirectional flow, the best known being by Shields in Germany in the 1930s. (The Shields diagram, in which the threshold of movement is represented by a curved band plotted on a graph of the "entrainment function" against the "grain Reynolds number", remains the favoured format of subsequent researchers for presenting their findings. See Box 4.2.)

The threshold of movement depends on the particle size, density, shape, packing and orientation in the bed. The first two are normally the most important factors, as larger and denser particles are able to resist higher flow velocities. In practical situations, however, the particles are not uniform, the bed has undulations that affect the local hydraulic resistance, and the local velocities and stresses vary spatially and temporally, so that the initiation of movement occurs intermittently and randomly.

Threshold of movement calculations are a key feature of various components of scour estimation, in particular for regime conditions and for contraction scour.

2.2.2 Cohesive sediments

Most fine-grained sediments possess some cohesion, the clay content being of great importance. Raudkivi (Breusers and Raudkivi, 1991) states that a clay content of only 10 per cent is sufficient to "assume complete control" and notes that submerged fine-grained sediments can become cohesive as a result of biological action, such as the growth of algae.

Cohesive sediments typically require relatively large forces to detach the particles and initiate movement, but relatively small forces to transport the particles away. Hoffmans and Verheij (1997) describe the process as follows:

> Experiments...have shown that the scour of clay soils with a natural structure...occurs in several stages. In the initial stage loosened particles and aggregates separate and those with weakened bonds are washed away. This process leads to the development of a rougher surface. Higher pulsating drag and lift forces increase the vibration and dynamic action on the protruding aggregates. As a result the bonds between aggregates are gradually destroyed until the aggregate is instantaneously torn out of the surface and carried away by the flow.

There has been some recent research relating competent velocities to the soil cohesion and other parameters, but the erosion properties of cohesive bed materials are not yet fully understood, particularly in relation to local scour around structures. As an interim measure, an approximate method for estimating local scour in cohesive materials is proposed in Section 4.3.1.

2.2.3 Sediment strata and layering

If the bed materials are stratified, then the layers need to be considered one by one, working downwards, to establish whether the scour is likely to terminate in that layer or progress through to the next. In cases where a more resistant layer overlies a less resistant layer, then a conservative approach needs to be taken regarding the risks of the scour breaking through the more resistant layer into the less resistant layer.

2.2.4 Armouring

Armouring is a process whereby a widely graded channel bed may be preferentially eroded, with the small particles being washed away and the larger particles progressively settling down to a lower elevation until they form the entire surface of the bed and are resistant to further erosion under the prevailing flow conditions. Reorientation of lens-shaped or elongated stones can also be a feature of river bed armouring.

Armouring can be relevant in the consideration of all forms of scour in gravel and cobble bed rivers, but is most likely to be an issue when evaluating regime conditions, degradation and contraction scour. The general approach is to consider the scour in successive depth bands, estimating the particle sizes and corresponding volumes of material that are liable to be washed away in each layer, then revising the grading of the next layer to reflect the addition of the uneroded material from above. This differential scouring process continues until there is a complete layer of material large enough to resist further erosion.

If a major flood produces more severe flow conditions than those under which the armouring developed, there is a risk that the armour layer could be washed away. This would leave the underlying, non-armoured material exposed and could result in a rapid increase in the depth of scour. Similar problems can occur if sections of channel bed are dredged, or reinstated with excavated material. For these reasons, a cautious approach should be adopted when estimating the possible effects of bed armouring on scour depths, especially in cases of clear-water scour, when there is no supply of potential armour material from upstream.

2.3 TIME EFFECTS IN SCOUR PROCESSES

2.3.1 Clear-water and live-bed scour

Scour at a particular location may occur in relatively low flows, before there is general movement of sediment on the river bed. This initial scouring is known as “clear-water” scour. As the flow increases, so this clear-water scour increases and may, in many granular bed materials, reach a significant depth before there is general sediment movement in the river. Once the flow has increased to the point at which general bed movement occurs, there is then a supply of sediment from upstream to offset the local removal of material. Scouring under these conditions is referred to as “live-bed” scour. When the flow reduces, clear-water scour may occur again.

Both contraction scour and local scour may occur under clear-water and live-bed conditions. The water in clear-water scour may not literally be clear, as it may contain finer sediments that remain wholly in suspension and do not affect the scouring processes at the bed.

At the transition from clear-water scour to live-bed scour, the supply of material from upstream may exceed the rate of local erosion, resulting in a reduction in the depth of the contraction or local scour. In many cases, there can thus be an initial maximum scour depth at about the point when general movement of the river bed begins, followed by a reduction in scour depth as the flow increases and more material is supplied from upstream. With uniform granular materials, this initial clear-water depth of scour may be reached at moderate flow rates and may not be exceeded even under severe flood conditions. In the case of graded sediments, however, the scour depth at the limit of clear-water flow is normally less than it would be with a uniform material; increases in velocity that produce live-bed scour are likely to result in greater depths of erosion, but the maximum value is normally no greater than the maximum occurring with a uniformly graded sediment.

In some circumstances, for example in channels with coarse or armoured bed material or in vegetated floodplains, or in moderate floods, only clear-water scour occurs. Clear-water scour approaches the maximum depth asymptotically, generally over a longer period than live-bed scour, and it may take several floods before the maximum is reached. Live-bed scour can reach its maximum much more rapidly and, in cases where the upstream channel contains dunes, the depth tends to fluctuate about a mean (equilibrium) value.

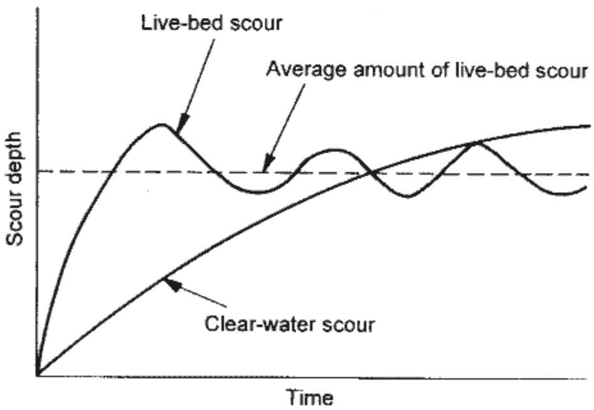


Figure 2.5 *Comparison of development of clear-water and live-bed scour*

Figure 2.5 compares the development of clear-water and live-bed scour. Richardson and Davis (1995) state that the maximum local pier scour under clear-water conditions is about 10 per cent greater than the equilibrium (mean) scour depth under live-bed conditions. They also quote a typical fluctuation of live-bed scour in sand rivers of about 10 per cent either side of the equilibrium value, indicating that the maximum values of clear-water and live-bed scour are about equal.

2.3.2 Transient scour

Observations of scour holes made after flood events can be misleading. Maximum scour depths are likely to occur near the peak of a flood, and during the recession stages the holes may be partially refilled. This phenomenon, which has been observed from the strata exposed in trial pits in scour holes, is principally a feature of local, contraction, bend and confluence scour in live-bed conditions, where bed material is transported from upstream. However, it can also occur over time following clear-water scour, due to the deposition of finer material or slumping of the scour slopes.

It is important to ensure that, where field measurements are used as part of the scour assessment, the basis of the field measurements is properly documented and understood. Equipment is available that allows either continuous monitoring of scour at structures, or that records the maximum scour reached during an event, but there are practical problems that can affect its successful deployment. Further information is given in Appendix 1.

2.3.3 Scour hole evolution

For local scour holes, four phases of evolution have been identified in model tests of clear-water scour (Hoffmans and Verheij, 1997):

- initiation
- development
- stabilisation
- equilibrium.

During the initiation phase erosion is rapid, and material eroded from the upstream slope of the scour hole goes into suspension. In the development phase, the scour hole deepens substantially. The upper part of the upstream slope is in approximate equilibrium and development occurs beyond it, enlarging the hole while its shape remains approximately constant. The development of the hole results in a progressive reduction in near-bed velocities and the rate of erosion.

During the stabilisation phase, the rate of erosion at the base of the hole is very small, but erosion continues at the more exposed position near the top of the downstream slope, resulting in the scour hole lengthening and equilibrium being nearly achieved. In the equilibrium phase, the dimensions of the scour hole are virtually fixed.

In the case of structures such as piers and piles, scour usually first develops along either side of the structure and is deepest at the points of maximum width where the flow velocities are highest. As the holes deepen, they increase in extent and (unless the structure is extremely wide) join to form a single scour hole around the upstream end of the structure. This hole then continues to deepen towards the equilibrium shape, with the maximum depth normally occurring on the centreline of the upstream face (unless the flow is not well aligned with the axis of the structure).

2.3.4 Tidal effects

In tidal situations, scour may be developed principally as a result of the tidal flows, which occur to some degree almost all the time, in contrast to fluvial flood flows, which are generally limited to short durations, particularly in the case of rare events.

All forms of scour may be subject to tidal effects, with the potential for scour to evolve in steps in response to the tidal flows. In some cases, such as contraction scour, the scour is likely to progress in a similar manner in either flow direction; for local scour around a bridge pier, the equilibrium shape of the scour hole differs for each flow direction; in other cases, such as scour downstream of a sill or apron, scour is likely to occur in different locations on the flood and ebb tides. Consideration needs to be given to the following factors in evaluating scour in tidal locations:

- the range of tidal amplitude between neaps and springs
- the peak tidal velocities and the relationship between flood and ebb velocities
- the peak velocities in fluvial floods and storm surges and their superimposition on the tidal velocities
- the different locations and/or shapes of scour likely under different flow directions
- the cumulative effect of a series of tides
- the potential for scour development in a single spring tide and/or fluvial flood.

2.4 FAILURES DUE TO HYDRAULIC ACTION

This section reviews the types of failures that may occur to structures as a result of hydraulic action. Scour is a potential contributory factor in all cases, and is believed to be the most common cause of failure or damage to bridges during floods, although the immediate cause may be another factor, such as water pressures on a bridge deck, or debris impact forces. Examples and statistics of failures are presented next, with information on failure mechanisms associated with scour and other hydraulic factors being given in the following sections. The discussion is based mainly on bridges, but the same considerations generally apply to other structures.

Richardson and Davis (1995) cite several studies of bridge failures in the USA, some of which are summarised below.

- A 1973 national study of 383 bridge failures caused by floods showed that 25 per cent involved pier damage and 72 per cent involved abutment damage.
- A more extensive national study in 1978 indicated that local pier scour and abutment scour were about equal in occurrence.
- The 1993 flood in the upper Mississippi basin caused 23 bridge failures, of which 14 were from abutment scour, two from pier scour, three from a combination of abutment and pier scour, two from lateral migration, one from debris load and one unattributed.

In a report for the Transport and Road Research Laboratory, HR Wallingford (1991) listed the following notable bridge failures due to scour:

- Schoharie Creek bridge, New York State, USA (1987): 10 killed (see Appendix 2)
- Glanrhyd railway bridge, Wales (1987): train ran off partially collapsed bridge, four killed (see Appendix 2)
- Wraysbury railway bridge, England (1988): train derailed (see Appendix 2)
- River Ness railway viaduct, Scotland (1989): failed during flood
- Hatchie River bridge, Tennessee, USA (1989): three spans collapsed, eight fatalities (see Appendix 2)
- Okuragawa road bridge, Japan (1989): collapsed, 10 killed
- Kufstein motorway bridge, Austria (1990): pier sank and rotated several metres due to scour
- Ireland (1986): failure of or damage to 18 small bridges in Wicklow, Leitrim and Waterford, mostly due to scour.

Melville and Coleman (2000) cite 31 case studies of bridge failures and damage in New Zealand due to scour since the 1930s and state that "at least one serious bridge failure each year (on average) can be attributed to scour of the bridge foundations". The 31 case studies are related to the following primary causes:

• pier failure	13
• erosion of approach or abutment	8
• degradation	4
• debris or aggradation	6

Examples of the cases cited by Melville and Coleman (2000) are:

- Bull's Road bridge failure (1973), due to scour reaching almost to the bottom of the pier support piles and to oblique flow against the pier, resulting in hinging between the pier and the piles and collapse of the bridge deck (see Appendix 2).

- Waitangitaona River road bridge failure (1982), in which one pier and the two adjacent deck spans were lost due to scour, debris accumulation and oblique flow against the pier.
- Waipaoa River rail bridge failure (Cyclone Bola, 1988), in which the abutment and three adjacent piers were affected by scour, the abutment being outflanked and then undermined by bend migration.
- Oreti River road bridge, affected by degradation (aggravated by gravel extraction upstream) since construction in 1955 and requiring a downstream rock weir to stabilise the bed (see Appendix 2).
- Bullock Creek road bridge subject to aggradation caused by upstream landslides, reaching 1 m above the deck during a flood in 1983, after which the bridge was removed and replaced at a new site.

A recent UK study (J Benn, *personal communication*), using library searches and questionnaire surveys of bridge owners, identified 104 bridge failures or partial failures due to scour and floods since 1880, of which 54 could reasonably be assigned to a scour failure and 11 to water pressure. Of the 54, it is notable that 31 were due to undermining or slippage of the approach embankment, rather than scour failure of the actual bridge piers or abutment. One single event in August 1948 accounted for the loss of 11 railway bridges between Berwick and Dunbar. A notable finding of the study was that failure of normally dry access bridges and cattle creeps was more common than failure of bridges over watercourses.

2.4.1 Scour failures

Most rivers have beds and banks of potentially mobile material, which may be subject to erosion in floods, leading to a fall in bed level or retreat of a bank. Scour in the vicinity of structures may be due to the combined effects of natural, contraction and local scour, and may also be increased by the effects of navigation.

A bridge constructed on spread footings is at risk of scour failure when the scour reaches the level of the base of the footing. However, it could be at risk with less scour if the substructure is subject to lateral ground pressures and water forces .

Scour adjacent to piled foundations may result in a loss of skin friction and load-bearing capacity. The piles may also be subject to unplanned bending stresses, from lateral loads and hydrodynamic forces.

Local scour at a bridge pier is normally greatest near the upstream nose of the pier, which may lead to the pier being undermined first at the upstream end and thus tilting. Similar effects may occur at abutments or with groups of piles. Variations in bed and flow conditions may also lead to tilting in other directions.

Climate change, if it produces larger peak discharges or changes other relevant factors such as the typical annual hydrograph, could result in increased risks of scour failures in the future.

2.4.2 Bank erosion and channel migration

Many rivers tend to change their course with time. A bridge or other structure that is sited to suit one location of the main channel may become progressively vulnerable to scour failure as the river attempts to migrate. Abutments or piers located on the original floodplain, if not designed to accommodate channel migration, may be undermined or otherwise destabilised if this occurs.

Changes in channel alignment can also be caused by poorly designed training works or by the uncoordinated construction of other structures upstream; examples of the latter include jetties, and rock weirs or groynes built to provide improved conditions for fish and other fauna. Maintenance repairs involving the placing of rock protection around bridge piers can reduce the flow area of the main span and lead to flow being diverted towards other channels or openings.

2.4.3 Hydraulic forces on piers

Water flowing past a pier or abutment exerts forces that increase markedly as the velocity increases (by about the square of the velocity). In an ideal situation, the forces are in line with the axis of the channel and piers, but in some cases channel movements and other factors may affect the flow direction and the resulting forces to a degree not anticipated in the design. The ability of the structure to withstand the hydraulic forces depends on the foundation design and may be compromised by the scour which occurs. Debris accumulation may also contribute.

2.4.4 Hydraulic forces on bridge deck or arch

If the water level reaches the soffit level of the bridge deck, or gets much above the springing level of an arch, the flow starts to exert much greater forces on the bridge structure. Such forces can be dangerous because of the overturning moment applied to the foundation, the risk of lifting a simply supported bridge deck off its supports, or the risk of reducing the compression forces around an arch. The risks of failure would be greater if aggravated by scour, particularly if the scour happens to be greater on the downstream side of the bridge.

2.4.5 Debris

The accumulation of debris against bridges and other structures can significantly affect the hydraulic behaviour, the amount of scour and risks of failure. For small single-span bridges, blockage by up to 90 per cent of the bridge opening has been known; this can lead to large rises in upstream water levels, flooding and overtopping. Accumulation of debris against a bridge structure can increase the amount of scour due to:

- the increased effective width of the pier (which is a significant factor in the amount of scour)
- the increased velocities resulting from the flow constriction and the rise in upstream head.

Debris can also be a contributory factor in bridge failure due to:

- increased drag and hydrodynamic forces
- impact forces resulting from the debris colliding with the piers and/or deck.

2.4.6 Ice

Ice can impose forces against structures due to its expansion during freezing, but this appears unlikely in the fluvial environment. Probably the greater risk comes from the impact of sheets of ice carried in the flow after the ice starts to thaw and break up.

2.4.7 Failure mechanisms

Table 2.1 lists potential failure mechanisms for hydraulic structures subject to hydraulic loading, most of which are directly or indirectly related to scour and all of which could be aggravated by scour.

Table 2.1 *Failure mechanisms for hydraulic structures*

General group	Possible mechanisms
Primary structural movement or failure	• pier settlement due to loss of support to foundation • pier tilting, or tilting of a group of piles • abutment settlement and/or tilting • piers, abutments or footings damaged by hydraulic loading, perhaps aggravated by debris accumulation • piers, abutments or footings damaged by collision, sediment abrasion or impact from boulders • superstructure/deck sliding off supports due to hydraulic/debris loading and/or collision • superstructure/deck damaged by collision of debris or vessel • scour hole or washout of embankment behind abutment
Secondary structural movement or failure	• structural damage to superstructure/deck caused by twisting from differential settlement of piers and/or abutments • superstructure/deck falling off abutment or pier due to adverse tilt of support, increasing gap between supports • superstructure/deck buckling or riding up over support due to reduced gap between supports • superstructure/deck sliding off supports, due to tilting of supports • collapse of highway into embankment scour hole or washout

3 Inputs to the design process

The following sections give details of the sources and types of data needed for estimating potential depths of scour at structures (Chapter 4) and for designing associated scour protection works (Chapter 5).

3.1 LEVEL DATUMS

As explained in Section 2.1, the overall depth of scour at a structure can be considered as being made up of three components: natural scour, contraction scour and local scour. It is extremely important when calculating the three components to adopt a consistent system for expressing bed levels and scour depths. Although this may seem a straightforward issue, confusion can potentially arise because some calculation methods determine the scour depth as the distance from the "initial" bed level to the bottom of the scour hole, while others give the overall depth from the water surface to the bottom of the scour hole. The "initial" bed level depends upon which element of the scour calculations is being considered. As an example, in the case of contraction scour, it would be the bed level after allowance has been made for the effects of natural scour. In a major flood this is likely to be below the long-term (or regime) value. Similarly, the level of the water surface varies according to the particular flow conditions; therefore, if scour depths are being considered for two possible flood events, the datums for determining the two values of scour depth would differ.

For these reasons, in this manual, the final results of scour calculations are expressed in terms of absolute levels (relative to an appropriate datum) and not in terms of water depths or depths below initial bed level.

3.2 DISCHARGE

In cases of fluvial flow, it is normally necessary to determine values of discharge in the river or channel for floods with different probabilities of occurrence or return period. The return period, N, of a flood is most conveniently defined as the average time interval between years in which a given value of discharge is reached or exceeded. The reciprocal of the return period therefore represents the probability of that discharge being equalled or exceeded in any year.

Detailed methods of determining values of flood discharge are outside the scope of this manual, but the following techniques can be used.

1 Direct measurements at the site using current meters or other flow gauging techniques. It is unlikely that the data would have been obtained in flow conditions as severe as the specified design flood so the results usually need to be extrapolated. Guidance on how to take stream flow measurements and process the results is given in Parts 3 and 3F of British Standard BS 3680 (1986).

2 Analysis of historical records from flow gauging stations upstream or downstream of the site. The period for which the records are available is often considerably less than the return period of the design flood, so statistical methods must be used to estimate flow rates for rarer events. Information on these techniques is given in the *Flood estimation handbook* (Institute of Hydrology, 1999) and textbooks on hydrology (eg Wilson, 1990). If the length of the reach of river between the gauging station and the site is significant, or it contains a tributary (or distributary), the values of discharge need to be suitably adjusted. An approximate estimate can be obtained by multiplying the discharge at the gauging station by the ratio between the catchment area at the site and that at the gauging station. However, for greater accuracy, it is necessary to carry out flood-routeing calculations (using one-dimensional or two-dimensional numerical models) to allow for inflows from the catchment and for the attenuation effects that reduce the magnitudes of flood peaks as they travel downstream.

3 Statistical methods based on correlations between flood discharge and catchment characteristics such as area, slope, soil type etc. These methods usually only give values of discharge and not information about the duration and shape of the flood hydrographs. In the UK, estimates can be obtained using the information given in the *Flood estimation handbook*.

4 Rainfall/runoff methods using statistical data on rainfall and information on the catchment characteristics. This information is used to predict the unit hydrograph for the catchment (ie its response to a unit amount of rainfall) and is then combined with the rainfall data to determine the shape of the flood hydrograph and the value of peak discharge. Information on suitable rainfall/runoff models for the UK are given in the *Flood estimation handbook*.

5 Comparison with available data on flood discharges for adjacent catchments or those with similar characteristics.

6 Calculation of flow rates in the river from survey data defining the cross-sectional shape and longitudinal gradient of the channel and adjacent floodplains (where appropriate) – see Boxes 3.1 and 3.2 for more details.

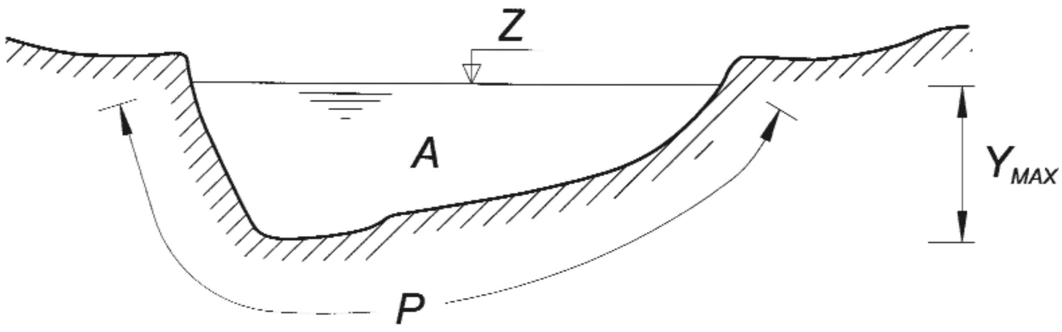
For major schemes where suitable records from flow gauging stations are not available, it is recommended to use more than one of the alternative methods to check that the estimates obtained are consistent and realistic.

In the case of structures subject to tidal flows in estuaries, the design discharge can be made up of three possible components:

- a fluvial flow corresponding to the design flood conditions in the upstream catchment
- a tidal component produced by astronomical tides occurring approximately twice a day, with the tidal range and the maximum tidal velocity varying in complex spring and neap cycles over periods of months and years
- a surge component caused by low atmospheric pressure or onshore winds raising sea levels and pushing additional water inland.

The relative importance of these components depends on the size of the river, the magnitude of the tidal range and the relative distance of the site from the upstream tidal limit. In the UK, the tidal component is likely to be dominant if the width of the river or estuary is greater than about 500 m at the site position. It can be noted that the fluvial flow increases the velocities occurring during the ebb portion of a tide and correspondingly decreases the velocities during the incoming (flood) tide. On a large lowland river, the duration of the main flood peak is likely to cover both the ebb and flood portions of a tide.

Box 3.1 *Calculation of discharge in rivers and channels*



A well-established method of calculating the discharge, Q (in m³/s), in a channel is through application of the Manning equation for flow resistance:

$$Q = \frac{1}{n}\frac{A^{5/3}}{P^{2/3}} S_e^{1/2} \quad \textbf{(3.1)}$$

where (as illustrated above) A is the cross-sectional area of flow (m²) and P is the length (m) of the wetted perimeter of the channel corresponding to a given water level Z (m above datum), at which the depth of flow to the lowest point in the invert of the channel is Y_{max} (m). The hydraulic resistance of the channel is defined in terms of the Manning coefficient n.

S_e is the energy gradient along the channel between two cross-sections 1 and 2 given by:

$$S_e = \frac{E_1 - E_2}{L_{1,2}} \quad \textbf{(3.2)}$$

where $L_{1,2}$ is the distance (m) between the two sections (measured along the direction of flow) and with Section 1 upstream of Section 2. The energy head E (m) is defined as:

$$E = Z + \frac{V^2}{2g} \quad \textbf{(3.3)}$$

where g is the acceleration due to gravity (= 9.81 m²/s) and V (in m/s) is the mean flow velocity in the channel, averaged over the cross-sectional area of the flow, so that:

$$V = \frac{Q}{A} \quad \textbf{(3.4)}$$

If the flow conditions and the channel geometry do not vary very much between the two sections, an approximate estimate of the discharge may be obtained by assuming that the energy gradient, S_e, is equal to the average longitudinal gradient, S, of the bed of the channel. However, controls such as weirs and channel constrictions downstream of the site can cause backwater effects that prevent S_e being even approximately equal to S. In these cases, backwater calculations may be carried out using a suitable one-dimensional numerical model to determine the flow conditions at the site more accurately.

Photographs of different types of channel with their corresponding values of n are given by Chow (1959) and, for UK rivers, by Hollinrake and Samuels (1995). If direct information is not available about the resistance characteristics of a river, an estimate can be made using the following formulae given by Samuels *et al* (2000):

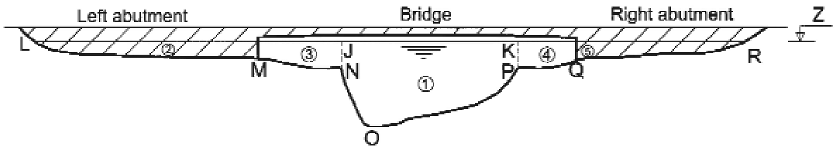
$$n = 0.183S^{0.22}, \text{ provided } R > 1 \text{ m} \quad \textbf{(3.5)}$$

where R is the hydraulic radius of the flow defined by:

$$R = \frac{A}{P} \quad \textbf{(3.6)}$$

For grassed floodplains without major obstructions, values of resistance can be of the order of n = 0.05.

Box 3.2 *Discharge in compound channels*



For cases where out-of-bank flows occur during flood conditions, it may be necessary to determine how the total discharge in the river is divided between the main deep-water channel and the floodplains on either side. Also, in certain cases of contraction scour at bridges, the amount of flow collected by embankments and channelled through the main opening may need to be calculated. The combination of a deep channel with one or more shallower channels having different values of Manning resistance coefficient, *n* (see Box 3.1), is termed a compound channel.

The following is an approximate method for calculating flow rates in the type of compound channel shown above. The channel cross-section is viewed from upstream and for illustrative purposes shows a bridge with abutments set back from the main channel on either floodplain. Channel 1 is the main deep-water channel (termed the "incised" channel) and the bridge opening spans channels 3, 1 and 4. Embankments on parts of the floodplain cause the flows in channels 2 and 5 to be diverted through the main opening of the bridge.

The Manning resistance coefficient of the incised channel is n_c and that of the floodplains is n_f. It is assumed that the level of the water surface, *Z* (m above datum), is the same across the full width of the river and that, upstream of the bridge, the floodplain flows are parallel with that in the main channel, that is, each channel has the same value of energy gradient, S_e. As explained in Box 3.1, if the flow conditions along the reach of river are fairly uniform, S_e may be assumed to be approximately equal to the longitudinal gradient, *S*, of the main channel invert. Applying Manning's equation to each channel separately and summing the flows gives the total discharge in the river as:

$$Q = Q_1 + Q_2 + Q_3 + Q_4 + Q_5 \qquad \textbf{(3.7)}$$

where for the main channel:

$$Q_1 = \frac{1}{n_c}\frac{A_1^{5/3}}{P_1^{2/3}} S_e^{1/2} \qquad \textbf{(3.8)}$$

in which A_1 is the flow area contained by the main channel up to the water level, *Z*. The perimeter P_1 is the distance corresponding to the line J–N–O–P–K; the inclusion of the lengths J–N and P–K as part of the boundaries of the channel is to allow for the drag resistance exerted on the flow in the main channel by the slower moving flows on the floodplains.

The flow in each of the floodplain channels is given by the general formula:

$$Q_m = \frac{1}{n_f}\frac{A_m^{5/3}}{P_m^{2/3}} S_e^{1/2} \quad \text{for } m = 2 \text{ to } 5 \qquad \textbf{(3.9)}$$

A_m is the flow area within a channel up to the water level, Z. The wetted perimeter, P_m, is the length of the floodplain boundary within a channel: thus, for channel 2, P_2 is equal to the length of the line L–M, and for channel 3, P_3 is equal to the length M–N (note that the line J–N is not included for the floodplain channel).

Using equations (3.7) to (3.9) it is therefore possible to calculate the total discharge in the river and the division of flow between the various parts of the compound channel. The problem is more complex in a meandering river, because the path length followed by the flow in the main channel is greater than that on the floodplains (so the values of energy gradient, S_e, are not necessarily equal). However, the meanders also tend to increase the energy losses on the floodplains. For this reason, it is normally recommended to use the same value of S_e for all the component channels and to calculate it using the path length measured along the main deep-water channel.

For a major scheme subject to significant tidal flows, it is usually necessary to carry out a detailed study to quantify the three components of tidal discharge and consider the joint probabilities of their occurring simultaneously. As an example, unnecessarily severe design conditions might be specified if they were to be based on a 100-year fluvial flood occurring at the same time as the highest astronomical tide and a maximum storm surge, since the return period of this combined event would be very much greater than 100 years. If scour is a significant factor in the design of a tidal scheme, the cost of the required study should be small in relation to the benefits resulting from a more realistic design specification.

3.3 CROSS-SECTIONAL AND PLAN GEOMETRY

To evaluate the potential for scour at hydraulic structures in rivers and other channels, it is necessary to have detailed and up-to-date information on the cross-sectional shape and plan geometry of the channels, and also of the floodplains if out-of-bank flows are expected to be significant. Possible sources of information include:

- a new topographic and bathymetric survey specially commissioned for the project
- data from previous surveys, if available
- large-scale survey maps
- aerial photographs
- satellite images.

For most major projects it is usually necessary to commission a new survey to obtain the specific information needed. However, comparison with older data can be important in identifying longer-term changes and trends in the position and shape of the main channels, particularly for meandering or braided rivers. The amount of survey information needed depends to a certain extent on the nature of the engineering works but, as a rough guide, cross-sections of the main channel should normally be obtained at intervals of about 50 m in the immediate vicinity of the works. For the floodplains, a larger spacing of about 100–150 m might be appropriate if their geometry is fairly regular. Contour surveys of the banks will also be necessary in the vicinity of the structure if it is necessary to design protection works.

For some projects, it may be necessary to carry out numerical backwater calculations to establish design water levels at the site (see Section 3.4). The spacings needed between cross-sections farther away from the site should be selected depending on the slope of the river. Some recommended values are given in Table 3.1.

Table 3.1 *Recommended intervals for survey cross-sections*

Channel gradient	Recommended interval (m)
1/300 to 1/1000	75 to 100
1/1000 to 1/3000	200
1/3000 to 1/10 000	500
Less than 1/10 000	1000

These figures are for relatively uniform channels and additional cross-sections should be added to pick up any special features or sudden changes. The distance over which the survey data is needed depends on the identification of a suitable point for the start of the backwater calculations (eg a gauging station, weir or similar control point). If the river is steep enough to produce supercritical flow in the channel (see Box 3.4), the survey data should be obtained in the upstream direction because the water levels at the site are then determined by the upstream flow conditions.

3.4 WATER LEVELS AND FLOW DEPTHS

Methods of obtaining information on design water levels at a site include the following.

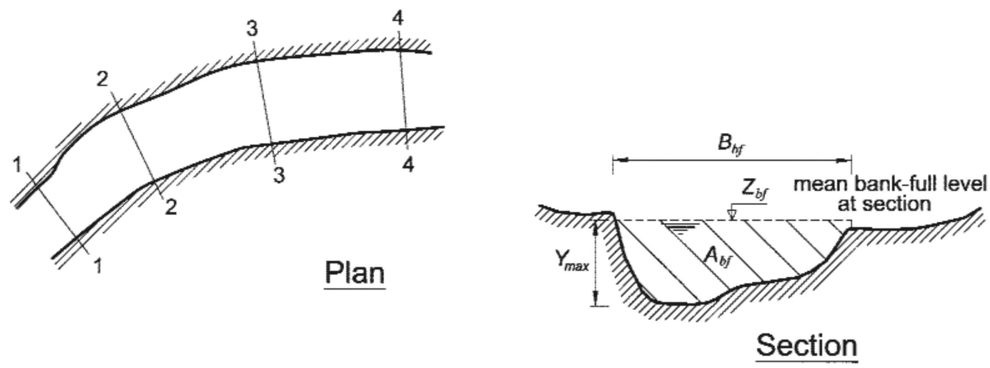
1. Analysis of historical data on water levels, if these are available from readings at gauge boards or gauging stations located nearby.
2. Use of tide tables, if the site is located in an estuary or coastal area.
3. Hydraulic calculations using estimates of flood discharge in conjunction with maps and survey data defining the cross-sectional geometry and longitudinal gradient of the river.
4. Anecdotal/informal information provided by local people on past floods (eg levelling of points such as trash marks defining the extent of a flood or heights reached on walls of buildings).

In all these cases, it is very important to check and compare the datums used for any information obtained on water levels. Data from different sources are often based on different topographical surveys, some of which may be old and poorly documented. Mistakes and confusion about the relationships between level datums tend to be common and can have serious consequences for the design and construction of a project. Simple common-sense checks between data sets can often highlight any serious inconsistencies. The datums used in tide tables and navigation charts for particular ports or estuaries are usually chosen so as to indicate minimum water depths for navigation purposes, with positive values on a chart representing the depth of the bed below the chart datum; it is therefore important to establish carefully the relationship between the chart datum and the national datum used for levels on land.

In the case of fluvial rivers, Method 3 above is normally necessary to predict water levels for design flood conditions if the return period of the event is significantly greater than the period for which records are available. For a relatively straight and uniform reach of river, reasonable estimates of flood level may be obtained using the Manning resistance equation and data on the longitudinal gradient of the channel (see Boxes 3.1 and 3.2). However, if the channel geometry varies significantly with distance or there is a downstream control, such as a weir or a junction with a major tributary, it may be necessary to carry out backwater calculations, either by hand or using a suitable computer program.

Computer programs may be classified according to the number of dimensions in which properties of the flow such as the velocity are calculated. Thus, one-dimensional (1-D) numerical models predict the variation of mean velocity (and water level) with distance along the channel. 2-D numerical models are able to take account of differences in local depth-averaged velocity across the width of a channel or floodplain. Full 3-D solutions also provide information on the variation of flow velocity with depth within the flow. Generally, the cost and complexity of a study increases significantly in moving from a 1-D to a 2-D simulation. A 1-D model often gives satisfactory results for fluvial rivers whose cross-sectional properties do not vary too rapidly with distance along the channel. However, in wide estuaries and in fluvial rivers with complex channel and floodplain geometries, it may be necessary to use a 2-D numerical model or a physical model to obtain reliable estimates of flood level. Use of 3-D numerical models is not normally justified, unless there is also a need to obtain detailed information about local flow conditions at particular locations or structures within the flow.

Box 3.3 *Definitions of channel shape and flow depth*



The width, depth and cross-sectional area of natural channels usually vary with distance, but for purposes of determining flow conditions and estimating scour it is necessary to define representative values. The following definitions are used in this manual.

It is assumed that cross-sectional information is available at several points along the river or channel in the vicinity of the site, as shown above. Provided that the variations between the cross-sections are not too large, the following procedure is recommended for calculating representative figures for the bank-full values.

1 At each cross-section, identify the level at which water would first begin to spill out of the incised channel onto one or both floodplains. Plot the average best-fit straight line through these points and use this line to determine a separate value of mean bank-full level, Z_{bf} (m), at each cross-section.

2 Use the cross-sectional data for each cross-section to determine: the actual bank-full width, B_{bf} (m); the cross-sectional area, A_{bf} (m^2), of the channel within the width B_{bf} up to the level Z_{bf} ; and the maximum channel depth, Y_{max} (m), between the level Z_{bf} and the lowest point in the cross-section.

3 From the individual values of width $B_{bf,1}$, $B_{bf,2}$, etc, calculate the average value $\overline{B}_{bf}$ and in a similar way the average flow area $\overline{A}_{bf}$; these averages can be taken as representative values of the bank-full geometry. A similar averaging technique can be used to determine representative values of channel geometry for other specified water levels and also for the floodplains.

4 Calculate the mean channel depth, Y_{bf} (m), measured from top of bank, as:

$$Y_{bf} = \frac{\overline{A}_{bf}}{\overline{B}_{bf}} \quad \textbf{(3.10)}$$

5 A measure of the cross-sectional shape of a channel is given by the shape factor, Ψ, defined as:

$$\Psi = \frac{Y_{max}}{Y_{bf}} \quad \textbf{(3.11)}$$

where Y_{bf} is calculated from equation (3.10) using averaged data from all the cross-sections. However, since for scour calculations it is usually necessary to consider the lowest likely bed levels, it is recommended to calculate Ψ using the largest value of Y_{max} observed at any of the cross-sections included in the averaging process.

6 For convenience, the bars indicating averaged values of A_{bf} and B_{bf} will be omitted from equations used in later sections of this manual.

If the planning, design and construction of a major project are likely to last for more than about two years, it is highly recommended that water level gauges be installed at the site as early as possible in the programme and a regular system of monitoring arranged. If the site cannot be attended on a regular basis, simple gauges that record the maximum water level occurring between readings can provide very useful information about flood events.

For structures in tidal flows, the heights of astronomical tides can usually be determined from navigation tables or from water levels measured by a local tide gauge. Care is needed to establish what is the correct relationship between the chart datum used for navigation purposes and the national datum. When determining the maximum design water level for protection works, it is normally necessary to add allowances for the effects of possible storm surges and fluvial floods (see Section 3.2). When estimating possible scour depths, it is worth noting that it is more appropriate to use the mean tide level (plus allowances for surges and flood flows) as the representative design value, because this is the level at which the maximum tidal velocities normally occur.

Flow depths can be calculated in a variety of ways, but particular definitions used in this manual are included in Box 3.3.

3.5 FLOW VELOCITIES

Information on flow velocities at and around structures is needed so that amounts of scour may be estimated and protection systems sized to withstand the erosive forces imposed. For fluvial rivers and channels, it is unlikely that flow measurements have previously been made under the flood conditions adopted for design. It is therefore usually necessary to estimate the flow velocities from the calculated values of design discharge, Q (m^3/s), and design flood level, Z (m above datum), from which the corresponding area of flow, A (m^2), can be determined using the known cross-sectional shape of the channel (see Sections 3.2 to 3.4). The mean velocity, V (m/s), over the full cross-section of the flow is given by:

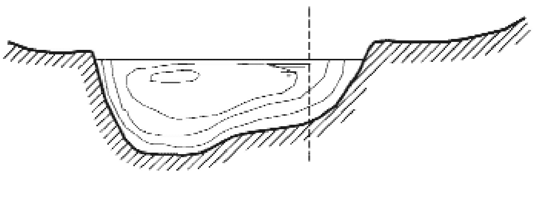
$$V = \frac{Q}{A} \qquad \textbf{(3.4)}$$

In the case of a compound channel consisting of an incised channel with adjacent floodplains, the methods given in Box 3.2 can be used to determine the value of Q in each part of the channel and the corresponding values of V.

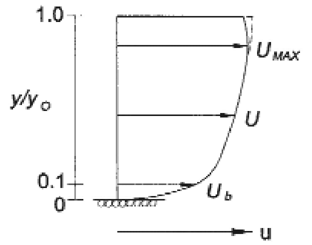
For design purposes it is normally necessary to use the value of local velocity adjacent to the structure, which in general is not equal to the section-mean velocity V. Information on typical velocity distributions in channels and around structures can be used to determine multiplying factors that can be applied to the value of V; details of these factors are given in Box 3.4.

Field measurements of flow velocity at the site can be useful even if the conditions under which they were measured are not as severe as specified by the design case. Velocities in the main channel of a river with floodplains tend not to increase substantially when the river is out-of-bank. Therefore, data obtained at or near bank-full conditions can provide a good indication of velocities that would occur in larger floods.

If velocity measurements are made at points across the width of a river, the information can be used to determine multiplying factors for relating local values of velocity to the section-mean velocity, V. This avoids the need to rely on assumed factors, such as those given in Box 3.4. Site-specific data are particularly useful in cases where bends or channel restrictions produce locally non-uniform flow. Important information can also be obtained from a field survey about flow directions within the river cross-section. Structures such as the piers and abutments of bridges, cofferdams and guide banks need to be well-aligned with the flow, otherwise the depths of scour and the amount of afflux (or head loss) produced by the structure may be increased considerably.



Contours of velocity *u*



Vertical velocity profile

It is necessary to distinguish between three definitions of flow velocity in a channel.

1 The **section-mean velocity**:

$$V = \frac{Q}{A} \tag{3.4}$$

2 The **local point velocity**, u (m/s), measured in the horizontal direction and at a height of y above the bed at a point where the local water depth is y_o (see above Figure). The particular case of the velocity at a height above the bed of y/y_o = 0.1 is defined in this manual as the bed velocity, U_b .

3 The **local depth-averaged velocity**, U (m/s), calculated from the bed to the water surface using the formula:

$$U = \frac{1}{y_o}\int_0^{y_o} u\, dy \tag{3.12}$$

The reference velocity used in several formulae for designing revetments is the depth-averaged value, U_t (m/s), measured at the toe of the revetment. As a guide, Neill (1973) recommends that:

- for straight channels: $U_t = \frac{2}{3}V$ **(3.13)**
- for the outside of bends and locations where the flow is attacking the bank: $U_t = \frac{4}{3}V$ **(3.14)**

The above values of velocity are all time-averaged values. A measure of the degree of turbulence within the flow is the root-mean-square fluctuation in local velocity, u_{rms} (m/s), relative to the time-averaged value of u at that point. The non-dimensional **turbulence intensity**, I, is defined as:

$$I = \frac{u_{rms}}{u} \tag{3.15}$$

The value of turbulence intensity at a height above the bed of y/y_o = 0.1 is used as a reference value and denoted by I_b. Based on field measurements by Escarameia and May (1992), the following typical values of I_b are suggested:

Location	**Turbulence level**	**I_b**
Fairly straight lengths of channel or revetment	Normal	0.12
Around bends and at upstream ends of revetments	Normal (higher)	0.20
Around structures such as piers, caissons and cofferdams	High	0.35
Downstream of zones of energy dissipation (eg from weirs and structures producing high-velocity jets)	Very high	0.5–0.6

The depth-averaged velocity, U, varies across the width of a channel and tends to be a maximum where the flow is deepest. As a rough guide, the maximum value of the ratio U/V in a fairly straight length of channel is normally about 1.15–1.25 (see, for example, Knight *et al*, 1994). Higher values may occur around bends or where there are constrictions in the flow.

continued on next page...

The presence of an isolated structure, such as a bridge pier, causes a local acceleration in the flow; for purposes of design, it may be assumed that the maximum depth-averaged velocity, U_S (m/s), around the structure is twice that of the undisturbed flow just upstream of the structure, ie:

$$U_S = 2U \tag{3.16}$$

The variation in local velocity with height above the bed depends on the degree of boundary layer development in the flow. For a fairly regular channel with a fully developed boundary layer, the velocity distribution can be described approximately by the formula:

$$\frac{u}{U} = \frac{8}{7}\left(\frac{y}{y_o}\right)^{1/7} \tag{3.17}$$

At a height above the bed of y/y_o = 0.1, this gives a value for the bed velocity of U_b = 0.82U.

Flow accelerations caused by structures, or changes in channel roughness at the upstream ends of protection works, tend to disrupt the previously developed boundary layer and lead to higher velocities being experienced close to the bed and sides of the channel. In these types of situation, the velocity distribution can typically be described by the formula:

$$\frac{u}{U} = 1.10\left(\frac{y}{y_o}\right)^{1/10} \tag{3.18}$$

At a height above the bed of y/y_o = 0.1, this gives a value for the bed velocity of U_b = 0.87U.

With open channel flow, the effects of secondary currents and the air–water interface normally cause the maximum velocity to occur below the surface at a value of about y/y_o = 0.9–0.95 (see above figure). The velocity profiles given by Equations (3.17) and (3.18) are therefore incorrect near the surface, but the overall effects of the error are usually small.

A key parameter that determines the magnitude of changes in water level and flow velocity around structures is the **Froude number**, F_r, which is defined as:

$$F_r = \sqrt{\frac{BQ^2}{gA^3}} \tag{3.19}$$

where A (m^2) and B (m) are, respectively, the values of flow area and surface width corresponding to the discharge, Q (m^3/s), in the channel.

If F_r < 1, the flow is termed **subcritical**. In a subcritical flow, energy losses due to the presence of a structure tend to cause the downstream water level to be lower and the velocity to be greater than upstream of the structure. Also, backwater calculations to determine water levels at the site need to start from a suitable downstream point and work backwards towards the site (see Section 3.4).

If F_r > 1, the flow is termed **supercritical**. In a supercritical flow, energy losses due to the presence of a structure tend to cause the downstream water level to be higher and the velocity to be less than upstream of the structure. There is also the possibility with a supercritical flow of a hydraulic jump forming upstream of the structure and of strong surface waves propagating downstream. In erodible channels, a hydraulic jump is likely to be undesirable because it can increase local scouring of the bed and its position may vary unpredictably with flow conditions. If the flow in a channel is supercritical, calculations to determine water levels at the site need to start from a suitable upstream point and work downstream towards the site (see Section 3.4).

If F_r = 1, the flow is termed **critical flow**. This is normally only a transitional state and occurs, for example, as flow is accelerated through a constriction, or passes from subcritical conditions upstream of a weir to supercritical conditions downstream.

The number of points at which measurements of flow velocity and direction are made should be chosen depending on the complexity of the river channel and the locations of the structures. For a fluvial river less than about 5 m deep, it might be sufficient to make one measurement at each selected position across the channel; the value of velocity at a height equal to 0.4 of the flow depth above the bed will give a reasonable estimate of the local depth-averaged velocity, U, at that point. For deeper channels, two or more measurements in the vertical at each position may be appropriate. It is recommended that one measurement be taken at a height equal to 0.1 times the flow depth above the bed (to provide a measure of the near-bed conditions) and another at a height of 0.9 times the flow depth (where the maximum velocity in a vertical section typically occurs). Further guidance on the selection of appropriate measurement positions is given in BS 3680 (1986).

For major structures in tidal conditions, it is recommended that field measurements of flow velocity normally be carried out unless detailed information from a calibrated numerical model is already available. Tide heights are predictable, so the survey can be planned to coincide with a high spring tide that can be expected to produce the largest flow velocities. Measurements should be made at more than one level in each vertical because inertial effects and density differences between fresh and saline water can cause significant variations in speed and direction at different depths and at different times throughout a tidal cycle. In particular, the difference between the mean flow directions in the ebb and flood phases of a tide may not necessarily be 180°.

3.6 BED MATERIAL

Information on the sediment characteristics of the bed and the banks is necessary in order to assess scour potential and design protection works.

For alluvial and gravel rivers, in which the bed material is usually mainly non-cohesive, the characteristics are normally obtained by sieving samples and determining the grading curves of the material. The grading is usually expressed in terms of the percentage weights of a sample passing different sizes of sieve. For example, 65 per cent of a sample by weight would be finer than the D_{65} size (in m or mm) of the sediment. Samples should be collected from several points in the area of interest to detect any important spatial variations in grading and to ensure that the results are representative. As well as the grading of the sediment, it is also necessary to determine the dry density (or specific gravity) of the particles. Information on how to collect and analyse bed samples is given in Parts 10C, D and E of BS 3680 (1986).

When estimating scour, it is not advisable to rely on any armouring of the surface layer that may have developed, because this layer of coarser material may be washed away in a major flood, leading to the more erodible underlying material becoming exposed to the flow (see Section 2.2.4). It is therefore, recommended that bed samples should normally be taken from a depth of about 0.5 m below the surface, so that they are representative of the underlying material. If it is believed that the ground conditions vary significantly with depth, core samples should be taken down to below the lowest expected scour level. Such samples are often obtained as part of the site investigations for the foundations of structures, but it is important to ensure that some cores are taken at points that are significant for scour. Also, the main layers within these additional cores should be analysed to determine their size gradings and cohesive properties, rather than relying on a simple visual description of the samples.

In rivers with coarse gravels or boulders, it may not be feasible to sieve or remove the samples. In such cases, it is necessary to rely on direct measurements of the dimensions of the stones. Care should be taken to identify a representative range of sizes; for irregular or angular large stones, the overall maximum, minimum and intermediate dimensions should be recorded. For defining materials such as riprap and rock armour used for protection works, it is normally convenient to specify the larger gradings in terms of the weights of the individual stones. Thus, for example, 65 per cent by weight of a sample would have a weight less than the W_{65} value of the grading. The weight, W, is usually expressed in units of kilogram-force (rather than in the dimensionally correct units of Newtons), and is often abbreviated to kilograms only; the latter convention is followed in this manual.

In estuaries, the bed material usually consists of mixtures of mud and sand and has cohesive properties. The erosive resistance of muds can vary significantly with depth within the bed and the period over which they have been able to consolidate. Bed samples should be collected and analysed to determine their density and moisture content. The sizes of the coarser constituents can be determined by sieving, but fine silts are usually categorised in terms of their settling velocity, which is measured using standard types of settling column (see Part 10C of BS 3680, 1996).

3.7 DESIGN RETURN PERIODS

An important factor that needs to be decided at an early stage in a project is the design return period used to determine the flow conditions for estimating scour and designing protection works. For major bridges in the UK, a flood return period of N = 100–200 years is often assumed. However, a more considered and objective approach is recommended for major schemes, where failures due to scour could have serious consequences in terms of safety of people and disruption to transport or infrastructure. The two factors that need to be taken into account are the design life, L_y (in years), of the structure and the degree of safety required against the design flow conditions being exceeded during the design life. The degree of safety can be expressed in terms of the probability, P_r, of the exceedance occurring, with a value of 0.0 corresponding to zero risk and a value of 1.0 corresponding to absolute certainty of exceedance. The factors N, L_y and P_r are connected by the following formula:

$$P_r = 1 - \left(1 - \frac{1}{N}\right)^{L_y} \qquad \textbf{(3.20)}$$

from which it can be shown that the relationship between the return period and the design life varies with the required degree of safety as follows:

$P_r = 0.60$	$N \approx 1.1\ L_y$
$P_r = 0.50$	$N \approx 1.5\ L_y$
$P_r = 0.40$	$N \approx 2.0\ L_y$
$P_r = 0.30$	$N \approx 2.8\ L_y$
$P_r = 0.20$	$N \approx 4.5\ L_y$
$P_r = 0.10$	$N \approx 9.5\ L_y$

It is important to realise that it is impossible to provide complete assurance against a certain set of design conditions being exceeded. However, the issue needs to be considered in a quantitative way as part of the overall risk assessment procedures (see Chapter 6). Also, it should be realised that the probability of a design flood being exceeded does not equate to the probability of a structure failing due to scour. Many other factors are involved, including the reliability of the prediction methods for estimating the depth and extent of scour.

When designing a major scheme, it is usually appropriate to specify a higher degree of safety (or level of confidence against damage) for the principal structures than for the training or protection works. Some damage to the latter in major floods may be acceptable, provided they do not endanger the principal structures. This approach can be readily followed using Equation (3.20): all the components of the scheme would have the same design life, L_y, but the specified probability of exceedance can be varied depending on the importance of the component. More detailed guidance on the choice of design return period is given in Section 6.3.2.

4 Estimation of scour

4.1 NATURAL SCOUR

As explained in Section 2.1, natural scour is considered in this manual to cover general changes in bed level that could be expected to occur at a site due to natural factors, without any additional effects produced by the presence of structures. Obtaining reliable estimates of the likely depth of natural scour is difficult, because of the lack of well-developed quantitative methods and because long-term changes can be the result of interactions between different scouring processes. Each of the main causes of natural scour are considered in the following sections.

4.1.1 Degradation of channel

Degradation leads to a general lowering of bed level along the length of the main deep-water channel and normally occurs over a period of years. No simple procedures for estimating changes in bed level are available because degradation is caused by large-scale imbalances in sediment load and sediment supply, and is also strongly dependent on the particular geotechnical properties of the channel. Two methods of estimating scour due to channel degradation are suggested.

The first method is to use historical data on bed levels in the vicinity of the site to estimate the long-term rate of change and to project this trend forward over the design life of the project. Also, a careful visual inspection should be carried out over several kilometres upstream and downstream of the site to identify whether there are signs of channel deepening progressing downstream (eg as a result of a structure reducing the sediment transport rate) or of a step reduction in bed level cutting its way upstream (termed a nickpoint recession). If these types of change are detected, a longitudinal survey should be carried out to quantify the likely change in bed level that will occur when the adjustment in channel gradient and depth reaches the site. If the magnitude of the change cannot be accommodated in the design of the structure, measures to stabilise the channel profile should be adopted (for example, by constructing a weir to prevent a nickpoint progressing farther upstream). Another important factor that can cause channel degradation is the removal of sand or gravel for construction or industrial purposes. If this is occurring, either indiscriminately or through the granting of abstraction licences, the potential long-term effects on bed levels need to be assessed.

The second method of estimating scour due to channel degradation is to use a numerical morphological model to predict long-term changes. This type of model calculates flow depths and velocities along a reach of river and from this information also determines rates of sediment transport. Variations in the transport rate between points along the reach are then used to calculate changes in bed level over time; allowance for the effects of sand or gravel extraction can also be made. One-dimensional models are computationally efficient and make it possible to simulate annual cycles of flow and to repeat them for a number of years to determine long-term changes in average bed level. Using cross-sectional information on typical channel shapes, the average levels can then be factored to obtain estimates of the lowest bed levels that may occur across the width of a channel (see Section 4.1.3).

Two-dimensional morphological models may be used where it is wished to obtain more detailed information on bed levels in a localised area. However, predicting how the cross-sectional shape of a channel will develop over a long period of time is difficult because random changes occurring naturally or introduced by instabilities within the numerical scheme used in the computer model can have a large impact on the final results.

The opposite case of channel aggradation does not normally need to be considered when assessing scour hazards because it is associated with general rises in bed level. However, if silting-up is occurring in a reach of channel, the deposition may not occur uniformly but be concentrated at particular points. As a result, the deposits may deflect the flow and increase the possibility of the main deep-water channel altering its course (see next section).

4.1.2 Lateral channel migration

Two main types of channel migration need to be considered for rivers and estuaries: erosion of the banks of the main incised channel causing a lateral movement of the whole river; and movement of one or more deep-water channels within a braided river or estuary.

Meandering rivers

As a result of bank erosion, the meander bends tend to move downstream, so that over time the position of the incised channel at a particular cross-section may vary transversely within the wider floodplain. This could result in part of a structure or scheme becoming exposed to deeper scour depths than were allowed for at the design stage. A particular problem can occur with bridge crossings, because lateral movement of the deep-water channel away from the main bridge opening is likely to lead to the channel attacking one of the approach embankments, potentially leading to a breach and cutting of the crossing. Once this has occurred, major engineering works may be necessary to restore the river to its original course through the main bridge opening.

The possible extent of lateral migration is related to the shape of the meander pattern. Analysis of the morphology of river channels indicates that there is a strong correlation between the mean bank-full width, B_{bf}, of the main channel (see Box 3.3) and geometric properties of the meanders such as the axial wavelength, λ_a (in m), the width of the meander belt, β (in m), and the radius of curvature at the apex of a bend, r_c (in m) – see Figure 4.1 for definitions. Data given or quoted by Leopold and Wolman (1960), Henderson (1966), Jansen *et al* (1979), Richards (1985), and Garde and Ranga Raju (1985) suggest the following relationships:

$$\frac{\lambda_a}{B_{bf}} \approx 12 \quad \text{typically, but can vary between 7 and 18} \tag{4.1}$$

$$\frac{\beta}{B_{bf}} \approx 2-4 \quad \text{typically, but up to 30 for strongly meandering rivers} \tag{4.2}$$

$$\frac{r_c}{B_{bf}} \approx 2-3 \tag{4.3}$$

These relationships can be expected to apply to alluvial rivers whose morphological development is not affected by man-made changes such as training works, or by geological factors such as the presence of rock outcrops. Although Equations (4.1) to (4.3) can be used to estimate possible amounts of lateral migration, it is important to study carefully the existing plan geometry of the channel upstream and downstream of the site, as this is likely to give the best indication of changes that may occur in the future.

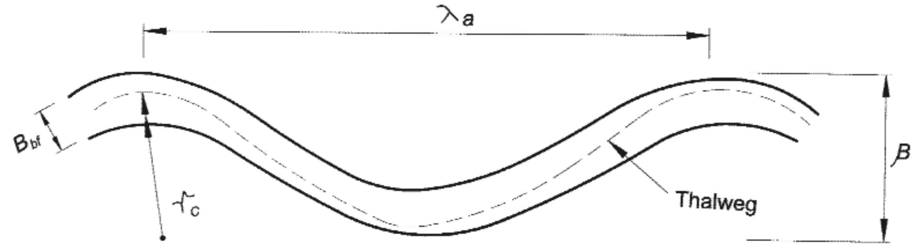


Figure 4.1 *Geometry of river meanders*

The rate of bank erosion varies greatly with the flow conditions, the size of the river and the type of material forming the banks. For UK conditions, an initial indication may be obtained using the following correlation established by Hooke (1980) between the average rate of bank retreat, E_R (in m/year), and the catchment area, A_c (in km^2), of the river basin at the point in question:

$$E_R = 0.025\,A_c^{0.45} \tag{4.4}$$

Typical values of E_R measured in catchments in south-east Devon were in the range 0.01–1.2 m/year. Much higher erosion rates can occur in large rivers with alluvial banks of low cohesive strength. Data given by Ackers and Charlton (1970) for seven rivers in the USA and India included four cases where rates were of the order of 50 m/year or more, with the highest figure being 244 m/year on the Missouri river in the USA. Due to the large uncertainty in possible values, as much use as possible should be made of historical data on channel movement in the vicinity of the scheme.

Movement of deep-water channel within cross-section

Braided rivers tend to be unstable and individual channels can form, move or disappear within a single major flood. Also, in rivers between fairly straight banks, the thalweg (the line of maximum depth) may sometimes develop a meandering pattern with bars forming on alternate sides of the channel. This type of internal meander tends to move gradually downstream causing the point of maximum depth in a particular cross-section to vary with time. For these reasons, it would normally be prudent to assume when designing or assessing a structure that the point of deepest natural scour could occur at any location across the width of the main river at some time during the life of the structure.

Exceptions to this general recommendation can be considered if changes in cross-sectional geometry are constrained by training works or geological features. Also, it may be reasonable to assume that at a bend the deepest channel will tend to remain near the outer bank, provided that the position of the bend itself is likely to remain stable.

In the case of estuaries, the positions of deep-water channels can vary both seasonally and over longer periods of time. Flow patterns in wide rivers or estuaries are complex; at some sites, the location of the deep-water channel may switch from one bank to the other in response to seasonal changes in the balance between freshwater flows and incoming tidal flows. Many estuaries are surveyed fairly regularly for navigation purposes, so good historical data may often be available on long-term patterns and trends. However, the effects on channel geometry and flow patterns of any dredging activities or aggregate extraction should also be assessed.

4.1.3 Regime conditions and long-term scour

As explained in Section 2.1.1, a river is said to be in a regime state when representative features of its cross-sectional shape remain reasonably constant, averaged over several years. In this condition, the channel is considered to have adjusted in such a way that its flow capacity and its sediment-transporting capacity are in a time-averaged equilibrium with the flows and sediment loads produced by the catchment that it drains. If a river has insufficient capacity, it will try to adjust to a more stable state by increasing its longitudinal gradient and/or changing its cross-sectional shape. The equilibrium condition is determined by the complex inter-relationships between flow rate, flow resistance and sediment transport rate.

Formulae for predicting the properties of alluvial channels in regime were originally developed using data from irrigation canals (in India and Egypt) that appeared to operate in a steady state without significant variations in channel size and depth. Correlations were established between the cross-sectional geometry, the longitudinal gradient, the flow rate and the type of sediment forming the bed and banks. The concept was extended to natural rivers by looking for similar correlations between flow rate, sediment type and channel geometry for rivers that appeared to have reached a quasi-equilibrium state. The characteristic features were considered to be the shape and size of the main channel when flowing at just bank-full conditions (ie the values of flow area, A_{bf}, flow width B_{bf} and mean bank-full depth Y_{bf} considered in Box 3.3) and also the longitudinal gradient, S of the channel. The representative flow rate has been defined somewhat differently by different researchers but it is generally considered to be the "dominant" or "channel-forming discharge", Q_{bf}, corresponding approximately to bank-full conditions for the main incised channel. For natural rivers, Q_{bf} is typically found to be the peak flow rate occurring in a flood with a return period of about 1.6 years (which corresponds statistically to the most probable maximum annual flood).

Of the various regime formulae available, those due to Lacey (1930), Lacey (1933), Simons and Albertson (1963) and Charlton (1982) are given in Box 4.1. Lacey's results are widely used, while those of Simons and Albertson take more detailed account of the effects produced by different types of bed and bank. Charlton's method is specifically applicable to gravel-bed rivers. It needs to be appreciated that the regime methods for alluvial channels are mainly based on data from irrigation canals and that there is, therefore, some uncertainty about their application to other types of channel. However, they do provide a useful indication of whether or not a natural channel is close to its likely equilibrium condition.

A more recent approach to regime theory is to combine predictive equations for channel resistance and rate of sediment transport and to assume that the regime condition occurs when the channel has adjusted to its most efficient configuration. Efficiency has been defined by various researchers in different ways, for example as minimum total stream power (ρQS), minimum unit stream power (ρqS) or maximum sediment transport rate. Although there are detailed differences between these assumptions, they appear to lead to similar results. Based on the maximum sediment transport assumption, White *et al* (1981) produced tables relating the dimensions of regime channels to dominant discharge for a range of different sediment sizes.

Box 4.1 *Formulae for regime channels*

The purpose of the following equations is to predict the long-term characteristics of alluvial channels produced by constant or naturally varying discharges. In the case of rivers, the geometry of the main incised channel is assumed to be produced by the bank-full discharge, Q_{bf} . Therefore, the predicted dimensions of the channel correspond to the bank-full condition (see Box 3.3). It is important to note that water depths predicted by regime equations are overall values measured from the water surface to the bed of the channel. Therefore, the change in predicted regime depth resulting from a variation in discharge is not, in general, equal to the change in bed level (since an increase in discharge normally leads to an increase in the height of the water surface).

LACEY (1930)

The original form of regime equation proposed by Lacey was:

$$V = 0.646\left(f\,R_R\right)^{1/2} \qquad \textbf{(4.5)}$$

where V (m/s) is the mean flow velocity in the incised channel (see Box 3.4), R_R (m) is the hydraulic radius (Equation 3.6) and f is termed the silt factor, which is related to the mean size, d_{mm} (in mm), of the alluvial material forming the bed by:

$$f = 1.76\sqrt{d_{mm}} \qquad \textbf{(4.6)}$$

The length of the wetted perimeter, P_R (in m), of the incised channel in the regime condition is related to the discharge, Q (m³/s), in the channel by:

$$P_R = 4.83\,Q^{1/2} \qquad \textbf{(4.7)}$$

Except for small watercourses, the regime width, B_R (m), of the channel can often be assumed to be effectively equal to P_R. For a large river or a channel that is relatively wide in relation to its depth, the mean flow depth, Y_R (m), for regime conditions is given reasonably closely by:

$$Y_R \approx R_R \qquad \textbf{(4.8)}$$

It follows from Equation (4.5) that Y_R is related to the discharge intensity q (m²/s) per unit width of channel ($q = Q/B$), by:

$$Y_R = 1.34\frac{q^{2/3}}{f^{1/3}} \qquad \textbf{(4.9)}$$

This is sometimes termed the two-dimensional form of the regime equation and can be used for estimating contraction scour (see Section 4.2).

LACEY (1933)

The length of the wetted perimeter, P_R (in m), of the incised channel in the regime condition is obtained as in Lacey (1930) from Equation (4.7).

The corresponding cross-sectional area, A_R (m²), of the flow is given by:

$$A_R = 2.28\frac{Q^{5/6}}{f^{1/3}} \qquad \textbf{(4.10)}$$

with the silt factor determined from Equation (4.6). It follows from Equations (4.7) and (4.10), that the regime values of the hydraulic radius and the mean flow depth (for wide channels) can be determined from:

$$Y_R \approx R_R = 0.472\frac{Q^{1/3}}{f^{1/3}} \qquad \textbf{(4.11)}$$

The longitudinal gradient, S_R to which the channel can be expected to adjust when it reaches the regime condition is given by:

$$S_R = \frac{f^{5/3}}{3170\,Q^{1/6}} \qquad \textbf{(4.12)}$$

SIMONS AND ALBERTSON (1963)

The geometric factors defined above are predicted from similar types of equation to those of Lacey, but with the addition of coefficients (J_1 to J_5 and m) that allow for different types of channel bed and bank:

$$P_R = J_1\, Q^{1/2} \qquad \textbf{(4.13)}$$

$$B_R = 0.98\, P_R + 0.7 \qquad \textbf{(4.14)}$$

$$A_R = J_2\, Q^{0.87} \qquad \textbf{(4.15)}$$

$$R_R = J_3\, Q^{0.36} \qquad \textbf{(4.16)}$$

$$Y_R = 1.21\, R_R \quad \text{for } R_R < 2.1 \text{ m} \qquad \textbf{(4.17)}$$

$$Y_R = 0.93\, R_R + 0.6 \quad \text{for } R_R > 2.1 \text{ m} \qquad \textbf{(4.18)}$$

$$S_R = \frac{1}{R_R^2}\left(J_4 V\right)^m \qquad \textbf{(4.19)}$$

or, alternatively:

$$S_R = J_5 \left(\frac{V^2}{g\, Y_R}\right)\left(\frac{\nu}{V\, B_R}\right)^{0.37} \qquad \textbf{(4.20)}$$

where ν is the kinematic viscosity of water (= 1.14×10^{-6} m²/s at 15° C). All quantities in the equations should be in SI units, as defined above. Values of the coefficients vary according to the type of channel as follows:

Channel type	Description
1	Sand bed and banks
2	Sand bed and cohesive banks
3	Cohesive bed and banks
4	Coarse non-cohesive material
5	Same as type 2, but with heavy sediment loads (eg 2000–8000 ppm)

	Channel type				
Coefficient	**1**	**2**	**3**	**4**	**5**
J_1	6.34	4.71	4.0–4.7	3.2–3.5	3.08
J_2	2.6	2.25	2.25	0.94	—
J_3	0.4–0.6	0.48	0.41–0.56	0.25	0.37
J_4	0.107	0.093	—	0.208	0.103
J_5	3.03	1.89	1.14	—	—
m	3.03	3.03	—	3.45	3.45

CHARLTON (1982)

The following formulae apply only to gravel-bed rivers.

(a) ***Deep channels*** – for $3 \le Y_R/d_{90} \le 80$

$$B_R = 3.74\, K_G\, Q^{0.45} \qquad \textbf{(4.21)}$$

$$Y_R = \left(0.114\, K_G^{-1.82} Q^{0.42} d_{65}^{-0.38} d_{90}^{0.24}\right) - d_{50Z} \qquad \textbf{(4.22)}$$

$$S_R = 0.15\, K_G^{-1} Q^{-0.76} B_R^{0.76} d_{65}^{1.38} d_{90}^{-0.24} \qquad \textbf{(4.23)}$$

In the case of gravels, the particles can be very angular with significant differences between the dimensions measured along the major, intermediate and minor axes. In the case of Equations (4.22) and (4.23), the representative sediment sizes d_{65} and d_{90} refer to values (in m) measured along the intermediate axis, with respectively 65% and 90% of the gravel particles by weight being finer than these sizes. The sediment size d_{50Z} (in m) is measured along the minor axis, with 50% of the gravel particles by weight being finer than this size. The value of the non-dimensional factor K_G depends on the type of vegetation on the banks of the channel: for grass and light vegetation, $0.9 < K_G < 1.3$; for trees and heavy vegetation, $0.7 < K_G < 1.1$. For a channel that is in a regime state with a width, B_R (in m), corresponding to the bank-full discharge, Q_{bf} (in m³/s), the value of K_G can be determined from Equation (4.21) and then used in Equations (4.22) and (4.23).

(b) ***Shallow channels*** – for $Y_R/d_{90} < 3$

$$B_R = 3.74 K_G\, Q^{0.45} \qquad \textbf{(4.21)}$$

$$Y_R = \left(0.477\, K_G^{-1.82} Q^{0.25} d_{65}^{-0.22} d_{90}^{0.55}\right) - d_{50Z} \qquad \textbf{(4.24)}$$

$$S_R = 0.068\, K_G^{-1} Q^{-0.45} B_R^{0.45} d_{65}^{1.22} d_{90}^{-0.55} \qquad \textbf{(4.25)}$$

The values of d_{65}, d_{90}, d_{50Z} and K_G are determined in the same way as for deep gravel-bed channels (see Section (a) above).

Recommended procedure

1. The purpose of the following procedure is to identify whether a reach of natural river or canal is close to its regime condition. If it is not, significant degradation, aggradation or width adjustment may occur if the channel is erodible and able to evolve towards the regime state. The procedure is also useful for establishing the most appropriate regime equations for estimating short-term scour (Section 4.1.4) and contraction scour (Section 4.2).
2. Calculate from the equations in Box 4.1 the dimensions (P_R, B_R, A_R, R_R, Y_R and longitudinal slope S_R) of the regime channel corresponding to Q_{bf}. Note that Y_R is the value of the mean water depth measured from the bank-full level (see Box 3.3). If there is evidence that a channel has remained fairly stable over a long period of time, this may be used to help establish a reasonable estimate of the silt factor, f, in Lacey's equations, or to decide between possible alternative descriptions of the bed and banks in Simons and Albertson's method. Alternatively, tables such as those produced by White *et al* (1981) may be used to estimate the regime geometry.
3. If the predicted cross-sectional shape of the regime channel is significantly different from the existing geometry, the channel may not be stable and may tend to evolve over time towards the regime shape (unless prevented by geological features or training works). If the actual value of bank-full width, B_{bf}, is less than B_R, the banks may suffer erosion; similarly if Y_{bf} is less than Y_R, the channel may deepen.

4.1.4 Short-term scour during floods

During a major flood, higher-than-average flow velocities may cause a short-term lowering of bed levels within an incised channel if the bed material is erodible. 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, however, it is recommended to assume 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 is difficult to predict with certainty because information on rates of natural scour is very limited. A key factor to be remembered is that a general lowering of bed level within a particular channel reach 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 the transport rates produced by a higher flow velocity does not itself cause scour, unless there is an imbalance between the amounts of sediment in transport at the upstream and downstream ends of the channel reach. However, any existing imbalances tend to be accentuated during floods, leading to more rapid short-term changes. Particular consideration should be given to cases where natural features or structures, such as barrages or weirs, act to limit the amount of sediment entering reaches containing structures that could be at risk from scour. Similarly, the removal of a downstream control on the flow (for example, through operation of gates or as the result of localised scour) could increase downstream flow velocities and sediment transport rates, leading to rapid 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 morphological type described in Section 4.1.1. Morphological 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. The data requirements and costs involved in this type of numerical modelling may not be appropriate for smaller or more localised projects, however. Wherever possible, the ability of a morphological model to reproduce past changes in channel geometry should be checked before it is used to predict short-term or long-term changes in the future.

The more usual approach to assessing short-term natural scour is to rely on an extension of regime theory. The basic assumption made is that, during a major flood, the main incised channel tends to increase in size towards the regime geometry corresponding to the peak flow rate. It is unlikely that a full adjustment to the higher regime condition will be achieved during an individual flood, not least because the appropriate changes in channel width and longitudinal gradient take a considerable time, and may be prevented by man-made training or control works. Nevertheless, because it is not possible to be certain how far any short-term changes will progress, it is customary to assume that the full regime condition corresponding to the design flood would be reached.

Methods for estimating amounts of natural scour during floods are not well established or documented, and it is usually necessary to make assumptions and simplifications. The following procedure is suggested, but alternative methods may be used based on previous experience. 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, if they are likely to carry significant flows during major floods. 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. If the section of channel being considered is subject to contraction scour due to the presence of a structure, the following calculation steps may be omitted and the overall depth of short-term scour in the contracted section determined using the procedure recommended at the end of Section 4.2.

Recommended procedure

1 Check from Section 4.1.3 whether the existing cross-sectional shape of the main incised channel (see Box 3.3) is in reasonable agreement with the dimensions predicted by the regime equations (see Box 4.1) for the bank-full flow rate, Q_{bf} . If one particular method gives a better fit to the observed geometry than the others, it is recommended that method be used for the following estimation of short-term changes. If none of the methods shows reasonable agreement, the results from the following steps should be interpreted with caution.

2 It is assumed that the design flood discharge, Q_D (m^3/s) , for the river has been determined (see Sections 3.2 and 3.7), together with the corresponding water level, Z_D (m above datum). Part of this flow would be in the main incised channel and part on any adjacent floodplains. The rate of flow, Q_1 (m^3/s), within the main incised channel can be calculated as follows using the information on compound channels given in Box 3.2. Label the main incised channel (having a surface width equal to the bank-full width, B_{bf}) as channel number i = 1 and other higher level or floodplain channels as i = 2, 3, .. , N. For the design flood level Z_D, calculate the conveyance parameter, C_i, for each sub-channel from:

$$C_i = \frac{1}{n_i} \frac{A_i^{5/3}}{P_i^{2/3}} \tag{4.26}$$

using the guidance on how to determine the flow area, A_i (m^2), wetted perimeter, P_i (m) and Manning roughness coefficient, n_i, given in Box 3.2. The flow rate, Q_1, conveyed by the main incised channel can then be found from:

$$Q_1 = Q_D \frac{C_1}{\sum_{i=1}^{i=N} C_i} \tag{4.27}$$

3 Use the regime equations in Box 4.1 to calculate predicted dimensions (B_R, P_R, A_R and Y_R) of the incised channel if it were able to reach an equilibrium condition corresponding to the flow rate, Q_1.

4 In most cases, the predicted channel width, B_R (m), will not equal the actual bank-full width, B_{bf} (m), in the vicinity of the structure being considered. As explained above, it is unlikely that the channel width would be able to adjust to the appropriate value during the limited time that the flood lasts. However, if B_R is significantly greater than B_{bf}, some bank erosion is likely to occur and protection works may be required in the vicinity of the structure.

5 The assumption is now made that the flood flow will scour the bed of the incised channel so as to provide the cross-sectional area, A_R (m^2), predicted by the regime method (that is, giving the same predicted mean velocity). 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. Therefore, the required cross-sectional area, A_s (m^2) of the scoured channel below bank level is:

$$A_s = A_R - B_{bf}\left(Z_D - Z_{bf}\right) \quad \textbf{(4.28)}$$

The average bed level, Z_b (m above datum), across the width of the incised channel is given by:

$$Z_b = Z_{bf} - \frac{A_s}{B_{bf}} \quad \textbf{(4.29)}$$

The lowest bed level in the cross-section, Z_{min} (m above datum), can be estimated using the value of the channel shape factor, ψ, determined from Equation (3.11) in Box 3.3 so that:

$$Z_{min} = Z_{bf} - \psi\,\frac{A_s}{B_{bf}} \quad \textbf{(4.30)}$$

Alternative assumptions can be made about how the area of scour below the normal bed level would be distributed across the width of the channel (see Neill, 1973), but the above method has the advantage of using information on the actual cross-sectional characteristics of the channel.

6 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 might be 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).

7 If the predicted amount of bed scour is substantial and it would have a significant effect on flow capacity, it may be necessary to determine a new estimate of the design flood level, Z_D, and repeat steps 2 to 6.

4.1.5 Bend scour

Flow around a bend in a channel creates a non-uniform flow distribution, with velocities around the outside of the bend being higher than in an equivalent straight section of channel. The flow distribution also produces a secondary current in the form of a spiral flow that transports sediment at the bed towards the inside of the bend. As a result, the flow depth around the outside of a bend is usually greater than the average depth in an equivalent straight channel, with the depth around the inside of the bend being correspondingly less than average.

The first step in estimating the maximum natural scour depth occurring in a bend is to determine from Equation (4.29) the mean bed level, Z_b (m above datum) produced in a straight section of channel by the design discharge, Q_D. The lowest bed level, Z_{bend} (m above datum) around the bend is given by:

$$Z_{bend} = Z_D - (1 + \xi)(Z_D - Z_b) \qquad \textbf{(4.31)}$$

where Z_D is the water level corresponding to Q_D. The bend shape factor, ξ, can be estimated from the following formula due to Thorne *et al* (1995):

$$\xi = 2.07 - \log_{10}\left(\frac{r_c}{B_{bf}} - 2\right) \quad \text{for } 2 < \frac{r_c}{B_{bf}} < 22 \qquad \textbf{(4.32)}$$

This best-fit equation was derived from laboratory and field data, with flow depths up to 17 m and sediment sizes between 0.3 mm and 63 mm. The error band for the predicted values of ξ was estimated to be ± 25 per cent.

4.1.6 Confluence scour

When flows from two rivers or channels meet at a confluence, the mixing of the two flows creates shear layers and increased turbulence. As a result, considerable extra bed scour can occur in the zone immediately downstream of the confluence. Similar effects may also occur where two channels in a braided river combine. The lowest bed level, Z_{con} (m above datum), is given by:

$$Z_{con} = Z_D - (1 + \eta)(Z_D - Z_b) \qquad \textbf{(4.33)}$$

where the confluence scour factor, η, can be estimated from the following formula based on data due to Klaassen and Vermeer (1988) and Ashmore and Parker (1983):

$$\eta = c_o + 0.037\theta, \quad \text{for } 20^o \le \theta \le 80^o \qquad \textbf{(4.34)}$$

where θ is the angle (in degrees) formed by the junction of the two rivers. The value of the coefficient c_o varies from 1.29 for rivers with fine sand to 2.24 for rivers with coarse gravel. Equation (4.34) does not take account of the relative magnitude of the flows in the two channels and may not be reliable if the ratio between the larger and the smaller flows exceeds about 2.2. The value of Z_b used in Equation (4.33) should be taken as the mean bed level that would be produced by the design flow in the channel immediately downstream of the confluence (ie the sum of the flows in the upstream channels for the design condition).

4.2 CONTRACTION SCOUR

Structures built within a channel can have two separate effects in terms of flow conditions and scour. The structures may reduce the available cross-sectional area and, for a given total discharge, lead to higher average flow velocities in the channel. If the channel is formed in erodible material, the higher velocities may cause general lowering of the bed across the full width of the channel, in addition to that produced by natural scour; this additional effect is termed contraction scour. Depending on the material forming the sides of the channel, contraction scour may also be accompanied by bank erosion. In addition to the overall effects on the channel, structures such as bridge piers, abutments, caissons and cofferdams produce localised increases in flow velocity and turbulence levels, and these can lead to the development of additional scour around and immediately downstream of the structures. This is termed local scour and is considered in Section 4.3.

The following procedure for estimating contraction scour is based on a similar approach to that given in Section 4.1.4 for determining natural scour during design flood conditions. As before, the calculation method aims to determine scour levels in the main incised channel (although the method can be adapted to other situations). Two types of contraction effect can occur singly or in combination. First, a scheme such as a bridge crossing of a river often includes approach embankments that block parts of the floodplain. For out-of-bank conditions, flows on the floodplains are intercepted and diverted through the main bridge opening containing the incised channel. The flow rate in the incised channel is therefore greater than it would be if the river were in its natural state, and this can lead to the occurrence of greater scour depths. The second type of contraction scour is due to the presence of structures in the incised channel that reduce the available cross-sectional area and thereby result in a general increase in flow velocity within the channel.

Recommended procedure

1 Check from Section 4.1.3 whether the existing cross-sectional shape of the main incised channel (see Box 3.3) is in reasonable agreement with the dimensions predicted by the regime equations (see Box 4.1) for the bank-full flow rate, Q_{bf}. If one particular method gives a better fit to the observed geometry than the others, it is recommended to use that method for the following estimation of contraction scour. If none of the methods shows reasonable agreement, the results from the following steps should be interpreted with caution.

2 It is assumed that the design flood discharge, Q_D (m^3/s), for the river has been determined (see Sections 3.2 and 3.7), together with the corresponding water level, Z_D (m above datum), just downstream of the structure(s) being considered; the water level within the contracted section is assumed to be equal to Z_D.

3 If the floodplains are blocked or partially blocked by embankments (and Z_D exceeds the bank-full level, Z_{bf} (m above datum)), estimate the flow rates that will be passed by any flood-relief culverts through the embankments and subtract them from Q_D. The remaining flow, Q_{DC} (m^3/s), is the design flow rate passing through the main opening in the embankment at the contracted section.

4 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. The rate of flow, Q_1 (m^3/s), within the main incised channel should be calculated using equations (4.26) and (4.27), as described in step 2 of Section 4.1.4 and with Q_{DC} replacing Q_D. Note that, 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.

5 Use the regime equations in Box 4.1 to calculate predicted dimensions (B_R, P_R, A_R and Y_R) of the incised channel if it were able to reach an equilibrium condition corresponding to the flow rate, Q_1.

6 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 B_{bf} less the width, measured transverse to the flow, of any obstructions in the incised channel). As explained in Section 4.1.4, it is unlikely that the channel width would be able to adjust to the appropriate value during the limited time that the flood lasts. 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.

7 The assumption is now made 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) predicted by the regime method (ie giving the same predicted mean velocity). 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.

Therefore, the required cross-sectional area, A_s (m^2), of the scoured channel below bank level is:

$$A_s = A_R - B_1\left(Z_D - Z_{bf}\right) \quad \textbf{(4.35)}$$

The average bed level, Z_b (m above datum), across the width of the incised channel in the contracted section is given by:

$$Z_b = Z_{bf} - \frac{A_s}{B_1} \quad \textbf{(4.36)}$$

The lowest bed level in the cross-section, Z_{min} (m above datum), can be estimated using the value of the channel shape factor, ψ, determined from Equation (3.11) in Box 3.3 so that:

$$Z_{min} = Z_{bf} - \psi \frac{A_s}{B_1} \quad \textbf{(4.37)}$$

Alternative assumptions can be made about how the area of scour below the normal bed level will be distributed across the width of the channel (see Neill, 1973), but the above method has the advantage of using information on the actual cross-sectional characteristics of the channel.

8 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 might be 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).

4.3 LOCAL SCOUR

4.3.1 General features

As explained in Section 2.1.3, the presence of a structure in flowing water causes three-dimensional changes in the velocity field around the structure. As shown in Figure 4.2, three important effects are:

- increases in local flow velocity in the horizontal plane as the flow locally accelerates around the upstream end of the structure. For structures with circular or rectangular plan shapes, the maximum velocity at the widest point of a structure is approximately equal to twice the velocity of the approaching flow just upstream of the structure
- development of vertical velocities caused by the upstream face of the structure, deflecting part of the flow downwards towards the bed
- separation of flow from downstream parts of the structure, leading to formation of a turbulent wake in which vortices with rotating cores of higher-velocity water are carried downstream by the flow.

All these effects tend to increase the erosive forces exerted by the flow on the bed of the channel. Therefore, in erodible materials, scour depths in the immediate vicinity of a structure are normally greater than elsewhere in the channel. At structures such as bridge piers and cofferdams, downward flow into the scour hole formed around the upstream face of the structure gives rise to a strong horseshoe vortex (see Figure 4.2). The high velocities, and resulting low pressures, within the core of the vortex produce a vacuuming effect that is efficient at removing sediment and that can lead to the formation of steep-sided scour holes around the structure.

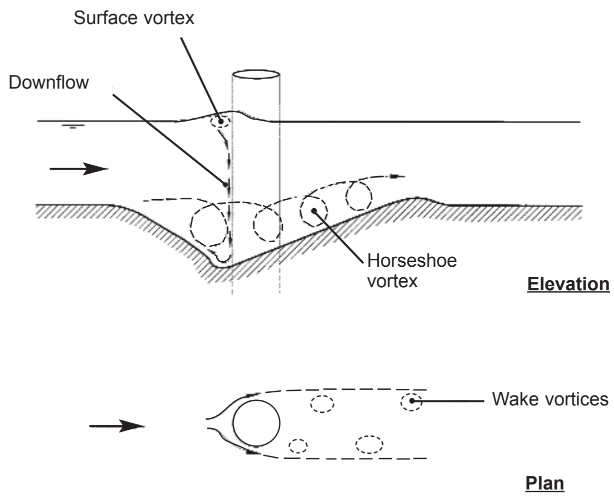


Figure 4.2 *Typical flow pattern around a structure*

When considering local scour, it is important to differentiate between clear-water scour and live-bed scour. Clear-water scour occurs when the flow velocity upstream of a structure is less than the threshold value needed to cause movement of the bed material. Scour develops around the structure if the local intensification of the flow is sufficient to exceed the threshold condition, with sediment eroded from the scour hole being transported a short distance downstream of the structure before depositing again. The depth and lateral extent of the scour hole continue to increase until the flow is no longer able to remove material from the hole. The rate at which the scour hole develops becomes slower as it becomes deeper, and the time taken to reach the final equilibrium value can be very long (of the order of weeks).

Live-bed scour occurs when the flow velocity upstream of a structure is greater than the threshold value needed to cause movement of the bed material. The scour hole develops in a similar way as it does in clear-water scour, except that there is a continuous supply of bed material entering the hole around its upstream edges. As a result, the equilibrium depth of scour is reached when the rate at which the flow can transport sediment out of the hole matches the rate at which material is entering from upstream, that is, when the net rate of sediment movement from the hole is zero. Since bedload transport in a channel is normally accompanied by the formation of ripples or dunes, the rate at which material enters the scour hole fluctuates with time and this causes a corresponding fluctuation in the value of equilibrium scour depth.

For a given size and shape of structure, the key factor determining the depth of local scour is the ratio U/U_{TC}, where U (m/s) is the depth-averaged velocity just upstream of the structure, and U_{TC} (m/s) is the value of U corresponding to the threshold condition for the start of bed material movement. Methods of calculating values of U_{TC} for non-cohesive sediments are given in Box 4.2. Typical variations of the equilibrium scour depth with the value of U/U_{TC} are shown in Figure 4.3 for the case of a bridge pier, where Y_S (m) is the depth of scour measured below the upstream bed level, and H (m) is the horizontal width of the structure measured normal to its longitudinal axis.

Curve 1 in Figure 4.3 illustrates the case of a non-cohesive sediment with a fairly uniform particle size. It can be seen that the equilibrium scour depth reaches a maximum when $U/U_{TC} = 1$, when the bed of the channel upstream of the structure is at the threshold of movement. For values of U/U_{TC} greater than unity, the scour depth initially decreases and then increases again to a depth equal to about 90 per cent of the value at the threshold condition; time-varying fluctuations about this limit are produced by the movement of bed features such as dunes passing through the scour

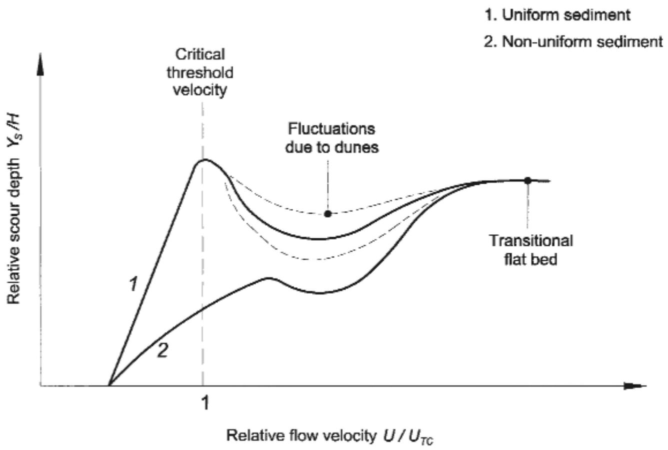


Figure 4.3 *Variation of scour depth with flow velocity for bridge pier*

Curve 2 in Figure 4.3 shows the case of a non-cohesive sediment mixture with a fairly wide size grading (with the value of U_{TC} corresponding to that for the mean particle size, d_{50}). Since each size fraction has its own corresponding value of threshold velocity, the sediment mixture does not give rise to a sharply defined maximum scour depth at $U/U_{TC} = 1$, but instead demonstrates a more general trend of increasing scour depth with increasing velocity. However, at high enough values of U/U_{TC} the scour depth becomes similar to that obtained with the uniform sediment.

A key finding established from research on local scour is that curves of the type shown in Figure 4.3 are generally independent of the actual sizes of the structure and the sediment particles provided that the values of equilibrium scour depth and flow velocity are calculated in terms of the ratios Y_S/H and U/U_{TC}. However, the relative scour depth does depend on the relative flow depth, y_o/H, where y_o (m) is the local flow depth measured from the bed of the channel just upstream of the scour hole. For relative flow depths less than about $y_o/H = 3$, a reduction in flow depth produces a reduction in equilibrium scour depth.

For structures such as bridge piers, abutments, caissons and cofferdams, it follows from Figure 4.3 that the maximum local scour depth that can occur is the equilibrium value obtained with a uniformly graded sediment at the limit of clear-water scour ($U/U_{TC} = 1$). This applies even if flow velocities considerably exceed the threshold velocity, U_{TC} , during a flood. The limiting value of Y_S (for the appropriate flow depth, y_o) therefore provides a safe estimate for design purposes. The main effect of flow velocity is on the rate at which the scour develops. If the value of U is only slightly greater than U_{TC}, the duration of the design flood may not be sufficient for the local scour to reach at all close to the equilibrium value. If the flow velocity is considerably higher than U_{TC}, however, scour depths approaching 80–90 per cent of the maximum possible value may be able to be reached within one or two days.

The above information on the development of local scour was mainly obtained from investigations in which the bed sediments were non-cohesive. Much less is known about local scour in cohesive materials.

In the case of estuaries, the sediments usually consist of fine sands, silts and muds and can demonstrate significant cohesive properties. The cohesive effects are produced by electro-chemical forces acting between the fine-grained particles and by biological slimes. The critical shear stress, τ_c (N/m^2), needed to initiate erosion of a cohesive bed is therefore greater than the value predicted by the Shields method (see Box 4.2) for a non-cohesive material with the same particle sizes. As well as the size grading, the value of τ_c depends on the bulk density of the sediment, which varies with the depth of the material below the surface and the time for which it has been deposited. Typical values of τ_c for sediments in estuaries are of the order of 0.1–0.2 N/m^2, with a maximum range of 0.02–5.0 N/m^2. Where possible, field samples of cohesive sediments should be obtained and their shear resistance to flow measured in a laboratory flume. More information on methods of estimating values of τ_c is given by Whitehouse *et al* (1999). When the value of τ_c has been determined, the corresponding depth-averaged flow, U_{TC} (m/s), at the threshold of movement of the sediment can be found from Equations (4.42) to (4.47) in Box 4.2.

In the case of rivers, the erosion resistance of sands and gravels can be significantly increased by the presence of clays. The depth-averaged velocity, U_{TC}, needed to initiate erosion at a point in the cross-section depends on the local water depth, y_o (m), and the composition of the material. Typical values of U_{TC} given by Hoffmans and Verheij (1997) are listed in Table 4.1.

Table 4.1 *Critical velocities to initiate erosion of cohesive materials in rivers*

Type of material	**Water depth y_o (m)**	**Threshold velocity U_{TC} (m/s)**
Loamy sand, light loamy clay with low compaction	1	0.4
Heavy loamy clay of low density	3	0.5
Low-density clay	10	0.6
Light loamy clay with medium compaction	1	0.8
Heavy loamy clay of medium density	3	1.0
Clay of medium density	10	1.3
Light loamy clay (dense)	1	1.2
Heavy loamy clay (dense)	3	1.5
Hard clay	10	1.9

Based on data given by Hoffmans and Verheij (1997)

Studies of the erosion of cohesive materials suggest that, once the shear forces exerted by the flow on the sediment bed exceed the appropriate threshold value, the particles in the surface layer are eroded and are transported by the flow in a similar way to non-cohesive sediments. The cohesive forces between the eroded particles or flocs do not re-establish themselves unless the material is able to consolidate again in areas or periods of quiescent flow. In the same way as with non-cohesive sediments, the scouring process proceeds downwards until the fluid stress acting at the bottom of the scour hole is no longer greater than the shear resistance of the material.

There is little comparative information on depths of local scour at structures in cohesive and non-cohesive sediments. For purposes of design, it is assumed in this manual that the equilibrium depth of local scour in a cohesive material is the same as would occur in a non-cohesive material having the same ratio between the depth-averaged flow velocity, U, and the critical threshold velocity, U_{TC}. Thus, as an example, a silty mud may be equivalent in terms of scour to an appropriate size of non-cohesive sand, while a loamy clay may be equivalent to a particular size of gravel. Suitable values of U_{TC} for cohesive sediments can be determined using the information given above. The main difference between a cohesive and an equivalent non-cohesive material is that the time needed for the cohesive material to reach its equilibrium scour depth is likely to be longer.

No general quantitative methods appear to have been developed for estimating depths of local scour around structures founded on rock. If the surface of the rock can be fractured or softened by the action of flowing water, the scour will tend to deepen gradually over time and may ultimately reach a significant proportion of the depth that would occur in a loose erodible material. Typical values of depth-averaged velocity, U_{TC} , needed to erode rock are in the range 1.5–2.5 m/s for soft rock and 3.0–4.5 m/s for hard rock. To obtain an approximate estimate of maximum scour depth, it is suggested that the depth be estimated assuming the material to be non-cohesive with a particle size equivalent to that of the blocks or stones into which the rock will tend to be broken or eroded by the water. The time for the development of the scour tends to be very much longer than in the case of loose erodible materials, however. In suitable circumstances, a programme of regular monitoring and reactive maintenance may be the most appropriate approach for dealing with scour problems of this type (see Section 5.8).

In Sections 4.3.2 to 4.3.8, information is given on ways to determine depths of local scour at different types of structure. Where discussed in the appropriate section, the following general procedure should be used to determine the lowest level to which the local scour can be expected to reach.

General procedure for estimating levels of local scour

1 First estimate the bed level, Z_o (m above datum), that can be expected to occur just upstream of the structure during the design flood – use either the procedure in Section 4.1.4 if the structure is located in an unrestricted section of natural channel, or the procedure in Section 4.2 if the structure is in an area subject to contraction scour. For design purposes, it should normally be assumed that $Z_o = Z_{min}$, the lowest predicted level of natural or contraction scour, unless there are good reasons for believing that the deepest point in the cross-section will not be able to migrate to the position occupied by the structure (for example, because of training works, geological features or large physical separation).

2 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:

$$y_o = Z_D - Z_o \qquad \textbf{(4.38)}$$

and also the corresponding value of the local depth-averaged velocity, U (m/s), for that flow depth.

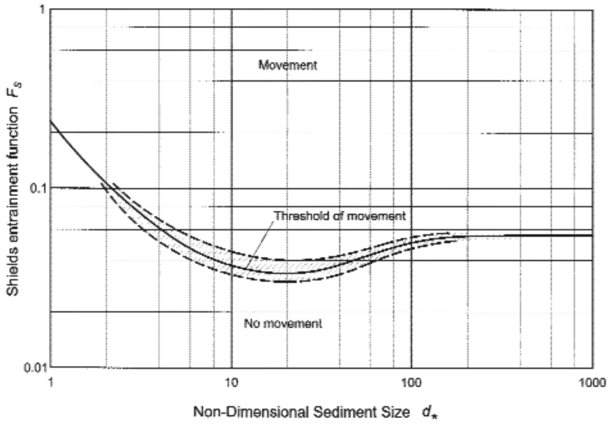
3 Determine the value of the depth-averaged critical threshold velocity, U_{TC} (m/s), using the information in Box 4.2 and this Section. If $U/U_{TC} < 1$, the local scour will be of clear-water type; if $U/U_{TC} > 1$, there will be general bedload movement of sediment in the vicinity of the structure and the scour will be of live-bed type.

4 Calculate the maximum depth of local scour, Y_S (m), at the structure using the information from Sections 4.3.2 to 4.3.8 as appropriate.

5 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:

$$Z_{LS} = Z_o - Y_S \qquad \textbf{(4.39)}$$

Box 4.2 *Calculation of flow velocity at threshold of bed movement*



Modified Shields curve for the threshold of movement of sediment particles

The following procedures can be used to determine the critical threshold velocity, U_{TC} (m/s), which is the depth-averaged flow velocity required to initiate bedload movement of non-cohesive sediment particles over a loose deposited bed of similar material. The preferred method is based on the use of experimental results and analysis due to Shields (1936). Initial estimates of U_c may also be obtained from simpler formulae such as those due to Hanco (1971) and Ackers and White (1973), but these results may not necessarily be valid for all combinations of flow condition and sediment size.

SHIELDS METHOD

Shields (1936) demonstrated that the threshold of movement of non-cohesive sediment particles on a loose bed of the same material could be defined in terms of two non-dimensional quantities. These are the Shields entrainment function:

$$F_s = \frac{\tau_c}{\rho g \left(\frac{\rho_s - \rho}{\rho}\right) d} \quad \mathbf{(4.40)}$$

and the particle Reynolds number:

$$R_{e*} = \frac{U_* \, d}{\nu} \quad \mathbf{(4.41)}$$

where:

τ_c (N/m^2) is the value of the critical shear stress exerted by the flow on the sediment bed at the threshold of particle movement

ρ is the density of water (= 1000 kg/m^3 for fresh water)

g is the acceleration due to gravity (= 9.81 m/s^2)

ρ_s (kg/m^3) is the dry density of the sediment particles (typically in the range 2400–2650 kg/m^3 for inorganic sands and gravels)

d (m) is the representative particle size; for a graded material, this can be taken as the d_{35} size (ie, 35 per cent of the material by weight is finer than the d_{35} size)

ν is the kinematic velocity of water (= 1.14 × 10^{-6} m^2/s at 15°C)

U_* (m/s) is the shear velocity of the flow defined by:

$$U_* = \sqrt{\frac{\tau_c}{\rho}} \quad \mathbf{(4.42)}$$

The relationship between F_s and R_{e*} established by Shields can be more conveniently expressed as an equivalent relationship between F_s and the non-dimensional particle size, d_*, given by:

$$d_* = \left[\frac{g}{\nu^2}\left(\frac{\rho_s - \rho}{\rho}\right)\right]^{1/3} d \quad \mathbf{(4.43)}$$

Experimental data for the threshold of movement tend to show a certain amount of variability, as indicated in the above plot of F_s versus d_*; the mean line through the data defines what is commonly termed the Shields curve.

continued on next page...

The shear stress exerted by flow on a sediment bed is made up of two parts: grain shear stress is due to the hydraulic resistance of the individual particles; form shear stress is due to the resistance produced by larger bed features such as dunes and ripples. Shields' analysis was based on data for flat beds so the values of τ_c and U_* should be determined from the value of the grain shear stress. The relationship between the shear velocity and the depth-averaged flow velocity, U (m/s), at the point being considered can be found from the Colebrook-White equation, which for fully developed open channel flow can be written in the form:

$$\frac{U}{U_*} = -\sqrt{32}\,\log_{10}\left(\frac{k_s}{12\,y_o} + \frac{0.222\,\nu}{y_o\,U_*}\right) \qquad \textbf{(4.44)}$$

where y_o (m) is the local flow depth at the point being considered, and k_s (m) is the effective roughness of the sediment bed. For a uniformly graded sediment, $k_s = d$; for a graded material $k_s \approx 2\,d_{50}$, where d_{50} (m) is the mean particle size.

The value of the depth-averaged velocity, U, at the threshold of movement is termed the critical threshold velocity, U_{TC}. The following procedure can be used to find the value of U_{TC}.

1 For the known density and representative size of sediment (see definition of d above), calculate the value of the non-dimensional sediment size, d_*, from Equation (4.43).

2 From the above plot, determine the corresponding value of the Shields entrainment function, F_s, and use Equation (4.40) to find the value of the critical grain shear stress, τ_c (N/m^2), at the threshold of movement.

3 Calculate the corresponding shear velocity, U^* (m/s), from Equation (4.42).

4 Use Equation (4.44) to find the value of depth-averaged flow velocity, U (m/s), corresponding to U_*. At the threshold of movement, $U = U_{TC}$.

It can be seen from the Shields curve that for non-dimensional particle sizes, d_*, greater than 140, the value of the entrainment function at the threshold of movement becomes approximately constant with $F_s \approx 0.056$. It follows that for gravels larger than about 6 mm, Equations (4.40) and (4.42) can be simplified to give the following formula for the shear velocity, U_*, at the threshold of movement:

$$U_* = \sqrt{0.056\,g\left(\frac{\rho_s - \rho}{\rho}\right)d} \qquad \textbf{(4.45)}$$

The corresponding depth-averaged flow velocity U_{TC} can then be found from Equation (4.44).

HANCO (1971)

The critical threshold velocity, U_{TC} (m/s), can be estimated directly from:

$$U_{TC} = a\sqrt{g\left(\frac{\rho_s - \rho}{\rho}\right)d_{90}}\;\left(\frac{y_o}{d_{90}}\right)^{0.2} \qquad \textbf{(4.46)}$$

The coefficient in the equation has a value of a = 1.0 for d_{90} > 0.7 mm, and a = 1.2–1.4 for d_{90} < 0.7 mm.

ACKERS and WHITE (1973)

An alternative formula for estimating U_{TC} is obtained from the Ackers-White sediment transport equation for the limiting condition of zero transport:

$$U_{TC} = \sqrt{32\,g\left(\frac{\rho_s - \rho}{\rho}\right)d_{35}}\;\left[\log_{10}\left(\frac{10\,y_o}{d_{35}}\right)\right]\left[0.23\left\{g\left(\frac{\rho_s - \rho}{\rho}\right)\frac{d_{35}^3}{\nu^2}\right\}^{-1/6} + 0.14\right] \qquad \textbf{(4.47)}$$

6. The point of deepest scour normally occurs near the upstream end of a 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.

7 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.

8 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 (see item 6).

4.3.2 Bridge piers, caissons and cofferdams

Many laboratory studies have been carried out to measure depths of local scour around discrete structures such as bridge piers, caissons and cofferdams. Investigators have established different relationships between the factors affecting the equilibrium depth of scour (see Section 4.3.1), but for this manual the approach originally developed by Breusers *et al* (1977) is preferred as having the best theoretical and experimental basis.

When comparisons are made between measurements of scour at full-size structures in the field and predictions from appropriate laboratory-based formulae, it is usually found that the prototype depths of scour are less than, or at the most equal to, those predicted. This is to be expected for a variety of reasons: the flood may not have lasted long enough for the equilibrium scour depth to have been reached; the flow conditions may not have corresponded exactly to those that would have produced the maximum amount of scour; or the scour holes may have partly filled in again by the time the measurements were taken. The balance of evidence is, therefore, that predictions from appropriate laboratory-based formulae provide an upper-limit estimate of maximum scour depths than can be expected to occur at full-size structures.

The non-dimensional approach developed by Breusers *et al* (1977) has been followed and developed by other researchers such as Melville and Sutherland (1988), May and Willoughby (1990), and Breusers and Raudkivi (1991), and the following design equation combines some of their main results. Comparisons with field data indicate that this general approach provides conservative but reasonable estimates of maximum scour depths at prototype structures. The equilibrium depth of scour, Y_S (m), is measured below the level of the bed upstream of the structure and is primarily related to the horizontal width, H (m), of the structure measured normal to its longitudinal axis; in the case of a circular pier, H is equal to the pier diameter, D (m). The non-dimensional scour ratio, Y_S/H, is related to other characteristics of the flow by the equation:

$$\frac{Y_S}{H} = S_F \,.\, \Phi_{shape} \,.\, \Phi_{depth} \,.\, \Phi_{velocity} \,.\, \Phi_{angle} \qquad \textbf{(4.48)}$$

S_F is a factor of safety that can be applied to best-fit predictions of maximum local scour depth to take account of random variations in scour development and uncertainties in flow conditions. Suitable values of S_F recommended by Johnson (1992) are given in Table 4.2. Each case needs to be considered individually, but a value of S_F = 1.6 has been found to correspond to maximum observed depths of scour in laboratory studies.

Table 4.2 *Values of factor of safety,* S_F

Percentage number of cases in which predicted depth of scour might be expected to be exceeded	S_F
50	1.00
10	1.20
1	1.40
0.1	1.60
0.01	1.75
0.001	1.85

The factor, Φ_{shape}, takes account of the effect that the shape of the structure has on the depth of local scour. The numerical value of Φ_{shape} is equal to the best-fit estimate of the scour ratio, Y_S/H, that will occur in deep water (see following definition) when the depth-averaged flow velocity, U, is equal to or greater than the critical threshold velocity, U_{TC} (see Section 4.3.1), and the longitudinal axis of the structure is parallel to the direction of the approaching flow. Based on information collated by Hoffmans and Verheij (1997), Table 4.3 gives values of Φ_{shape} for different types of prismatic structure that project through the full depth of the flow.

Table 4.3 *Values of shape factor,* Φ_{shape}*, for structures*

Type of structure	Φ_{shape}
Cross-sectional shape in plan	
Circular	1.5
Lenticular	1.0–1.2
Elliptic	0.9–1.2
Square	2.0
Rectangular	1.5–1.8
Rectangular with semi-circular nose	1.35
Rectangular with chamfered corners	1.5
Rectangular nose with wedge-shaped tail	1.3
Rectangular with sharp nose 1:2 to 1:4	1.0–1.15
Cross-sectional shape in elevation	
Pyramid (widest at base)	1.15
Inverted pyramid (narrowest at base)	1.8

Based on data given by Hoffmans and Verheij (1997)

Values of Φ_{shape} and the effective width, H_e (m), of more complex piled structures normally need to be determined from tests with physical models. Some representative results for specific geometries are shown in Figure 4.4 and may be used as an initial guide. The value of H_e should be used in place of H in Equations (4.48), (4.49) and (4.50).

The factor Φ_{depth} takes into account the effect of relative water depth on the depth of scour and can be calculated from the following formula due to May and Willoughby (1990):

$$\Phi_{depth} = 0.55\left(\frac{y_o}{H}\right)^{0.60}, \text{ for } y_o/H \leq 2.7 \qquad \textbf{(4.49a)}$$

and:

$$\Phi_{depth} = 1.0, \text{ for } y_o/H > 2.7 \qquad \textbf{(4.49b)}$$

where y_o (m) is the local water depth upstream of the structure (taking account of any natural scour and local scour). It can be seen from Equation (4.49a) that scour depths around large structures such as caissons and cofferdams can be significantly reduced if the depth of water is relatively small compared with the transverse width of the structure.

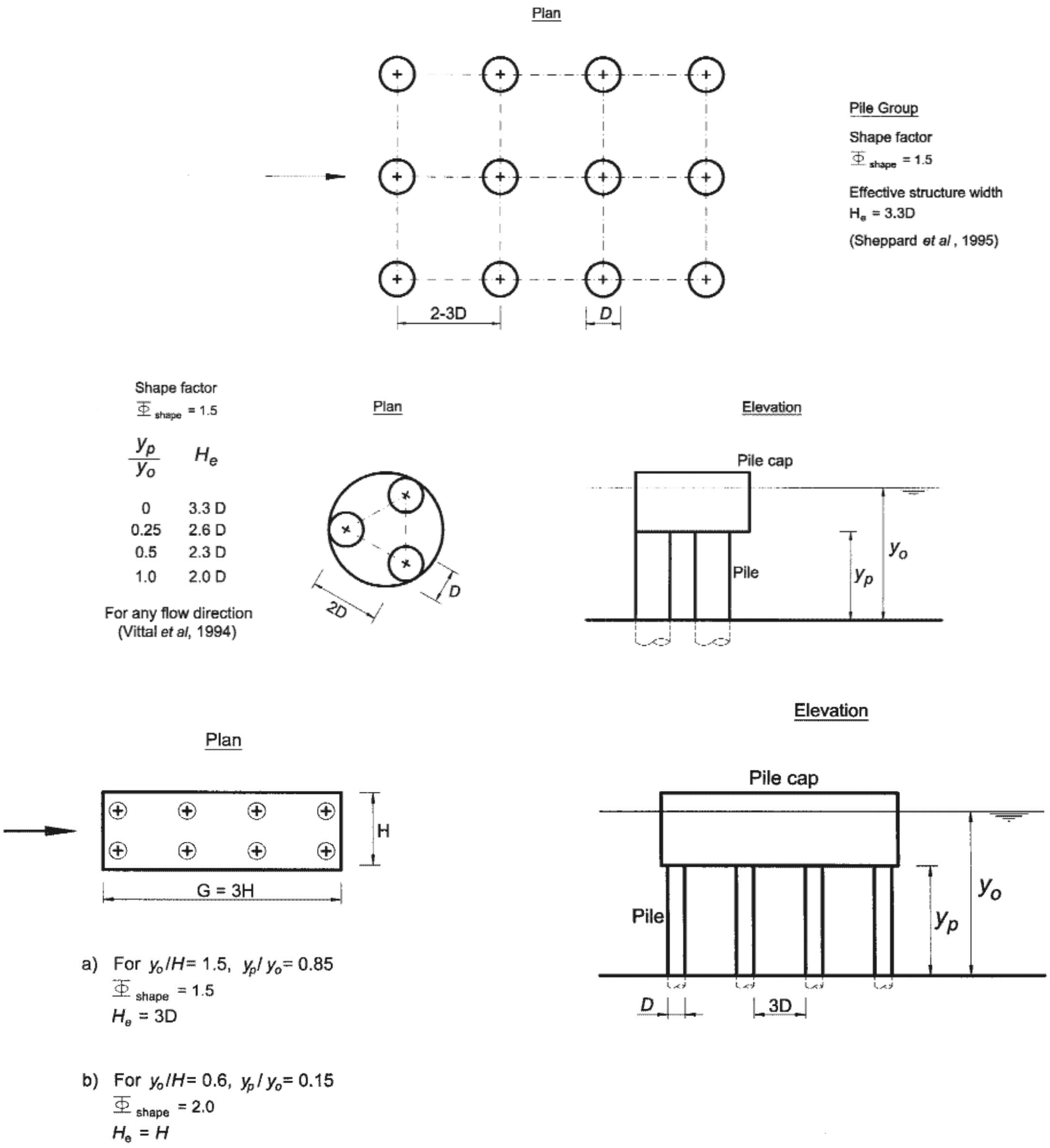


Figure 4.4 *Typical values of shape factor and effective width for local scour at piled structures*

The factor $\Phi_{velocity}$ describes the effect of flow velocity on scour depth and can be determined from:

$$\Phi_{velocity} = 0\,, \quad \text{for } U/U_{TC} \le 0.375 \tag{4.50a}$$

$$\Phi_{velocity} = 1.6\left(\frac{U}{U_{TC}}\right) - 0.6\,, \quad \text{for } 0.375 \le U/U_{TC} \le 1.0 \tag{4.50b}$$

$$\Phi_{velocity} = 1.0\,, \quad \text{for } U/U_{TC} > 1.0 \tag{4.50c}$$

As explained in Section 4.3.1, Equations (4.50a to c) describe the behaviour of uniformly graded sediments and may, in some conditions, overestimate the depth of scour for widely graded sediments. However, an upper limit of $\Phi_{velocity} \approx 0.9$ is likely to be reached with widely graded sediments if the value of U/U_{TC} exceeds about 3–4.

The final factor in Equation (4.48) is Φ_{angle}, which allows for the increase in scour that occurs if the longitudinal axis of the structure is at an angle to the approaching flow. The value of the factor can be calculated from the following formula due to Froehlich (1988):

$$\Phi_{angle} = \left[\cos\alpha + \left(\frac{G}{H}\right)\sin\alpha \right]^{0.62} \qquad \textbf{(4.51)}$$

where G (m) is the length of the structure measured along its longitudinal axis at right angles to the width, H (m), and α is the angle between the longitudinal axis of the structure and the direction of the approaching flow.

The method of applying Equation (4.48) to determine the lowest bed level due to local scour at a structure is explained in the general calculation procedure given at the end of Section 4.3.2.

4.3.3 Abutments

If an abutment of a bridge projects into the main channel spanned by the bridge, the flow passing around the abutment is accelerated and may be capable of causing additional local scour if the bed or banks of the channel are formed of an erodible material. An abutment located on part of the floodplain outside the main channel may also be subject to some localised scour and this should be considered in relation to the existing or planned level of the foundations. In general, the amount of local scour on a floodplain is likely to be much less than in a main channel because the flow velocity and water depth are smaller and also because the floodplain may be protected by grass or other vegetation.

Experimental studies have shown that local scour at abutments exhibits several similarities to local scour at isolated structures such as bridge piers and cofferdams. The depth of scour tends to be proportional to the transverse width of the abutment (that is, the amount by which it projects into the main flow) and also depends on whether the abutment is bluff-bodied or streamlined. Tests carried out by Kandasamy (1989), and reported by Breusers and Raudkivi (1991, Chapter 4), showed that the scour was a maximum when the upstream flow conditions corresponded to the limit of live bed-scour with $U/U_{TC} = 1$ (see Section 4.3.1); for values of U/U_{TC} between 1 and 4, the scour depth varied between about 72 per cent and 88 per cent of the maximum value.

When using formulae to predict scour at abutments, it is important to check whether the estimate is a combined figure that takes account of both the local scour at the structure and the more general scour due to the contraction produced by the abutment. Many of the laboratory studies have considered only simplified geometries, such as abutments projecting into rectangular channels with no adjacent floodplains. It is therefore recommended to use the more general approach in Section 4.2 to determine the amount of contraction scour and then to add a separate allowance for the local scour at the abutment. On this basis, the following modification of a formula developed by Hoffmans (1995), and quoted by Hoffmans and Verheij (1997), is suggested:

$$\frac{Y_S}{H_a} = S_F \cdot \Phi_{abut} \cdot \Omega_{depth} \cdot \Phi_{velocity} \qquad \textbf{(4.52)}$$

where Y_S (m) is the maximum depth of local scour at the abutment measured below the upstream bed level (calculated taking separate account of any natural scour or contraction scour), H_a (m) is the horizontal width of the abutment projecting into the flow and G_a (m) is the length of the abutment in the direction of flow (see Figure 4.5 for some typical geometries). Appropriate values for the factor of safety, S_F, are given in Table 4.2.

Values of the shape factor, Φ_{abut}, for various types of abutment are given in Table 4.4.

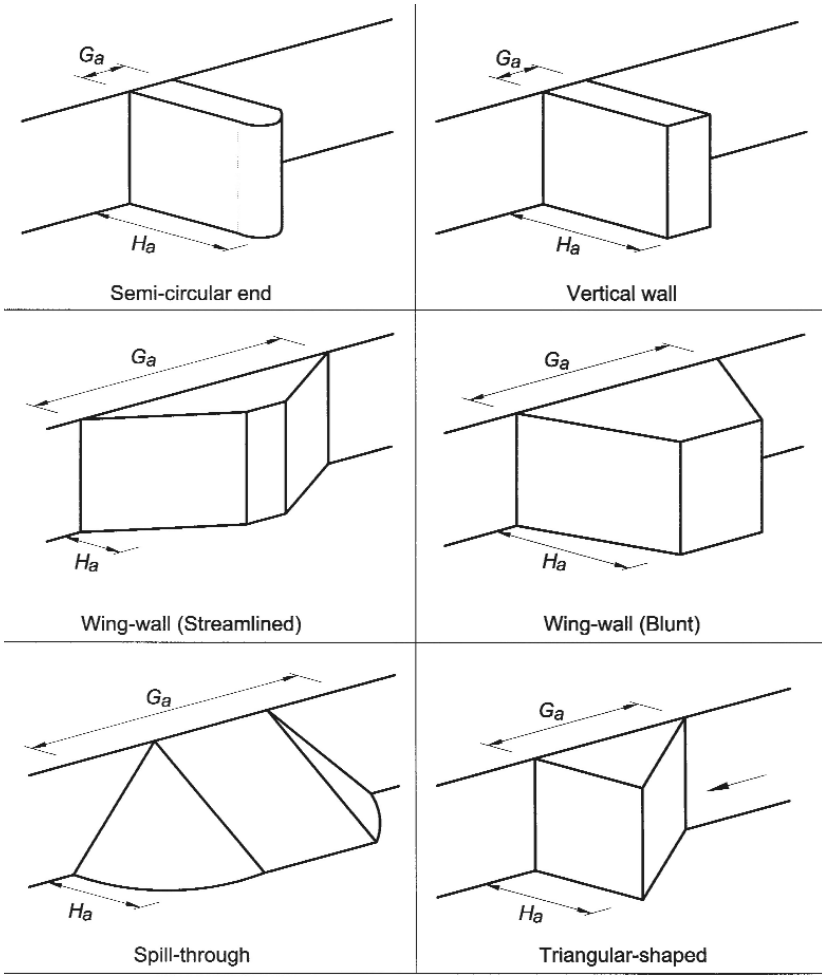


Figure 4.5 *Typical geometries of bridge abutments*

Table 4.4 *Values of shape factor, Φ_{abut}, for abutments*

Type of abutment	H_a/G	Angle of wing wall to line of abutment	Shape factor Φ_{abut}
Semi-circular ends	< 3		1.5
	3–5		2.25
	> 5		3.0
Vertical-wall abutment			3.0
Wing-wall abutment	0.2	45°	0.75
(streamlined)	0.3	35–45°	1.25
Wing-wall abutment	0.5–1.5	30°	1.5
(blunt)	1.5–2.5	30°	2.0
Spill-through abutment			
1.5H:1.0V	0.2		0.75
1.0H:1.0V	0.2		1.0
1.0H:1.0V	0.5–1.5		1.5
Triangular-shaped abutment		45°	1.0

Based on data given by Hoffmans and Verheij (1997)

The depth factor, Φ_{depth}, is given by the hyperbolic tangent function:

$$\Omega_{depth} = \tanh\left(\frac{y_o}{H_a}\right) = \frac{\exp\left(\frac{2y_o}{H_a}\right) - 1}{\exp\left(\frac{2y_o}{H_a}\right) + 1} \tag{4.53}$$

where y_o (m) is the corresponding local flow depth measured from the bed level just upstream of the abutment. If $y_o/H_a > 2$, the depth factor tends closely to a value of $\Phi_{depth} = 1$; if $y_o/H_a < 0.4$, the factor can be approximated satisfactorily by $\Phi_{depth} = y_o/H_a$.

The factor $\Phi_{velocity}$, which takes account of the effect of flow velocity on the depth of local scour at the abutment, is determined from Equations (4.50a to c).

4.3.4 Guide banks and revetments

The construction of longitudinal training works such as guide banks or revetments along a section of river or tidal estuary may result in the depth of scour adjacent to the works being greater than would be the case if the banks were in a natural state. The presence of a revetted bank can have two separate effects on local flow conditions.

First, the bank may alter the magnitude and direction of the flow velocity adjacent to it. This is most apparent at the upstream ends of guide banks which are usually radiused so as to convey flow smoothly through the main bridge opening and also prevent the banks being outflanked by upstream migration of the natural channel. Flow curvature around the rounded nose of a guide bank accelerates the flow and disrupts the protective boundary layer, causing the nose to be subject to more "attacking" flow conditions than are experienced by straight sections of bank. Changes in the slope or surface roughness of a bank may also alter the local flow velocity. Thus, constructing a length of vertical or steeply sloping guide wall with a smooth finish would cause the bed at the toe of the wall to be subject to higher velocities and scour than would have been the case with the original natural bank. Conversely, if a section of natural bank were replaced by a revetment of the same slope but with an armour layer of greater roughness, local flow velocities near the toe of the bank might be reduced.

The second effect that a revetted bank may cause is a change in the level of turbulence within the flow near the bank. If the upstream nose of a guide bank is incorrectly aligned to the approaching flow or is too sharply curved, the flow may separate and produce rotating eddies that are carried downstream; these can increase turbulence levels significantly and lead to enhanced scour. For straight sections of bank, the turbulence level is affected by the surface roughness of the armour layer; if the revetted bank is smoother (and straighter) than the natural bank, the level of turbulence and the amount of scour may be reduced.

It is apparent from this description of the processes involved that a factor such as the surface finish of a revetment can have opposing effects in terms of local scour. On the one hand, a high roughness tends to reduce velocities close to the bank (which is beneficial), but on the other it tends to increase turbulence levels (detrimental). Few systematic studies on changes in scour depth caused by the presence of revetments have been carried out, so it is not possible to quantify the effects of the different factors. Guidelines used by UK engineers tend to be based on Indian data obtained from field measurements of scour at training works in large alluvial rivers; see Chapter VI of the *Manual on river behaviour, management and training* by Varma *et al* (1989) for details. Although this field information is valuable, it is not possible to be certain how much of the observed scour was caused by the revetments and how much by other possible factors such as contraction scour and bend scour; also the applicability of the design recommendations to smaller rivers or to those having different characteristics from the Indian rivers that were surveyed has not been established.

Estimates of scour depth should be made using both methods given below and the larger of the two values assumed for design, unless scour data from the site or from comparable schemes support the alternative estimate.

Recommended procedures

Method 1 – **Joglekar (1971)**

1 For the main channel formed by the guide banks or revetted banks, calculate the flow rate, Q_1 (m^3/s), carried by the channel under the design flood conditions, using steps 1 and 2 in the recommended procedure in Section 4.1.4.

2 Calculate the regime depth, Y_R (m), corresponding to the flow rate Q_1 using the following regime equation due to Lacey (1933), see Box 4.1:

$$Y_R = 0.472 \frac{Q_1^{1/3}}{f^{1/3}} \tag{4.54}$$

3 The depth, Y_{rev} (m), from the water surface to the point of lowest scour adjacent to the revetment or guide bank can be estimated from:

$$Y_{rev} = \sigma_1 \, Y_R \tag{4.55}$$

where the factor σ_1 has the following values:

Type and location of structure	Values of σ_1 Mean	Range
Nose of guide bank	2.75	1.6–3.9
Transition from nose to straight section of guide bank	1.5	1.25–1.75
Straight section of revetment or guide bank	1.25	1.0–1.5

Values of σ_1 towards the upper limit of the appropriate range should be chosen if the conditions are considered to be particularly severe, for example, a sharply radiused nose causing flow separation, or the presence of a steeply sloping wall increasing the local flow velocity.

4 If the elevation of the water surface under the design flood condition is Z_D (m above datum), the lowest scour level, Z_{LS} (m above datum), adjacent to the revetment or guide bank is given by:

$$Z_{LS} = Z_D - Y_{rev} \tag{4.56}$$

Method 2 – not applicable to nose of guide banks.

1 Determine from the recommended procedure of Section 4.1.4 (see steps 1 to 7) the lowest bed level, Z_{min}, to be expected in a natural channel having the same cross-sectional characteristics, and making allowance for any additional contraction, bend or confluence scour (see sections 4.1.5, 4.1.6 and 4.2 respectively).

2 Limited available data indicate that the extra scour produced by the presence of the revetment or guide bank may be in the range of σ_2 = 0.1–0.2 times the flow depth measured from the level Z_{min}. A value of σ_2 towards the upper limit of the range should be chosen if the conditions are considered to be particularly severe, for example, a sharply radiused nose causing flow separation or the presence of a steeply sloping wall increasing the local flow velocity.

3 The lowest scour level, Z_{LS} (m above datum), adjacent to the revetment is determined from:

$$Z_{LS} = Z_{\min} - \sigma_2 \left(Z_D - Z_{\min} \right) \quad \textbf{(4.57)}$$

4.3.5 Spur dikes (groynes)

Spur dikes (or groynes, as they are alternatively termed) are structures constructed transverse to the line of a bank to protect the bank from erosion or as part of a system of training works for stabilising the planform of a river or tidal channel (see Figure 4.6). Spur dikes act either by deflecting high-velocity flow away from an erodible bank or by stabilising the position of a deep-water channel at a certain distance from the bank. The structures may either be impermeable (eg formed with dumped rock or of embankment type with a soil core protected by rock armour) or permeable (eg constructed using timber, steel or concrete piles) so as to allow some flow parallel to the bank, but at a low enough velocity to prevent erosion and/or encourage sediment deposition. Care needs to be exercised in the use of spur dikes to ensure that they do not simply transfer erosion from one location to another, or initiate unforeseen changes in the general channel morphology. Guidelines covering the geometric layout of spur dikes (eg their length, longitudinal spacing and angle relative to the bank) are given by Jansen *et al* (1979), Varma *et al* (1989), Bettess (1990) and Lagasse *et al* (1995).

By acting on the flow around them, spur dikes tend to increase local velocities and turbulence levels in their vicinity. The structure of the dike itself may be liable to erosion; flow moving parallel to the bank is intercepted and accelerates along the upstream face of the dike towards the nose. Depending on the layout of the training works, flow separation at the nose of a dike can set up a strong recirculation in the downstream "shadow" zone that it creates; in certain situations, this reverse flow may be capable of causing erosion along the line of the bank and the downstream face of the dike, and also at the junction between the two (which can sometimes be a particular point of weakness). The high velocities and strong curvature of flow near the nose of a dike can cause significant scouring of the adjacent channel bed. Unless the foundations of the structure are deep enough or are suitably protected, the end section of dike may be undermined by local scour; in the case of dikes of the embankment type, loss of protection at the nose will expose the erodible core and could lead to a progressive failure of the whole structure.

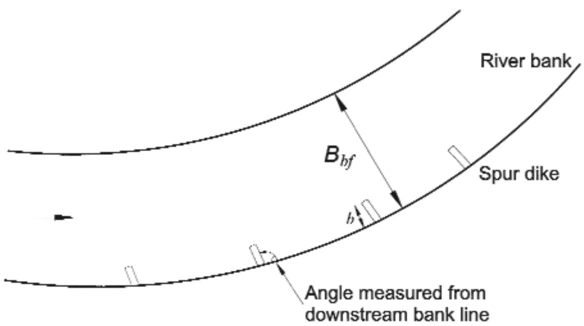


Figure 4.6 *Typical plan layout of spur dikes*

Two alternative methods are given below to estimate the maximum depth of scour at the nose of a spur dike or groyne. The first method is based on an analysis by Inglis (1949) of prototype scour data from alluvial rivers in India and Pakistan. The second method developed by Ahmad (1953), and extended by Breusers in Chapter 4 of Breusers and Raudkivi (1991), takes account of a wider range of factors, but is based on laboratory studies carried out in flumes with rigid vertical walls. Estimates of scour depth should be made using both methods and the larger of the two values assumed for design, unless scour data from the site or from a comparable scheme support the alternative estimate.

Recommended procedures

Method 1 – **Inglis (1949)**

1 For the main incised channel in which the spur dike is assumed to be located, calculate the flow rate, Q_1 (m^3/s), in the channel under the design flood conditions, using steps 1 and 2 of the recommended procedure in Section 4.1.4.

2 Calculate the regime depth, Y_R (m), corresponding to the flow rate Q_1 using the following regime equation due to Lacey (1933), see Box 4.1:

$$Y_R = 0.472 \frac{Q_1^{1/3}}{f^{1/3}} \tag{4.54}$$

3 The depth, Y_{dike} (m), from the water surface to the point of lowest scour at the nose of the spur dike can be estimated from:

$$Y_{dike} = \sigma_3 \, Y_R \tag{4.58}$$

where the factor σ_3 has the following values:

Type of spur dike	Value of σ_3
Straight dike, angled upstream and with short sloping nose (1.5V:1H)	3.8
As above, but with long sloping nose	2.25

4 If the elevation of the water surface under the design flood condition is Z_D (m above datum), the lowest scour level, Z_{LS} (m above datum), at the nose of the dike is given by:

$$Z_{LS} = Z_D - Y_{dike} \tag{4.59}$$

Method 2 **– Ahmad (1953), Breusers and Raudkivi (1991)**

1 For the main incised channel in which the spur dike is assumed to be located, calculate the flow rate, Q_1 (m^3/s), in the channel under the design flood conditions, using steps 1 and 2 of the recommended procedure in Section 4.1.4.

2 Calculate the average discharge, q_1 (m^2/s), per unit width in the contracted section of the main channel at the nose of the spur dike:

$$q_1 = \frac{Q_1}{(B_{bf} - b)} \tag{4.60}$$

where B_{bf} (in m) is the bank-full width of the incised channel and b (m) is the distance that the spur dike projects into the channel measured normal to the line of the bank.

3 The depth, Y_{dike} (m), from the water surface to the point of lowest scour at the nose of the spur dike can be estimated from:

$$Y_{dike} = 2.3 \; \gamma_1 \, \gamma_2 \, \gamma_3 \; q_1^{2/3} \tag{4.61}$$

where the factors γ_1, γ_2 and γ_3 allow for the effects of the following factors:

γ_1 angle of the dike relative to the line of the bank (0° ≡ pointing downstream, 180° ≡ pointing upstream)

γ_2 shape of the dike in section

γ_3 location of the dike in the channel.

Angle of dike to bank	30°	45°	60°	90°	120°	150°
Value of γ_1	0.8	0.9	0.95	1.0	1.05	1.1

Shape of dike (in section)	Value of γ_2
Vertical board	1.0
Narrow vertical wall	1.0
Wall with 45° side slopes	0.85

Location of dike	Value of γ_3
Straight channel	1.0
Concave side of bend	1.1
Convex side of bend	0.8
Downstream part of bend, concave side:	
Sharp bend	1.4
Moderate bend	1.1

4 If the elevation of the water surface under the design flood condition is Z_D (m above datum), the lowest scour level, Z_{LS} (m above datum), at the nose of the guide bank is given by:

$$Z_{LS} = Z_D - Y_{dike} \tag{4.62}$$

4.3.6 Gates and rectangular culverts producing 2-D jets

Gates and rectangular culverts can produce high-velocity horizontal jets that are capable of causing local scour if the bed of the channel downstream of the structure or protective apron is erodible (see Figure 4.7). Experimental studies have shown that the maximum depth of scour is proportional to the thickness of the two-dimensional jet and is also determined by the ratio between the flow velocity in the jet and the critical threshold velocity needed to initiate movement of the sediment in the bed. The depth and shape of the scour hole also vary depending on whether the jet is submerged or unsubmerged by the downstream tailwater level.

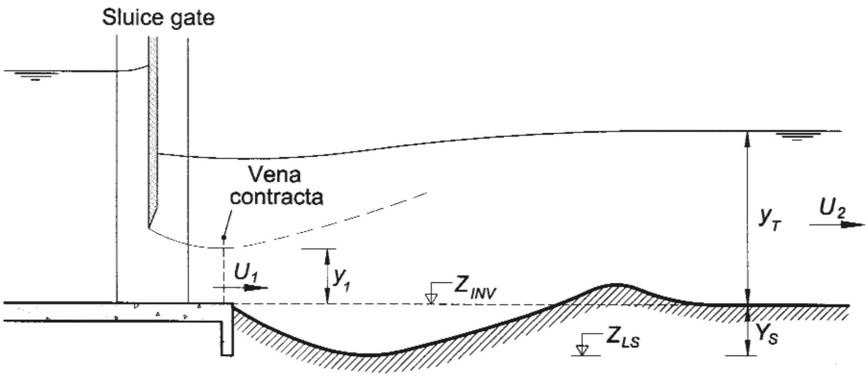


Figure 4.7 *Scour produced by 2-D jet*

Various equations for predicting the dimensions of the scour holes in non-cohesive sediments have been developed, mainly on the basis of laboratory studies, and have been evaluated by Hoffmans and Verheij (1997). The following formula due to

Hoffmans (1997) is suggested for obtaining a preliminary estimate of the maximum depth of local scour, Y_S (m), below the invert of the apron or outlet channel (which is assumed to be at the same level as the unprotected bed downstream):

$$Y_S = \left(\frac{50}{\kappa}\right)\left(1 - \frac{U_2}{U_1}\right) y_1 \qquad \textbf{(4.63)}$$

where y_1 (m) is the vertical thickness of the jet and U_1 (m/s) is the depth-averaged flow velocity within the jet. For flow discharging on to an unprotected bed from a rectangular culvert that is flowing full, y_1 is equal to the height of the culvert barrel. For a vertical sluice gate, y_1 is the thickness of the vena contracta downstream of the gate, which is typically of the order of 0.6 times the vertical opening of the gate; U_1 should also be calculated at the vena contracta. The velocity, U_2 (m/s), is the depth-averaged velocity downstream of the scour hole, as determined by the tailwater conditions in the channel. The non-dimensional scour factor, κ, depends on the d_{90} size (in mm) of the sediment particles in the bed and can be evaluated from the following equations:

$$\kappa = 2.95\, d_{90}^{1/3} \text{ , for } 0.1 \text{ mm} < d_{90} < 12.5 \text{ mm} \qquad \textbf{(4.64a)}$$

$$\kappa = 6.85 \text{, for } d_{90} > 12.5 \text{ mm} \qquad \textbf{(4.64b)}$$

It is assumed that the sediment is non-cohesive and with a density similar to that of sand or gravel.

If the invert level of the culvert or of the outlet channel or apron is Z_{inv} (m above datum), the level of lowest scour, Z_{LS} (m above datum), downstream of the structure is given by:

$$Z_{LS} = Z_{inv} - Y_S \qquad \textbf{(4.65)}$$

In the case of submerged horizontal jets, the overall length of the scour hole (measured from the end of the structure to the point where the level of the bed is equal to the invert level of the apron or outlet channel) is about 5–7 times the value of Y_S. The maximum width of the scour hole is about 3–4 times Y_S. The point of maximum scour depth is located a distance of about $2Y_S$ downstream from the end of the structure; the slope of the upstream face of the scour hole may lead to some undermining of the structure if it is not protected by a cutoff wall projecting below the level of the bed. If a jet is unsubmerged, the scour hole tends to be proportionately longer and shallower. As a guide, Rajaratnam and MacDougall (1983) found that the scour depth produced by an unsubmerged jet was approximately 50 per cent of that for an equivalent submerged jet, with the length of the scour hole being about 25–50 per cent larger.

If the above calculations indicate that local scour could present a serious hazard to a structure, it is recommended to carry out a more detailed analysis using alternative scour formulae such as those detailed by Whittaker and Schleiss (1984), Breusers and Raudkivi (1991) and Hoffmans and Verheij (1997). In the case of major projects or non-standard designs of outlet structure, a mobile-bed physical model may be appropriate to determine the amount and extent of local scour.

4.3.7 Circular and square culverts producing 3-D jets

Similar considerations to those described in Section 4.3.6 apply to the local scour produced by a jet from a circular or square culvert discharging over an unprotected erodible bed (Figure 4.8). In the case of a three-dimensional horizontal jet, an estimate of the maximum depth of scour, Y_S (m), below the invert level of the culvert can be estimated from the following equation due to Ruff *et al* (1982):

$$Y_S = 2.07 D \left(\frac{Q}{\sqrt{g\, D^5}}\right)^{0.45} \qquad \textbf{(4.66)}$$

where D (m) is the diameter of a circular culvert. For arch, square or slightly non-square culverts producing three-dimensional jets, it is recommended to use an equivalent value of D giving the same cross-sectional area as the actual culvert. The level of lowest scour, Z_{LS}, downstream of the structure is found from Equation (4.65), where Z_{inv} (m above datum) is the invert level of the culvert at the outlet.

The overall length of the scour hole produced by a three-dimensional jet is about $7Y_S$ (measured from the end of the structure to the point where the level of the bed is equal to the invert level of the apron or outlet channel) and the overall width of the scour hole is about $5Y_S$. The slope of the upstream face of the hole may lead to some scour undermining the end of the culvert if it is not protected by a cutoff wall projecting below the level of the bed.

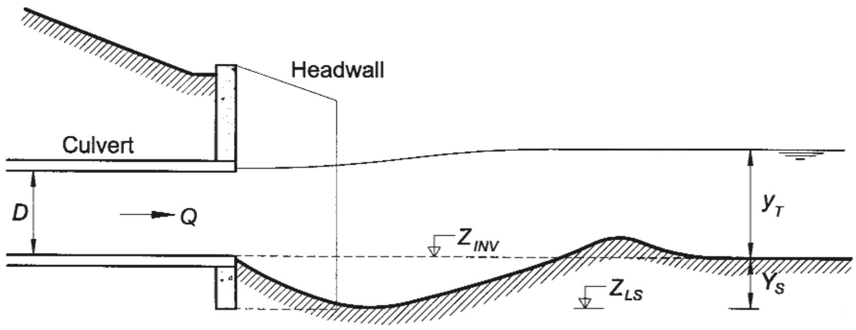


Figure 4.8 *Scour produced by 3-D jet from culvert*

If the above calculations indicate that local scour could present a serious hazard, it is recommended to carry out a more detailed analysis using alternative scour formulae such as those detailed by Breusers and Raudkivi (1991) and Hoffmans and Verheij (1997). In the case of major projects or non-standard arrangements, a mobile-bed physical model may be appropriate to determine the amount and extent of local scour.

4.3.8 Weirs and drop structures producing plunging jets

Similar considerations to those described in Section 4.3.6 apply to the local scour produced by a plunging jet from a weir or drop structure discharging on to an unprotected erodible bed (see Figure 4.9). An estimate of the maximum depth of scour, Y_S (m), below the general bed level can be estimated from the following equation due to Fahlbusch (1994) and Hoffmans (1997):

$$Y_S + y_T = \left(\frac{20}{\kappa}\right)\sqrt{\frac{q\,U_1 \sin\delta}{g}} \qquad \textbf{(4.67)}$$

where y_T (m) is the downstream tailwater depth measured from the unscoured bed level and q (m^2/s) is the flow rate per unit width discharged by the structure. U_1 (m/s) is the average velocity of the plunging jet entering the tailwater and δ is the angle between the jet and the water surface at this point. The scour factor, κ, depends on the d_{90} size (in mm) of the sediment in the bed of the channel and is calculated from Equation (4.64).

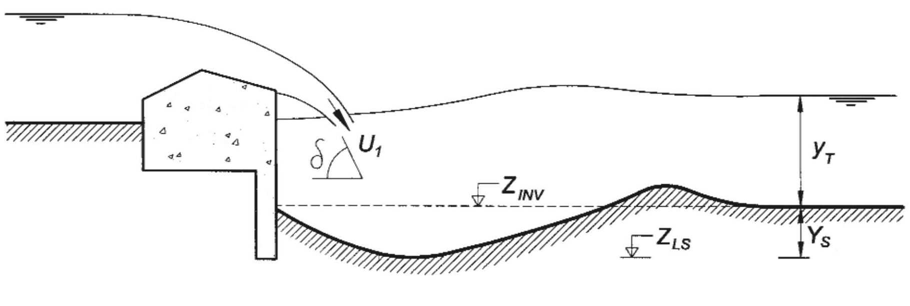


Figure 4.9 *Scour produced by plunging jet*

The tailwater provides a cushion that reduces the erosive effect of the plunging jet, and it can be seen from Equation (4.67) that an increase in y_T produces a corresponding decrease in the scour depth, Y_S. The level of lowest scour, Z_{LS} (m above datum), downstream of the structure is found from Equation (4.65) using Y_S from Equation (4.67), with Z_{inv} (m above datum) being the level of the downstream bed in the absence of local scour.

If the above calculations indicate that local scour could present a serious hazard, it is recommended to carry out a more detailed analysis using alternative scour formulae, such as those detailed by Breusers and Raudkivi (1991) and Hoffmans and Verheij (1997). In the case of major projects or non-standard arrangements, a mobile-bed physical model may be appropriate to determine the amount and extent of local scour.

4.4 LOCAL SCOUR IN TIDAL CONDITIONS

The design recommendations on local scour given in Section 4.3 are based on the assumption that the flood flow conditions last long enough for the scour to reach close to the maximum possible equilibrium values. In many cases, this is a conservative assumption, but there is usually insufficient information about the erosion processes and the patterns of possible floods for it to be safe to rely on a less severe design criterion.

An exception to this general rule occurs in cases of local scour around structures in tidal channels. In this type of situation, the characteristics of the tidal flows can be determined from field measurements or numerical models with reasonable accuracy, and the additional effects of freshwater floods and tidal surges are predictable and often relatively limited. More importantly, the astronomical tides produce regular reversals in flow direction approximately twice every day. As a result, for large structures such as bridge piers, cofferdams and caissons, it is unlikely that the scour hole around the upstream end of the structure will be able to reach the equilibrium values of size and depth within the duration of a single half-tide (about six hours). When the flow direction reverses, scour starts to develop at the opposite end of the structure, with the original hole being partly filled in by the newly eroded material. There is, therefore, a continuing dynamic process, with the scour holes at either end of the structure never reaching the equilibrium conditions that would be achieved with a steady unidirectional flow.

No reliably verified methods of predicting local scour in tidal conditions have yet been developed. However, a laboratory study carried out by Escarameia and May (1999) indicates that the key factor is the ratio between the duration of the tidal cycle and the "half-life" of the scouring process, defined as the time that would be taken for the scour hole to reach half the equilibrium scour depth occurring with an equivalent unidirectional flow. As the value of this ratio increases, the scour occurring under tidal conditions approaches closer to the maximum equilibrium value. Factors that increase the characteristic "half-life" of the scouring process are a reduction in the flow velocity and an increase in the size of the structure. Data from the laboratory study and preliminary field measurements suggest that maximum scour depths occurring around bridge piers in tidal conditions may typically be of the order of 50–75 per cent of the maximum equilibrium values for cases where the peak tidal velocity is approximately equal to the critical threshold velocity for movement of the bed sediment.

4.5 PIPELINE CROSSINGS

Three options for pipeline crossings of rivers and tidal channels can be considered.

The first is to support the pipeline on piers (or another structure such as a bridge), so that it is above the highest predicted flood level, with an additional freeboard allowance for waves and floating debris. In this case, scour levels at the piers should be determined using the information given in Section 4.3.

The second option is to lay the pipe on the bed of the channel or in a shallow trench and to protect it from scour by means of riprap or another suitable protection system. During flood conditions, the armour layer above the pipe may become exposed and effectively form a bar or sill across the channel. The possible effects of the bar on the morphology of the river or tidal channel need to be carefully considered.

The third option is to lay the pipe sufficiently below the normal bed of the channel for it not be exposed to flow by natural scour occurring during floods up to and including the design condition. Since the presence of the pipeline will not normally change the properties or cross-sectional shape of the channel, the lowest design scour level can be determined using the estimation methods for natural scour given in Section 4.1.

4.6 CLOSURES IN FLUVIAL AND TIDAL CHANNELS

During the construction of barrages or control structures across rivers or tidal channels, the available flow width may be gradually reduced by the construction of cofferdams and embankments, or by the placing of caissons. Flow velocities through the remaining opening or openings are likely to be considerably increased before complete closure is achieved, and may give rise to significantly increased depths of scour in the channels and around the structures. Each stage of the closure process should be carefully analysed to determine the most critical conditions and to determine whether suitable scour protection measures are necessary. Depths of scour can be estimated using the calculation methods for natural, contraction and local scour given in Sections 4.1, 4.2 and 4.3 respectively.

4.7 ASSESSMENT OF EXISTING STRUCTURES

4.7.1 Existing assessment procedures

Most of the major bridges in the UK are the responsibility of two bodies, the Highways Agency and Railtrack. Both organisations have developed procedures for assessing existing structures. Although these procedures are available to practitioners in the industry, they have not been published, so (with the permission of the Highways Agency and Railtrack) summaries of the scope and philosophy of the procedures are provided below.

For the sake of completeness, the procedure normally adopted in the USA, which appears in *HEC18* (Richardson and Davis, 1995), is also summarised below.

Highways Agency procedure

The Highways Agency advice note (*Assessment of scour at highway bridges*, Binnie Black & Veatch, 1998) is intended to provide a means of assessing the potential for scour to damage an individual bridge. The outcome of the procedure is a “priority rating”, which takes account of the importance of the bridge as well as the assessment of scour in relation to the foundation conditions. The methodology includes relatively simple assessments of the potential amount of scour, but does not provide a quantitative measure of the risk of failure. The two-stage assessment comprises:

Stage 1 Collection of data, site inspection and preliminary inspection. If there are features that make the scour risks very low, then the assessment is complete; otherwise it proceeds to Stage 2.

Stage 2 Calculation of potential scour depth taking account of the estimated flood magnitude and the hydraulic conditions at the bridge; comparison with foundation depth etc and determination of the priority rating.

The advice note is intended as a stand-alone document, giving sufficient information to allow the assessment to be carried out largely without reference to other documents, the exceptions being the relevant hydrological data, Ordnance Survey mapping and (for tidal sites) the tide tables. Accordingly, much of the contents are not directly related to the assessment of scour. In the interests of clarity, little theoretical background is given and alternative methods are not compared. The Stage 2 procedure may be summarised as:

- estimation of the magnitude of the flood of 200 years return period
- calculation of the corresponding flow depths and velocities
- determination of the potential depths of scour adjacent to the piers and abutments
- calculation of the priority rating.

The advice note gives numerate consideration only to contraction and local scour, although the procedure requires the assessor to look for indications of general degradation. The average contraction scour is calculated from a graph of competent velocities (taken from Neill, 1973) and then factored to give the maximum in the cross-section. Local scour for bridge piers is based on the work of Melville and Sutherland (1988), while local scour at abutments is based on *HEC18* (Richardson and Davis, 1995).

The advice note includes detailed step-by-step procedures, together with standard reporting and calculation forms and a worked example.

A trial of the procedures was undertaken by Jeremy Benn Associates (2000) on a selection of 20 bridges, using teams with varying levels of experience. It was found that the advice note is effective in "identifying those structures most prone to scour and hence requiring further investigation. It is not, and was not intended to be, a full scour risk assessment." Recommendations resulting from the study included:

- the advice note should only be applied as part of a wider scour management strategy, covering such issues as the identification of bridges for assessment, a method of assessing the risk of damage or failure and guidance on scour countermeasures
- computerisation of the assessment procedure
- consultations with other bridge owners.

Railtrack procedure

The Railtrack procedure (HR Wallingford, 1993) is a development from an earlier report for British Rail (*Handbook No 47*, HR, 1989) and includes:

- discussion of possible causes of failure
- guidelines to assess individual structures with respect to scour, based on bridge geometry and river and catchment characteristics
- recommendations concerning data to be recorded and maintained for each bridge.

The guidelines are intended to allow a bridge to be inspected and assessed quickly, without using special equipment. Quantitative methods, covering contraction and local scour, are given only for bridge piers and abutments projecting into the main channel, although guidance is also given on factors affecting features in the floodplain and embankments. The general procedure is covered by nine steps.

1 Identify the elements subject to hydraulic action.

2 Collect data, inspect site and take measurements if not available.

3 Review history and consider any special factors.

4 Classify the river, based on type of river, bank stability and flashiness.

5 Categorise elements of bridge.

6 For piers and abutments in the main channel, calculate scores for features, which are combined to give a "risk number", calculate scour depth and "priority rating".

7 Modify priority rating according to river type, foundation and bank materials, and any existing protection.

8 For features outside the main channel, carry out qualitative assessment.

9 Decide on further action, depending on priority rating.

Little theoretical background is given and the sources of the relationships used are not quoted. The report includes detailed step-by-step procedures, calculation forms and worked examples. The authors expected that the priority scoring might require revision after calibration against a number of test cases.

United States procedure

There has been a great deal of interest in the subject of bridge scour in the USA (see, for example, Richardson and Lagasse, 1999) probably as a result of the wide range of climatic and morphological conditions, and the greater number of larger and more mobile rivers than found in most other countries, including the UK. The development of scour evaluation procedures in the USA has been sponsored mainly by the Federal Highway Administration.

HEC18 Evaluating scour at bridges (Richardson and Davis, 1995) is the third edition of this important document, the first edition which appeared in 1991 having superseded a previous publication by the US Department of Transportation (1988). As well as covering the evaluation of existing bridges, *HEC18* contains detailed descriptions of scour phenomena and presents alternative calculation methods, so has acted as a valuable source in preparing this manual. Two chapters are specifically devoted to the assessment of existing bridges, from the strategic consideration of the overall stock to the evaluation of an individual bridge, in five principal steps.

1 Screening all bridges and placing them into five categories (low risk, scour-susceptible, scour-critical, unknown foundations and tidal).

2 Preliminary office and field study to prioritise the scour-susceptible bridges and those with unknown foundations.

3 Scour evaluations of bridges in the step 2 prioritised list, leading to entries in the bridge inventory. These evaluations select those bridges that should be redefined as "scour-critical".

4 For scour-critical bridges define an action plan.

5 For the remaining bridges on the inventory, prioritise scour evaluations according to the importance of the bridges.

Detailed guidance is given for the screening and prioritisation process, bridge inspections and evaluations, including an inspection checklist and a coding system for recording the findings in the national "structure inventory and appraisal" database.

4.7.2 Scour risk management system

Existing scour assessment procedures generally include a preliminary screening stage, to prioritise structures needing more detailed appraisal, which itself requires some data to be collected and may include a preliminary site appraisal. After the scour appraisals have been made, the findings need to be followed up with action, which may include implementation of remedial measures, monitoring and/or further more detailed studies.

Table 4.5 *Proposed scour risk management strategy*

Stage	Aim	Steps
1 Compile asset register	To establish inventory of all assets that could be at risk of scour	Identify all structures potentially in the influence of water (either in the main channel or in areas that may flood) Include structures that may be affected if assets (such as flood banks) owned by others were to fail or be overtopped Carry out preliminary screening of flood potential on each structure using historic flood data and structure dimensions, placing structures into appropriate preliminary categories as they are entered into the register Exclude from the scour risk register those structures not considered to be likely to be affected by water
2 Initial scour assessment	To quantify the potential depth of scour at each structure and compare this with the level of protection provided	Estimate flood flows up to 200 years return period Estimate potential scour depths If foundation depth is unknown, estimate from bridges of similar construction, if reasonable to do so Categorise structure as high risk if potential scour depth is greater than or similar to foundation depth or if insufficient data are available for the assessment and proceed toStage 3; otherwise categorise structure as medium or low risk and proceed to Stage 4.
3 Detailed scour assessment	For those structures considered most at risk, carry out a more detailed scour assessment and implement protection measures as needed	Detailed survey to establish present conditions, including foundation depth Detailed appraisal of potential scour depth Confirm or revise risk categorisation of structure and determine if additional scour protection works are required Design and implement scour protection measures and/or monitoring as necessary
4 Long-term management	To ensure that any changes to the asset or developments which affect it are monitored and their effect on scour risk assessed	Monitor structures according to Stage 2 and Stage 3 assessments Prepare and implement a programme of regular reassessments of structures Implement procedures to monitor and assess the effects of modifications to the river and catchment, include procedures to add new or newly affected structures to the scour risk register

Based on the suggestions of Whitbread *et al* (2000)

The overall process can be considered as comprising a "scour management system", which would normally form part of the owner's overall asset management system. Whitbread *et al* (2000) noted that "little formal guidance has been proposed for bridge owners wishing to manage their bridge assets in an effective and safe manner" and suggested that an effective management system should:

- "recognise that all bridges over watercourses are potentially at risk of scour
- make an assessment of the level of risk
- ensure that appropriate action is taken to accommodate the cost of damages."

The system would not guarantee zero risk of scour failures, but would demonstrate that the owner had fulfilled the duty of care expected. Based on their experience of scour assessment programmes, Whitbread *et al* (2000) proposed a four-stage scour assessment strategy, which is broadly endorsed by the present manual and presented (with some minor revisions to generalise the scope) in Table 4.5. Stages 1, 2 and 4 would be mainly (but not entirely) desk-based studies, making use of existing data. The Stage 3 studies should be undertaken by specialist river engineers, with specialist structural advice. The Stage 2 studies would also benefit from similar specialist inputs.

It is believed that the above overall strategy would be compatible with the current Highways Agency and Railtrack guidance documents.

4.7.3 Scour appraisal procedure

Screening

Screening, which forms part of Stage 1 of the strategy outlined in Section 4.7.2, is the first part of the scour appraisal process. The categories can be varied to suit the owner's needs and any legal requirements, but at a minimum would simply be that the structure is either potentially affected by water during floods or is very unlikely to be so affected. The categorisation suggested in *HEC18* (Richardson and Davis, 1995), which comprises the following four non-tidal categories, appears to have much to commend it:

- low risk
- scour-susceptible
- scour-critical
- unknown foundations.

A checklist of the features that are likely to result in structures being categorised as "scour-susceptible" or "scour-critical", is given as Table 4.6.

Table 4.6 *Checklist for the vulnerability of bridges and other structures to the risk of scour*

Main feature	Factors
Existing or historic scour	• Evidence of current scour or a history of scour, as identified from experience, maintenance records, bridge inspections or anecdotal data
Erodible channel bed, plus structure features which make it vulnerable	• Structure foundations comprise spread footings or short piles • Superstructure vulnerable to collapse in event of foundation movement • Bridges with inadequate flood openings or vulnerable to collection of debris and ice • Other structures that form an excessive obstruction to flow
Mobile river	• Active degradation and aggradation processes • Significant lateral movement or erosion of banks • Located in a delta area • Steep slopes and high flow velocities • Sand/gravel mining from or near the river • A history of floods damaging riparian structures, roads etc
Adverse flow characteristics	• Proximity of a confluence • At or near a bend

Based on the approach in *HEC18* (Richardson and Davis, 1995)

Prioritisation

The primary purpose of the screening process is to decide which structures are to be placed in the register, excluding only those that can be confidently stated to be unaffected by water and scour risks, even during severe floods. Extending the process to give a preliminary appraisal of scour-susceptibility, as in Table 4.6, forms a useful prelude to Stage 2 of the risk management strategy in Table 4.5. This preliminary appraisal may provide the basis for a "fast-track" to Stage 3 for structures which are immediately seen to warrant a detailed scour appraisal, for example "scour-critical" structures, whereas "scour-susceptible" and "unknown foundations" structures would first proceed to Stage 2.

Prioritisation procedures may also be used in order to determine the sequence for carrying out the Stage 2 and Stage 3 assessments. Suitable scoring systems could be devised to suit the requirements of the owner, taking account of such issues as:

- the likelihood of a collapse in a major flood
- whether collapse would be progressive or sudden
- the potential danger to the public
- for a bridge or other infrastructure asset, its local, regional and national importance, taking account of the existence of alternative routes.

A key consideration in devising a prioritisation procedure is that it should be rapid to implement, so that it does not unduly divert resources from the scour appraisals themselves.

Table 4.7 *Checklist of information to be gathered and reviewed as part of the initial data review*

Data	Issues to be considered
Previous appraisals etc	• Has an engineering evaluation of scour previously been made? If so, was the structure considered vulnerable to scour, what measures were taken for monitoring, remedial measures etc?
Construction records (including any site investigations and any remedial works records)	• Adequacy of information regarding structure dimensions etc • Type of foundation and associated vulnerability to scour • Any evidence of nature of riverbed material
Maps and aerial photographs	• What do historic maps and aerial photographs tell us about the stability of the river channel and past changes to the structure?
Riverbed surveys	• Have riverbed surveys been undertaken? • Do the surveys show evidence of changes in riverbed level or movement of the river channel since construction or between successive surveys, giving indications of aggradation, degradation and channel instability? • Any evidence of scour holes around piers and abutments?
Planning and preparations for field appraisal and inspections	• Scope required, bearing in mind result of screening • Any particular matters to look out for (such as condition of previous remedial measures, aggravation of previously noted problems)? • Boat needed? • Equipment needed for survey (waders, surveyor's level or theodolite, staff, tapes, rods, poles, sounding lines, sonar etc)? • Does the structure require a survey (to make good any shortcomings in the construction records)? • Underwater inspection needed (either for condition of structure or depth of scour)? • For tidal sites, determine suitable timing in relation to tidal flow conditions • Consider timing of surveying and underwater inspections in relation to engineer's field appraisal

Initial data review

This initial review of the data available for the structure is intended to take place in the office, with the purpose of establishing what data are available, what preliminary impressions can be gained from them and what additional data are needed in order to carry out the scour assessment. The data review helps direct the field appraisal, ensuring that the additional data which will be needed from the field appraisal – and from land-based, water-based and underwater inspections of the structure – are identified in a timely and effective manner, minimising the call for return visits and abortive work.

Table 4.7 contains a checklist of the information that should be gathered and reviewed before making the detailed field appraisal and conducting the structural inspection and scour assessment. In cases including a detailed prioritisation procedure, much of these data would have already been gathered and reviewed as part of that procedure.

Field appraisals of scour

The field appraisal of scour is primarily concerned with looking for evidence of scour and reviewing features of the channel and its relationship to the layout of the structure that affect the potential for scour. A record must be made of the present conditions, preferably including photographs, for comparison with previous and future conditions. Matters concerned with the inspection of the condition of the structure are covered in the next chapter.

Table 4.8 presents a checklist of the matters that should be considered in the field appraisal of scour. A survey of the structure (if needed to make up shortcomings in the construction records) may be carried out as part of the field appraisal, or undertaken in advance and the results made available. The scope of the field appraisal and associated inspections may be curtailed in the case of structures that are to be subject to an initial scour assessment (Table 4.5, Stage 2), with the full field appraisals and inspections only undertaken for detailed scour assessments (Table 4.5, Stage 3).

Structural inspections

The structural inspection is intended to provide a record of present conditions and identify features of its design that may render the structure particularly vulnerable to scour, and also to look for signs of distress or damage that may indicate it has been affected by existing or past scour. Table 4.9 provides a checklist of issues to be considered.

Underwater inspections

Some of the most important matters for the field appraisal of scour is to establish the present cross-section of the river channel in relation to the location and elevations of the structure, the presence of existing scour and the condition of any existing scour protection measures. If any of these cannot be established by observations and measurements made from the banks, riverbed and superstructure, perhaps due to excessive water depth, the use of divers to make an underwater inspection must be considered. Table 4.10 provides a checklist for underwater inspections. Where conditions allow, photographs should be taken.

Table 4.8 *Checklist for field appraisals of scour*

Location	Evidence to be looked for
Upstream conditions	
Channel banks	• Stability indicated by undisturbed natural vegetation, intact bank stabilisation measures etc
	• Instability indicated by bank sloughing, undermining etc
Main channel	• Clear and open with good approach conditions, or meandering or braided with adverse approach direction?
	• Presence of islands, bars, debris, fences etc that may affect flow
	• Aggrading or degrading riverbed or migrating channel
Floodplain	• For bridges, presence and condition of relief arches or other bypass route for floodplain flows
	• Developments on floodplain that may affect flows
Other	• Debris
	• Upstream features that may affect flow conditions
Conditions at structure	
Piers, abutments etc	• Scour
	• Debris
Pile caps, footings etc	• Exposure by scour
Superstructure (eg bridge deck)	• Likelihood of inundation by floodwater
	• Whether it would obstruct flood flows and be vulnerable to debris accumulation
Channel protection etc	• Integrity of riprap, guide banks etc
Flow cross-section (for bridges)	• Does the flow cross-section appear adequate in relation to the upstream channel and floodplain?
Downstream conditions	
Channel banks	• Stability indicated by undisturbed natural vegetation, intact bank stabilisation measures etc
	• Instability indicated by bank sloughing, undermining etc
Main channel	• Clear and open with good "getaway" conditions, or with features that retard the flow?
	• Aggrading or degrading riverbed or migrating channel (degradation could increase scour risks in the future)
Other	• Downstream features that may lower tailwater levels and increase velocities

Condensed from *HEC18*, Table 15 (Davis and Richardson, 1995)

Table 4.9 *Checklist for structural inspections*

Feature	Evidence to be looked for
General	• Settlement (look for discontinuities in line and level, cracking and spalling)
	• Section loss, abrasion from bedload etc
	• Collision damage
	• Corrosion
Piers, abutments, wingwalls	• Rotational movement (check with plumb line)
	• Relative movements, including against superstructure
Superstructure	• Evidence of misalignment or movement relative to supports
	• Vulnerability to crushing or loss of support from relative movement
	• Vulnerability to displacement by water or debris loads, including impact
Foundations	• Damage to scour protection measures
	• Exposure or threat of exposure of footings, piles etc

Adapted from guidance in *HEC18* (Davis and Richardson, 1995)

Table 4.10 *Checklist for underwater inspections*

Feature	Evidence to be looked for
River cross-section at structure	• Changes since construction or since previous inspection • Aggradation or degradation • Contraction scour
Around piers and abutments	• Existing scour holes (depth and shape) • Refilled scour holes
Foundations	• Exposure of footings, pile caps and piles • Any damage, abrasion etc
Scour protection measures	• Any movement or loss of riprap, gabions etc

Adapted from guidance in *HEC18* (Davis and Richardson, 1995)

Scour assessment

Once all the available information, including the construction records, previous studies, field appraisals and inspection reports, has been assembled, the scour assessment can be undertaken. If the staged assessment approach given in Table 4.5 is being followed, the scour assessment would be either an initial (Stage 2) assessment or a detailed (Stage 3) assessment.

The calculations needed to estimate the potential total scour, in either case, are described earlier in Chapter 4. If the actual scour (where known) approaches or exceeds the computed potential total scour, the available information should be reviewed to explore the opportunity for refining the estimates through back-analysis and calibration against actual floods. Table 4.11 provides a generalised procedure for detailed scour assessments.

Table 4.11 *Generalised procedure for detailed scour assessments*

Subject	Steps involved
Hydrology and hydraulics	• Estimate design flood of required return period • Determine values of any lesser floods for which scour estimates are also required (remember that lesser discharges may sometimes result in more severe scour conditions) • Estimate flow conditions in general river section in vicinity of structure, taking no account of the structure, but allowing for any downstream control etc
Scour	• Estimate natural scour • Estimate contraction scour and distribution across channel width • Estimate local scour • Compute total scour and plot on channel cross-section • Compare computed total scour with actual scour and reassess calculations accordingly
Foundations	• Determine foundation depths/profiles from best available data and plot on cross-section for comparison with total scour
Scour potential	• Categorise structure as "high", "medium" or "low" priority for action, according to ratio between foundation depth and computed scour depth

5 Scour protection

5.1 DESIGN PHILOSOPHY

Scour protection is incorporated in a structure to make it less vulnerable to failure or damage by scour. Figure 5.1 shows some of the consequences of scour damage. The measures taken may be incorporated at the design stage, as an integral part of a new structure, or they may take the form of remedial works when scour problems arise or are anticipated from scour assessments. Scour protection can be divided into three main categories:

- scour reduction measures
- structural measures
- scour protection measures.

Scour reduction measures aim to improve flow conditions at a structure to reduce the magnitude and effects of scour. The measures used include suitable location of the structure and its structural elements and streamlining individual components such as piers. Because these measures actively reduce scour, rather than simply protecting a structure against it, they should always be considered first in the design process and then revisited during the design of other protection measures. Scour reduction measures are discussed in Section 5.2.

Structural measures involve designing the structure's foundations to withstand the expected depths of scour and the consequent additional forces on the structure. In general, structural measures are the safest and most reliable way to ensure that scour does not threaten a structure's integrity. For bridge scour, experience of actual flood damage has shown that generally it is more cost-effective to provide a foundation that will not fail than to use other protection measures. Structural measures are discussed in Section 5.3.

Scour protection measures limit the extent to which scour can occur, by providing a more robust and erosion-resistant surface (for example rock, concrete or gabions) than the existing bed or bank material. Scour protection measures are discussed in Section 5.4.

A fourth category of scour protection is monitoring, followed by remedial works when needed, and perhaps closure of the structure during periods of high flows. This category is generally only appropriate for existing structures where active or immediate remedial works cannot be justified on safety or cost grounds.

The choice and design of scour protection measures are usually iterative, as they depend, among other things, on the structural design of the structure, the costs and benefits of the measures, and the probability and consequences of failure. For these reasons it is essential that scour protection be considered as an issue at the conceptual design stage. Decisions taken then, particularly concerning structural aspects, can reduce the options for scour protection at a later stage, leading to designs that are possibly less safe and more expensive. Collaboration at an early stage between the structural, geotechnical and hydraulic engineers is therefore important.

Figure 5.1 *Effects of scour damage (photographs courtesy of Czech Railways Infrastructure Division)*

For major structures, an economic analysis and risk analysis of scour protection options should be carried out. For smaller structures, it is often worthwhile carrying out simplified analyses of economics and risk so that different options can be compared and the adoption of specific protection measures justified. Sections 6.3 and 6.4 provide guidance on risk assessment and economic analysis techniques that are suitable for different scales of structure. When several existing structures are owned by the same operating authority, a system of scour assessment, identification of scour protection works and prioritisation is recommended as a procedure within an economic and risk framework. Experience of this type of scour assessment in the UK from the two major bridge asset owners, Railtrack and the Highways Agency, is discussed in Section 4.7, together with with US practice.

When considering the need for scour protection, it is important not to overlook structures that are not normally in contact with a river or other channel. Structures such as access tunnels through road embankments, for example, may be at risk from scour from flows during flood conditions.

Scour protection should not be considered an element that can be neglected or forgotten during the life of a structure. Most scour protection measures require inspection and maintenance throughout their life, and this should be taken into account at the design stage and also while developing and preparing operation and maintenance manuals and procedures. It is important that bridges, for example, are inspected and assessed for the risk of scour throughout their life, as the risks can change and develop over long as well as short timescales, in response to changes in channel alignment, riverbed degradation or the construction of other structures that may affect scour.

It can be easy to overlook or forget an aspect of scour protection design that may affect the success of the design. To reduce this risk, a checklist is provided at the end of this chapter, summarising the items that should be considered during design (Box 5.2).

Box 5.1 *Scour protection key points*

1. Scour protection can be divided into three main categories:
   - scour reduction measures
   - structural measures
   - scour protection measures

   Monitoring (and reaction) can be considered as a fourth category for existing structures.
2. Scour reduction measures are important, as they are the only ones that actively reduce scour.
3. Structural measures are often the most reliable and economic methods for protecting against scour.
4. The choice of measure is iterative and a combination of measures is often an appropriate solution.
5. Consideration of scour protection early in the structural design process is essential.
6. Economic analysis and risk analysis can be used as tools in deciding which measures to use.
7. Inspection, monitoring and maintenance of scour protection measures over the life of the structure need to be considered and incorporated in the operation and maintenance (O&M) practices for the structure.
8. Structures away from the normal course of a river may be at risk during floods and require scour protection.
9. A checklist of design considerations is provided as an aide-memoire at the end of this chapter (Box 5.2).

5.2 SCOUR REDUCTION MEASURES

Probably the best measures for avoiding scour problems, and often the most cost-effective, are those that reduce the factors that give rise to the scour. By carefully considering the hydraulic conditions that will prevail at the structure over the long term, it is often possible to ensure good flow conditions at the structure throughout its life. Improving flow conditions leads to less turbulence and hence less scour. A common cause of scour is river instability that changes flow conditions at a structure from those prevailing at the time of construction. It is outside the scope of this manual to provide detailed methods for studying river stability and designing measures to reduce instability. Instead, the principal factors affecting river stability are discussed in this section. The main types of scour reduction measure are described, and references given to more specific literature for detailed design. Many of the topics described in this section are covered in more detail in *Stream stability at highway structures* (Lagasse *et al*, 1995) and *River training techniques* (Przedwojski *et al*, 1995).

Examples of potential scour reduction measures are given in Table 5.1. It is recommended that these issues be reviewed before and during the design of other scour protection measures.

Table 5.1 *Issues to consider and measures to reduce scour*

Type	Issues to consider and measures to reduce scour
Location	• avoid locating structures at a confluence • avoid locating structures at a bend • consider the stability of the channel vertically and laterally • check channel stability using aerial and satellite photography, historic maps • consider river morphology
Hydraulic design	• size of waterway opening • size and number of relief openings • overtopping of approach embankments • channel improvements
Streamlining structural elements	• sloping abutments cause less scour than vertical abutments • pier groups • pier shapes • overall structure alignment and alignment of elements
River training	• transverse – groynes • longitudinal – river training • vertical (bed elevation control) – sills or weirs
Deflectors	• vanes and sacrificial piles • debris and ice deflectors • guide banks

5.2.1 Location

Experience suggests that most scour problems arise from two causes. The first of these is lack of consideration during design of the alignment of the structure in relation to flow conditions. The second cause is movement of the river channel laterally or vertically during the life of the structure. Figure 5.2 highlights the way that lateral movement of a channel can undermine infrastructure. It is therefore essential that the location of the structure and the long-term stability of the river are considered in sufficient detail during design. The design of a structure should take account of long-term morphological effects and, where possible, optimise hydraulic conditions to reduce scour. This applies even to the relatively stable rivers encountered in the UK. The location of structures is often dictated by factors other than hydraulics. Nevertheless, relatively small changes in location can significantly reduce the incidence of scour. Conversely, quite small changes in the river laterally can significantly increase scour at poorly located structures.

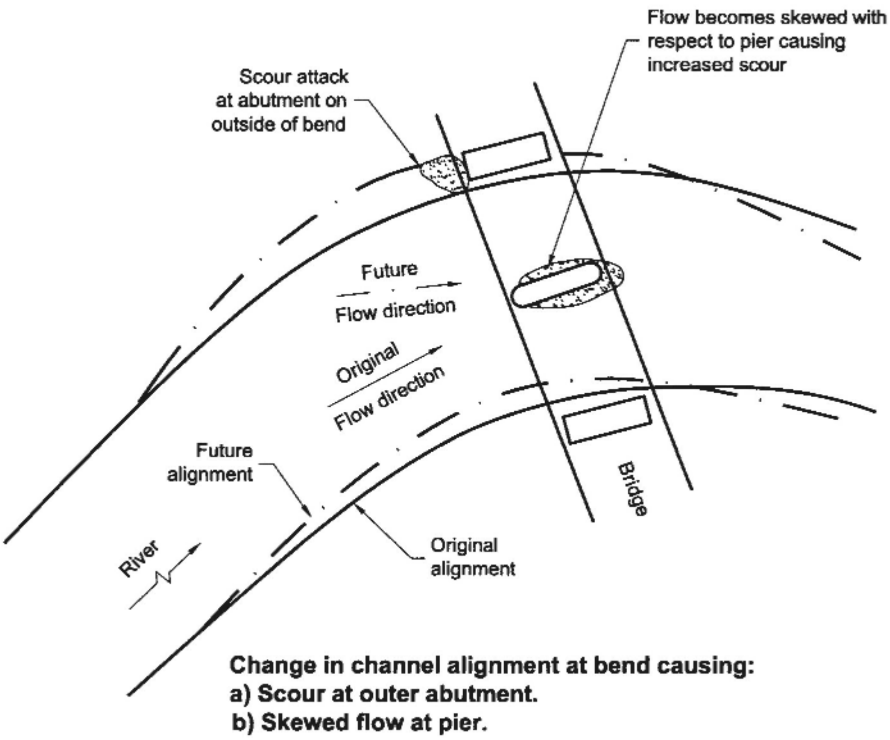
Figure 5.2 *Road undermined by lateral erosion from an upland wadi*

Structures located at or near sharp bends often experience scour problems caused by lateral movement of the channel. Locations on straight reaches or gentle bends are therefore preferred. In meandering channels it may be possible to locate the structure at a nodal point – a point where the river is known to be stable despite changing meanders elsewhere. Structures located at the confluence of two or more channels experience increased scour due to the interaction of different flows.

Alluvial fans present problems for the location of structures, as they experience frequent channel instability in terms of aggradation, degradation and sudden changes in channel course. It is normally preferable to locate bridge crossings at the head or apex of the fan, as this is where the channel is likely to be more stable.

Where a structure is located close to an existing structure the interaction of flow patterns between the structures can exacerbate scour. Problems may arise where a downstream structure is within the extent of the scour hole of the upstream structure. Alternatively, the upstream structure may affect the flow approaching the downstream structure. For example, it may concentrate flow towards a pier or abutment on the downstream structure. It is also important to be aware that the removal of a nearby structure can affect flow conditions. For example, a new bridge located upstream of an old small arch bridge may experience increased scour if the old bridge is removed. Two factors may be involved in the increase in scour. First, bed degradation could occur if the bed levels were previously controlled by the invert of the old bridge. Second, if the old bridge had caused a significant blockage of the river, removal of it could increase flow velocities at the new bridge upstream.

Some common problems with the location of structures are shown in Figure 5.3.



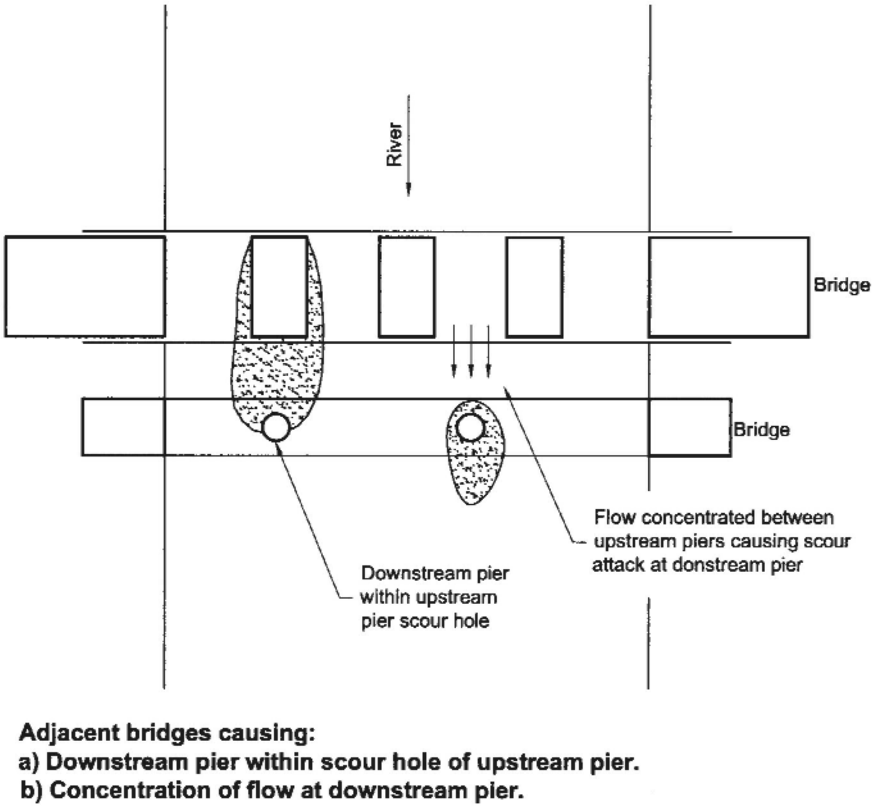
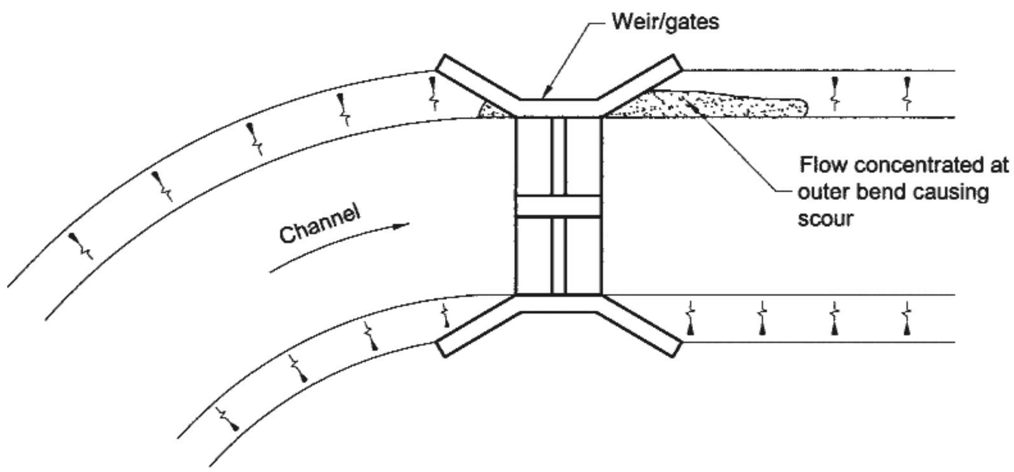
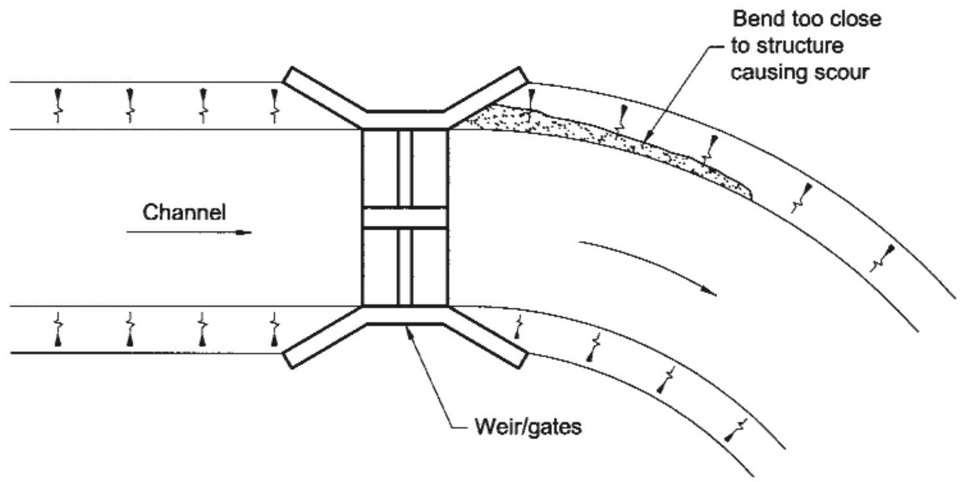


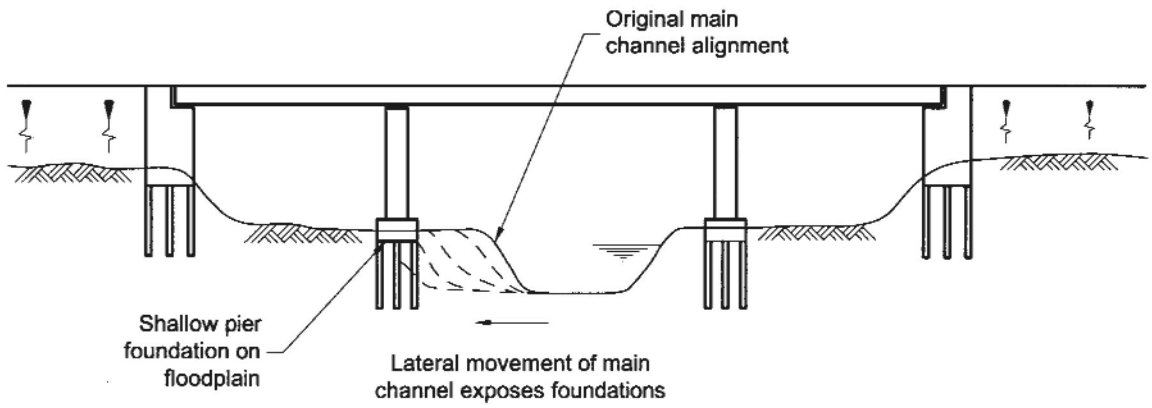
Figure 5.3 *Common location problems* (continued on next page...)



Poor upstream alignment: bend on approach causing scour at outer side of bend.



Poor downstream alignment: bend within area of energy dissipation.



Lateral movement of main channel exposing shallow foundations on floodplain

Figure 5.3 *Common location problems* (continued)

It is difficult to predict the long-term changes that could occur in a river over the design life of a structure. Some changes that affect the structure may be unforeseeable, such as the construction of nearby structures or changes in dredging practices. Nevertheless, time should be allowed during design for gathering information on the historic changes in the river as well as considering possible future changes. Historical records that can help in assessing the lateral stability of the river include aerial photography and satellite imagery (for large rivers), as well as comparison of old maps. In the case of existing bridges, records of bridge inspections that include photographs or details of river alignment can be invaluable.

Valuable information can be gained from nearby structures. Research into the condition of nearby structures, the type of scour protection used at them and any scour problems experienced often proves useful. In many cases, historical data may be sparse or inconclusive, and knowledge and experience of the factors affecting river stability will be needed to assess possible changes in the river's morphology. Physical or mathematical modelling can provide additional tools for assessing the benefits of differing locations and alignments. Physical [word missing?] (or detailed mathematical modelling) should be considered for the following:

- major structures on alluvial rivers or channels
- major tidal crossings and barrages
- complex flow conditions that cannot readily be modelled using simple 1-D models or do not readily match well-researched scour problems.

In brief, the factors that affect river stability can be categorised as either geomorphic or hydraulic. Each can be affected by both natural changes and man-made changes.

Geomorphic factors include river size, river type (ephemeral, flashy, perennial), river shape, bed and bank material, sinuosity, location within catchment. The potential for lateral erosion increases with river size, as indicated by data in Section 4.1.2. Ephemeral rivers only flow for part of the year in direct response to precipitation. They tend to be relatively large and unstable, often with a rapidly shifting thalweg, large sediment load and rapid degradation. Perennial rivers tend to be more stable during floods, with changes occurring over a longer timescale. However, flashy perennial rivers can be unstable.

Bed material is a factor in stream stability, with beds of cohesive material tending to be more stable than those of non-cohesive material. The greatest susceptibility to scour or instability is found in beds composed of sand or silt material. Lateral movement of meanders can change the alignment of flow at structures over time. While it is very difficult to predict accurately the rate and shape of channel movement, it is useful to consider the main modes of behaviour of meander development. These are shown in Figure 5.4. Figure 5.5 shows how even on a relatively stable UK river meander development can lead to failure of a bridge abutment.

Hydraulic factors affecting river stability include channel roughness, bedforms, flood frequency, and flow conditions. In sand-bed rivers, bed material is being moved continually, giving rise to different bed configurations or bedforms such as bars, dunes and ripples that propagate along the bed. The greatest changes in channel geometry occur during periods of high flows. It has been shown that about 90 per cent of channel changes occur during flood conditions, when the flow exceeds the dominant or normal discharge.

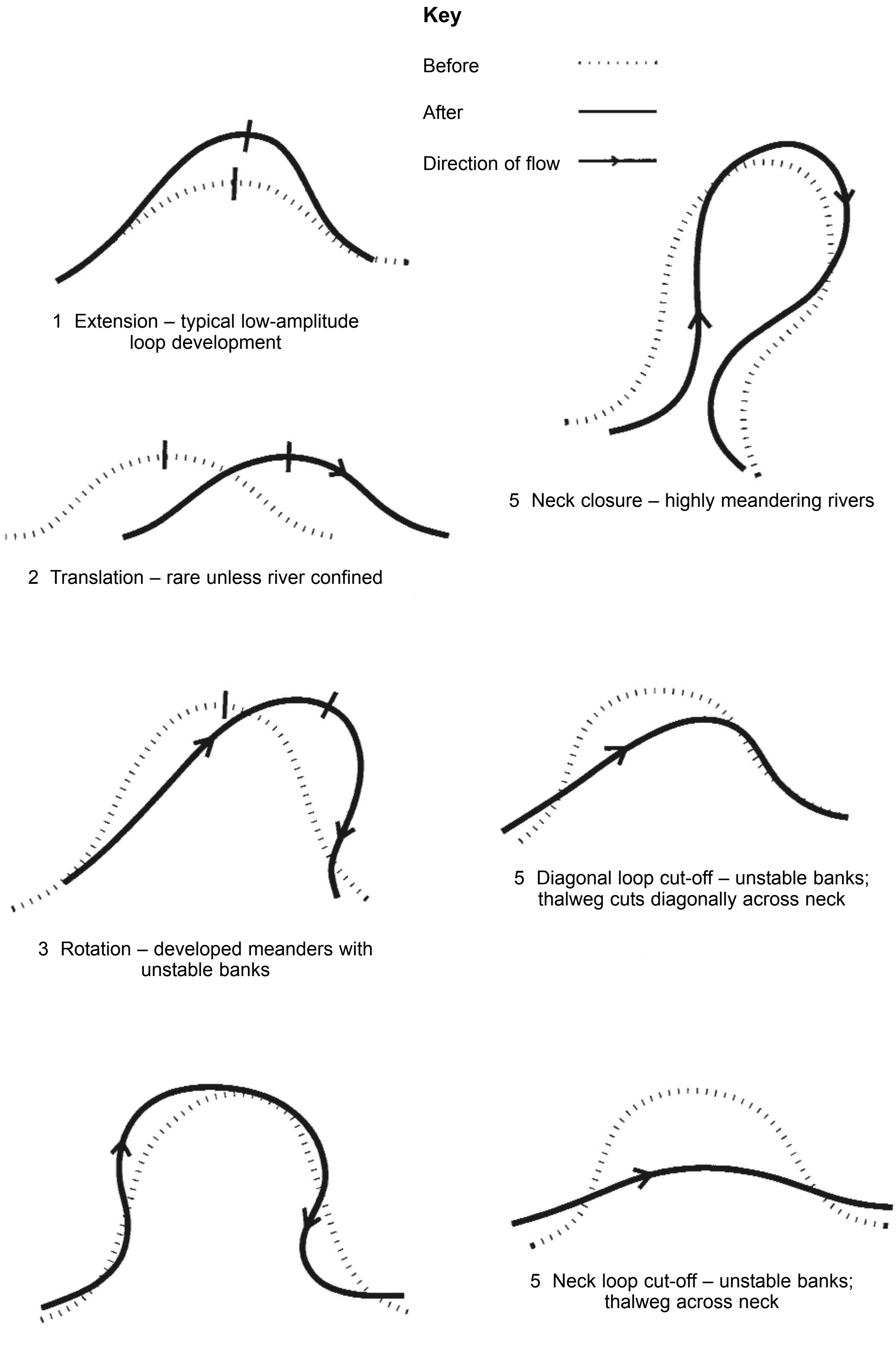


Figure 5.4 *Meander development (after Lagasse* et al*,* HEC20*, 1995)*

View from upstream

View of right abutment

Figure 5.5 *Scour at bridge abutment caused by meander development (photographs courtesy of ATPEC Ltd)*

It is therefore important to consider river behaviour over a wide range of flow conditions, and in particular under flood conditions.

Man-made changes to a river or catchment can affect the geomorphic and hydraulic factors described above, and thus affect river stability. Such changes include channel widening, realignment, and changes in agricultural practices. For example, deforestation for agriculture within the catchment of geologically young rivers can cause particularly severe and rapid changes in river morphology (Figure 5.6).

Figure 5.6 *Degradation and lateral erosion caused by deforestation*

5.2.2 Hydraulic design

Several factors in the hydraulic design of structures can significantly affect scour. Although it is outside the scope of this manual to provide details of the hydraulic design of bridges and hydraulic structures, some of the main issues affecting scour are discussed below. The hydraulic design of bridges is covered in greater detail in *Guide to bridge hydraulics* (Neill, 1975), *Hydraulic factors in bridge design* (Farraday and Charlton, 1983), and *Bridge hydraulics* (Hamill, 1999). Methods of estimating the difference between water levels upstream and downstream of a structure (the afflux) are given in *Hydraulics of bridge waterways* (US Department of Transportation, 1970) and *Afflux at arch bridges* (Brown, 1987).

The size of structure opening is dictated partly by cost and partly by hydraulics. Generally, the aim is to reduce the structure size within the hydraulic constraints. In many cases, hydraulic requirements override cost considerations in order to meet planning constraints. In the UK, for example, planning constraints often require a structure to be made larger to limit afflux upstream, so that flood risk is not increased. As well as affecting afflux, the size of the opening influences the magnitude of contraction scour. The benefits of the reduced costs of a smaller structure must therefore be weighed against not only afflux but also an increase in contraction scour. Greater contraction scour may require either deeper structural foundations or enhanced scour protection. The cost of structural measures to combat contraction scour may in some cases exceed the cost savings from reducing the size of the structure.

If an approach embankment to a bridge crosses a floodplain that carries a significant amount of flow during floods, a suitable route for the conveyance of that flow must be provided. It is generally preferable to provide bridges or culverts, known as relief openings, within the embankment to pass floodplain flows through the embankment. If relief openings are not provided, all flow has to return to the main channel and be passed through the main bridge crossing. There are four disadvantages to doing this:

- the main bridge crossing has to be larger to pass these flood flows
- water levels on the floodplain may be increased, which may not be acceptable to the regulatory authorities or to the owners of properties and agricultural land
- the approach embankments may be susceptible to scour from flow parallel to the embankments returning to the main channel
- the abutments (and possibly the piers) of the main bridge crossing may suffer increased scour from the turbulence and vortices generated by the interaction of the main channel flow with the returning floodplain flow.

The size, location and number of relief openings should be determined by hydraulic design. For both the main bridge crossing and the relief openings a few large openings are generally preferable to a larger number of small openings. Scour protection at relief openings may be necessary to prevent erosion of the channel inverts and the adjacent edges of the embankment. If such erosion occurs, increased flow may be attracted towards an opening, leading potentially to a washout of that section of embankment.

Problems at a crossing of a meandering channel with a floodplain can arise where the predominant flow direction (when the river is flowing in-bank) differs from when there are greater flows and the river is "out of bank". This tends to occur if the main channel at the crossing is not parallel to the edges of the floodplain. In such cases scour should be considered under both flood and bankfull conditions, to determine the worst case for scour. The choice of shape and alignment of elements of the structure may need to be revised to avoid flow being skewed under one of the flow conditions.

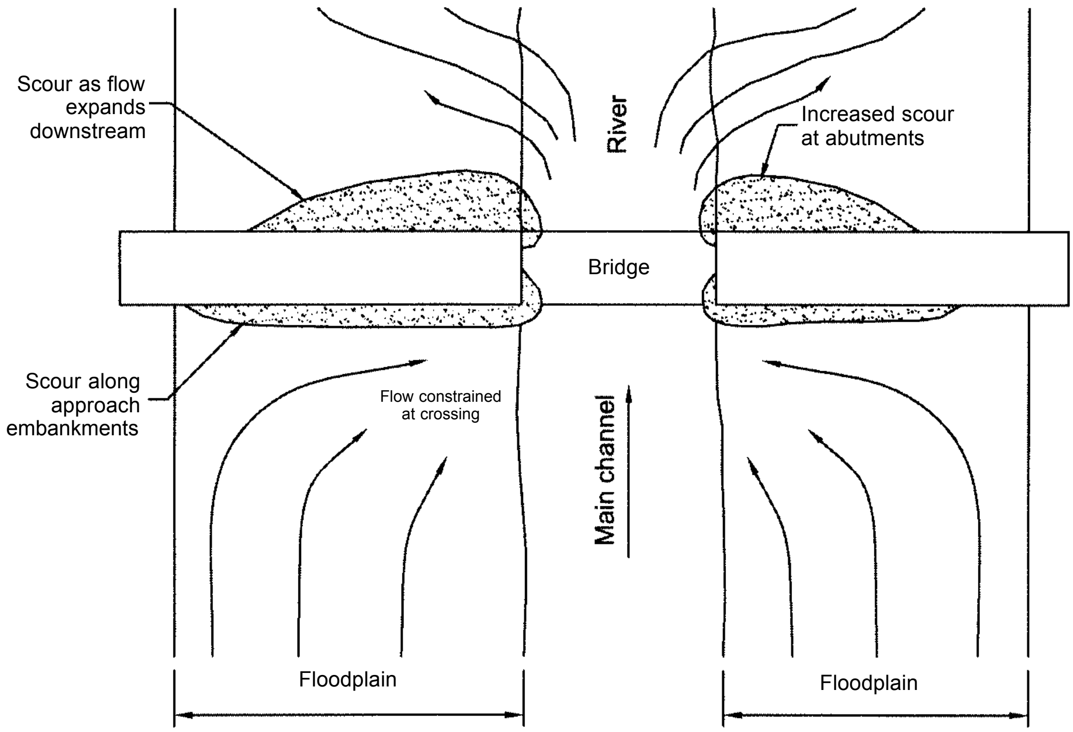
Where there is a possibility of flow being concentrated on one side of a hydraulic structure, the increased scour potential should be assessed carefully. Crossflows may be generated upstream and downstream of structures, particularly where flow is not evenly distributed across the channel, and this may lead to confluence scour. The design and intended operation of the structure should be reviewed if the scour generated by concentration of flow or by abrupt changes in flow intensity is likely to be significant. Concentration of flow may occur because of the location (eg on a bend) or as a result of operational requirements (such as closure of one part of a structure for flushing sediment or other maintenance activities). Such operational requirements may not be immediately obvious during design; nevertheless, it is important to consider the risk of their occurrence because of correct or incorrect operation of the structure.

In some circumstances it may be possible to reduce the risk of the main bridge being washed away during a flood by setting the level of the approach embankments below that of the bridge deck. This has two benefits:

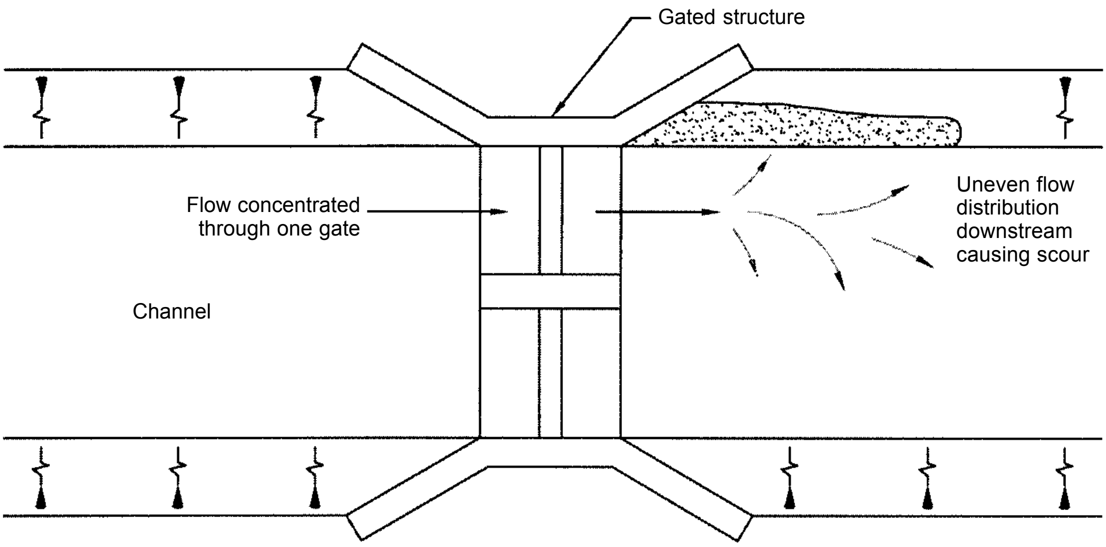
- flows at the bridge are reduced or alleviated, reducing scour and the risk of failure
- failure of the embankments is initiated before failure of the bridge deck, with a lower risk of loss of life and a lower rebuild cost.

Against these advantages, there is a risk of the river changing course and realigning through the approach embankments (avulsion), which should be avoided. This form of construction can be suitable for both major bridge crossings and small-scale crossings. It is particularly appropriate where the bridge and embankment are not raised above a floodplain and hence are likely to be flooded frequently, for example a farm track giving access across a floodplain. It is also appropriate in very flashy rivers such as the geologically young rivers mentioned in the previous section. In these cases the occasional rebuilding of an approach embankment may be more cost-effective than constructing the substantial foundations, river training works and scour protection works to withstand these highly unpredictable rivers.

Some common scour problems generated by hydraulic factors are shown in Figure 5.7

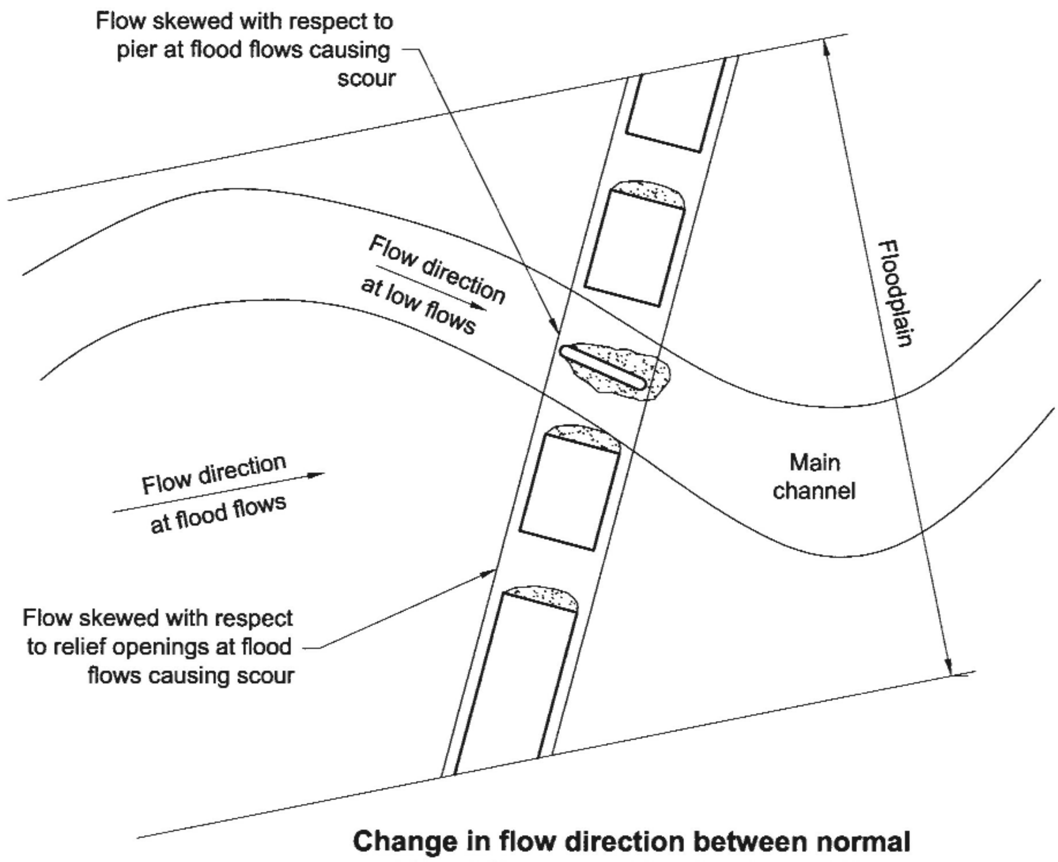


Lack of relief openings on floodplain

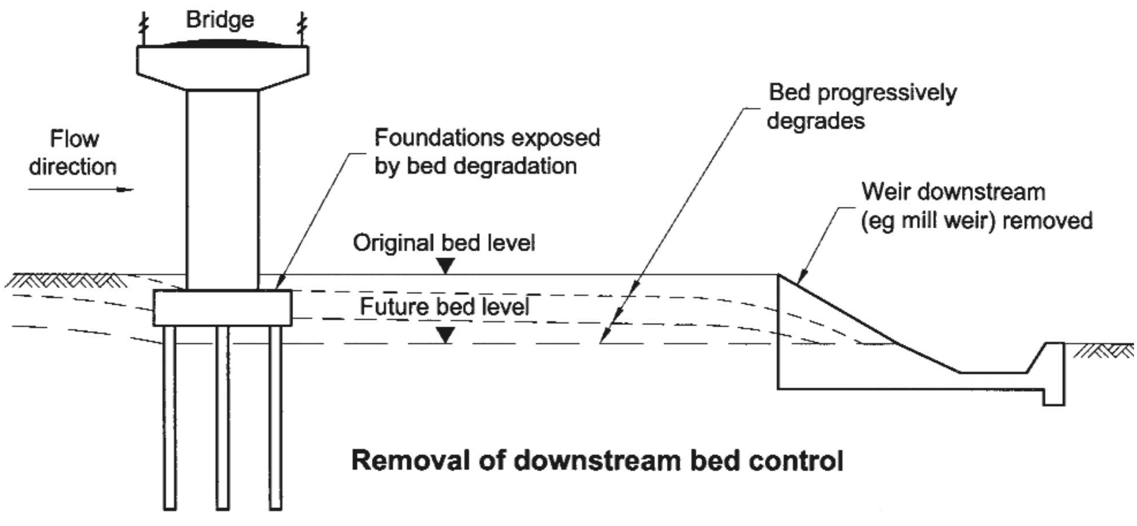


Uneven flow distribution across channel

Figure 5.7 *Common hydraulic problems* (continued on next page...)



Change in flow direction between normal and flood flows resulting in skewed flows at piers, abutments and relief openings.



Removal of downstream bed control

Figure 5.7 *Common hydraulic problems* (continued)

5.2.3 Streamlining structural elements

A relatively cheap and effective method for reducing scour can be to streamline the structural elements of a bridge or hydraulic structure. The main aim is to avoid rapid flow expansion or contraction, and to reduce turbulence by encouraging parallel streamlines. Of particular concern is flow separation causing eddy formation. It is important that structural and hydraulic engineers should work closely together from the earliest stages of a project, so that decisions about issues such as the position, size, shape and alignment of piers and abutments are made taking proper account of the hydraulic factors. If this is not done, satisfactory streamlining and alignment of the structural components may not be possible at a later stage in the design process. Figure 5.8 illustrates the erosive effects of flow separation downstream of a bridge.

Figure 5.8 *Scour caused by turbulence downstream of a bridge. The provision of a solid invert to a bridge almost inevitably leads to scour downstream. The gabion wall on the left of the picture has been added as an emergency measure to stabilise the bank*

Section 4.3.2 (Table 4.3) gives pier shape factors for different pier shapes. Reference should be made to these factors to optimise the design of the pier shape. Lesser factors represent less scour for a given pier width. Rectangular piers are particularly poorly streamlined, whilst the best hydraulic performance is given by rectangular piers having a wedged-shaped nose (known as "cutwaters") and by elliptical piers or lens-shaped piers. However, where the river may change its angle of approach over the life of the structure, the benefit of using a pier of rectangular or elliptical shape can become a liability. In such a case a circular pier, or a series of circular piles with a pile cap above water supporting the piers, may be more appropriate, because the amount of scour is then independent of the angle of approach of the flow.

The hydraulic benefits of streamlining can be significantly reduced by the effect of debris accumulation. Even streamlined piers collect debris (for example a tree truck spanning across more than one pier). Where debris accumulation is likely to be a problem, debris deflectors can be used (see Section 5.2.5). An example of significant blockage by debris is shown in Figure 5.9.

Figure 5.9 *Blockage by debris (photograph courtesy of Czech Railways Infrastructure Division)*

The hydraulic performance of a pier can be improved if the nose is tapered in side view so that the upstream face leans away from the flow; this causes the horseshoe vortex to be directed upwards from the bed and results in less local scour than an equivalent pier with a vertical leading edge (see Table 4.3). Conversely, if the pier has an inverse taper with the front face angled downwards into the flow, the amount of local scour is increased. Scour depth is generally directly proportional to pier width, so reducing the widths of piers or pile caps can yield benefits in terms of scour reduction. The influence of pile groups on scour depth is indicated by Figure 4.4 in Section 4.3.2.

Abutments can also be streamlined to reduce scour. In a similar manner to piers, Section 4.3.3 (Table 4.4) gives abutment shape factors, showing that a sloping ("spill-through") abutment causes significantly less scour than a vertical wall abutment. In addition, angled wing walls (typically set at 30–75º to the longitudinal flow direction) or curved wingwalls improve the hydraulic performance of vertical wall abutments. Under normal circumstances, angled wingwalls are adequate and, where turbulence due to separation of flow is unlikely to be a significant problem, wingwalls at 90º to the longitudinal flow direction are also acceptable.

Bridge deck and culvert soffits are normally given a freeboard above design water level. However, where this is not possible and where the bridge deck could become submerged by an extreme flood (in excess of the design event) it may be appropriate to streamline the underside of the bridge deck by rounding the upstream and downstream faces. This also encourages debris to pass under the deck, rather than collecting at the upstream face and causing a blockage to flow and consequent increased scour.

The same principles of streamlining should apply to culverts and bridges through approach embankments on floodplains. Flow velocities on the floodplain tend to be lower than in the main river. However, because the flow upstream and downstream of these crossings is not confined to a defined channel, there can be rapid contraction and expansion of flow, leading to scour of the embankment on either side of the culvert and scour of its bed. The hydraulic design of these structures needs to be as well thought out as that of the main crossing.

For other hydraulic structures, such as at culvert inlets or outlets and at weirs or gates, the same principles of streamlining apply, with the main aim of avoiding rapid contraction or expansion of flow, so as to reduce flow separation. The transition between a sloping channel bank and the vertical structure wall is often a source of turbulence due to flow expansion and contraction. In most cases, providing a gradual transition from vertical walls to sloping channel is uneconomic. However, where high turbulence can be expected, certain shapes of wingwall can improve flow conditions and reduce flow separation. Examples of transition shapes with their relative benefits are shown in Figure 5.10.

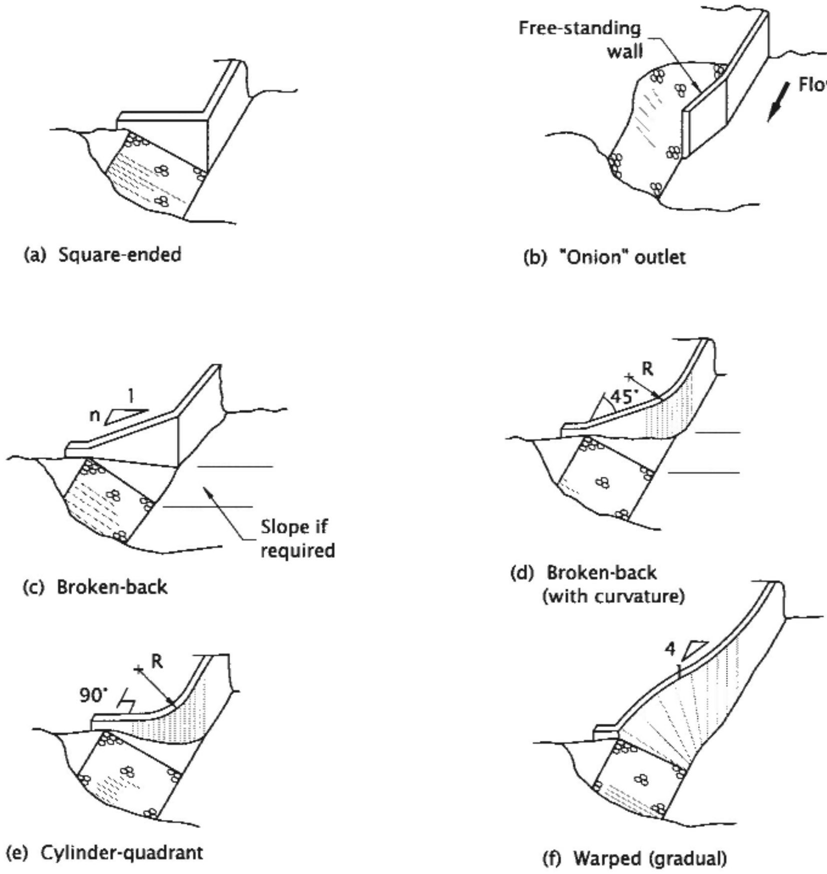


Figure 5.10 *Examples of transitions between vertical and sloping banks (after Hemphill and Bramley, 1989)*

5.2.4 River training

In cases where a bridge or hydraulic structure is located on a river or channel that is unstable, river training works should be considered. The purpose of river training works is to constrain the river locally to reduce instability and thus pass flows through the structure under good hydraulic conditions. There are three main types of river training:

- longitudinal structures, which are parallel to the flow and define the river banks and prevent lateral movement
- transverse structures, approximately perpendicular to the flow, to deflect flow away from a bank, reducing flow velocities at that bank of the river, thereby reducing lateral movement and encouraging build-up of sediment
- bed control structures, mainly taking the form of sills or weirs, which fix bed levels, so reducing degradation of the river bed upstream of the sill.

Longitudinal river training protects the river banks from erosion. They are often useful for velocity control at expansions to avoid separation of flow and eddy formation downstream of abutments. Erosion protection systems include riprap, gabion mattresses, concrete blocks (interlocking or articulated) and sheet piling. In addition, various bio-engineering solutions using soil reinforcement and vegetation cover are coming in to more widespread use, generally in locations of low flow velocities (less than about 2 m/s).

Transverse river training works, known as groynes, are normally constructed from stone, earth, sheet piling or timber cribwork, extending out into the channel from a bank that is at risk of erosion. They are most commonly used on wide braided or meandering channels. They are less suitable for use where the channel is less than about 40–50 m wide and where bend radii are less than about 100 m. Scour occurs at the head of the groyne and hence this area should be suitably protected. An example of the use of short (stub) groynes is shown in Figure 5.11.

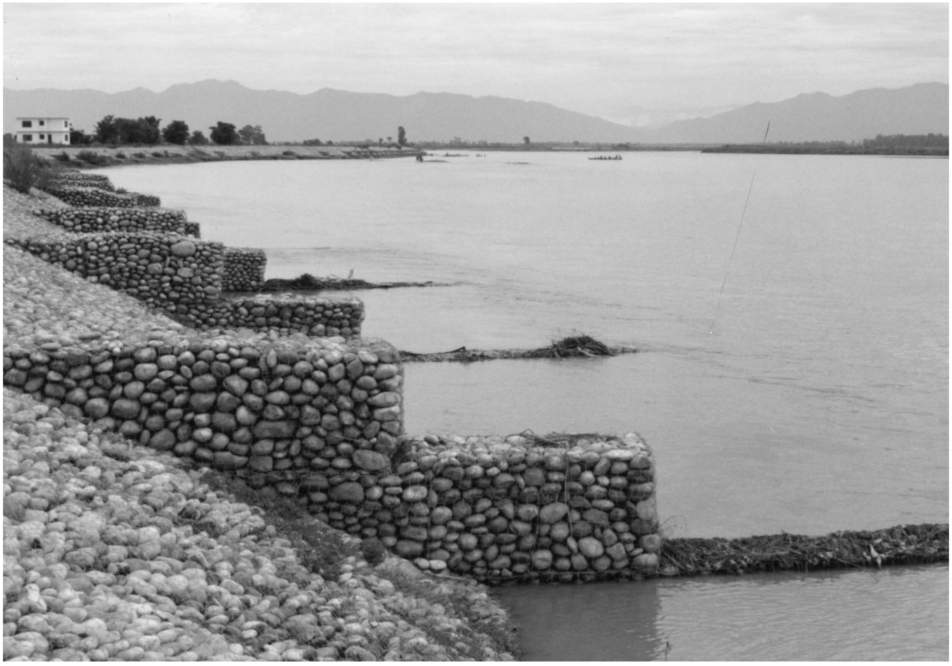


Figure 5.11 *Stub groynes used for river training*

The construction of sills (or weirs) in the river bed creates local fixed points in the river bed, preventing further erosion of the bed at that point. Some erosion can still occur in the reach upstream of the sill, until a stable bed slope is achieved. Weirs can also be used to perform a similar function, although they reduce flow velocities and increase water levels upstream, and may therefore be unacceptable. The reduced velocity also has the benefit of reducing scour. Bed control is probably most often used on steep upland rivers where there is active bed erosion, although they may be used wherever bed degradation is likely. A typical sill immediately downstream of a crossing is shown in Figure 5.12.

Figure 5.12 *Bed control using a downstream sill. Note that poor construction has led to failure of the pitched stone apron, and that scour downstream of the sill – if left unchecked – could undermine the sill*

There are risks associated with the use of bed control as a scour reduction measure. In particular, if the weir is located at some distance downstream of the structure its purpose may be forgotten over time and in the future it may be removed, leading to further bed degradation. Bed degradation caused by, for example, the removal of a weir downstream may be capable of causing the failure of a structure. Furthermore, weirs are themselves susceptible to scour and, if not adequately designed, can fail, again leading to bed degradation.

River training techniques (Przedwojski *et al*, 1995) gives comprehensive details and design methods for different river training techniques. *River and channel revetments* (Escarameia, 1998) provides design procedures for different types of revetment systems. *Waterways bank protection: a guide to erosion assessment and management* (Environment Agency, 1999) gives details of mainly small-scale (longitudinal) bank protection solutions using both bio-engineering and structural techniques (focusing on the UK).

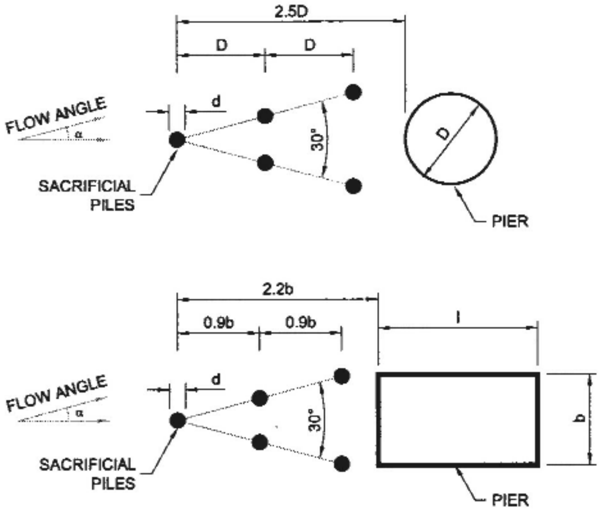
5.2.5 Deflectors

The aim of deflectors is to direct currents away from the critical structural elements of a bridge in order to reduce scour depths at those elements. Types of deflectors include sacrificial piles, vanes, ice and debris deflectors and guide banks. Care must be taken to avoid deflected flow causing scour elsewhere.

Sacrificial piles

Sacrificial piles are piles installed upstream of bridge piers to interrupt the downward current at the pier that gives rise to horseshoe vortices. The piles also induce deposition in the local scour hole at the pier, by creating a wake region downstream of the piles. The usual configuration is a set of three or five piles in a "V" formation with the apex of the V pointing upstream. The piles themselves are subject to significant scour.

Although sacrificial piles have been tested under different laboratory conditions, fewer field data are available to confirm their effectiveness. Their use at full scale, other than in field trials, is limited, although further developments are continuing. Up to 50 per cent reduction in local scour has been shown from experiments. This appears to be backed up by a field study on the Big Sioux River, South Dakota (Chang and Karim, 1972), which indicated a reduction in scour of up to 44 per cent.



NB: LAYOUTS ARE THOSE THAT HAVE BEEN USED FOR LABORATORY RESEARCH.
ALTERNATIVE LAYOUTS HAVE ALSO BEEN MODELLED

FLOW ANGLE (α)	REDUCTION IN MAXIMUM SCOUR DEPTH*
0	41%
20	23%
30	26%

* VALUES QUOTED ARE FROM LABORATORY EXPERIMENTS BY HADFIELD (1997) FOR CLEAR WATER SCOUR $\frac{V}{V_c} = 0.9$

Figure 5.13 *Indicative layouts of sacrificial piles*

The effectiveness of the sacrificial piles is dependent on the flow angle and the flow intensity. Where flow velocities are greater than the threshold velocity for bed movement – that is, under live bed conditions – the effectiveness of sacrificial piles is reduced. The reduced effectiveness is due to the passage of bedforms (dunes and ripples) causing greater scour depths. For flow angles greater than about 20 per cent the piles have little effect under live-bed conditions, whilst under clear-water conditions their effectiveness is significantly reduced, as they are protecting less of the pier. The top of the piles does not need to be above the water level, though this may be preferable to reduce the risk of damage to boats. There is little apparent difference in scour reduction between a submerged pile and one protruding above water.

Suggested sacrificial pile layouts for circular and rectangular piers are shown in Figure 5.13, along with an indication of their effectiveness based on laboratory data. A typical layout is shown in Figure 5.14. Sacrificial piles are only recommended where the flow is likely to remain aligned with the pile or pier arrangement and for relatively low flow intensities (that is, under clear-water scour conditions).

Figure 5.14 *Sacrificial piles upstream of a bridge*

Vanes

Vanes (also known as "Iowa vanes") are vertical plates that are used in two distinct applications. They are installed either upstream of bridge piers or adjacent to channel banks.

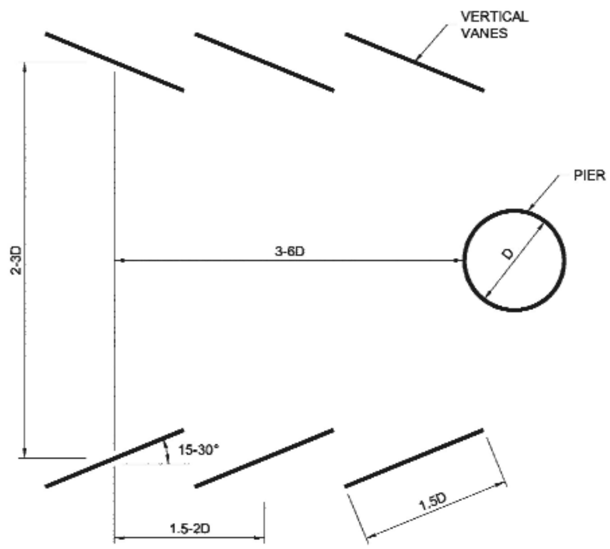


Figure 5.15 *Indicative layout of vanes. The dimensions given show typical the range of dimensions tested in the laboratory*

At piers they are placed in lines of three or more either side of the pier, with the vanes angled inwards in the downstream direction. The concept behind them is similar to sacrificial piles: to deflect the approaching flow and so interrupt the downward current at the pier. The concept was developed in the USA. However, as for sacrificial piles, guidance on their use for piers is still in the early stages of development and is mainly based on laboratory experiments. Reductions in average scour depths at piers of between 30 per cent and 50 per cent in theory appear feasible under live-bed conditions.

However, the reduction in the maximum scour depth is considerably less and, like sacrificial piles, vanes may be more appropriate for clear-water scour conditions. An indicative layout used in laboratory testing is shown in Figure 5.15.

Their purpose at channel banks is to interrupt the flow approaching the channel bank. This provides an area of more tranquil water next to the bank and diverts the bedload towards the bank, countering the natural erosive secondary current. In such situations, they are appropriate for live-bed conditions and their use has been studied in the Netherlands.

Debris deflectors

Debris accumulating around bridge piers can significantly increase scour. Two effects of debris accumulation can increase scour. The debris can increase the effective width of a pier and thus generate a larger and deeper scour hole. In addition, by obstructing flow near the water surface, a sluicing action is generated with increased flow velocity under the debris. In unstable channels (where debris is particularly common anyway) blockage of the channel may also encourage lateral movement or shifting of the channel. In certain climates, ice accumulation can have the same effect. To avoid scour induced by debris accumulation, debris deflectors can be installed upstream of the bridge; Figure 5.16 illustrates the concept. Quantitative information on their effectiveness is hard to obtain and debris accumulation has not normally been taken into account in scour estimates. The main use of debris deflectors is on existing bridges, where there is known to be both a scour and debris accumulation problem.

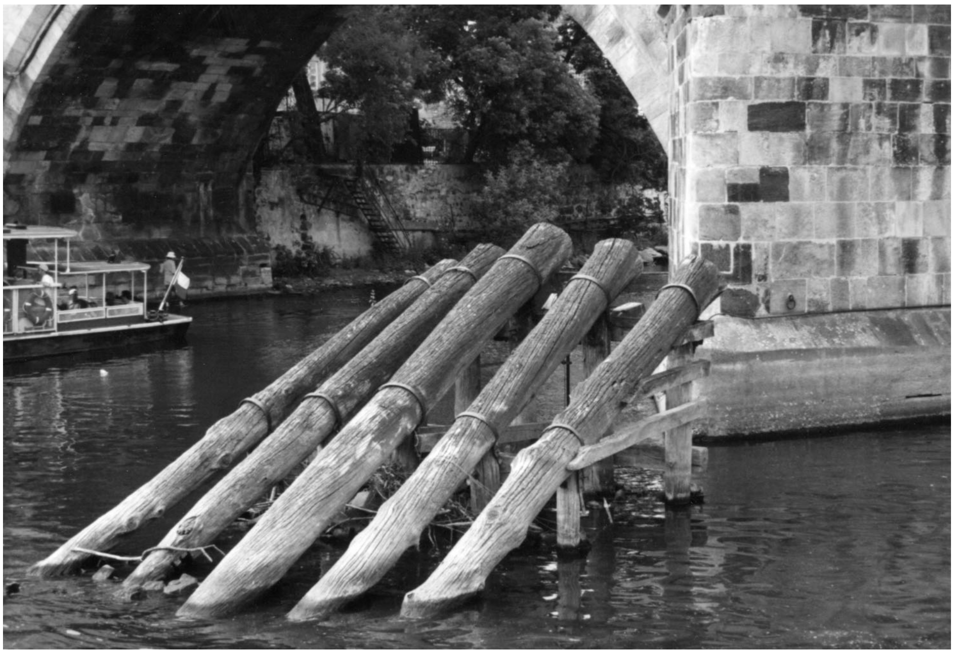


Figure 5.16 *Debris/ice deflector upstream of a bridge pier*

Guide banks

Guide banks are earth or rock embankments (and sometimes walls) that extend upstream – and normally downstream – from the ends of bridge abutments or approach embankments to align flow through the bridge opening, thereby improving flow conditions at the bridge. Scour is thus moved away from abutments to the ends of the guide banks. Although normally constructed when a new bridge is built, they can also be added to an existing bridge crossing to reduce its susceptibility to scour.

An example of the use of a guide wall to channel flow under a bridge is given in Figure 5.17. They are most useful in cases where significant floodplain flow returns to the main channel, rather than passing through relief openings. Przedwojski *et al* (1995) suggest their use where more than about 15 per cent of the discharge flowing on the floodplain has to return to the main channel and pass under the main crossing.

Figure 5.17 *Guide wall on approach to a bridge across an alluvial fan. This view is looking upstream from the bridge. The wall channels water flowing from left to right, to avoid outflanking of the bridge*

The guide banks are normally elliptical in plan, to provide a smooth transition and contraction of the flow through the bridge opening. In India and Pakistan, they tend to be straight, with curved sections only at the upstream and downstream ends of the guide banks. At a skewed crossing across a floodplain that carries large flows, there may be a need to protect the approach embankment (with erosion protection or groynes) and the back face of the guide bank from the eddies generated.

Guide banks are normally constructed in pairs either side of the main channel but, if there is significant out-of-bank flow on only one of the floodplains, then a single guide bank may be sufficient. The section of the guide bank downstream of the crossing is sometimes omitted. Its purpose is to reduce scour due to separation and eddy formation and thus to reduce the flow velocity smoothly to that of the downstream channel.

A typical guide bank layout is shown in Figure 5.18. As general guidance, the length of the upstream guide bank is about 0.75 to 1.25 times the main waterway width and the length of the downstream guide bank 0.25 to 0.4 times the main waterway width. However, the length required is more correctly related to the proportion of flow returning from the floodplain to the main channel and the flow velocity. A minimum guide bank length of 15 m is recommended (Lagasse *et al*, 1995).

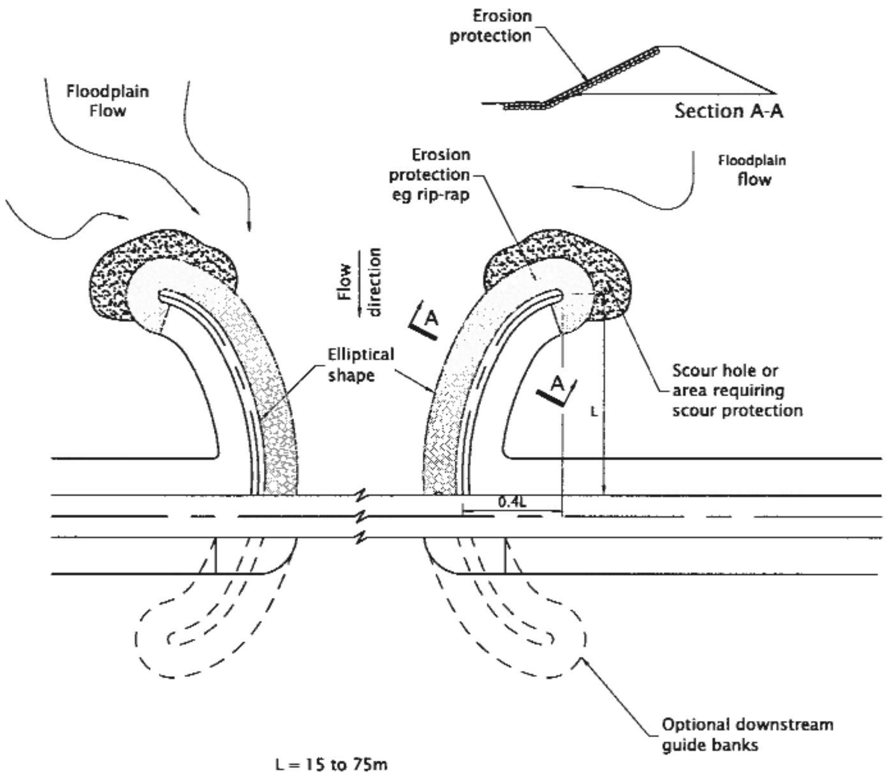


Figure 5.18 *Typical guide bank layout*

There is limited information on the performance of guide banks, and model testing may be appropriate for large structures. The design of guide banks is covered in more detail by Lagasse *et al* (1995) and Przedwojski *et al* (1995).

5.3 STRUCTURAL MEASURES

5.3.1 General guidance

The main structural measure to combat scour is to locate foundations well below the expected scour depth. For purposes of structural design, it should be assumed that the full scour depth has developed and that, over the extent of the scour hole, all bed material above the level of lowest scour has been removed and is not available for bearing or lateral support. The design should also take into account the change in hydrodynamic forces that will act on the pier as a result of the changed flow area.

The sensitivity of the structural integrity of the structure to changes in the maximum scour depth should be checked. In the case of bridges, the method for estimating maximum scour depths gives an upper bound on predicted scour depths. US design practice (Richardson and Davis, 1995) is that the structure and foundations of highway bridges should have a safety factor against collapse of 1.5–2.0 in terms of the loads occurring in a design flood with a 100-year return period. In addition, the design should be checked to ensure that the main structure can survive a so-called "superflood", with the load factor against collapse remaining greater than 1.0; in US practice the superflood is defined as the 500-year event. The factors of safety specified for a particular bridge should be based on the importance of the crossing and the degree of uncertainty in the design.

The extent of scour holes is discussed in Section 4.3. The extent of local scour holes should be plotted on a cross-section of the river to check for interaction or overlap between piers or between piers and abutments, as shown in Figure 5.19. Where scour holes interact, the total scour depth may be greater than that predicted for a single hole.

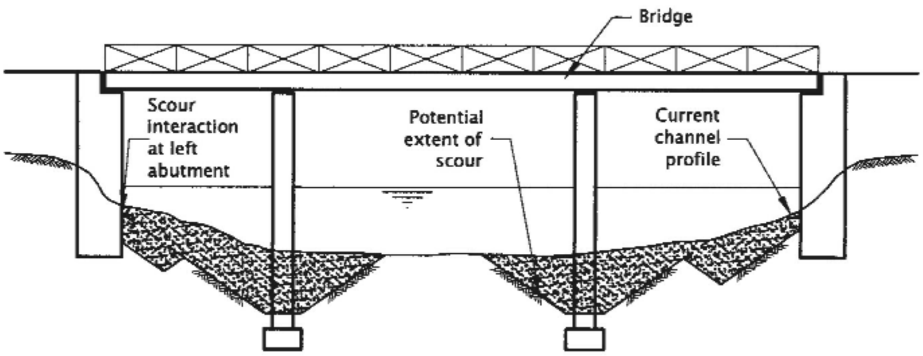


Figure 5.19 *Plotting the extent of local scour holes*

Foundations generally take the form of spread (pad) footings or deep piled foundations. Experience indicates that spread footings have in the past been more susceptible to catastrophic failure due to scour than piled foundations. A study in New Zealand of scour damage to railway bridges over the past 100 years confirmed the increased risk to footings compared with piled foundations (Holmes, 1974). Of the bridges that had suffered damage, 35 per cent had spread footings, yet the overall proportion of railway bridges in New Zealand having this type of foundation is relatively small. In part, may be accounted for by previous under-estimation of scour depths, so footings have been under-designed. However, it also reflects the fact that, once footings are exposed and undermined, failure is more likely than for a piled foundation – in the latter, even if scour depths are larger than predicted, some residual strength is likely.

During pile design, particularly where scour is likely to be large or is difficult to predict, consideration should be given to using fewer longer piles rather than many short piles. Shorter piles more rapidly reach their ultimate load condition than longer piles, as scour increases. Also, a larger number of piles is likely to present a greater obstruction to flow and thus cause deeper scour than fewer (though larger) piles.

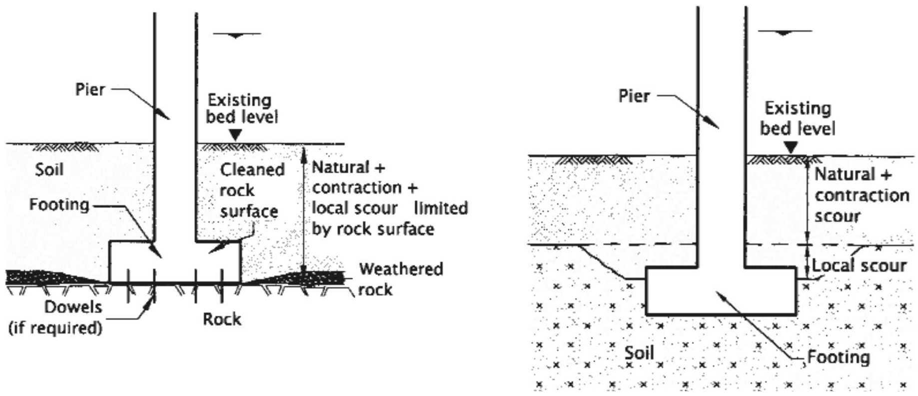
For foundations on floodplains, consideration should be given to designing them at the same elevation as the pier foundations in the main channel if there is the likelihood of channel migration across the floodplain over the life of the structure. The same principle applies to the abutments of clear spans. This is less likely to be a requirement for the UK, where rivers are generally relatively stable and major channel realignment is rare. Where migration is a possibility but less likely, then the benefits of providing full-depth foundations on the floodplains must be compared with the cost of construction and weighed against the alternatives. If migration of the channel is considered to be a possibility, it may be appropriate to use river training to reduce this risk. The bridges spanning the larger rivers of the Indian sub-continent generally have associated river training works (guide banks, groynes, bank protection) to keep the river on course. In other cases, where investment in extensive training works cannot be justified, it is often appropriate to monitor channel changes and react to local problems as they arise. Monitoring should be carried out on both a regular basis – the frequency being dictated by the stability of the channel, based on knowledge of the site – and after major floods. Survey data should be stored in a standardised and accessible format. A simple risk assessment or cost benefit analysis could be used to consider these issues (see Sections 6.3 and 6.4).

Consideration should be given to setting the level of the approach embankments lower than the main bridge crossing to reduce the risk of failure of the main bridge, as described in Section 5.2.2.

5.3.2 Spread footings on soil

Recommendations for the location of footings are given below (based on US recommendations in *HEC18, Evaluating scour at bridges*, Richardson *et al*, 1995) and illustrated in Figure 5.20:

- the base of the footing should be located at least below the total scour depth (sum of natural scour + contraction scour + local scour)
- the top of the footing should be located at least below the level of the natural scour plus contraction scour depth
- if the top of the footing is located above the existing bed level or above the level of natural scour plus contraction scour, then the footing will present a greater obstruction than the pier and result in an increased scour depth.



Spread footing on sound rock   Spread footing on soil

Figure 5.20 *Recommended footing locations*

If placed below the level of natural and contraction scour, the footing itself can act as a scour protection measure. It reduces local scour by interrupting the downward currents that would generate the horseshoe vortices that lead to local scour. Section 5.4.12 provides details of the dimensions and elevation of footings and their effect on reducing local scour.

5.3.3 Spread footings on rock

The first aspect to establish when designing spread footings on rock is whether the rock is susceptible to scour, that is, whether it is erodible or non-erodible. It is not appropriate to consider a single rock property to determine the scour potential of rock. The US Department of Transport gives some guidance on the properties to consider for defining whether a rock is susceptible to scour (US Department of Transportation, 1991). This guidance is summarised in Table 5.2.

Where rock is highly resistant to scour the base of the footing should be placed on the cleaned rock surface. Small embedments or keys are generally best avoided, as blasting can damage the rock structure beneath, making it more susceptible to scour. Where footings require lateral restraint on smooth rock surfaces, steel dowels should be drilled and grouted into the rock below the base of the footing.

Table 5.2 *Guidelines for assessing the erodibility of rock*

Rock property	Scour criteria
Rock quality designation (RQD) (ASTM D6032)	An RQD less than 50 indicates a rock that should be considered as a soil in terms of its scour potential
Unconfined compressive strength (ASTM D2938)	Samples with unconfined strengths below 1724 kPa (250 psi) are not considered to behave as rock
Slake durability index (SDI) (International Society of Rock Dynamics)	The SDI test is used on metamorphic and sedimentary rocks such as slate and shale: an SDI value of less than 90 indicates poor rock quality
Soundness (AASHTO T104)	Threshold loss rates of 12 per cent (sodium) and 18 per cent (magnesium) can be used as an indication of scour potential
Abrasion (AASHTO T96)	Rock with losses of greater than 40 per cent should be considered as erodible

Where spread footings are founded on potentially erodible rock, the rock formation needs to be carefully assessed for scour. A good-quality rock formation may be present beneath a thin weathered layer and footings should be founded on this. Where deep deposits of erodible rock exist, footings should be placed below the predicted scour depth. Excavation of weathered rock should be carried out with care. Where blasting is needed, it should be carried out using light closely spaced charges. Overbreak below the footing base should be avoided.

5.3.4 Piled foundations

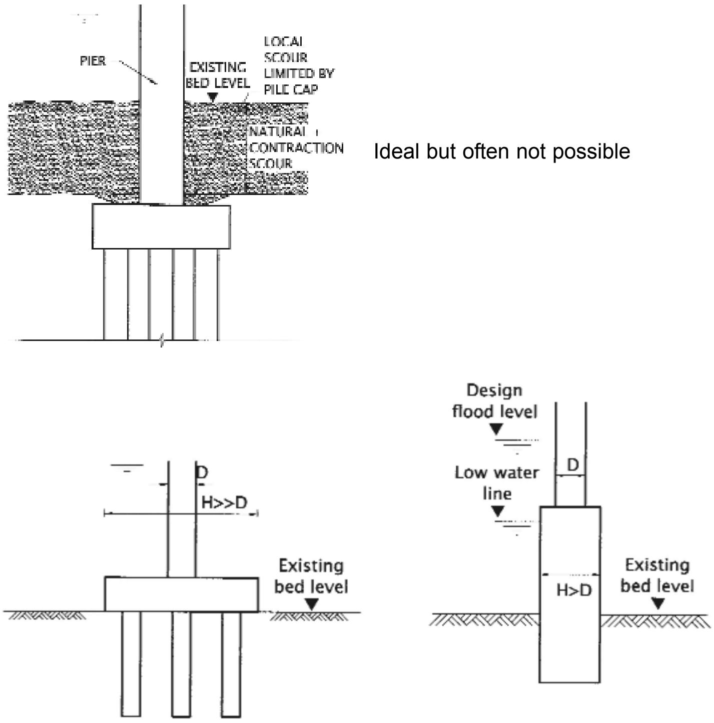
It is suggested that the general principles given below and illustrated in Figure 5.21 be followed:

- the top of the pile cap should normally be placed either below the level of natural scour plus contraction scour or above water level. If the pile cap is between these levels, the depth of local scour will be determined by the size, spacing and positions of the piles and pile cap, and not by the geometry of the pier (see Figure 4.4 for examples)
- if exposure of the piles by scour is unacceptable, the pile caps should be located below the maximum scour depth
- if the piles are exposed, structural checks in the fully scoured situation are required and monitoring may be appropriate (see Section 5.8).

Where the recommendations above cannot be applied for practical or cost reasons, it may be necessary to accept that part of a pile cap may become exposed to the flow. Also, in deep rivers, it is sometimes the practice to set the pile caps well above the bed, so that they can be constructed out of the water during periods of low flow. In these situations, the estimates of scour around the piers must take into account any potential additional scour caused by the presence of the pile cap and by the piles themselves. Structural checks are also necessary to ensure that the piles can safely withstand the amount of exposure to the flow, including loss of ground support.

In a similar way to a footing, a pile cap may act as a scour protection measure reducing local scour. Section 5.4.14 provides more details of how pile caps can affect local scour.

Depth of local scour determined by pile cap or caisson size



Depth of local scour determined by pile cap or caisson size

Figure 5.21 *Principles of pile and pile cap location*

5.3.5 Structural repairs

If scour problems occur, major structural measures tend to be less easy to implement than scour protection measures. The most obvious structural measure that can be carried out to an existing structure is underpinning, although reinforcement and extension of foundations can also be carried out. Specialist advice should be sought for underpinning.

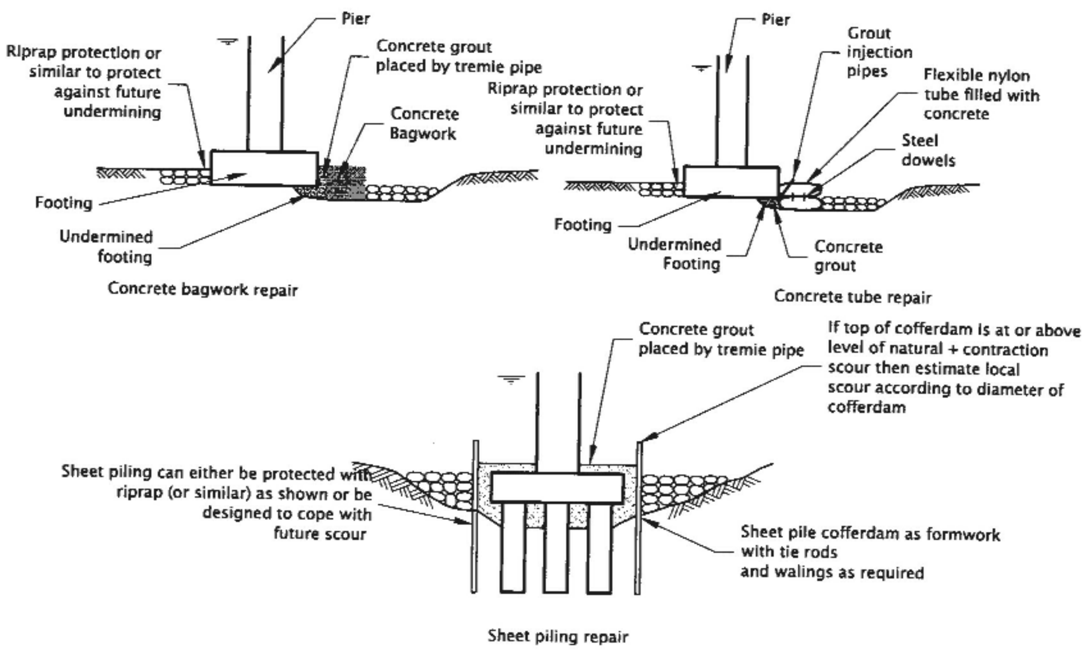


Figure 5.22 *Typical structural repairs*

Repair of footings or pile groups exposed by scour can be carried out. Two types of repair are commonly used: the restoration of load-bearing capacity by filling voids under footings, and the protection of exposed pile groups. Repairs usually involve some scour protection measures to reduce the risk of further undermining occurring. The division between structural repair and scour protection is therefore not always clear-cut. Some general types of foundation repairs are shown in Figure 5.22. Repairs are usually carried out using one or more of sheet piling, bagged concrete and injected concrete grout. These are combined with scour protection measures such as those described in Section 5.4. Figure 5.23 shows concrete repairs to an abutment.

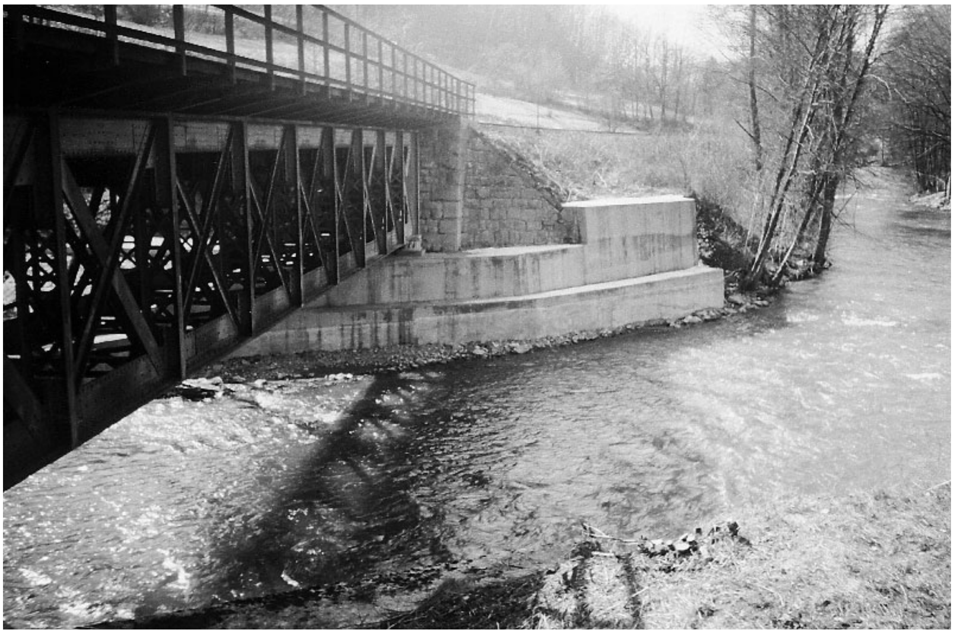


Figure 5.23 *Concrete repairs to a bridge abutment (photograph courtesy of Czech Railways Infrastructure Division)*

The design of repairs must take into account the danger of increasing the effective cross-section of a pier or other structure and thus inducing additional local scour. Provided that repairs are placed below the level of the combined effect of natural and contraction scour, then the repairs should avoid scour-inducing currents and may reduce local scour by interrupting the downward currents at the piers. If it is not practical to limit the repairs to below the level of the combined effect of natural and contraction scour, then scour depths may need re-estimating using the guidelines in Chapter 4, based on a wider effective pier dimension. In addition, the designer should be aware of the danger of scour repairs at one pier altering currents at other piers or structures, which could increase scour at those locations.

When underwater repairs are necessary, one of the first aspects to consider is whether or not to dewater to carry out the repair. Dewatering is usually effected by installing a sheet-piled cofferdam to allow the use of conventional above-water repair techniques. One advantage is that dewatering allows the quality of work to be better controlled. Where it is feasible, dewatering may nevertheless be an expensive option and may cause additional contraction scour while the cofferdam is in place. Dewatering costs were reported to comprise 40 per cent of repair costs on US Army Corps of Engineers projects (*Underwater bridge maintenance and repair*, Transportation Research Board, 1994). Specialist knowledge and the advice of specialist contractors is often required to make a judgement on the cost and feasibility of carrying out repairs underwater. The following sections describe some of the more common methods of structural repair.

Bagged concrete

Concrete in fabric bags can be used to protect the foundation material from further scour as well as providing formwork for placing concrete grout for restoring the bearing beneath a foundation. Normally the bags are filled with dry concrete before placing in position. The cement hydrates and hardens on contact with water. Bags should be anchored together and to the bed material or footing, for example using dowels. Synthetic fibre (such as nylon) fabric bags can be filled after they have been positioned underwater. The fibres allow bleedwater to be expelled. Because fabric bags are water-permeable the water/cement ratio may be low at the surface of the concrete and hence the strength at the surface may be low, though this is not generally a concern. A refinement of the concrete bag is the concrete tube. This is essentially similar to the bag, but is normally larger, sized to fit a void and allows the concrete to shape itself to fill the void. Various proprietary systems are available.

Advantages

- versatile: can be used for a wide range of applications and in a range of sizes
- handles and places relatively easily underwater
- can be installed by a small team with small-scale equipment
- eliminates the need for formwork.

Disadvantages

- strength and durability generally not as good as conventional *in-situ* concrete, because water/cement ratio cannot be controlled
- filled bags require dry storage before use
- watertightness and bond between bags not as good as conventional *in-situ* concrete
- potential for increased local scour if the cross-section of the pier or other structure is increased
- cement washout from the bags may cause pollution.

Sheet piling

In cases where concrete grouting is used to restore the bearing capacity of a footing or pile group, sheet piling can provide a suitable formwork for retaining the grout, as well as providing protection against further scour. The elevation of the top of the piling ideally should be below the level of the combined effect of natural and contraction scour for the reason described in previous sections. Where this is not possible, the effective pier diameter will be increased and greater local scour than for the existing pier may occur.

Advantages

- straightforward to design and usually straightforward to construct providing access for plant is possible
- economical.

Disadvantages

- may increase local scour
- use can be difficult where headroom is limited.

Concrete grout

Voids, eg below footings, can be filled with concrete grout. Where this is carried out underwater the grout can be injected using tremie pipes. Formwork is needed to contain the grout, which can take the form of concrete bags or sheet piles, as described above, as well as conventional formwork. Grouting is carried out from the bottom up. As grouting progresses the tremie pipe should remain embedded in the freshly placed grout. Formwork should be watertight to prevent loss of grout and held down by dowels or similar to prevent uplift generated by pressure of the grout. The top of the formwork should be protected to avoid the loss of fines and cement, but vented to indicate when the formwork is full. A typical arrangement for the top of the formwork may consist of a highly permeable fabric held down with wire mesh, in turn held down with plywood.

Advantages

- voids with difficult access can be filled
- water cement ratio and shrinkage can be reasonably well controlled
- bonding is good
- unaffected by dilution or washout
- formwork does not need dewatering
- straight, clean surfaces can be formed.

Disadvantages

- may be relatively expensive compared with other options
- the pressure induced by the pumped grout means that formwork needs to be particularly well anchored
- cement washout may cause pollution.

Further advice on underwater bridge repairs can be found in *Underwater bridge maintenance and repair* (Transportation Research Board, 1994).

5.4 SCOUR PROTECTION MEASURES

5.4.1 Choice of materials

Scour protection measures are designed to protect the channel bed and banks from the erosive forces causing scour. They fall into two main categories: 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 include:

- construction cost
- underwater or dry construction
- availability of materials
- construction and maintenance constraints (low headroom, access)
- channel stability laterally and vertically
- environmental considerations
- future maintenance costs and access.

A matrix indicating, from experience, which measures are suitable under which conditions is shown in Table 5.3.

Table 5.3 *Scour protection measures: selection checklist*

Legend ● Good/appropriate o May be appropriate × Do not use/not applicable H High M Moderate L Low	**Factors** Underwater construction	Repairs	Construction cost	Maintenance cost	Restricted access/headroom	Environmental suitability	High velocity flow	Vertical stream instability	Lateral stream instability
Flexible protection									
Riprap	●	●	L	M	o	●	●	●	o
Gabion mattresses and sacks	●/o	●	M	M	o	●	o	●	o
Gabion boxes	×	●	M	M	o	●	o	o	o
Articulated concrete blocks	o	●	H	M	o	o	o	●	o
Articulated grout-filled mattresses	●	●	M	L	●	o	o	●	o
Bituminous systems	×	●	L	M	o	o	o	●	o
Biotechnical solutions	o	●	M	M	o	●	×	●	o
Rigid protection									
Rigid grout-filled bags and mattresses	●	●	M	L	●	o	●	o	o
Concrete aprons	×	●	M	L	●	o	●	×	o
Stone pitching	o	o	M	M/H	o	o	●	o	o
Other									
Protective collars (piers only)	o	×/o	L	L	o	●	●	●	●
Pile caps/footings (piers only)	o	×/o	L	L	o	●	●	o	●
Sheet piling	●	●	M/H	L	o	●	●	●	●

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.

Systems using concrete as their main component, such as grout-filled mattresses, often provide an effective solution, particularly for repairs where access is restricted and where the repairs need to fill an irregular shape.

Working underwater presents particular problems with regards to quality control and health and safety.

In general, the flexible systems can accommodate larger changes in channel stability than rigid systems, and are preferred where there is significant channel instability. The rigid systems are generally more resistant to surface erosion, so can provide good protection against high velocity and high turbulence.

Maintenance of scour protection measures can be difficult. First, it is often difficult to inspect for damage, as the system may be underwater and covered with sediment. Second, access for repairs can be difficult and costly.

The erosion protection systems described in the manual present no major adverse environmental impact over and above the general environmental impact of construction within a river environment. The environmental aspects that need to be considered when carrying out such construction work are discussed in Section 6.1, which gives details of the environmental issues particular to each protection system.

A study of US practice (by Melville and Coleman, 2000) indicated that monitoring is the most widely adopted approach for dealing with scour at bridges. The most common type of scour protection measure is riprap, which was reported to be in use in about 70 per cent of cases. The other main methods, each comprising between 3 and 9 per cent of cases, involved extending pier footings or using gabions, groynes or concrete aprons. A similar study in New Zealand (Holmes, 1974) that assessed the cost of scour repairs indicated broadly similar findings, with riprap and gabion protection methods accounting for more than 70 per cent of total expenditure on scour protection.

The following sections give advice on the design and construction of the main types of scour protection system. For more detailed information reference should be made to Escarameia (1998). A protection system should normally be designed to resist movement/damage of the armour layer and prevent loss of underlying material in flow conditions up to and including those occurring in the design flood. Guidance on the definition of the design flood and choice of appropriate return period is given in Sections 3.7 and 6.3.2. Information on how to determine values of flow depth, velocity and turbulence level occurring in the vicinity of a protection system is provided in Chapter 3. In navigation channels the passage of boats can produce additional currents and waves, and where appropriate these should be taken into account in the design of scour protection systems; methods of estimating the magnitudes of the waves and currents due to boats in restricted waterways are given in PIANC *Supplement to Bulletin No 57* (1987).

5.4.2 Riprap

Riprap is the term used to describe loose quarry stone with a wide grading, laid as scour protection. It is one of the most versatile and commonly used types of revetment, as it can generally be readily sourced, easily placed and can be specified to suit particular flow conditions. It is flexible and can accommodate small ground movements and some loss of stones without failure. Suitably sized riprap is appropriate as protection up to very high velocities and turbulence. It can be used to protect banks with slopes up to 1V:1.5H, without requiring additional restraint. Because of the flexibility in the shape of the area that can be covered, it is useful for protecting small awkwardly shaped areas and transitions between hydraulic structures and natural channels.

Riprap can be placed by machine and does not require hand placing or compaction. It can be placed underwater, although good quality control is needed to ensure an even coverage, and it is normal to increase the thickness placed to compensate for the greater difficulty of accurate placing. It can be placed in flowing water, although care is needed to avoid segregation and loss of the smaller stone fraction. The layout of riprap protection is often designed so that there is a surplus of material at the edges, so that if there is scour adjacent to the protection, stone will fall into the scour hole but will continue to provide protection. This is known as a falling (or launching) apron and is described further in Sections 5.4.5 and 5.5.2.

Riprap is normally placed on a filter of either geotextile or granular material to prevent loss of the underlying material through the riprap voids. Guidance on filter design is given in Section 5.6.

In designing and detailing an erosion protection system it is important to understand the main mechanisms causing failure in order to combat them. Four main failure mechanisms can be identified from experience and research:

- hydraulic failure due to the size (in fact the weight) of individual stone being inadequate for the flow conditions, characterised by the scattering of riprap stone around the protected area and loss in thickness of the riprap
- winnowing failure caused by erosion of the underlying bed material through the voids of the riprap, due to failure or omission of filter layers, from inadequate riprap thickness due to under-design or poor placing or as a result of poor grading of the rock, characterised by the stones being submerged within the bed of the channel
- edge failure due to the erosion of a scour hole in the natural bed adjacent to the protection, with stones at the outer edge of the riprap falling into the hole and leading to progressive failure, characterised by scour around the protection and loss of riprap around the edge of the protection
- bed movement undermining, where significant natural scour takes place, if riprap is placed on or at the original bed level – this type of failure can appear similar to winnowing failure, although more extensive movement of stone usually occurs laterally.

Sloping riprap can suffer from two further failure mechanisms:

- translational slide of the riprap down the slope, which normally occurs if the angle of the slope is too steep or if the toe of the riprap has not been keyed in adequately – where riprap is laid on geotextile there may be less friction between the rock and underlying soil, thus increasing the risk of sliding
- rotational slip failure of the soil mass beneath the riprap owing to an unstable slope.

Apart from slide and slip failures, failure tends to occur gradually, allowing time for repairs to be carried out, provided that the failure process is observed early enough. Riprap has fallen out of favour in the United States because of some recent failures, but is widely used worldwide.

Where stone is scarce, randomly placed precast concrete blocks have been used instead. In coastal situations, specially shaped proprietary concrete blocks (eg tetrapods) have been used instead of rock armour. The special shapes are designed to interlock better than standard rock shapes, in order to improve resistance to wave attack. They have also been used, although less extensively, in tidal and fluvial situations.

Riprap can be grouted with cement or bitumen for a less flexible, less permeable but stronger revetment.

Sizing

Many formulae have been proposed for sizing riprap and, like scour estimation, designers have been faced with several possible solutions which, in some circumstances, give greatly differing results. The variations generally arise because of limitations in the extent of laboratory testing of flow conditions on which the formulae are based, over-simplification of the parameters that affect riprap stability, and the use of different safety factors. Research on stability has shown that the main parameters affecting the stability of riprap are:

- flow velocity
- flow conditions (degree of turbulence)
- stone properties (density, shape)
- the location of the riprap (bed or banks).

Design engineers and researchers do not agree about which formula or theory gives the "best" estimate of stone size for riprap; the processes are too complex to represent by simple formulae. Nevertheless, designers have to make decisions based on the best available guidance. This manual includes three widely accepted formulae that have been based on good quality data and extensive research. Each formula requires an estimate of flow velocity and the degree of turbulence likely to be experienced. Both of these to some extent are subjective, so it is recommended that all three formulae are used and the design is based on:

- the average answer, or
- the largest answer if the consequences of failure are severe, or
- the average of the lower two answers if the consequences of failure are not severe and/or there is a commitment to monitoring on a regular basis.

The size of a stone is generally defined in one of three ways: as the diameter, d_s, of a sphere of equivalent volume; as the size, d_n, of a cube of equivalent volume; or as the size, d, of a randomly shaped stone intermediate in shape between a sphere and a cube. It is important that the correct definition is used when converting a stone size determined from one of the formulae below to a stone grading, otherwise considerable errors in the stone weight specified could occur. A sphere with a diameter the same as the length of a side of a cube will weigh nearly 50 per cent less than the cube. The three definitions, d_n, d_s and d (in m), are related to each other and to the mass of the stone, W (in kg), by the following formulae:

$$d_n = \left(\frac{W}{\rho_s}\right)^{1/3}, \; d_n = 0.806 d_s \text{ and } \quad d_{n50} = 0.84 \text{ to } 0.91 d_{50} \qquad \textbf{(5.1)}$$

where ρ_s (in kg/m^3) is the dry density of the sediment particles. Stone masses (in kg) are commonly (but incorrectly) termed stone weights. In this manual, for familiarity, the term "stone weight" is used in this and subsequent sections to refer to stone mass.

The three equations presented are due to Escarameia and May (1992), Pilarczyk (1990) and Maynord (1995). The Escarameia and May (1992) equation is a form of the Izbash equation. It was developed from laboratory tests on riprap, concrete blocks and gabion mattresses. Work in the Netherlands on the stability of riprap, stone mattresses and concrete block mattresses led to the Pilarczyk (1990) equation. The third equation is used in the US Army Corps of Engineers procedures. It was developed and refined by Maynord over a number of years, using laboratory tests and full-scale trials. It should be noted that the Escarameia and May (1992) and Pilarczyk (1990) formulae define stone size in terms of d_{n50}, while the Maynord (1995) formula uses d_{s30}.

When sizing riprap for a scour protection system, the worst case conditions in terms of water depth and flow velocity should be established. During the design flood, the main incised channel tends to increase its cross-sectional area as a result of natural and contraction scour (see Sections 4.1.4 and 4.2), leading to a reduction in flow velocity for a given value of discharge. For design purposes, the riprap should be sized on the assumption that the discharge in the design flood may initially occur while the channel still has its "normal" or long-term cross-sectional area; this is likely to be more severe than the condition that will apply later in the flood, when scouring of the channel may have temporarily increased its cross-sectional area towards the regime value corresponding to the design discharge.

Escarameia and May (1992)

$$d_{n50} = C_I \frac{U_b^2}{2g(s-1)} \tag{5.2}$$

where: d_{n50} is the characteristic size (in m) of the stone (equivalent cube)

C_I is a coefficient that takes into account the turbulence intensity, TI:

$$C_I = 12.3\,TI - 0.20 \tag{5.3}$$

g is the acceleration due to gravity (= 9.81 m/s²)

s is the relative density of the stone ($=\rho_s/\rho$)

U_b is the velocity near the bed (at 10 per cent of the local water depth above the bed; see Box 3.4).

Values of TI can be found from Table 5.4. Values of U_b should be determined from the information in Section 3.5 and Box 3.4. In the case of a sloping bank, U_b is the value of bed velocity at the toe of the bank.

Table 5.4 *Values of turbulence intensity under different scour conditions*

Scour condition	**Turbulence**	***TI***
Fairly straight lengths of channel or revetment	Normal	0.12
Around bends and at upstream ends of revetment	Normal (higher)	0.20
Around structures such as piers, caissons and cofferdams	High	0.35
Downstream of zones of energy dissipation (eg from weirs and structures producing high-velocity jets)	Very high	0.50–0.60

Pilarczyk (1990)

$$d_{n50} = \frac{\mu}{(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_S} \frac{U^2}{2g} \tag{5.4}$$

where: dn_{50} (in m) is the characteristic size of stone (equivalent cube)

μ is a stability correction factor

μ = 0.75 for continuous protection

μ = 1.0 to 1.5 at edges and transitions

s is the relative density of the stone

Ψ_{CR} is a stability factor, for riprap Ψ_{CR} = 0.035

K_T is a turbulence factor (not equivalent to turbulence intensity TI)

K_T = 1.0 for normal river turbulence

K_T = 1.5 to 2.0 for high turbulence (downstream of stilling basins, at bridge piers)

K_Y is a depth factor, for high turbulence flows taking the value:

$$K_Y = \left(\frac{d_{n50}}{y_o}\right)^{0.2}$$

y_o (in m) is the local water depth

K_S is the slope factor, defined as the product of a side slope term (k_d) and a longitudinal slope term (k_l), ie $K_S = k_d\, k_l$

$$k_d = \cos\alpha \sqrt{1 - \left(\frac{\tan\varepsilon}{\tan\phi}\right)^2} \quad \text{and} \quad k_1 = \frac{\sin(\phi - \chi)}{\sin\phi}$$

where ε is the angle of the bank to the horizontal, ϕ is the internal angle of friction of the revetment and χ is the angle of the channel invert to the horizontal

U (in m/s) is the depth-averaged velocity above the protection or, for a sloping bank, the value of U at the toe of the bank (ie U_t in Box 3.4)

g is the acceleration due to gravity (= 9.81 m²/s).

To find the required stone size from Equation (5.4), first estimate a value of d_{n50} and use it to calculate K_h. Then check the estimate with the value given by Equation (5.4) and repeat the process until satisfactory agreement is obtained.

Maynord (1995)

$$d_{s30} = S_f\, C_s\, C_v\, C_t\, y_o \left(\sqrt{\frac{1}{s-1}}\, \frac{U}{\sqrt{K_1\, g\, y_o}} \right)^{2.5} \qquad \textbf{(5.5)}$$

where: d_{s30} (in m) is the characteristic size of stone (equivalent sphere) of which 30 per cent is finer by weight

$$d_{s30} \approx 0.70 d_{s50}$$

S_f is a safety factor

C_s is a stability coefficient

C_s = 0.3 for angular rock

C_s = 0.375 for rounded rock

C_v is a velocity distribution coefficient

C_v = 1.0 for straight channels

C_v = 1.25 downstream of hydraulic structures and at the end of groynes

C_t is a blanket thickness coefficient, normally 1.0 if the thickness designed is as recommended below; otherwise refer to Maynord (1995)

y_o (in m) is the water depth

s is the relative density of the stone

U (in m/s) is the depth-averaged velocity above the protection or, for a sloping bank, the value of U at the toe of the bank (ie U_t in Box 3.4)

K_1 is a sideslope correction factor given by:

$$K_1 = -0.672 + 1.492 \cot \varepsilon - 0.449 \cot^2 \varepsilon + 0.045 \cot^3 \varepsilon$$

where ε is the angle of the bank to the horizontal

g is the acceleration due to gravity (= 9.81 m/s²).

The minimum value of safety factor, S_f, recommended by Maynord is 1.1 but this assumes that more riprap movement is allowable than is assumed for the Escarameia and May (1992) or Pilarczyk (1990) equations. A higher safety factor of 1.5 is therefore suggested for design.

Grading

Riprap gradings are normally given by weight (actually mass in kg) rather than size. It is generally, though not universally, acknowledged that a well-graded riprap is preferable to a single-size grading. A possible explanation of this is that the smaller particles tend to jam within the interstices of the larger stones and thereby increase their resistance to movement by fluid forces and transmitted loads. The smaller particles also reduce flow velocities at the riprap/underlayer interface and help prevent the underlying material being eroded and drawn out through the protective armour layer. However, it should be noted that for coastal structures, such as rubble-mound breakwaters, a single-sized stone may be more suitable than riprap, because it is able to provide additional dissipation of wave energy within the body of the structure.

Available information on specifications for riprap show that there is no clear consensus about the shape of grading curve that should be specified for riprap. The grading should lie within an envelope of upper and lower gradation curves. However, the grading envelope should not be so restrictive that production costs would be excessive. The following criteria are suggested for the upper and lower riprap grading envelopes:

W_{50} lower should not be less than the stone weight adopted as the characteristic stone weight from the formulae above

W_{50} upper should not exceed 1.5 times W_{50} lower

W_{100} lower should not be less than two times W_{50} lower

W_{100} upper should not exceed five times W_{50} lower

W_{15} lower should not be less than one-sixteenth of W_{100} upper

W_{15} upper should not exceed W_{50} lower.

An alternative way of expressing this, in terms of the characteristic stone weight W_{n50} determined from the formulae above, is:

$W_{15} = 0.3\ W_{n50}$ to $1.0\ W_{n50}$

$W_{50} = 1.0\ W_{n50}$ to $1.5\ W_{n50}$

$W_{100} = 2.0\ W_{n50}$ to $5.0\ W_{n50}$.

Typical gradings are shown in Figure 5.24.

More restrictive gradings have been used or recommended elsewhere, but in practice it can sometimes prove difficult to conform to them at a reasonable cost.

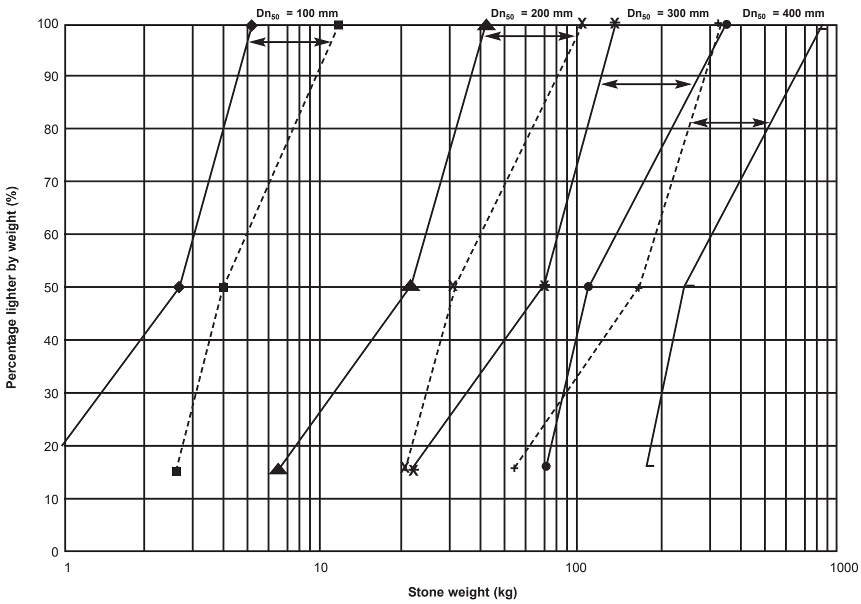


Figure 5.24 *Typical gradings for different stone sizes*

Angular stone is considered to be preferable to rounded stone due to improved interlocking, although some research has indicated that there is little difference in performance.

Thickness

A thickness of between $1.5 \times d_{50\,upper}$ and $1.0 \times d_{100\,upper}$ is recommended for normal conditions. Where riprap is placed underwater (in depths of greater than about 1.0 m) and in fast currents, this should be increased by about 50 per cent to $2.5 \times d_{50\,upper}$ or $1.5 \times d_{100\,upper}$.

Specification

Suitable specifications for riprap are given in CUR/RWS Report 169 (Centre for Civil Engineering Research and Codes, 1995).

5.4.3 Gabion mattresses, boxes and sacks

Gabions are wire mesh containers filled with stone. The flexibility of the mesh allows the containers to deform to the bed profile, while preventing the stone contained within from either shifting to expose the bed or from being removed from the revetment. Enclosing the stone within the mesh allows smaller sized stone to be used. Research comparing gabions with riprap also shows that a thinner revetment layer can be used. Thicknesses between one and two thirds of those required for riprap have been found from research.

The mesh in gabions is usually fabricated from steel wire, although polymer meshes are available. The wire is either woven or welded. Woven wire tends to be more flexible and may therefore be more appropriate where significant deformation may occur. Welded mesh is more rigid and is more suitable where it forms an earth-retaining function, rather than just an erosion protection function. Welded mesh containers are easier to handle and fill. Corrosion can be a problem, particularly in saline environments. The wire can be galvanised, or galvanised and then coated with PVC, for additional corrosion protection. For most river applications, PVC-coated wire should be specified. Gabions are less suited if there is significant wave attack because this tends to move the rock within the mattress, leading to stressing and abrading of the mesh. Their resistance to abrasion and wave attack can be improved by partially or fully grouting with mastic.

In gravel and cobble bed rivers, where abrasion may be a problem, the larger diameters of wire available (typically 3.0 mm) can be used. In abrasive conditions a second panel of mesh is sometimes laced onto the exposed face of the gabion mesh, at right-angles to the lower layer, to reduce the risk of holes developing in the facing mesh. However this is usually only considered where repairs are being carried out to an existing structure that has suffered abrasion since, if these conditions exist, then an alternative type of scour protection is probably more appropriate. Examples of the use of gabions are shown in Figure 5.25.

Local manufacturers in many countries can produce gabions, and the mesh can even be hand woven. However the quality of locally produced mesh can be variable and hence a careful specification is needed to ensure that the appropriate quality is supplied.

A wide range of sizes and shapes of gabion is available to suit different applications. Gabion boxes are normally produced with dimensions between 0.5 m and 2.0 m. These units are relatively large and inflexible and are used where flow velocities are high, for example as spillway linings, or where they perform a river-training or earth-retaining function.

Gabion protection to bridge invert

Gabion protection to bridge abutments

Gabion protection at energy dissipator

Figure 5.25 *Examples of gabion scour protection (photographs courtesy of Link Middle East Ltd)*

Gabion mattresses are more flexible than gabion boxes because they are thinner and because smaller stones and thinner wire are used in their construction. Due to their flexibility, mattresses are probably the most commonly used form of gabion for erosion protection. Where necessary, they can be pre-filled and placed under water, then wired together by divers. Mattresses are available in a range of sizes but typically have a thickness between about 0.15 m and 0.3 m. Where a protection thickness of 0.5 m or more is necessary, gabion boxes may be preferable to mattresses because of their greater strength. However, if additional flexibility is also needed, for example in a falling apron, the required thickness can be obtained by placing two mattresses on top of each other but not tied together. Farther away from the structure or area of strongest erosion, the protection can be reduced to a single layer of mattress. Gabion sacks range in diameter from about 0.5 m to 1.0 m and are generally 2–3 m in length. They are used mainly in locations where construction in the dry is not possible. The gabion sacks can be filled on land and then transported by barge and dropped or placed below water. Gabions can also be filled *in situ* on banks.

Gabions are normally filled by hand, although machine filling, particularly of gabion sacks, is possible. Careful construction is the key to a robust and successful gabion protection system.

There are several failure mechanisms for gabions that should be borne in mind; they can normally be avoided by good construction practice:

- failure of mesh leading to loss of stone from compartments within the protection system – causes include corrosion and abrasion, vandalism and theft, and damage during construction
- winnowing failure, due to erosion of the underlying bed material through the gabions – this can be caused by failure or omission of filter layers, or by inadequate gabion thickness for the flow conditions. This type of failure is characterised by gabions becoming submerged within the bed of the channel or by movement of one section of gabion relative to another
- edge failure, due to the erosion of a scour hole in the natural bed adjacent to the protection – although mattresses can accommodate significant movement, excessive movement can lead to mattresses breaking or being undermined
- excessive movement of stone within compartments. High flows will usually cause some slight displacement of stones towards the downstream end of each compartment. However, the amount of movement can become excessive if the velocities exceed those designed for, if the stone is poorly packed, or if the partitions forming the compartments are not spaced closely enough. The underlying material may then become exposed to current attack and, in extreme cases, the mesh may fail due to the additional stresses imposed on it.

Gabion sacks are not normally tied together in the same way as boxes or mattresses, so they have an increased risk of edge or winnowing failure.

A common concern is the long-term corrosion resistance of the gabion mesh. This can vary, but for most conditions a design life of several decades can be assumed. Other concerns are the problems of vandalism and theft. These problems tend to be relatively rare, but should be considered in known problem areas. In developing countries this can take the form of cutting out mesh for resale. In developed countries vandalism and theft of stone (for garden rockeries, for example) may be a problem. Riprap can suffer from the same problems of theft and vandalism. These problems can be overcome by random spraying of asphalt over the surface of the gabions or riprap, but this leaves them visually unattractive.

Under high flow velocities or high turbulence, using two mattresses, each of less than the required thickness laid on top of each other, provides better protection than a single mattress of the required thickness. Joints between mattresses should be staggered. Mattresses should be laid in such a way that the maximum dimension of a compartment in the direction of flow does not exceed 1.0 m. Internal diaphragms are normally provided to reduce the size of individual compartments and limit any movement of the stone. On a sloping bank, the diaphragms should normally be set parallel to the line of the top of the bank, while on a flat bed they should be at right angles to the direction of the flow.

Gabions are normally placed on a filter of either geotextile or granular material to prevent loss of the underlying material through the gabion voids. Guidance on filter design is given in Section 5.6.

Sizing

Suitable sizes of stone for use in gabions should first be determined on the basis of their stability against movement under design flow conditions; as described below, modified versions of Equations (5.2) and (5.4) for riprap can be used for this purpose. However, practical factors such as the dimensions of the gabion cells and the size of the mesh also need to be considered and may influence the choice of characteristic stone size – see the following sections on Grading and Thickness.

Alternatively, manufacturers can provide technical advice and information, and in some cases have produced design manuals based on experimental tests and practical experience. The design guidance is generally expressed in the form of limiting flow velocities for different thicknesses of gabion, with the stone sizes then being dependent on the mattress thickness and mesh size. The advice provided in this manual aims to combine these two approaches into practical guidance.

When sizing gabions for a scour protection system, the worst-case conditions in terms of water depth and flow velocity should be established. During the design flood, the main incised channel will tend to increase its cross-sectional area as a result of natural and contraction scour (see Sections 4.1.4 and 4.2), leading to a reduction in flow velocity for a given value of discharge. For design purposes, the gabions should be sized on the assumption that the discharge in the design flood may initially occur while the channel still has its "normal" or long-term cross-sectional area; this is likely to be more severe than the condition that will apply later in the flood when scouring of the channel may have temporarily increased its cross-sectional area towards the regime value corresponding to the design discharge.

Table 5.5 *Relationship between gabion thickness and mean velocity*

Mattress thickness (m)	Stone size d_{50} (mm)	Critical velocity (m/s)	Limiting velocity (m/s)
0.15–0.17	85	3.5	4.2
	110	4.2	4.5
0.23–0.25	85	3.6	5.5
	120	4.5	6.1
0.30	100	4.2	5.5
	125	5.0	6.4
0.50	150	5.8	7.6
	190	6.4	8.0

Based on information from Agostoni (1988)

Manufacturers' literature suggests "critical" and "limiting" velocities for different mattress thickness, as shown in Table 5.5. The critical velocity is the velocity to initiate movement of the stone within the gabion. The limiting velocity is the velocity at which the mattress reaches the acceptable limit of deformation. It would be normal to design to this critical velocity and assume the ability to accept higher velocities up to the limiting velocity before failure as a safety factor. The values given appear comparable to Equation (5.2) at normal levels of turbulence, but may be less appropriate for high turbulence conditions.

Equations for gabions due to Escarameia and May (1992) and Pilarczyk (1990) are similar in form to the corresponding Equations (5.2) and (5.4) for riprap, but have slightly different coefficients as shown below.

Escarameia and May (1992)

$$d_{n50} = C_I \frac{U_b^2}{2g(s-1)} \qquad \textbf{(5.6)}$$

where the parameters are as given in Section 5.4.2, except that:

$$C_I = 12.3\,TI - 1.65 \qquad \textbf{(5.7)}$$

Pilarczyk (1990)

$$d_{n50} = \frac{\mu}{(1-p)(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_S} \frac{U^2}{2g} \qquad \textbf{(5.8)}$$

where the parameters are as given in Section 5.4.2, except that:

Ψ_{CR} (the stability factor) = 0.07 for gabions

p is the porosity of the stone filling the gabions, which is unlikely to be greater than 0.4.

Grading and thickness

There is a lower bound on the stone size, which is limited by the requirement that the gabion mesh retains the stone. Gabion mesh is normally produced in two standard sizes 60 × 80 mm and 80 × 100 mm. The size is the nominal size of the openings in a panel of mesh. A third size, 100 × 120 mm, is also sometimes used for large box gabions. The smallest acceptable stone size is about 70 mm for 60 × 80 mm mesh and 90 mm for 80 × 100 mm mesh. The maximum stone size considered acceptable is about 250–300 mm – if any larger, close packing and handling become difficult. If there is a significant degree of wave attack, the stone size should not be less than about 150 mm, because smaller particles can tend to be sucked out through the mesh by the strongly fluctuating flow forces.

The thickness of gabion mattresses should be about 2.0 times the characteristic stone size used. In the case of sack gabions, it is suggested that in general the manufacturer's advice be sought on suitable thicknesses and diameters for particular applications.

Stone for gabions is normally nominally of a single size, within the sizes suggested below. The size of stone used in design is linked to the minimum and maximum acceptable stone sizes and the thickness of the mattress, as well as the characteristic stone size determined above.

Four principles should be observed:

- the minimum stone size should not be less than the characteristic stone size
- the maximum stone size should not exceed two thirds of the mattress thickness
- the minimum stone size should not be less than 70 mm for 60 mm × 80 mm mesh and 90 mm for 80 mm × 100 mm mesh
- the maximum stone size should not exceed 300 mm under any circumstances.

5.4.4 Articulated concrete blocks (cable-tied and interlocking)

Concrete block revetment comprises a single layer of precast concrete blocks laid on a geotextile or granular filter. The blocks are generally cellular, with up to about 20 per cent of their plan area open, although solid blocks are also manufactured. They may take the form of individual blocks that interlock with adjacent blocks (interlocking blocks), or they can also be linked into a mat using cables running through the blocks (cable-tied blocks). The cables are either galvanised steel or polyester. Mat sizes of about 6 m × 2 m are typical.

Cable-tied mats are placed by machinery, and can be placed under shallow depths of water. Interlocking blocks are normally placed by hand, but can only be placed in water depths of up to about 0.5 m of clear water; they are suited to locations where access by machinery is difficult. The mats and blocks adjust to minor movements, while retaining their interlocking characteristics, although large movements should be avoided. The interlocking provides strength and can be enhanced by blinding the blocks with gravel. Above water, the cells of the blocks can be filled with topsoil and seeded to improve both protection and appearance. Compared with interlocking blocks, cable-tied blocks are easier to lay, stronger and more resistant to failure, but are less adaptable for laying small areas or in confined areas. Concrete block construction is shown in Figure 5.26.

Figure 5.26 *Interlocking concrete block protection to bridge (photograph courtesy of the Environment Agency)*

This type of revetment has historically been used more in coastal applications to resist wave attack, although it is suitable for fluvial conditions and current attack. When used for scour protection it is important that the edges of the revetment are adequately toed into the underlying material to prevent the edges being undermined. This is particularly important for the leading edge of the revetment, which generally experiences movement of the adjacent bed material first. Where the bed of the river forms a series of migrating dunes, the edges of the revetment should be toed into the bed below the level of the dune trough.

Gaps at the interface between the concrete blocks and the structure being protected should be kept to a minimum. Any gaps that are present should be grouted, or otherwise filled, to prevent underlying material being lost. Cable-tied blocks should be anchored either to the structure or into the underlying material.

It is recommended that the edges of the revetment be securely anchored into the underlying material. This should avoid the revetment being lifted at the edges and, under high turbulence, "rolled up". Suitable anchors can generally be supplied by the block manufacturer for this purpose. Where the revetment covers a large area experiencing high flow velocities, anchors should also be installed at regular intervals under the revetment to avoid the mattress lifting away from the underlying material.

Some of the main modes of failure have been outlined in the paragraph above:

- development of gaps between the revetment and the structure, leading to loss of underlying material and progressive movement of the blocks.
- edge failure due to the erosion of a scour hole in the natural bed adjacent to the protection, causing blocks at the outer edge of the protection to fall into the hole and leading to progressive failure. Rapid collapse of interlocking blocks can occur where sloping protection is inadequately toed in – scour at the toe can lead to wholesale movement of the revetment down the slope
- undermining of blocks by bed movement – where the revetment is placed on or at the original bed level and significant natural scour takes place, only small movements of interlocking blocks can be accommodated before the underlying material is exposed. Significant bed movement should therefore be avoided
- lifting of unanchored edges of the revetment – without suitable anchoring, currents can lift the edges and, in extreme cases, roll up the whole revetment
- plucking out of individual blocks from a non-cabled mattress, due to the blocks being inadequately sized or insufficiently interlocked. Loss of blocks can be progressive and lead to rapid failure.

As with other revetments, a damaged or unsuitable filter layer can lead to winnowing failure of the underlying material through the blocks. Piping failure caused by build-up of pore pressure within the river bank is not generally a problem, because of the open nature of the blocks. Piping can be problem for solid blocks that have been grouted, but both the problem and this type of revetment are uncommon in fluvial situations. Under-drainage and pressure relief should be provided where needed to avoid piping failure.

Sizing

In the past, concrete block revetment has been used more in coastal situations, so manufacturers' literature most commonly gives design guidelines in terms of protection against wave attack, rather than current attack. Manufacturers should be consulted on the performance of their particular blocks, as these can vary in size, weight and thickness.

When sizing concrete blocks for a scour protection system, the worst-case conditions in terms of water depth and flow velocity should be established. During the design flood, the main incised channel will tend to increase its cross-sectional area as a result of natural and contraction scour (see Sections 4.1.4 and 4.2), leading to a reduction in flow velocity for a given value of discharge. For design purposes, the blocks should be sized on the assumption that the discharge in the design flood may initially occur while the channel still has its "normal" or long-term cross-sectional area; this is likely to be more severe than the condition that will apply later in the flood when scouring of the channel may have temporarily increased its cross-sectional area towards the regime value corresponding to the design discharge.

For determining the thickness of interlocking concrete blocks it is suggested that Equation (5.2) due to Escarameia and May (1992) be used, with the coefficient C_I for the effect of turbulence intensity *TI* (see Section 5.4.2) being given by:

$$C_I = 9.22\,TI - 0.15 \qquad \textbf{(5.9)}$$

The characteristic stone size d_{n50} used in the equation should be considered equivalent to the block thickness T (in m). Reference should be made to manufacturers' literature for the density of the blocks, which is usually about 1800–2000 kg/m^3.

It has been commonly accepted that cable-tied systems of concrete blocks have greater resistance to erosion than interlocking blocks due to the restraining forces provided by the cables. The strength provided by the cables is mobilised only when the blocks begin to move and lift away from the underlayer, by which time the revetment can be considered to be failing. The cables should be considered as a safety mechanism, therefore. For cable-tied blocks in high turbulence conditions, it is recommended that the Escarameia and May (1992) equation for interlocking blocks (see above) be used, with no account taken of the cable strength. Under normal or low turbulence conditions a block thickness formula developed by Pilarczyk (1990) for cable-tied concrete blocks is applicable. It is based on Equation (5.8) for gabions, but with different coefficients.

Pilarczyk (1990)

$$T = \frac{\mu}{(1-p)(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_S} \frac{U^2}{2g} \qquad \textbf{(5.10)}$$

where the parameters are as given in Section 5.4.2, except that:

T = block thickness (in m)

μ = 0.5 for continuous protection

μ = 1.0 for edges and transitions

Ψ_{CR} (the stability factor) = 0.07 for cable-tied blocks

p is the porosity of the blocks

It is likely that this equation will give a block thickness for cable-tied mattresses of the order of 75 per cent of the thickness for interlocking blocks (by a non-rigorous comparison of coefficients used in Equations (5.9) and (5.10)).

5.4.5 Falling aprons

The perimeter of a scour protection system is always a potential area of weakness, because erosion of the natural bed or bank adjacent to the protection can result in undermining and lead to the edge of the protection dropping into the developing scour hole. The change in surface roughness between the protection system and the natural bed or bank may also intensify the erosion, particularly if the top of the protection layer is higher than that of the adjacent bed. A common method of safeguarding against this type of failure is to install a falling apron (also known as a launching apron) around the perimeter of the main protection system. The apron consists of additional loose material such as riprap or a flexible mattress that can "deploy" by following the shape of the developing scour hole and armouring the sloping face so that it does not over-steepen or undercut the main protection. Sufficient material needs to be provided in the apron for it to be able to armour the sloping face down to the lowest anticipated scour level. Falling aprons are commonly used where it is not practicable to excavate or place protection material down to this lowest level. They have been used to protect against potential scour depths of 10 m or more, but large aprons require considerable care in their design and placing, to ensure that they will be able to deploy when required (perhaps many years after construction) and in the way intended.

Materials used in falling aprons need to be highly flexible so that they can adjust to substantial movements, while still providing continuous cover to the underlying natural material. The main choices are normally either riprap or gabion mattresses. Flexible concrete mattresses and cable-tied concrete blocks can also perform the same function, but have more limited flexibility. If suitable rock is not available, different sizes of concrete cubes randomly placed can be an option for use in areas where the turbulence of the flow is relatively low. More information on the design of falling aprons is given in Section 5.5.2.

5.4.6 Rigid grout-filled mattresses and bags

Bags, sacks and mattresses filled with cement grout or concrete can be a cost-effective method of scour protection. The bags may either be filled with a dry mixture that hydrates and hardens on contact with water, or grout can be pumped into the bags. In the past, the fabric used was hessian, but today synthetic fabrics such as polyester and polypropylene are used. The fabric acts as a shutter to retain the grout and form the shape of the revetment. For walls, bags may be used in a dry-stone wall type of construction, where it acts as an earth-retaining gravity structure. Mattresses (and less commonly bags) can also be laid on stable slopes and beds.

This type of scour protection is used particularly in underwater conditions and for temporary repairs. It is often used to form a concrete apron underwater, with the mattresses being filled from floating or land-based plant using tremie pipes. Adjacent mattress compartments can be connected with straps and ties or proprietary connectors. Mattresses can be strengthened with reinforcement bars or mesh. Examples of their use as bed and slope protection are shown in Figure 5.27.

The revetment is essentially an impermeable and inflexible revetment. Where a build-up of hydrostatic pressure could occur, the weight should be sufficient to avoid uplift or allowance should be made for pressure relief. If uplift is a problem the mattresses can be prefabricated with filter drains or drain holes to relieve pressures. As for concrete block revetment, some key design details should be observed:

- the edges of the mattress should be toed into the underlying material, particularly at the leading edge
- the toe of the mattress should be deeper than the trough of dunes, if these bedforms occur
- gaps should be avoided between the mattress and the structure; if the bagwork or mattress cannot be placed close enough to the structure, remaining gaps should be grouted
- the revetment should be secured with anchors, particularly at the edges.

The main mode of failure with this type of protection is undermining, leading to gradual collapse of individual compartments. Vulnerability is greatest where lateral or vertical channel movement can be expected, in which case flexible mattresses or bags may be a better alternative.

Where there is the potential for settlement of the underlying ground (for example, if the mattress is laid on made ground), it may be necessary to provide a flexible joint between the mattress and structure being protected, to allow for differential movement.

Grout-filled mattress protection to channel bank

Grout-filled mattress protection to sheet-pile wall

Figure 5.27 *Examples of grout-filled mattress protection (photographs courtesy of Proserve Ltd)*

Design

Because this type of revetment has, in the past, been mainly used for repair work by specialist contractors, design procedures have not been widely researched. The basis for design is therefore experience and commonsense. Manufacturers should be able to provide technical information on the range of conditions for which their products are suitable.

5.4.7 Flexible mattresses and bags

Alternatives to rigid grout-filled systems are flexible filled systems. Flexibility can be introduced to grout-filled mattresses by suitable connectors at joints and, to a certain extent, between pockets, by the use of continuous seams within a mattress. These mattresses can be strengthened with cables.

Bags, sacks and mattresses can also be filled with sand or locally won fill to provide a more flexible revetment than grout-filled systems, and where cement or rock is too costly or not readily available.

The use of large geotextile bags filled with sand has proved very useful for providing heavyweight erosion protection underwater. Bags as large as 0.5 m^3 or more have been used successfully. They can be placed individually by mechanical grab or in groups using split-bottom barges (as for dumped rock). The geotextile is usually a strong needle-punched fabric and the bags are sewn using high-strength binding. Underwater these bags can be used as a substitute for large rocks and can provide a particularly economic solution where sand is readily available and rock is not. The filled bags are reported to have a long life and can resist mechanical damage, although advice from manufacturers should be sought if the risk of damage is high (from trailing anchors, for example).

Whereas concrete grout-filled mattresses remain a robust protection system after the fabric bags deteriorate or are torn, sand-filled mats rely on the retention function of the fabric to be secure and are therefore less robust. Above water, the effects of ultraviolet deterioration and potential vandalism have to be taken into account. Below water, damage by dredging and abrasion has to be considered. Care must be taken to avoid damage during construction.

Sand-filled mats have been found to withstand velocities of about 2.5 m/s. For protection against higher velocities, large sand-filled bags or cement and concrete grout-filled bags and mattresses should be used.

5.4.8 Bituminous systems

Bituminous scour protection systems generally involve binding a loose material such as sand or stone with bitumen. The flexibility of the bitumen allows movement, while its binding properties give strength, meaning that less filler material and smaller stone sizes can be used. It is widely used in the Netherlands, particularly on wide rivers and navigation channels, as stone is relatively scarce. Bituminous systems can take various forms: open stone asphalt and sand asphalt, which are both permeable; dense stone asphalt and asphaltic concrete, which are impermeable.

Open stone asphalt is a mix of asphalt mastic and stone, usually in the proportions, 20 per cent and 80 per cent by mass respectively. The mastic is composed of bitumen, a filler (such as ground limestone) and sand. It is normally placed above water, although pre-prepared mats are available for underwater use, and can withstand high flow velocities. Research from the Netherlands indicates it can withstand velocities up to about 7 m/s. Open stone asphalt is particularly suitable for large-scale protection in wide navigable canals and rivers.

Sand asphalt comprises sand coated with 3–5 per cent by weight of bitumen. A higher proportion of bitumen (6 per cent) has also been used in the United States, where it is known as bulk asphalt. It can be placed above or below water, but is only suitable as protection in its own right for low flow velocities of less than about 2 m/s. It does find uses as a temporary protection measure to stabilise exposed slopes, however. Sand asphalt is also used as a filter layer, underlying open stone asphalt, for example. Because it can withstand higher velocities than a normal sand layer, it is used where currents would wash out a normal sand filter before an armour layer could be placed.

Dense stone asphalt is a mix of asphalt mastic and stone, with a larger proportion of asphalt mastic than for open stone asphalt. The normal proportions are 50–70 per cent stone and 50–30 per cent asphalt mastic by weight. It can withstand moderate flow velocities of 2–5 m/s and can be placed underwater. Since it is impermeable, pressure relief should be provided where excess porewater pressure could be generated.

Asphaltic concrete comprises a mix of sand, gravel, filler and 6–9 per cent bitumen by weight, the aim being to produce a dense mix with a low voids ratio. After laying, it should be compacted to increase its density. It is normally placed above water, unless prefabricated.

The following key design details should be observed to reduce the risk of failure:

- uplift pressures should be allowed for if an impermeable type is used
- suitable toe details are needed to avoid the revetment sliding down the slope of a bank
- a good formation should be provided to avoid excessive settlement
- joints between panels of revetment should be sealed, or adjacent panels should be overlapped with the lap facing downstream.

It should be noted that the effects of ultraviolet radiation and weathering reduce the flexibility of the bitumen over time. In UK conditions this is not normally significant.

Sizing

Guidance can be obtained from specialist contractors. As a general guide, the layer thickness for all types of bituminous system should be a minimum of two to three times the maximum stone size. With a typical maximum stone size being about 50 mm, this gives a layer thickness of 100–150 mm. Under severe current or wave attack, the stone size may be greater and a thickness up to about 250 mm may be required. Because of the difficulties of accurately laying revetment below water (except where using pre-formed mats), thicknesses in this case should be at the upper end of those recommended. For sand asphalt placed underwater, a thickness of 0.7 m is recommended. Since factors such as ageing, settlement and permeability are normally more important factors in the design of asphalt systems than resistance to erosion by currents, a full design method is outside the scope of this manual. Further guidance is given by PIANC (1987) and a Dutch publication produced by the Technical Advisory Committee on Water Defences (1985).

5.4.9 Biotechnical solutions

Under the broad heading of biotechnical solutions, a wide range of systems can be considered that combine the use of vegetation to bind or stabilise river banks with structural materials (natural or otherwise) that enable vegetation to become established and provide some form of scour resistance. Some of the systems are very traditional, such as fascine mattresses and faggots, whilst many have been recently developed with the advent of new fabrics such as geotextile fibre rolls. Some of the main forms of biotechnical solutions are described below. The perception has been that they are only suitable for very low velocities. Although this is true of some of the systems, others can offer a robust protection system at reasonably high velocities and turbulence levels.

Systems that are only suitable for low-velocity flow are not described below, as they are unlikely to be suitable for many scour protection situations. Concerns over their design life, lack of understanding on how to construct and maintain the systems and lack of information on their performance have hampered their use in the past. However, they are again becoming more widely used, and experience on their use is being developed. An example of their use as bank protection is shown in Figure 5.28.

Figure 5.28 *Construction of rock armour and willow faggoting repair to eroded river bank (photograph courtesy of Jeremy Benn Associates)*

Fascine mattresses

Fascine mattresses are three-dimensional woven mats consisting of bundles of willow brush and poles. They have been used for hundreds – probably thousands – of years as erosion protection and are mainly used underwater to protect the river bed near structures (eg downstream of weirs). The method has the benefit that bed material becomes trapped in the fascines, improving the protection. A form of willow mattress (also known as faggoting) can also be used above water with the benefit that live willow can be used. Once established, a dense root system binds the bank, affording additional bank stabilisation. The main problem with their large-scale use is obtaining sufficient materials and skilled labour.

Fascine mattresses have been successfully used recently in the Netherlands, where some of the traditional natural materials have been replaced by geotextiles. Although this results in a thinner mattress, the reduction in weight can make it harder to lay than traditional types. The newer mattresses are also more difficult to place in flowing water, but have been used effectively in canals and canalised rivers. The mattress is floated into position and then loaded with stone to sink it on to the channel bed.

Faggots

Faggots are bundles of willow (or thorn) poles laid in a variety of formations parallel and perpendicular to river banks and held in place by stakes and sometimes netting. They can be used to protect river banks both above and below water level. If live faggots are used, then additional protection is provided once the willows have established a root system. The faggots trap sediment or can be backfilled with excavated or dredged material.

Two- and three-dimensional soil reinforcement geotextiles

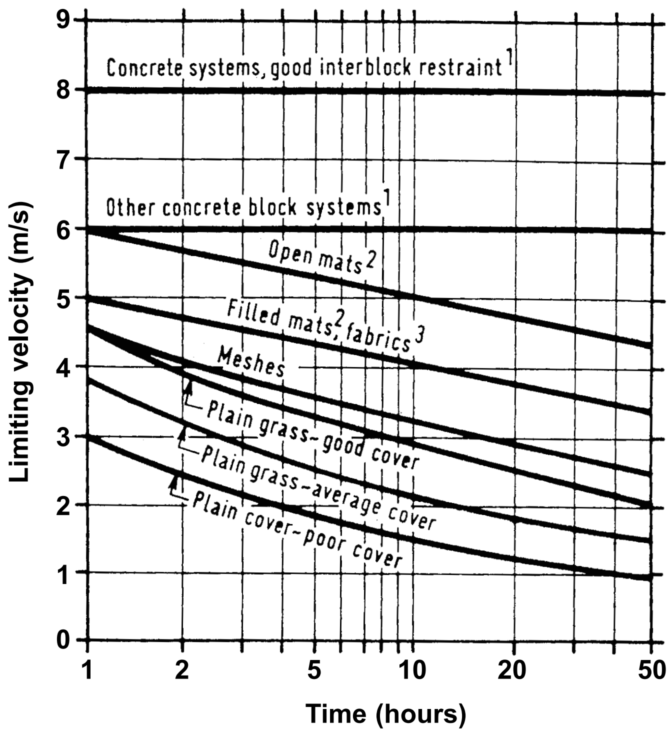
The erosion resistance of grass can be enhanced by the use of soil reinforcement mats. These can be two-dimensional fabrics or meshes or three-dimensional synthetic mats. The mats usually comprise a flat lower layer to provide good contact with the subgrade and an upper layer of entangled polypropylene (or similar) fibres. The mats are filled with topsoil after laying, and seeded. They can also be pre-seeded and pre-filled. Grass roots bind around the fibres and into the subgrade to form a continuous protection. The mats help to retain soil particles so assisting root structure. The mats can be further strengthened by being combined with a mesh reinforcement, such as steel gabion mesh or a polymer grid mesh. Because it is intended as a grass reinforcement, it is most commonly used above normal water levels and in ephemeral rivers, where grass can establish. It can also be used below water, where sediment is trapped by the mat. If used below water, additional strength in the form of a reinforcement mesh is usually required.

Two-dimensional fabrics and meshes perform a similar function to the three-dimensional mats. They retain the subsoil, but do not retain soil within their structure.

Design

The design of fascine mattresses and faggots is normally based on experience and commonsense. Hemphill and Bramley (1989) suggest that they are suitable for moderate to heavy current attack, with moderate defined as 1–2 m/s and heavy as 2–4 m/s. These should be assumed to be under low or normal turbulence conditions.

The effectiveness of soil reinforcement geotextiles depends on the flow velocity and duration of current attack. Over time, soil particles are gradually removed from the reinforcement and subsoil and the grass roots are weakened. Most of the research on the effectiveness of grass and soil reinforcement geotextiles has concentrated on steady flow parallel to the surface, rather than on turbulent scour conditions. Hence they should be used with caution, except under low turbulence conditions. A comparison of the limiting velocities for erosion resistance for unprotected grass and soil reinforcement geotextiles is given in Figure 5.29. The values apply to general species of grass and types of soil found in the UK and low to normal turbulence conditions. For further guidance refer to *The design of reinforced grass waterways* (Hewlett *et al*, 1987) and *Use of vegetation in civil engineering* (Coppin and Richards, 1990). For non-UK situations reference should be made to *Stability design of grass-lined open channels* (Templeton *et al*, 1987).



Recommended limiting velocities for plain grass, geotextile-reinforced grass and concrete block systems with unidirectional flow, subject to well-established good grass cover (except where otherwise specified) and the following specific notes:

1 Minimum superficial mass 135 kg/m^3

2 Minimum nominal thickness 20 mm

3 Installed within 20 mm of soil surface, or in conjunction with surface mesh

Figure 5.29 *Erosion protection of geotextile-reinforced grass (reproduced from CIRIA Report 116, Hewlett* et al*, 1987)*

5.4.10 Concrete aprons

Concrete aprons are commonly used to provide a highly scour-resistant surface. They are frequently used at the inlet and outlet of hydraulic structures and are also sometimes used to protect bridge inverts. Stilling basins could be considered as a specialised form of concrete apron, although their design is not covered in this manual. They are normally reinforced with nominal steel to resist cracking and any uplift forces. Mass concrete aprons are sometimes used, but generally only for small structures. Providing they are well laid, aprons can provide protection against very high velocities and high turbulence. For very high turbulence in gravel/boulder-transporting rivers, abrasion may require the adoption of greater than normal cover to reinforcement. Problems of cavitation experienced by high-head hydraulic structures are not encountered under the conditions covered by this manual. Concrete aprons are only normally laid in the dry. Where an underwater apron has to be laid, a grout mattress, described in Section 5.4.6 may be more appropriate.

Due to their high durability, concrete aprons rarely suffer direct erosion unless the concrete is of poor quality. Abrasion may be a problem, but generally only for spillways and high-head structures. The most common cause of apron failure is probably undermining. Aprons are inflexible, so any erosion of the underlying or adjacent bed material can expose the apron foundations. Once exposed, erosion can take place under the apron, which may be exacerbated by seepage through the underlying material. An example of an undermined apron is shown in Figure 5.30.

Figure 5.30 *Scour downstream of a small structure, highlighting the problem with an inflexible concrete apron*

Undermining can be avoided either by protection with an upstream and downstream concrete toe or a sheet-piled toe, or by laying the apron below the level of natural plus contraction scour. Alternatively, the apron can be combined with a flexible system such as riprap, with the riprap protecting the apron from undermining.

Uplift pressures can also cause the failure of thin concrete linings. In situations where water levels in the channel can fall rapidly, leaving an out-of-balance hydrostatic head under the concrete, entire panels can be lifted and damaged. In such circumstances, the weight of panels could be increased or the design should incorporate pressure relief holes with associated under-drainage so that excess pressures can be dissipated.

Design

Concrete apron design should follow normal standards and codes of practice for concrete structures. It is usual to provide a toe at the ends of the apron to reduce seepage under the apron and to protect against undermining. Care should be taken when designing aprons at structure outlets to ensure exit velocities are close to downstream channel flow velocities.

5.4.11 Stone pitching

Stone pitching is hand-placed stone, a traditional form of erosion protection that is still in widespread use worldwide. It usually consists of a single layer of single-sized stone, closely packed and laid on a gravel bed in dry conditions. It is labour-intensive, but needs little or no plant, so it is particularly suited to isolated locations, small areas and where labour is cheap and plentiful. When local stone is used, it can provide a more attractive finish than riprap or concrete block revetments. It can be placed with no mortar in the joints ("dry pitching"), or the joints can be grouted with a cement mortar or bitumen. The use of grouted stone pitching to protect a bridge invert is shown in Figure 5.31.

Grouted stone pitching increases the stability of the protection and reduces the risk of loss of individual stones that could lead to progressive collapse. It also allows smaller stone sizes to be used. Grouting the stone, however, makes it less permeable and less flexible. The pitching can be grouted to its full thickness for a strong but impermeable and inflexible revetment, or surface or pattern-grouted where a more permeable and flexible revetment is required.

Figure 5.31 *Grouted stone pitching to bridge invert (photograph courtesy of Czech Railways Infrastructure Division)*

Either grouted or dry, stone pitching is relatively inflexible and can only accommodate small movement or settlement. Because of its inflexibility, a sound, well prepared subgrade is important to minimise settlement. In addition, regular inspection is recommended, as loss of individual stones can rapidly develop into widespread collapse.

Placing the pitching on a gravel bed is important because it facilitates placing and provides a filter, reducing the risk of loss of subgrade. Typically, pitching stones would be laid to a depth of 0.2–0.4 m, on a gravel backing (20–50 mm size), 0.1–0.2 m thick. Slopes should not exceed 1V:1.5H and 1:2 is preferred.

Like other inflexible systems stone pitching mainly fails by undermining. As such, a secure toe is needed on slopes. It is not well suited to applications where large lateral or vertical movement of the channel could occur. For example, where laid on slopes, if the toe of the slope is inadequately keyed in, then scour at the toe can lead to wholesale loss of ungrouted stone. In similar conditions, slabs of grouted stone can collapse under their own weight in a cantilever action and slip down the bank.

Design

Guidelines on the design of ungrouted stone pitching are given by Indian Standard IS 8408 (1976) and an Indian Road Congress (1985) report. Equations for stable stone sizes recommended in these documents can be expressed in the following form:

$$d_{n50} = C_p \frac{U^2}{2g(s-1)} \quad \textbf{(5.11)}$$

where:

d_{n50} is the characteristic size (in m) of the stone (equivalent cube)

g is the acceleration due to gravity (= 9.81 m/s^2)

s is the relative density of the stone (= ρ_s/ρ)

U (in m/s) is the depth-averaged velocity at the toe of the bank (ie U_t in Box 3.4).

The coefficient C_p depends on the slope of the bank as follows:

slope 1V:1H C_p = 1.23
slope 1V:2H C_p = 0.74
slope 1V:3H C_p = 0.56

It is recommended that the weight of individual stones used for pitching should not be less than 40 kg. The thickness, t (m), of the pitched layer is normally between 0.3 m and 1.0 m, but may be increased for large guide banks in major rivers. Indian practice suggests the following formula for determining t, subject to the above limits:

$$t = 0.06\, Q^{1/3} \qquad \textbf{(5.12)}$$

where Q (m^3/s) is the discharge in the channel. A graded filter with a thickness of 200–300 mm should normally be used (see Section 5.6 for details of filter design).

In cases where the stone is fully grouted, the pitching can be considered as a monolithic structure, rather as individual elements, and should be designed accordingly. Where the stone pitching is surface- or pattern-grouted, it is less clear what reduction in stone size compared with riprap can be achieved. For these cases, it is recommended that the grouting is considered to provide an implicit additional safety factor that gives greater long-term durability, rather than explicitly taking account of it in sizing the stone pitching.

5.4.12 Sheet piling

Sheet piling is a semi-structural measure for protecting against scour. Instead of preventing erosion of the bed material and the development of scour holes, it prevents loss of material from beneath a structure and thus protects it from undermining. It can be used to protect pier and abutment foundations, as well as the upstream and downstream ends of hydraulic structures such as the end sills of stilling basins. The fact that it does not prevent the development of scour holes should be borne in mind when designing nearby works. The effect of allowing scour hole development is to improve energy dissipation and therefore reduce turbulence nearby. However the visual appearance of a scour hole may be unacceptable or cause alarm if the operator of the structure does not know or understand that scour hole development is allowable.

A similar alternative to sheet piling is to extend the concrete toe of a structure vertically down in the form of a cutoff. Because of the excavation and formwork required to construct a concrete toe, it may only be more cost effective than sheet piling for small depths (of say 0.5 m or less), or where the use of sheet piling is impractical, for example, because of poor access.

The design of the sheet piles should be in accordance with good piling practice such as the *Piling handbook* (British Steel, 1997) and codes of practice for earth retaining structures (eg BS 8002: 1994), taking into account the predicted scour depth. The factor of safety of cantilever sheet pile walls reduces rapidly as scour depth increases and hence embedment depth reduces. A suitable safety factor should be applied to take account of the uncertainty in the estimated scour depth (see Section 5.3.1).

5.4.13 Protective collars

Protective collars, also called deflectors, are a specialised form of protection suitable for pier protection only. They take the form of thin horizontal plates attached around the pier at or below bed level, to reduce the intensity of the downward current at the pier by deflecting and interrupting it. Research has indicated that they can reduce scour substantially. Both circular and egg-shaped collars have been tested, with little difference found between the two. Only relatively limited research has been carried out to date on their effectiveness. Research has been limited to clear-water scour conditions, and their performance under live-bed conditions is not known. It is recommended that collars only be considered for situations with low sediment transport.

There is insufficient research and practical experience to develop full design methods, but the following general principles can be used:

- the collar should be located at or below the original bed level (scour reduction increases with the distance of the collar below bed level; if the collar is too high secondary downward currents can develop below the collar)
- scour reduction is strongly influenced by collar diameter (a collar of diameter two times the pier diameter placed at bed level reduces the scour depth by about 20 per cent; a collar of diameter three times the pier diameter, at the same elevation, reduces the scour depth by about 50 per cent).

A tentative plot of the magnitude of scour reduction from protective collars is given in Figure 5.32, based on data analysed by Fotherby and Sterling Jones (1996).

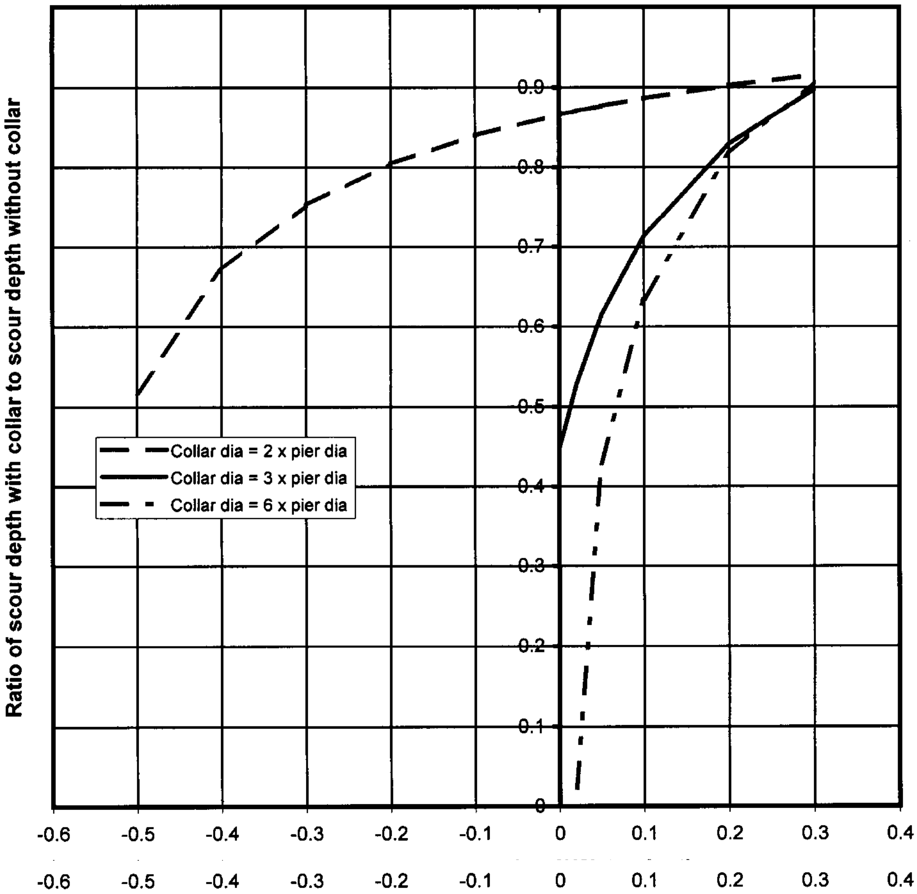


Figure 5.32 *Scour reduction due to protective collars (based on data by Fotherby and Sterling Jones, 1966)*

5.4.14 Pile caps and footings

Where pile caps and footings extend horizontally beyond bridge piers, they can act to reduce scour in the same way as protective collars (Section 5.4.12). There are situations where they can increase scour significantly, however, so it is important to understand how they affect scour depths. If they are located above the bed level, or above the level of natural and contraction scour, then they present an increased obstruction to the flow and thus increase scour depths. If placed below this level, then the downward current causing local scour at the pier can be interrupted and deflected by the top surface of the footing or pile cap to disrupt the formation of horseshoe vortices.

Although some research has been carried out to determine the benefits for scour depth reduction, the results are not yet extensive enough to enable firm design procedures to be formulated. In addition because of the risk that they will increase scour pile caps and footings should be used with caution and in conditions where scour can be well defined. Nevertheless, the following principles can be followed:

- the pile cap or footing should be located below the level of natural and contraction scour to have a beneficial effect
- scour reduction is strongly influenced by the diameter and elevation of the pile cap or footing.

5.5 DESIGN DETAILS

5.5.1 General

Although some details of design are particular to a given type of structure or type of scour protection measure, there are some general principles of good design that apply to all scour protection situations.

First, it is important to have a good foundation to scour protection. Settlement of the subgrade under the weight of scour protection is a factor in many cases of failure. This means that the subgrade should be firm, even and free from large boulders. A suitable filter must also be installed between the scour protection and the subgrade. This performs three functions:

- prevents loss of material from the subgrade due to flow velocities at the soil–water interface
- separates the scour protection and the subgrade to prevent movement of the subgrade into the scour protection and vice versa
- acts as secondary protection, reducing flow velocities at the subgrade surface and the migration of fines behind the filter.

The filter can be of granular type or a geotextile. Underlayer design is discussed further in Section 5.6.

Second, the edge detailing is particularly important for revetments. The edges are the most likely parts of the revetment to experience scour as the adjacent bed or bank material is eroded. Therefore, for sloping revetments, a well-designed toe is essential to protect the lower part of the revetment and to prevent the revetment slipping down the slope. For revetments on the bed, it is important that the edges are either designed to be flexible, so that they can move and retain their armouring function if the adjacent bed material is eroded, or that the edge is keyed into the bed material below the level of any scour. The transition to natural river bank should be carefully considered, with protection on river banks keyed into the bank. It is also important to consider whether seepage under a revetment could be a problem leading to undermining of its foundations. To combat this, cutoffs may be necessary.

In all situations, it is useful to consider the flow conditions in the vicinity of the structure. The aim should always be to ensure smooth flow transitions in the approach to and exit from structures, to reduce turbulence as far as possible. Smooth flow transitions can be encouraged by good design of the alignment and layout of the structure itself, by channel alignment, and by the alignment and layout of scour protection measures themselves.

The elevation of the scour protection should be carefully considered. Where bed degradation is possible, the scour protection should, where practical, be placed below the level of natural and contraction scour (see Sections 4.1 and 4.2). In some cases, this may be impractical because of the cost and the difficulty of excavating to this level. If this is so, then the top of the protection should if possible be set flush with the bed. When the scour protection is not below the level of natural and contraction scour, the type and extent of protection selected should take account of possible bed degradation occurring upstream or downstream and the possibility of the protection itself obstructing the flow and inducing additional scour.

For flexible protection, a launching apron should be provided that can fall into the scour hole. This provides protection between the levels of the original bed and scoured bed. Where the protection is inflexible, a toe or cutoff should be considered to prevent undermining. However, it should be noted that the performance of many of the scour protection measures significantly improves the deeper they are placed within the bed.

The practicalities of scour protection construction should be carefully considered at design stage, as there are often practical problems. Flow and water level conditions during the construction stage affect the choice and design of revetment and its filter layer. The construction sequence affects where and when a revetment can be laid. Access constraints may include low headroom when providing protection under a bridge, deep water and problems gaining access into the river for floating plant.

For impermeable revetments (and those that could become impermeable over time by blockage) consideration must be given to whether excess porewater pressures can build up behind the revetment. If so, suitable pressure relief should be provided.

5.5.2 Falling aprons

A falling apron can be placed so that it extends out horizontally from the main protection and is able to deploy by simply rotating downwards as the scour hole develops; this is suitable for loose materials such as riprap and for flexible mattresses. For loose materials only, an alternative is to place the required volume of material in a thicker layer at the edge of the main protection; deployment of the apron is then achieved by the material rolling downwards and outwards into the developing scour hole. The quantity of material in the apron should be sufficient to armour the side of the scour hole adjacent to the main protection down to the predicted lowest level of scour. Varma *et al* (1989) recommend that, for design purposes, the slope of the scour hole should be assumed to be between 1V:2H and 1V:3H. If the apron is curved in plan and is required to deploy outwards from the centre of curvature, the volume of material provided in the apron should take account of the greater perimeter of the scour hole at its base.

In the case of loose materials such as riprap, the thickness of the falling apron after it has deployed needs to be greater than in an engineered system such as a revetment if it is to provide a similar degree of scour protection. This is because the stones do not spread uniformly as the apron deploys and a certain proportion of them may be carried away by the flow. In addition, some segregation of the coarser and finer fractions of the grading may occur. For these reasons, it is recommended that the volume of material placed in a falling apron should be sufficient to produce a protective layer that is 1.5 times the thickness of an engineered protection system using the same material. The following guidelines for the design of falling aprons at the toes of bank revetments were developed by Gales (1938) from experience gained at the Hardinge Bridge over the River Ganges and appear to have been applied quite widely.

- Assume that the maximum predicted depth of scour below the level of the toe of the revetment is Y_S (in m), see Section 4.3.4, and that the thickness of the rock armour on the sloping face of the revetment is T (in m), see Section 5.4.2
- assuming that the face of the scour hole has a slope of 1V:2H, the slope length to be protected by the falling apron is $\sqrt{(1^2+2^2)}Y_S = 2.24\ Y_S$
- if the deployed apron is required to have a thickness of 1.5 T, the volume of material required is 3.35 $T\ Y_S$ m^3 per m length of straight bank
- Gales (1938) recommends that the material in the apron should be placed in a layer of constant thickness projecting a distance of 1.5 Y_S from the toe of the bank. It follows that the thickness of the apron as initially placed should = 2.24 T.

Alternative configurations of falling apron have also been used and further information can be found in Varma *et al* (1989). An Indian Road Congress (1985) report recommends that, at the toe of the bank, the bed should be locally excavated and filled with armour material in order to provide a keyed section to help maintain the stability of the bank if and when the apron deploys. The report suggests that the keyed section should be trapezoidal in cross-section extending to a depth of 0.75 m below the underside of the falling apron, with a vertical back face at the toe, a base width of 0.5 m (measured normal to the line of the toe), and a width of 1.0 m at the level of the underside of the falling apron. Whatever layout is adopted, it is important that the design should provide continuity of protection between the bottom of the revetment and the start of the apron, and that any geotextile or filter layer within the revetment should be retained in place at the toe.

If a granular underlayer is provided beneath the scour protection it will suffer the same problems as the scour protection of losses due to current forces, movement and segregation. Unless account is taken of this, by providing an excess quantity of underlayer material, then it is unlikely adequately to perform its filter function. As a guide, it is suggested that twice the normal underlayer thickness be provided. An alternative, particularly where the size of stone used for the scour protection is relatively small, is to redesign the riprap grading so that it contains a sufficient quantity of larger stone to resist the erosive forces but also sufficient fines for filter purposes. A greater than normal thickness should be provided (at least 50 per cent).

Where a geotextile underlayer is used, the problems of segregation or loss of material are not encountered, but there is the potential for damage of the geotextile and a significant risk that it will not take up the profile of the falling apron. For added safety it may be appropriate to redesign the riprap grading to perform a dual function as erosion protection and filter.

A range of possible layouts for toe details and falling aprons is shown in Figure 5.33.

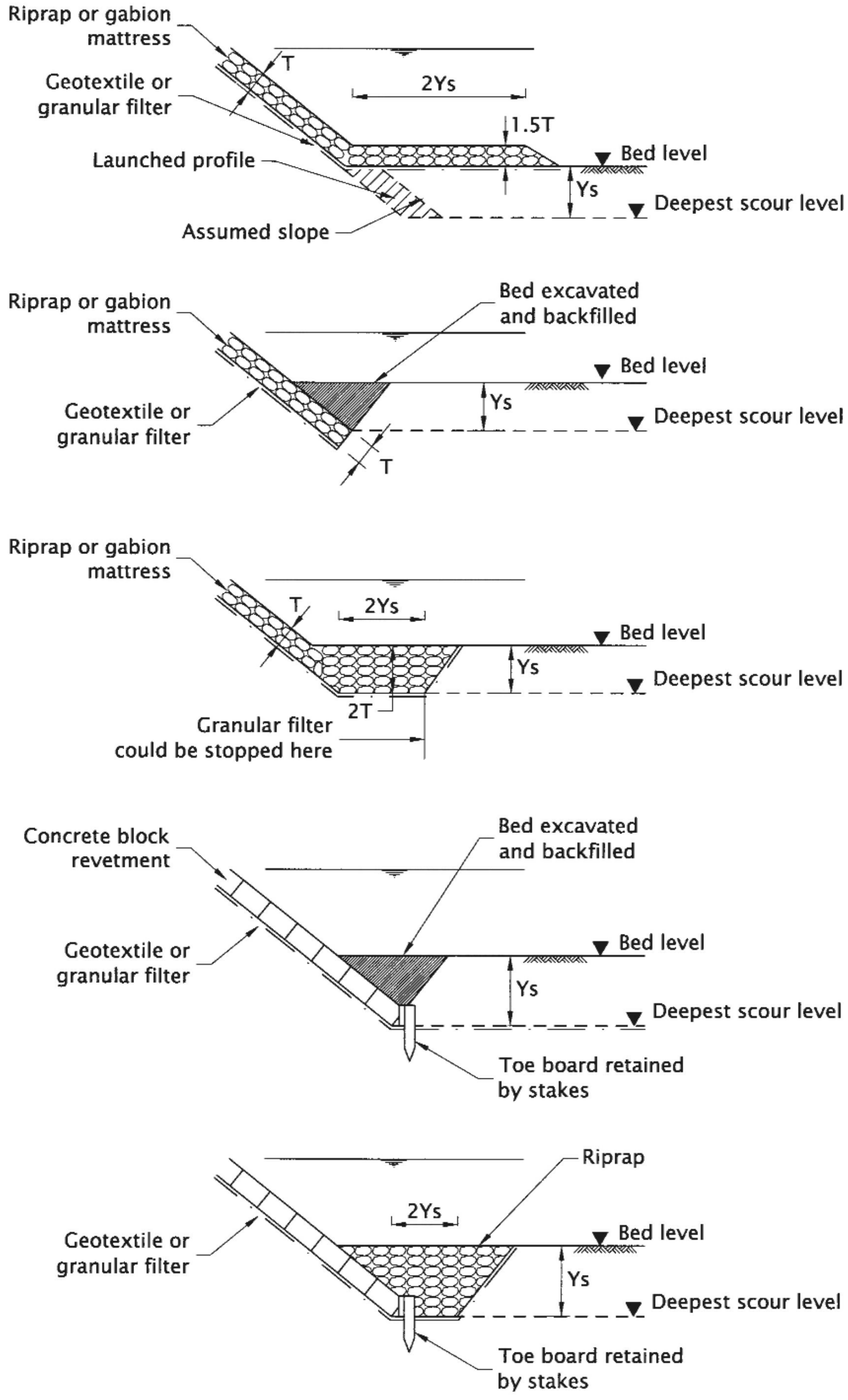


Figure 5.33 *Toe and falling apron details*

Where scour protection is inflexible, an additional area of protection can be provided which is turned down into the bed of the channel. This mimics the final launched position of a flexible protection. Alternatively, the edges can be protected by sheet piling or concrete cutoffs.

5.5.3 Bridge piers, caissons and cofferdams

Scour protection is necessary around a structure such as a pier, caisson or cofferdam if the predicted depth of scour (see Section 4.3.2) exceeds what is permissible for structural or other reasons, and if other options (see Section 5.2) for reducing the amount of scour are not feasible or economic. Having chosen the type and size of protection system, it is necessary to determine how far the protection should extend from the sides of the structure. Although this issue has been studied widely, both in the laboratory and in the field, a range of recommendations can be found in the literature. Figure 5.34 indicates the variety of recommendations.

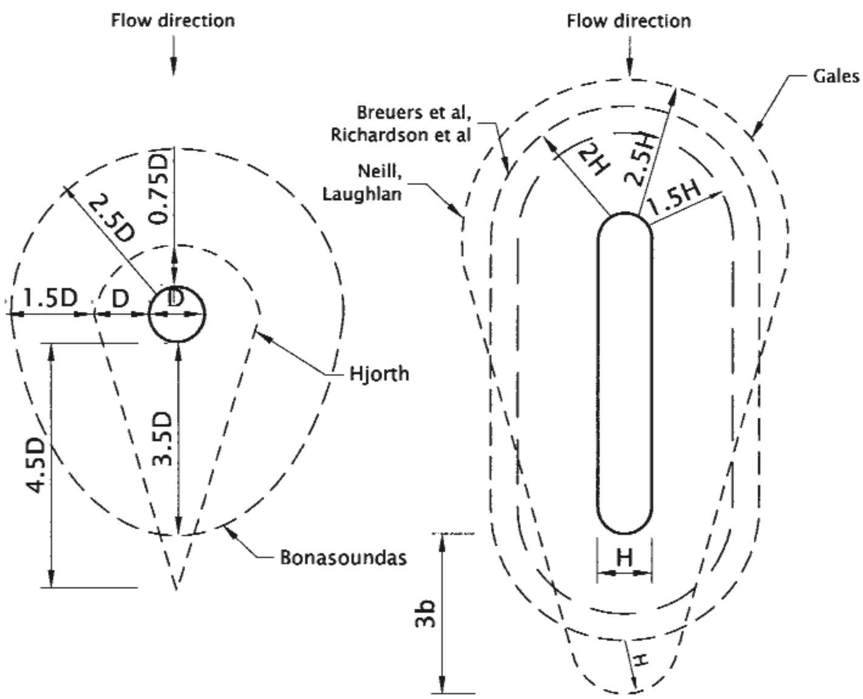


Figure 5.34 *Alternative recommendations for extent of scour protection*

Recent research from laboratory testing seems to indicate that scour is reduced to a minimal amount when scour protection extends to a distance 1.5 times the pier width from the face of the pier. To provide a safe design, it is recommended that protection should extend to a distance of 2.0 times the pier width from the face of the pier. For circular piers, the pier width should be taken as the pier diameter. For rectangular piers, the pier width should be taken as the width of pier normal to the flow (equivalent to the pier width if the flow is aligned with the pier but larger if the flow is not aligned). The recommendations are shown in Figure 5.35. This ties in with authoritative references such as *HEC18* (Richardson and Davis, 1995).

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. It should be noted that current US practice is to place the top of riprap protection at the bed elevation rather than below bed level. This is to allow the riprap to be inspected for damage. While this approach has some validity, there are nevertheless some concerns. It may not be feasible to inspect the riprap, due to siltation and the difficulty of inspection underwater. There is also an increased likelihood of damage to the protection.

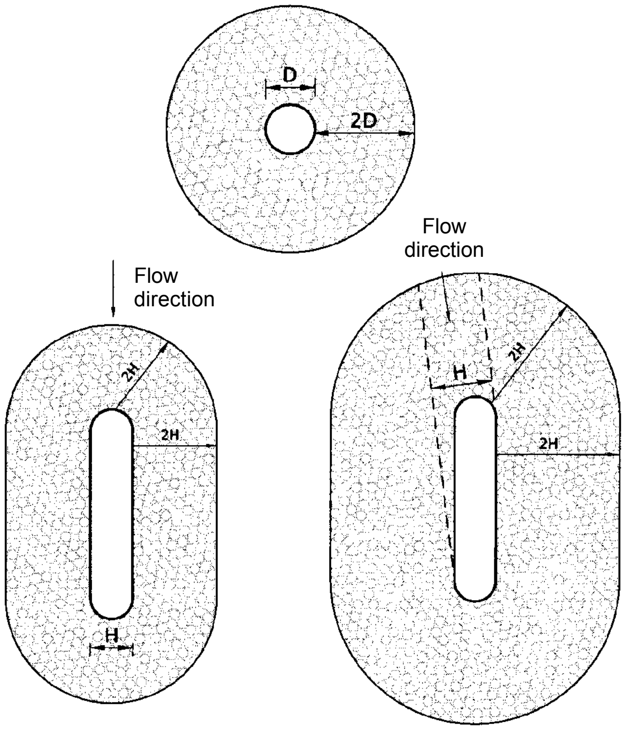


Figure 5.35 *Recommended scour protection extent at piers*

Figure 5.36 shows some of the correct and incorrect ways of arresting pier scour. Figure 5.37 illustrates some typical details of pier and abutment protection.

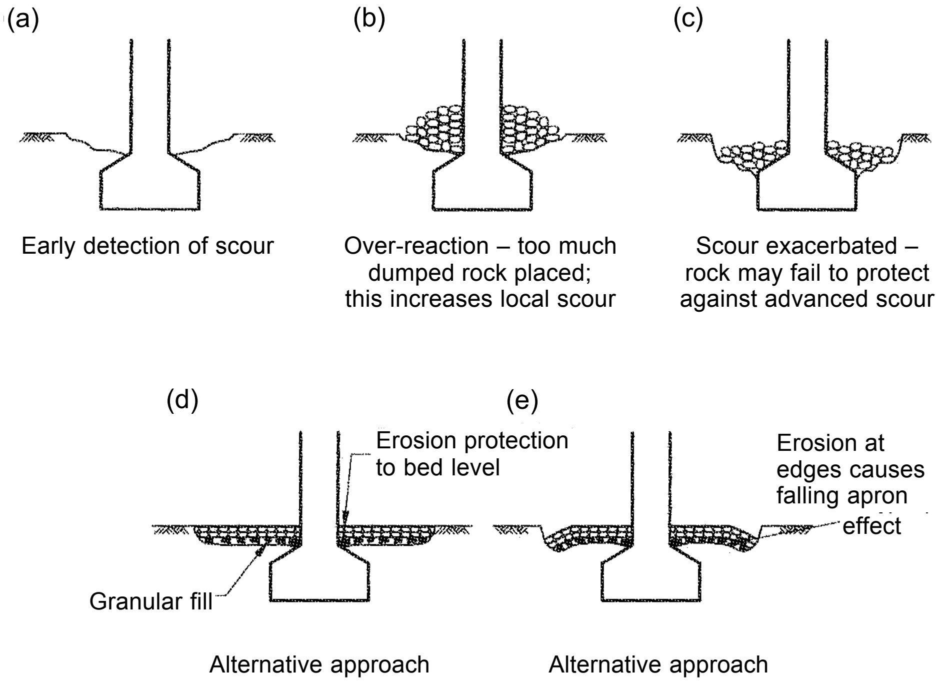


Figure 5.36 *Correct and incorrect ways of arresting pier scour*

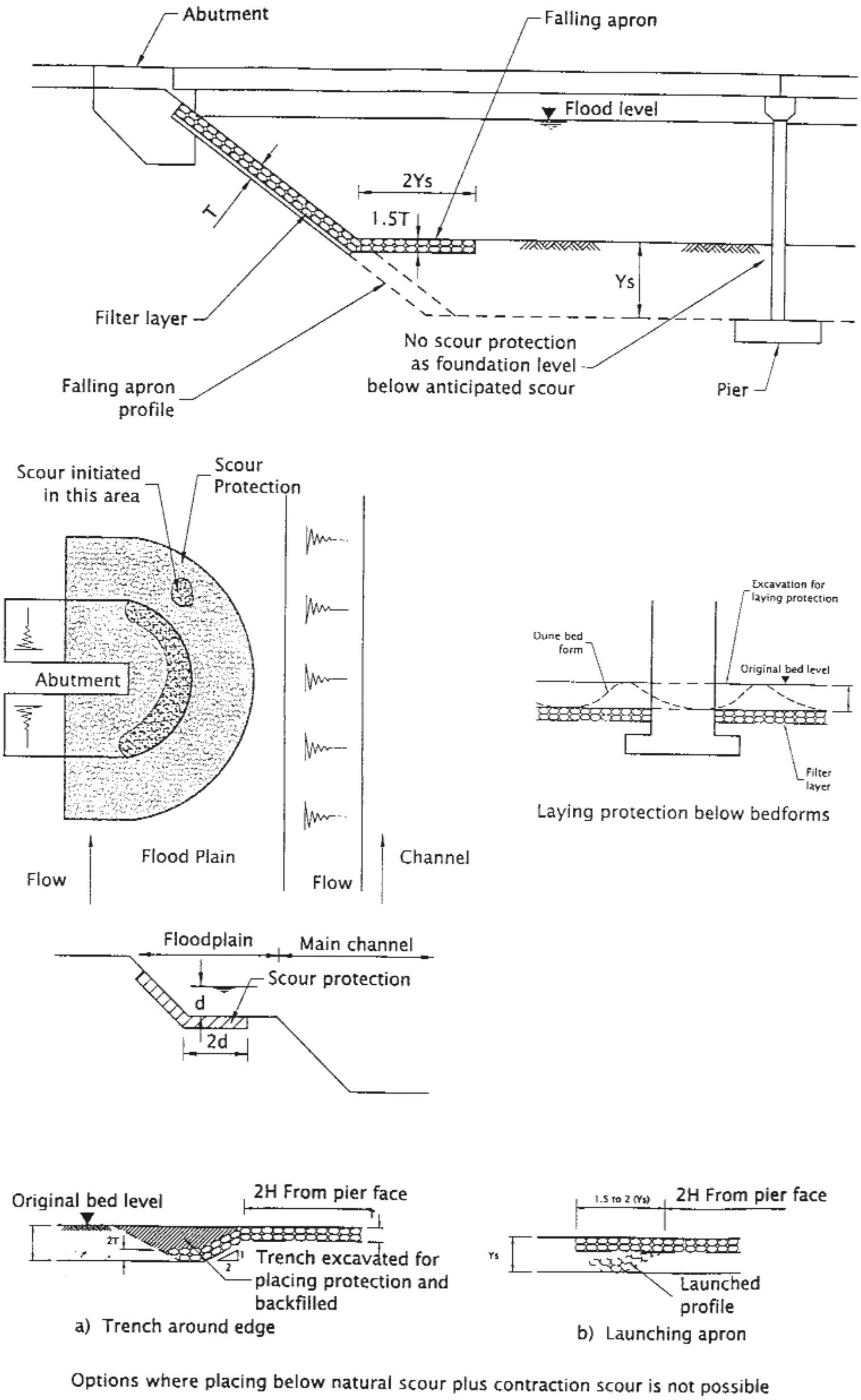


Figure 5.37 *Scour protection at bridge abutments and piers*

5.5.4 Abutments

There has been less research on the impact of different extents of scour protection at abutments. With sloping spill-through embankments, the area where scour is initiated and where scour protection is most likely to fail is usually on the sloping section near the downstream toe of the abutment. The upstream corner of the abutment is also vulnerable. For vertical abutments, the scour is normally initiated at the upstream corner of the abutment, as the flow accelerates through the opening.

5.5.5 Guide banks

Although guide banks are themselves a scour reduction measure, they in turn require protection from scour. The sizing and layout of guide banks have been described in Section 5.2 and Figure 5.18. It is normal to protect the exposed face of the guide bank with suitable scour protection. The most severe scour is expected to be experienced at the upstream head of the guide bank, where the transition occurs between normal flow in the channel and floodplain and constricted flow between the guide banks. Typical protection details should be as for sloping scour protection shown in Figure 5.33.

5.5.6 Spur dikes (groynes)

Spur dikes that are aligned perpendicularly to the flow or point upstream (used for retarding flows and building up sediment along river banks) are likely to experience scour around the nose of the structure. Spur dikes that point downstream (used for flow diversion or deflection) tend to experience scour along their upstream face as well as around the nose of the structure. Careful consideration should be given as to whether the spur dike will be submerged or non-submerged during floods. Impermeable groynes are often designed to be non-submerged, as otherwise they can be susceptible to scour along their length from flow over the crest. Permeable groynes are more suited to submerged conditions, as they generate less severe eddies. To reduce scour at the nose of a groyne, the crest can be sloped, with its elevation reducing with increasing distance from the river bank. This reduces flow concentration at the nose. Details as shown in Figure 5.33 apply.

5.5.7 Pipeline crossings

Scour protection of pipelines and pipe crossing serves three purposes:

- protection against scour of surrounding bed material
- protection against mechanical damage, for example by dredgers or trawlers or the river's bedload
- stability to prevent movement of the pipeline laterally or upwards.

Where riprap is used, it is essential that the stone is not dropped from too great a height, to avoid damage to the pipeline. The pipeline can be protected from impact damage by a layer of finer material such as gravel. A bedding layer beneath the pipeline may also be needed to provide an even formation. A filter fabric, fascine mattress or sand-filled mat laid over the gravel can avoid loss of the gravel under current action before the armour layer can be placed. Figure 5.38 shows some typical pipeline protection layouts.

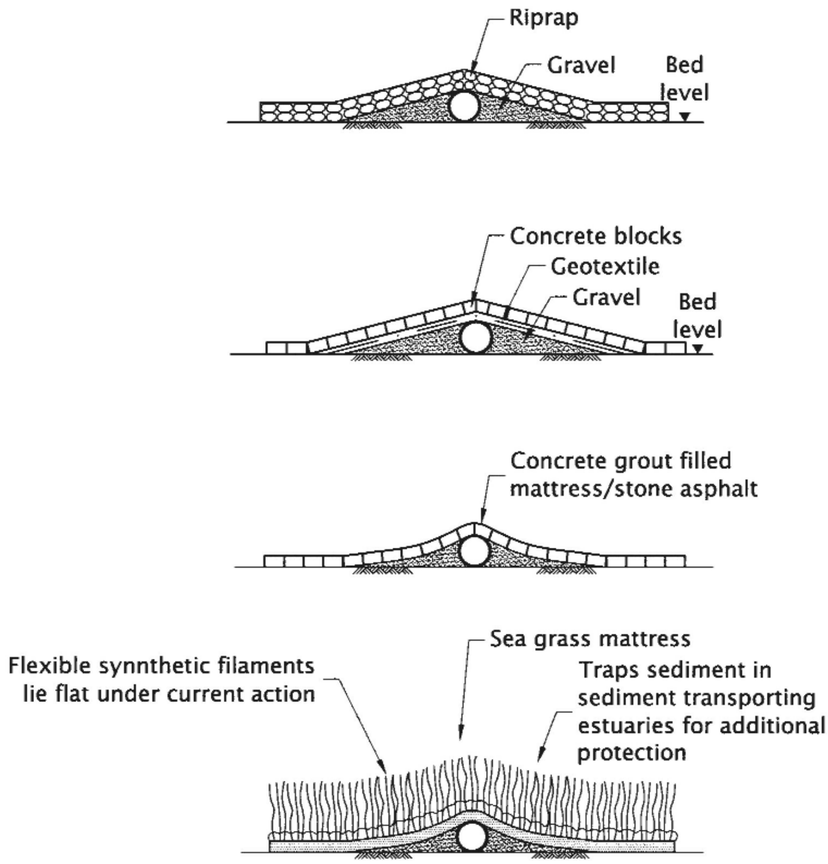


Figure 5.38 *Typical pipeline protection*

5.5.8 Culverts and other hydraulic structures

Relatively few studies have been carried out on scour protection downstream of culverts, but the following guidelines developed by Bohan (1970) may be used for initial assessments. The size of stone, d (m), required depends upon the degree of submergence produced by the tailwater in the downstream channel, so that:

$$\frac{d}{D} = 0.25\,F_c \text{ , for } y_T < D/2 \qquad \textbf{(5.13)}$$

$$\frac{d}{D} = 0.25\,F_c - 0.15 \text{ , for } y_T \geq D/2 \qquad \textbf{(5.14)}$$

where D (m) is the diameter of the culvert and y_T (m) is the tailwater depth measured relative to the invert level of the culvert. F_c is the Froude number of the flow discharging from the culvert and is defined by:

$$F_c = \frac{U_1}{\sqrt{g\,D}} \qquad \textbf{(5.15)}$$

in which U_1 is the mean velocity at the culvert outlet and g is the acceleration due to gravity (= 9.81 m/s^2). The research report did not define how the stone size, d, was measured but it is considered to have most likely been the d_{50} size. For arch, square or slightly non-square culverts producing three-dimensional jets, it is recommended to use an equivalent value of D giving the same cross-sectional area as the actual culvert.

The following results were also given by Bohan (1970) for the minimum length of scour protection, L_p (m), required downstream of the culvert outlet:

Subcritical conditions: $F_c \le 1$

$$\frac{L_p}{D} = 8 \tag{5.16}$$

Supercritical conditions: $F_c > 1$

$$\frac{L_p}{D} = 8 + 17 \log_{10} F_c \quad , \text{ for } \ y_T < D/2 \tag{5.17}$$

$$\frac{L_p}{D} = 8 + 55 \log_{10} F_c \quad , \text{ for } \ y_T \ge D/2 \tag{5.18}$$

At its downstream end the protection should be turned downwards into the bed over a length of about one culvert diameter. The precise layout required depends on the difference in velocity between flow in the culvert and flow in the channel downstream, the rate of expansion of the flow into the downstream channel and its alignment. If the channel immediately downstream of the culvert is made significantly steeper than the natural slope of the channel, large-scale erosion (termed “gully” erosion) may develop and work backwards towards the culvert, leading to failure of the protection system and, ultimately, to undermining of the culvert.

The above recommendations apply to culverts that discharge directly into a downstream channel. In certain cases, a more economic solution may be achieved if an energy dissipation structure is constructed downstream of the culvert to reduce the velocity of the flow entering the channel and thus the amount of bed protection required. Peterka (1978) gives guidelines on sizes of riprap for use downstream of stilling basins and energy dissipators.

Particularly severe problems of scour can be encountered where watercourses need to be channelled beneath roads in mountainous areas. Here, river training measures such as check dams, cascades and channel lining may be more effective means of protection than basic erosion protection. *Overseas Road Note 16* (Transport Research Laboratory, 1997) provides more details of the issues to be considered in mountainous areas.

At weirs and downstream of stilling basins or downstream of concrete aprons, scour protection should be provided to protect against scour of the downstream channel. The distance downstream over which protection should be provided depends, in a similar way to culverts, on the transition between flow conditions in the stilling basin or apron and conditions in the downstream channel. Particular consideration should be given to the design of protection for compound structures, where velocity differences across a structure can give rise to eddies, and to wide structures where cross-flows can generate eddies. Figure 5.39 indicates typical protection details downstream of a hydraulic structure. Additional recommendations on outlet arrangements and scour protection are given in *Design of small canal structures* (Aisenbrey *et al*, 1978).

Figure 5.40 illustrates a situation where energy dissipation was inadequate, resulting in highly turbulent conditions downstream.

Commonsense and experience should be used to determine the extent of scour protection. As a preliminary guide, the protection should extend downstream for about five times the head difference across the structure.

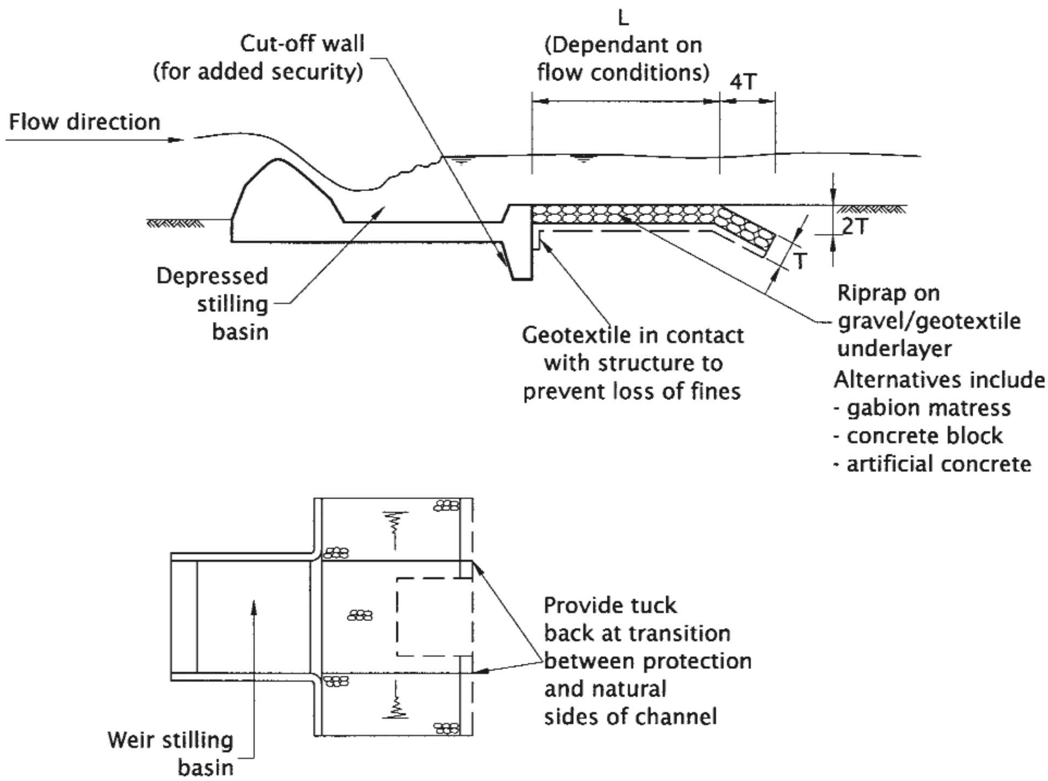


Figure 5.39 *Erosion protection downstream of weir structure*

Figure 5.40 *Scour downstream of an offtake structure, caused by the lack of scour protection coupled with inadequate energy dissipation*

5.6 FILTER DESIGN

5.6.1 General issues

Failure of scour protection works can often be linked to the poor design or construction of the filter between the scour protection and the subsoil. This can lead to loss of the subsoil through the protection, causing movement of the armour layer, further exposing more of the subsoil and in turn leading to more subsoil being removed. Progressive collapse then results.

The function of a filter is to prevent migration of the subsoil particles through the protection works, while allowing water flow across the soil boundary, to avoid the build-up of an unacceptable head across it. Two types of filter are commonly used: granular filters and geotextile filters. Composite filters combining both materials are also used to combine some of the useful properties of each material.

Table 5.6 *Choice of filter type*

Filter type	Properties
Granular	• deforms, so good contact maintained between subsoil and armourlayer • repairs are relatively easy and damage is sometimes self-healing • durable • excavation may be required to lay it • accurate placing difficult in deep water • difficult to place in high currents • careful placing needed to achieve required thickness • grading needs to be carefully controlled • multiple layers may be needed to meet filter requirements • required gradings may be difficult to obtain locally
Geotextile	• relatively low cost • large areas can be laid quickly • small construction thickness • non-woven types can cope with soil variations • difficult to lay in deep water • difficult to place in high currents • needs to be pressed evenly against the subgrade by the armour layer • damage can be difficult to repair • careful laying of geotextile and subsequent placing of armour layer needed to avoid damage. A bedding layer between geotextile and armour layer may be needed • openings can become blocked • long-term behaviour is less certain than granular filters
Composite	• can be useful in deep water and high current conditions where mattresses or sacks can be more readily placed than loose granular material or light geotextiles • can be useful in protecting a geotextile from damage by large riprap

The term geotextile is used to cover a wide range of synthetic grids, meshes and textiles. Two forms of geotextile are used in scour protection, woven and non-woven. Woven geotextiles are formed using regularly placed fibres orientated at right angles to give uniform hole sizes. Non-woven geotextiles are formed using randomly placed fibres, giving a range of hole sizes. Woven geotextiles are generally stronger than non-woven geotextiles and can be used as filters for soils of a particular size. They can also be appropriate where very high porosities are required. Non-woven geotextiles are generally considered more useful as filters in scour protection situations, because the hole opening sizes available cover a wider range of soil types. They can also stretch more before failure. By maintaining contact with the subsoil and the armour layer when stretched its filter function is not compromised.

Since the hydraulic response of both geotextiles and granular filters can be similar, the choice of materials is usually based on practical considerations. Table 5.6 summarises some of the main issues that should be considered.

5.6.2 Granular filter design

Granular filters are normally designed using grading criteria derived from Terzaghi's filter rules. Various criteria have been developed. An important criterion is for the grading envelope to be approximately parallel to that of the soil. The following criteria are recommended, based on CIRIA and CUR (1991) and CUR/RWS Report 169 (Centre for Civil Engineering Research & Codes, 1995):

For uniformly graded material:

For retention: $$\frac{d_{50\,filter}}{d_{50base}} \leq 5$$

For well-graded material:

For retention: $$5 \leq \frac{d_{50\,filter}}{d_{50base}} \leq 20$$

For piping: $$\frac{d_{15\,filter}}{d_{85base}} \leq 5$$

The criteria should be applied to the interface between the armour layer and the filter as well as to that between the filter and the base soil.

If the base material is gap-graded, then it should be considered as a mixture of two subgradings and the piping criterion should be based on the d_{85} of the finer of the two subgradings. This can be approximated to the d_{30} of the base material, so the piping criterion becomes as follows.

For gap-graded base material:

For piping: $$\frac{d_{15\,filter}}{d_{30base}} \leq 5$$

For all types of material to ensure adequate permeability:

For permeability: $$\frac{d_{15\,filter}}{d_{15base}} \geq 5$$

In addition, a uniformity criterion (also called a geometrically tight criterion) for the filter itself is often specified, to ensure that the finer particles of the filter are not removed through the voids between the coarser particles. This is particularly important where hydraulic loadings are high, such as occur in turbulent flow conditions.

For uniformity: $$\frac{d_{60}}{d_{10}} \leq 10$$

CUR/RWS Report 169 (1995) suggests the use of a more rigorous uniformity criterion developed by Kennedy and Lau (1985):

$$\left(\frac{F_{4d}}{F_d} - 1\right)_{\min} > 2.3$$

where F_d is the percentage (by weight) of the filter finer than a particle size d and F_{4d} is the percentage (by weight) of the filter finer than a particle size of $4d$. Different values of particle size d along the grain size distribution curve give different values of $((F_{4d}/F_d) - 1)$. The minimum value of $((F_{4d}/F_d) - 1)$ is at the flattest part of the grain size distribution curve.

The thickness of each filter layer should be greater than 100 mm and should be at least 150 mm where only one layer is required. Normally, a thickness of at least 200 mm to 250 mm should be used. The layer thickness should also not be less than the d_{100} size or 1.5 times the d_{50} size of the filter layer. Where placed underwater or in high currents the layer thickness should be increased by about 50 per cent.

5.6.3 Geotextile filter design

Geotextiles are designed primarily according to three criteria: soil retention, permeability and strength. Soil retention is related to the size of pores or holes in the geotextile, termed the characteristic opening size, which is normally defined in terms of O_{90} (90 per cent of the pores are smaller) and the O_{95} (95 per cent of the pores are smaller). The following criteria have been suggested in CUR/RWS Report 169 (Centre for Civil Engineering Research & Codes, 1995):

For geotextiles laid against non-cohesive, uniform soils:

$$\frac{O_{95}}{d_{85base}} < 1$$

For geotextiles laid against cohesive soils three criteria are specified:

$$\frac{O_{90}}{d_{10base}} < 1.5\frac{d_{60base}}{d_{10base}} \quad ; \quad \frac{O_{90}}{d_{50base}} < 1 \quad ; \quad O_{90} < 0.5\text{mm}$$

To satisfy permeability requirements the following criterion is suggested:

$$\kappa_g \geq M\, \kappa_s$$

where κ_s (in m/s) is the permeability of the soil, κ_g (in m/s) is the permeability of the geotextile and M is a coefficient which depends on the type of geotextile:

M = 10 for woven geotextiles

M = 50 for non-woven geotextiles.

The strength criterion is based on the need to avoid damage from the armour layer being placed onto the geotextile. Where the armour layer is particularly large it is normally preferable to lay a granular separation layer between the geotextile and armour layer to provide the dual function of protecting the geotextile and acting as a filter. Reference should be made to manufacturers' literature to determine the appropriate strength of a geotextile to cope with different sized stone being dropped onto it.

For more detailed guidance refer to the CUR/RWS Report 169 (Centre for Civil Engineering Research & Codes, 1995).

5.7 CONSTRUCTION ASPECTS

5.7.1 General

Construction of scour protection measures is often undertaken in difficult site conditions, such as fast flowing water, limited access and tidal variations. Because of this, it is particularly important during design to consider the practicality of the design and the problems and constraints likely to be encountered during construction. As well as practical construction constraints, there are likely to be environmental and health and safety constraints that can have a significant bearing on the design and construction. Further information is given in Sections 6.1 and 6.3.

For large projects it can be useful to construct trial sections of scour protection. This helps to perfect construction techniques, allows the practicality of construction to be tested, allows different construction methods to be tried out and provides a standard against which the rest of the protection can be compared. Trial sections are particularly common for gabion construction, where the success of the system can be strongly correlated to the quality of construction.

5.7.2 Underwater

Constructing scour protection underwater presents particular problems and can increase the cost of construction considerably in comparison with construction in the dry. Conversely, dewatering can be problematic and expensive and often proves uneconomic, because scour protection often requires a large plan area to be dewatered. Construction using land-based plant can normally be carried out straightforwardly in water depths up to about 1.5 m, providing currents are not high. Hand-placed revetments can be placed in water depths up to about 0.5 m, providing currents are low.

Because placing can be less carefully controlled, it is normal to require riprap thicknesses to be increased when placed underwater (see Section 5.4.2). Quality of placing is also much more difficult to supervise and monitor. Hydrographic surveys are normally used to check placed thicknesses. Inspection can often only be carried out by divers, often with limited or no visibility. Diving inspection can be appropriate at critical elements, such as piers, where it is important to check that there is a good connection between the scour protection and the pier surface.

The practicality of laying scour protection in the flow conditions that will be experienced at the site must be carefully considered. In particular, laying filter layers can be difficult under even moderate flow velocities. Particles of a granular filter layer tend to be displaced downstream. The length of displacement is dependent on the flow velocity, water depth and stone size. It can be estimated from the equation:

$$L = \frac{0.25\, y_o\, U_d}{\sqrt{d}}$$

where:

L (in m) is the distance the particle is displaced
y_o (in m) is the local water depth
U (in m/s) is the local depth-averaged velocity
d (in m) is the nominal stone size.

As an example, a coarse gravel filter with a nominal stone size of 50 mm, dropped from an excavator bucket 0.5 m above the bed in water flowing at only 1.5 m/s, will be displaced by about 0.8 m. This highlights the benefits of releasing granular fill and riprap as close to the bed as possible, rather than releasing them at the water surface. Geotextile filter layers can also be difficult to lay underwater and often need weighting down before and during laying. Because of this problem, sand-filled mats are sometimes used. These are geotextile mats filled with sand or fine gravel. The weight of sand ensures that the mats can be laid without movement by currents, while the geotextile provides the required filter properties. The sand can also act as a secondary filter.

The seasonal and daily timing of working hours can be important for underwater construction. In fluvial conditions, construction during drier seasons, when water levels and flow velocities are at a minimum and the risk of floods is less, is always preferred. This can reduce construction costs and increase the speed of construction considerably. In tidal locations, working hours may be restricted by tides, wave and wind conditions. In navigable waterways restrictions on timing, working hours and construction sequence may be imposed.

5.7.3 Availability of labour, plant and materials

The suitability of different types of scour protection is strongly influenced by the availability of labour, plant and materials. Some types of scour protection, such as gabions, interlocking blocks and stone pitching, require significant labour. Sheet piling requires the availability of plant, whereas other types of protection require certain materials to be available at reasonable cost, such as suitably sized rock for riprap or appropriate geotextiles. This may mean that in some countries and locations certain types of protection are very difficult to construct at reasonable cost, whereas other types are suited to the labour and materials available. Use of local labour, plant and materials is usually preferred for social, cost and environmental reasons. However, if there are doubts about the possible quality of the constructed works, it is important to specify higher factors of safety for the designs.

Figure 5.41 *Scour protection works being undertaken in ideal conditions*

5.7.4 Access

Access for construction of scour protection frequently presents problems. First, access for plant into and along the channel has to be considered. Because scour problems often occur in fast-flowing, deep rivers, the shallow water depths and tranquil conditions shown in Figure 5.41 are rarely encountered. When carrying out repairs to bridges there may be limited headroom, which may affect the choice of scour protection measure used. Figure 5.42 shows riprap being placed in limited headroom conditions.

Figure 5.42 *Placing riprap in restricted headroom (photograph courtesy of Jeremy Benn Associates)*

5.7.5 Gabion construction

Gabions are straightforward to erect and workers can rapidly learn how to carry out high quality construction. However, to be a successful system, gabions have to be carefully constructed 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.

5.7.6 Concrete mattress construction

Where mattresses are laid on slopes, injection of grout should proceed from the toe of the slope to the top of the slope. Prior to grouting, the mattress should be anchored at the top, either to a structure or using ground anchors. The mattress normally contracts in length as it fills with grout.

5.7.7 Riprap construction

Riprap should where possible 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.

5.7.8 Filter construction

It is important, particularly for geotextile filters, that they are laid on a firm, even formation, free from stones or holes. This ensures that there is good contact between the geotextile and soil, and avoids damaging the geotextile. Geotextile should be laid out so that it is in contact with the formation, avoiding folds, creases or loose areas.

Where laid above water, geotextile should be fixed with pins or stakes to avoid movement as the armour layer is placed on to it. Laying geotextile underwater is more difficult and often requires divers. The geotextile roll should be submerged and the leading edge of the geotextile weighted down with sandbags or loose stone. The geotextile can then be unrolled by divers along the direction of flow, weighing it down at frequent intervals. This is only suitable where current and wave action are very low and for relatively shallow water depths.

Specialist plant is available for unrolling geotextile directly from a pontoon, but is only likely to be economic for large-scale works. Where a pontoon is used for the scour protection works it can help to shield the geotextiles from some of the effects of current action. When laid on a slope, the geotextile can be unrolled down the slope and weighted down at frequent intervals to the bed of the river. Again this is difficult where currents are high. An alternative is to incorporate a weighted bolster of sand or stone into the toe of the geotextile. The geotextile can be fixed above water and then unrolled down the slope, with the weighted bolster toe helping to unroll the geotextile down to the bed of the river.

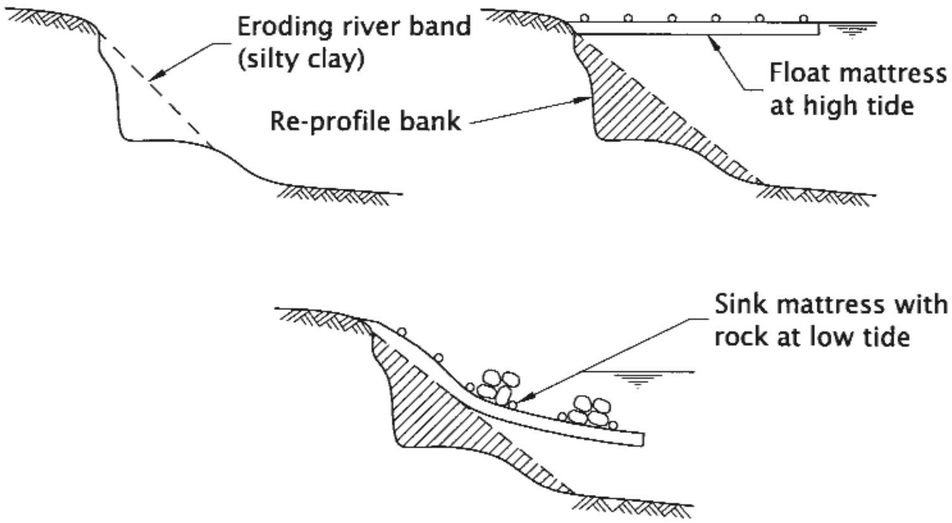


Figure 5.43 *Laying fascine mattress (tidal river)*

For high turbulence conditions the use of fascine mattresses or sand mats or bags can provide the strength and rigidity to enable placing. For example, a geotextile can be fixed to the underside of a fascine mattress, the mattress can be floated into position by pontoon and then sunk with stone. Figure 5.43 shows a typical method for laying fascine mattresses underwater.

The armour layer should be placed carefully on to the geotextile, neither being pushed across it using dozers, for example, nor being dropped from a height. To avoid damage, the movement of tracked plant across the geotextile should not be permitted. Geotextiles suffer ultra-violet (UV) deterioration, so should be covered when stored and, when laid, should not be left exposed for long periods, particularly in sunny conditions.

Joints in geotextile should be lapped by at least 0.3 m with the lap laid so that it closes under the action of the predominant flow. Under wave action laps should be laid so that they close under the action of downwash. Where high turbulence or wave action is expected the laps should be fixed down, for example using pins.

In high pH conditions (above 10) polypropylene is preferred to polyester, owing to its higher resistance to chemical attack.

Careful quality control of granular filters is required to ensure that a suitable grading is consistently being achieved at the quarry and that filter layers are of the required thickness. The armour layer should be placed carefully on granular filters to avoid displacement of material, which could reduce the thickness of the layer or even expose the subsoil.

5.8 SCOUR MONITORING

Many existing bridges may be susceptible to scour, and for many no information on their foundation construction survives. Given limited resources, it is often not cost-effective to carry out replacement or repair. In such cases, the structure must be monitored and inspected after flood events, and its use during floods may need to be restricted. Periodic inspections are also necessary to check for possible vandalism that might affect the safety or performance of structures and protection works.

Scour monitoring and inspection are not straightforward. During a flood, scour is not normally visible, and during the falling stages of a flood scour holes often fill in. It can be difficult, therefore, to assess the magnitude of scour holes and determine whether the bridge or hydraulic structure is safe.

Post-flood inspections can be carried out by divers, and, using probes, experienced divers are often able to distinguish between a filled-in scour hole and the undisturbed bed. Flow conditions may not be safe enough for diving for some time after the peak of a flood, however, so there can be a delay in determining whether a bridge is safe. If the bridge has been closed, this delay may cause considerable disruption and loss of revenue. Inspection of bridges over ephemeral rivers is easier if they can be inspected when dry, and again it can be possible to determine the depth of scour by comparing undisturbed bed material with material transported into scour holes.

5.8.1 Monitoring equipment

Instrumentation is becoming available that may allow practical real-time monitoring of scour depths at important bridges that are thought to be vulnerable to scour and may have to be closed when a certain scour depth is exceeded. Other devices allow the maximum depth of scour to be determined after the event. Clearly, the use of monitoring equipment would not be appropriate where the installation of physical measures is a practical and economically preferable alternative.

Brief descriptions of the available methods and equipment are given below, while further data on the development, laboratory testing, field trials and appraisals of the equipment are given in Appendix 1.

Sounding weights and probing

A lead sounding weight on a cable may be used to determine the bed profile. Unless carried out during the flood, which can be difficult and dangerous, the maximum scour depth is not determined.

Probing with a rod after the flood can be carried out underwater by divers. It may allow the user to detect the interface between an infilled scour hole and undisturbed bed material. It does not provide real-time information, however, and can be expensive and time-consuming if large areas have to be covered.

Sounding rods

Sounding rods are sleeved vertical or near-vertical metal rods connected to a bridge pier or other structure. They rest on the river bed and can slide vertically down as a scour hole develops. The basic concept appears sound and they have been successfully used in the field, but have some limitations. On the falling stage of the flood, the sounding rod is normally partially buried by sediment refilling the hole. In subsequent floods, therefore, the scour depths would only be measured if they are larger than in previous floods.

Concerns have been expressed that sounding rods can sink into non-cohesive bed material under their own weight, encouraged by vibration and rotation of the rod caused by vortex shedding. Suitably sized footplates to reduce bearing pressures can overcome this concern. (Reducing bearing pressures to about 3 kPa was found to solve the problem in sand bed material in one field trial.) Damage to the rod can also be a concern, either by bending under hydrodynamic forces that prevent it sliding down, or by impact damage caused by debris or ice. Bending and damage to the rod may be overcome by limiting the length of exposed rod and using a suitably robust rod and protective sleeve. Corrosion may also be a concern.

Sounding rods can be difficult to mount on structures with projecting footings or pile caps, which would either limit the measurable depth or require substantial mountings (slightly angled if necessary) to allow the rod to pass down beyond the obstruction. Drilling a hole through the projection so that the rod can pass through the obstruction is not recommended, as the rod is likely to become stuck; in any case, it would not normally then measure the maximum scour depth.

Sliding collars

The sliding collar device consists of a vertical support rod driven into the river bed with a horseshoe-shaped or circular collar around it. The collar rests on the river bed and slides down the support rod as a scour hole develops. On the falling stage of the flood, the collar is normally buried by sediment and its position is determined by sending a detector down the hollow support rod. In some installations, the pipe is plastic and a magnet or metal detector is used to detect the collar. In other installations, a metal pipe is used and the collar is radioactive. This type of device has been widely used, appears practical and is easy to install and operate. Its limitation is that it only measures the maximum scour depth, although additional collars can be placed around the support rod to monitor subsequent smaller scour holes.

As for the sounding rod device, the support rod, its support and cabling must be suitably robust, to resist hydrodynamic and impact forces. Potential problems are binding of the collar, by tilting or by intrusion of stones between the rod and collar, and corrosion, particularly in saline conditions, which may prevent free movement of the collar along the support rod.

Sonic fathometers

Sonic fathometers use reflected acoustic waves to detect the water–bed interface. They comprise a downward-pointing transducer mounted on the structure, usually within a protective housing and normally below the flood water level. Well-designed fathometers that are robust and easy to maintain can provide a durable and reliable method of scour monitoring. They have the advantage over other scour monitoring methods that they do not require equipment to be installed at the bed. They also provide full real-time monitoring of scour.

The presence of air bubbles can interfere with the reflected wave, so sonic fathometers should not be used in areas of high turbulence or where there is significant vortex shedding, such as the downstream face of piers or where there are sudden contractions or expansions of flow. The transducer head must also be streamlined to avoid flow separation. High turbidity and sediment transport may affect readings. In addition, a highly mobile bed can give erroneous readings. Debris accumulated against the structure may make the device inoperable by blocking the acoustic signal. This may generally be overcome by locating the transducer as close to the river bed as possible, away from the effects of debris. Maintenance may become difficult, however, and problems can occur when debris sinks to the bed. The transducer heads require regular maintenance to clean off accumulated biological organisms. As with the other devices, robust mounting and cabling are needed to cope with hydrodynamic and impact forces.

Buried instrumentation

This type of device is buried within the river bed and detects the bed–water interface as the scour hole develops. The method of detection varies between devices. The "tell-tail" developed by HR Wallingford uses a series of sensors mounted on a vertical rod. When exposed to flow, the sensors oscillate, triggering an alarm. The long-term durability and robustness of the sensors used are the main concerns with this type of device. Because they have to be installed below bed level, they can be difficult to install at existing bridges. They are more suitable at new bridges and in ephemeral rivers.

Box 5.2 *Scour protection: design considerations checklist*

A	**Location considerations**
1	Is the structure on a bend? Can the structure be located to avoid this?
2	Is the structure at a confluence? Can the structure be located to avoid this?
3	Is the structure skewed to the flow direction? Can the structure be aligned to avoid this?
4	Is the structure close to an existing structure? Could interacting flow patterns exacerbate scour? Will scour holes overlap?
5	Is the structure located on an alluvial fan? Can the structure be located to minimise the risk of channel re-alignment?
6	Is the channel unstable laterally? Can historic records, aerial photography or satellite imagery be used to determine this? Is a more detailed assessment needed?
B	**Hydraulic considerations**
1	Is the channel bed stable, aggrading or degrading? Can historical records be used to determine this? Is a more detailed assessment needed?
2	Are bed levels controlled by a structure downstream that could be removed over the life of the new structure?
3	Could future dredging affect bed levels?
4	Can contraction scour be reduced by widening the clear spans of the structure?
5	Has the flow alignment during major floods been considered?
6	Are flow conditions complex and difficult to predict? Is a physical or mathematical model appropriate to assess hydraulic conditions and scour depths/extents?
7	Are the number and size of openings at optimised? A small number of larger openings is preferable to a large number of small openings.
8	Will there be significant flow on the floodplain during high floods?
9	Are relief openings required through the approach embankment on the floodplain?
10	Are guide banks required to channel flow through the bridge?
11	Is the structure susceptible to significant blockage from debris or ice? Has this been considered when predicting scour depths
12	Can flow be concentrated on one side of the structure and are cross-flows likely that would exacerbate scour at that side of the structure (eg gated weir)?
13	Is the downstream channel straight?
14	Are there similar structures in the area? How have they performed in relation to scour?
15	Is river training needed to control the river alignment?
16	Is bed level control needed to control degradation?
17	Do downstream areas subject to flow separation and eddy formation require protection?
C	**Structural considerations**
1	Can abutment scour be reduced by using spill-through abutments instead of vertical abutments?
2	Can the risk of abutment failure be reduced by setting back the abutments from the main channel?
3	Are piers aligned to the flow? Is there is a risk that the channel will change its approach alignment to cause skewed flow? If so, can circular piers be used to make this unimportant?
4	Is the pier thickness the minimum practical to reduce scour?
5	Are piers streamlined?
6	Could the bridge deck become submerged during extreme floods? Could it be streamlined to reduce debris accumulation and the vertical contraction?
7	Can the risk of failure of the bridge deck during extreme floods be reduced by setting the approach embankments lower than the deck to allow flow to spill over?
8	Can the foundations be designed to resist scour by structural measures? Has an appropriate safety factor been applied to account for possible under-prediction of scour depth?
9	Has the risk of failure of different types of foundation (eg footings or piled) been considered? A small number of long piles will cause less scour than a large number of small piles if exposed.
10	Can abutments be set back from the channel to reduce the risk of failure in the event of a river bank collapse?

continued on next page...

11 Where pier or pile groups are used, has the estimate of scour depth taken account of interactions between component structures?

12 Can pile caps and footings be located below the level of natural plus contraction scour?

13 Can pile cap or footing shape be minimised and streamlined to reduce scour?

14 Has the interaction between scour holes been considered by plotting scour extents on a river cross-section? Can the piers or abutments be located to avoid overlapping scour holes?

15 Are the foundations of structures on the floodplain at risk if the main channel changes alignment?

D Tidal considerations

1 What combinations of tidal range, fluvial flow and storm surge should be adopted for design?

2 How will tidal conditions affect the working methods and working hours?

3 Is wave action likely to be significant?

4 Will tidal currents cause scour? What information is available on tidal currents?

5 Could the long-term movement of creeks and channels expose foundations and increase local currents at structures?

E Construction considerations

1 Where erosion protection measures are used, what will be the construction sequence?

2 Can erosion protection be placed in the dry? How will this be achieved?

3 If construction will be under water, how will it be placed? What plant is needed to do this?

4 What materials are available for erosion protection measures? Are they of suitable quality and cost? Have haulage routes and storage requirements on site been considered?

5 Has the time taken to construct the measures been considered?

6 At what time of year should the erosion protection measures be constructed to minimise risks due to high flows? What hydraulic conditions will prevail: velocity, water levels etc?

F Maintenance and monitoring considerations

1 How will scour be monitored in the short and long term? Are diving inspections needed? Is permanent monitoring required?

2 Will inspection be carried out by persons with sufficient experience/knowledge to identify hydraulic problems and assess their risk?

3 How will erosion protection measures be maintained? Has access, availability of materials, practicality of placing been considered?

4 If the certainty of regular inspections and maintenance cannot be assured, has a higher factor of safety been adopted in design?

G Risk considerations

1 Has the return period of the design flood been chosen to reflect the acceptable risk of failure of the structure? Does the client/operator understand this premise?

2 Does the adequacy of the proposed design depend on any factors over which there is some uncertainty? Can this uncertainty be reduced by further assessment or by adopting a different design?

3 What is the risk of flooding during construction; how would this affect the construction of scour protection measures?

H Environmental considerations

1 Could vandalism or theft reduce the effectiveness of the erosion protection measure used?

2 Does the construction method proposed carry an increased risk of a pollution incident. What measures have been put in place to avoid pollution?

3 What measures has the environmental assessment suggested to avoid or mitigate damage to the environment ? How can these best be implemented?

4 Are there opportunities for environmental enhancements?

I Safety considerations

1 Is there the possibility that work could be carried out from an unstable river bank?

2 How will different types of protection by constructed? How will safety during construction affect the choice of protection?

3 How will different methods of working affect the level of risk to safety?

4 How will maintenance of the scour protection works be carried out safely?

6 General issues

6.1 ENVIRONMENTAL FACTORS

6.1.1 The requirements for environmental assessment

It is common practice to investigate the environmental impact of engineering works as part of the project preparation process. In the European Community (EC), the preparation of an environmental assessment (EA) is a regulatory requirement for certain types of scheme. For some schemes, including those for which scour protection could form a component, it is at the discretion of the planning authority as to whether an EA is required. Outside the EC, the term "environmental impact assessment" (EIA) is the term in common usage, and the same or similar regulatory requirements hold.

It is impossible in this manual to give specific advice about the need for an EA, but there should be a presumption in favour of undertaking some form of assessment, even where this is not a legal requirement. A full understanding of the environment in which the scour protection works are to be constructed is an essential stage in reaching the right answer. An environmental assessment normally comprises three components:

- a baseline survey to define the current environment
- the assessment of the adverse and beneficial impacts of the scheme on the current environment
- the consideration of measures to avoid or mitigate the adverse impacts.

Widespread consultation is an integral part of the process and there is usually a predefined list of bodies that must be consulted. In addition, consent may be required for the work. In England and Wales, under Sections 109 and 110 of the Water Resources Act 1991, consent is required from the Environment Agency for works in, over or adjacent to a statutory main river. It should be noted that separate consents may be required for the permanent and the temporary works. On ordinary watercourses the consent of the drainage authority is required under Section 23 of the Land Drainage Act 1991. In Scotland consents are required from local authorities and the Scottish Environmental Protection Agency (SEPA) under the Land Drainage and Flood Protection Scotland Act 1997. In Northern Ireland the Rivers Agency should be consulted.

As well as any statutory consultees, it is appropriate to consult with other interested parties at an early stage, so that the important issues may be determined. These might include local wildlife and conservation groups, angling clubs, riparian owners, amenity and recreation bodies, and navigation organisations. The aspects of the environment that would normally need to be considered include:

- ecology (flora and fauna)
- air, water and soil quality (pollution, contamination etc)
- human impacts (noise, visual intrusion, traffic disruption etc)
- geomorphological and landscape (sediment transport, drainage etc)
- archaeological, historical and cultural aspects
- social and socio-economic aspects.

6.1.2 Environmental impacts of scour

Scour can have both beneficial and adverse environmental impacts. Since it can be a natural or a man-made phenomenon, it can be seen as both a natural process of change in a river as well as a negative consequence of human interference with the environment. Whether a natural or a man-made change, its impacts can be detrimental to the current environment.

Scour can result in the wholesale loss of habitat for fish and invertebrates and can destroy in-river and bank vegetation. At the same time, the eroded material can be deposited to form shallow margins at river banks that are ideal habitats for a range of flora and fauna. Scour can also involve the movement of large quantities of sediment that can affect habitats downstream, for example by smothering fish spawning grounds. There are recent cases of scour protection being needed to protect contaminated land adjacent to a river, to avoid contaminated soil being transported downstream and causing pollution.

Scour normally has a negative impact on archaeological sites, many of which are sited near rivers. However, erosion and scour have often uncovered previously unknown historic riverside sites and yielded many finds.

The human and socio-economic impacts of scour are generally negative and are usually the driving force behind scour protection. These range from bridge damage that might cause disruption to traffic, to damage to an irrigation structure that could lead to loss of agricultural production.

6.1.3 Environmental impacts and mitigation

Rivers and estuaries are often environmentally sensitive areas, and any construction works in or adjacent to them runs the risk of causing an adverse environmental impact. Design and construction of such works should take account of the issues that are particular to river and coastal environments and that may not be relevant or so important for land-based construction.

The construction of scour protection measures can involve the removal of significant quantities of river bed and bank material. The way in which this is removed should be considered carefully to avoid releasing sediment or contaminated material. In some circumstances it can be beneficial to retain this material, which can then be replaced over the scour protection works. As well as providing a minor improvement in scour protection, it assists the rapid re-establishment of habitats.

Enhancement measures at scour protection works may be limited in type owing to the high-velocity flow likely to be encountered and the need to streamline flow near the works. Measures put in place away from the scour protection works may therefore be more suitable in many cases.

Soil or fill imported for scour protection works should be checked to ensure that it is not contaminated. Grass seed mixes should be specified to give an appropriate composition in terms of durability, maintenance and variety to suit the location.

Some of the greatest environmental risks and impacts are likely to occur during the construction of the works. Typical impacts include the risk of oil or fuel spillage, loss of amenity, traffic nuisance, noise and dust. Many of these impacts can be adequately mitigated by the enforcement of site rules and by good construction practices. Further information can be found in the CIRIA *Environmental handbooks for building and civil* *engineering projects*, Publications C512, C528 and C529 (Venables *et al*, 2000) and from CIRIA C532 *Control of water pollution from construction sites – guidance for consultants and contractors* (Masters-Williams *et al*, 2001).

The impacts of some of the main types of scour protection are briefly discussed below with suggestions made on mitigation and enhancement measures.

Riprap

Riprap can be visually intrusive where used in inappropriate locations. For example, in lowland areas where there is no locally occurring stone, its use may appear out of place. Elsewhere, particularly in upland cobble and boulder rivers, it can blend well with its surroundings. Use of local stone is preferred where possible (and is normally cheaper anyway) to reduce the length of haul routes (and hence traffic disruption) as well as to match the visual appearance of the surroundings. Stone would normally be sourced from an existing quarry, but where opening a new quarry is considered, the impacts of this in terms of noise, waste, effect on landscape and traffic must be assessed.

Riprap can provide a good habitat for invertebrates. The stone is normally inert and does not pose a pollution risk, but in sensitive areas it may be necessary to test for the potential for harmful leachates. Its appearance can be masked by filling the void spaces above water with soil or gravel. Replacement of any excavated material from the river bed and banks can re-establish habitats and hide the riprap.

Gabions

The same issues apply to gabion stone as apply to riprap. The gabion mesh and the more linear form of gabions gives a slightly less natural appearance than riprap, although the mesh is often practically invisible from a distance. Soils or excavated material can be replaced over the gabions. In addition, gabions are now available that include planting pockets or pre-seeded mats that can greatly enhance their appearance. Gabion construction can be labour-intensive, which can be an advantage for the local economy in regions where labour is cheap.

Concrete blocks

Concrete blocks in themselves have a less natural appearance than stone, but this can be appropriate in some urban environments. The cells of the blocks are often filled with topsoil and seeded (or filled with pre-seeded topsoil); these rapidly give the appearance of a grassed river bank so that, after a few years, the presence of the concrete blocks may only be detected by close inspection.

Grout-filled bags and mattresses

These do not have a natural appearance, but can be appropriate in urban environments and where predominantly viewed from a distance. They do not generally provide attractive habitats. There is a risk of pollution from cement washing out of the bags when laid.

Bituminous systems

These do not have a natural appearance, but can be appropriate in urban environments and where predominantly viewed from a distance. They do not generally provide attractive habitats. Open stone asphalt has been extensively used on coastal and estuarine flood embankments, and can provide a more acceptable appearance through colonisation by local flora.

Biotechnical solutions

The incorporation of vegetation into scour protection measures offers the opportunity for a natural appearance and varied habitats, for example by trapping sediment. Their use can encourage management of local resources such as coppices and wetlands and can help to maintain local craft skills.

Concrete aprons

Concrete aprons are predominantly used below water and out of sight. They tend not to offer opportunities for habitat creation, although there are always ways in which a barren concrete surface can be improved to incorporate more attractive habitats.

Stone pitching

The issues for riprap also apply to the stone for stone pitching. Pitching is rarely covered by soil. Instead, in the right context, it can provide an interesting and appropriate architectural feature. However, it offers limited opportunities for habitat creation. Its construction is labour-intensive, which can be an advantage for the local economy.

Sheet piling

Sheet piling is a relatively hard type of protection with a functional appearance. It can be softened with planting pockets attached to the sides of the piles or, in some circumstances, low-level (below water) piling can be used, with softer protection (fibre rolls etc) above.

6.2 TEMPORARY WORKS

Temporary works required during the construction of a scheme are liable to suffer flow-induced scour in exactly the same way as permanent structures. Careful consideration should be given to possible scour risks as part of the safety assessment needed for the temporary works, as well as to help ensure that the structures are not damaged or delayed by unexpected problems due to scour.

The estimation of potential amounts of scour and the design of possible protection works should be carried out using the information and guidance given in Chapters 3, 4 and 5. An important issue is the selection of a suitable return period for the design flood needed to estimate flow conditions at the temporary structures. Following the recommendations in Section 3.7, the return period, N (years), can be calculated from Equation (3.20) using the known design life, L_y (years), of the temporary works (ie, the period for which they will be in place) and the required degree of safety (expressed in terms of the probability, P_r, of the design flood being exceeded in that period). To establish the most economic solution, it is recommended to consider a range of P_r values to determine how much additional scour (and additional cost) would result if a higher degree of safety were to be specified.

If structures such as bridge piers are built inside cofferdams, the temporary works usually have more severe effects in terms of flow blockage and scour than the permanent works. Also, associated features such as temporary embankments or piled jetties used for access, or floating platforms for installing equipment, can significantly increase the amount of blockage and divert flow from one part of a channel to another. A careful analysis, therefore, needs to be made of the various stages of construction to determine the most critical flow conditions for individual structures. Account should also be taken of the likely concurrence of each construction stage with seasonal variations in flow.

Sheet-pile cofferdams that are rectangular or square in plan are usually the easiest to construct, but they are also the worst possible shapes in terms of local scour (see Section 4.3.2). Constructing a streamlined nose to the upstream end of the cofferdam may allow the depth of toe-in required at the base of the sheet piles to be reduced, leading to a cheaper solution overall. In rivers where the main flow direction is not well-defined, or in estuaries where the direction may vary throughout the tidal cycle, circular cofferdams are likely to be the best option to reduce scour. For given flow conditions and water depths, a circular cofferdam reduces depths of local scour by about 25 per cent compared with a rectangular or square cofferdam of the same transverse width.

If it is decided to install bed protection works around a temporary structure such as a cofferdam, these should be designed and installed according to the recommendations in Chapter 5. In fast-flowing conditions, local scour can develop within a a few hours, so it may be necessary to install the protection system before starting sheet-piling. The blockage effects of temporary works may also increase flow velocities in other parts of the channel and lead to contraction scour of the bed and/or increased erosion at the banks. Protection works for these areas may, therefore, also be necessary during the construction stage.

As mentioned above, temporary works may cause larger depths of contraction scour and local scour than will occur once the permanent works have been completed. The scour holes tend to fill in again over time, the rate depending on the amount of natural bedload transport that occurs. In some situations, however, it may be necessary to return the bed of the channel to its original state as soon as the temporary works are removed and to provide a protective layer so that the fill material is not easily eroded. A typical example is provided by the case of a bridge pier where the pile cap is designed to remain below the level of the bed, with a more slender pier projecting through the full depth of the water. If the pile cap remains buried, the local scour depth is determined by the dimensions of the pier. However, if the pile cap is partially exposed to the flow, its greater size results in an increased depth of local scour that will, in turn, expose more of the pile cap. In this case, filling the scour hole formed around the pier by a temporary cofferdam enables the long-term depth of local scour to be minimised.

6.3 RISK ASSESSMENT

6.3.1 Why is risk an issue for scour at structures?

Scour arises from a combination of natural and man-made hazards. The hazards giving rise to scour and the consequences of scour affect decision-making processes in:

- the budget and programme for maintaining and improving a set of existing assets, such as road or railway bridges
- the overall design of a new bridge or structure
- the choice and details of scour protection options.

In addition, the process of constructing scour protection works in rivers and estuaries presents its own risks, whether these be financial or related to time, quality, environmental or health and safety factors. Risk assessment allows the hazards to be identified and the consequences to be recognised. The uncertainty in the performance of the structure can also be considered. It allows a variety of options to be compared using a common framework, taking into account factors other than cost alone. An understanding of the risks allows them to be managed. The risks may be managed by avoiding or mitigating them by creative design and construction, or it may be decided to share, transfer or accept the risks.

Risk assessment and risk management should be integral to the following processes:

- *Development of monitoring, maintenance and remedial works programmes for existing structures.* This process identifies those structures that have the highest priority for action and those that require further investigation, with the overall aim being to produce a consistent and well-planned programme. This type of prioritisation has been described in detail in Section 4.7 with reference to UK and US practices.
- Definition of appropriate design conditions. The risk assessment allows explicit account to be taken of uncertainties in the following factors: the maximum scour that a structure is likely to experience; the corresponding scour-induced loads on the structure; the probability that the design conditions will be exceeded; and the consequences resulting from such exceedances.
- Construction risk management. Working in a river and estuarine environment presents a range of risks to the successful completion of a project. These risks need to be managed, to ensure that construction is completed safely, to time, within budget, to the specified quality and with an acceptably small impact on the environment.

In addition, risk assessment may be encountered in the following circumstances:

- a wide-ranging risk assessment for infrastructure planning that could cover topics such as financial risks, risks to meeting completion targets, health and safety risks and environmental risks
- comparison of different scour protection methods to determine the most appropriate design taking into account such aspects as cost, the risk of failure, environmental factors, health and safety issues, quality control, duration and ease of construction
- determination, for an asset or series of assets, of the relationship between cost of scour damage and frequency, to enable assessment of likely maintenance expenditure.

It is sometimes assumed that methods of risk analysis are complex and therefore inappropriate for situations such as scour, where the factors leading to failure are uncertain and difficult to quantify. However, a range of techniques can be applied that are both qualitative and quantitative at both a broad-brush and a detailed level. It is essential to use the technique that is most appropriate to the scale and importance of the project or structure.

The following sections give brief details of the main risk assessment techniques that may be useful for the assessment of scour and suggest in what circumstances they may be appropriate. Further guidance on risk management in the general field of construction can be found in CIRIA SP125 (Godfrey, 1996). Guidance on construction risk specific to the riverine environment can be found in *Construction risk in river and estuary engineering* (Morris and Simm, 2000). Some of the risk assessment techniques outlined in Section 6.3.3 that are particularly applicable to the riverine environment are discussed in more detail in *Flood and coastal defence: project appraisal guidance: approaches to risk* (MAFF, 2000).

6.3.2 Deciding on a design return period

The design life of a structure is commonly between 30 and 150 years, depending on the size and nature of structure being designed. A structure would normally be designed to withstand a flood flow of a given magnitude, the design flood, which has a certain probability of occurring. The probability is normally expressed in terms of return period, with a return period of N years likely to be exceeded, on average, once in N years.

It is not normally appropriate – although in practice it is often done – to use the design life of a structure as the return period of the design flood. The danger of doing so is illustrated by considering a structure with a design life of 100 years, which is designed for a 100-year return period flood. This would have a 63 per cent chance of experiencing a flood of that magnitude or greater over its life.

A more rigorous approach is required when deciding an acceptable probability of the design flood being exceeded over the structure's design life. The probability of exceedance, P_r, the design life, L_y, and return period, N, are connected by the following formula:

$$P_r = 1 - \left(1 - \frac{1}{N}\right)^{L_y}$$

from which it can be shown that the relationship between the return period and the design life varies with the acceptable degree of risk as follows:

$P_r = 0.60$	$N \approx 1.1\ L_y$
$P_r = 0.50$	$N \approx 1.5\ L_y$
$P_r = 0.40$	$N \approx 2.0\ L_y$
$P_r = 0.30$	$N \approx 2.8\ L_y$
$P_r = 0.20$	$N \approx 4.5\ L_y$
$P_r = 0.10$	$N \approx 9.5\ L_y$

If, for example, a 20 per cent chance ($P_r = 0.2$) of the design flood being exceeded during the 50-year design life of a structure is judged acceptable, then a design flood with a return period of $N = 225$ years should be used. The relationship in Equation 6.1 is shown graphically in Figure 6.1. Due to safety factors included in the foundation design, a small exceedance of the design flood should not normally result in the failure of a bridge; this can make it more difficult to judge what constitutes an appropriate probability of exceedance for use in design.

In the UK, a flood return period, N, of 100–120 years is often used, although a range from about 30 years for minor rural roads to 150 years or more for motorway or trunk roads is also typical. In the United States, bridges are generally designed to withstand a 100-year flood with a load factor of about 1.5–2.0 against collapse; a separate check is also made to ensure that the bridge will survive what is termed a "superflood", with the load factor for the structure remaining greater than 1.0. The superflood is normally taken to be the 500-year return period event. The stability of foundations at existing bridges is also checked for the superflood. Equation 6.1 indicates that, for a typical bridge with a 100-year design life, US practice can be considered as equivalent to accepting an 18 per cent probability that failure of the bridge might occur during this period.

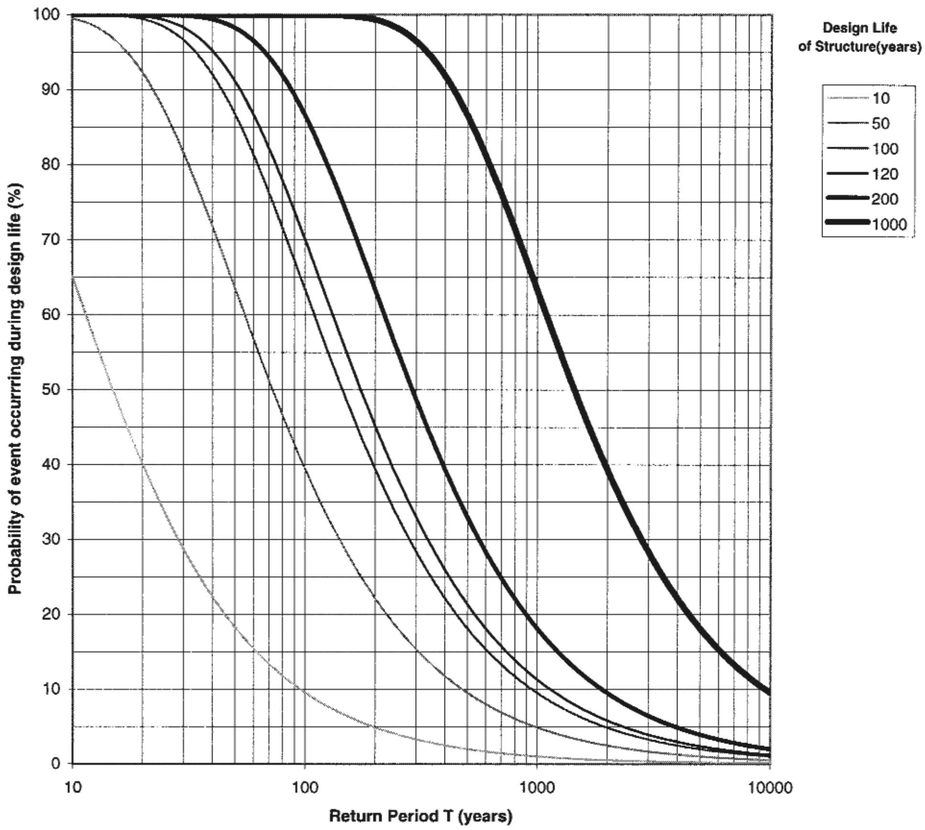


Figure 6.1 *Choice of design return period*

The US approach of designing highway bridges for a certain magnitude of flood and then checking that failure will not occur in a more severe event (ie the superflood) has merits, and it is suggested that this type of approach be adopted more generally, except in the case of major projects where more detailed analysis may be appropriate. However, the US practice of defining the design flood as the 100-year event and the superflood as the 500-year event may not be appropriate in all cases. Table 6.1 indicates how a distinction can be made between different types of structure when determining the acceptable probability of occurrence of the design flood and the superflood.

Table 6.1 *Indicative values of return period for design*

Type of structure	Return period, N, as multiple of design life, L_y, of structure for specified probability of exceedance, P_r			
	Design flood		Superflood	
	P_r	N	P_r	N
Minor access crossing (eg farm access)	0.6	1.1 L_y	0.4	2.0 L_y
Minor road crossing	0.5	1.5 L_y	0.3	2.8 L_y
Major road crossing	0.4	2.0 L_y	0.2	4.5 L_y
Motorway, trunk road crossing, railway crossing	0.3	2.8 L_y	0.1	9.5 L_y

It should be noted that the figures given in Table 6.1 are only indicative. It is recommended that asset owners should define their own design criteria using a risk assessment based on the values and consequences of failure for their particular assets.

In the case of scour protection systems and associated training works, it is normal practice to design them to resist erosion or movement of armour material for flow conditions up to and including those occurring in the design flood, but to accept that there might be some damage (and need for subsequent repair) in more severe events. If there is a likelihood of the protection being completely lost in the superflood, it is necessary to check the stability of the structure and its foundations on the assumption that the full potential depth of scour might occur.

6.3.3 Construction risks

In designing and undertaking scour protection works, construction, health and environmental hazards will be encountered or created. These may well be different from those more generally encountered, particularly as the hazards of working in, or close to, water and the possibility of causing pollution must be addressed. Within the UK the methods of assessing these hazards and their consequential risk are well established and, in many cases, enforced by legislative requirements. It is essential that the reader is aware and complies with the legislation and understands the hazards particular to the riverine environment. These issues extend beyond the scope of this manual, and the reader is recommended to consult specialist books which deal with these matters. Guidance on the health and safety risks of working on or near water is given in *Site safety for the water industry* (Finney *et al*, 1997) and in *Site safety handbook* (Bielby, 2001).

Working in a water environment is more prone to disruption from external factors such as weather. The consequences of these disruptions can be an increased time for project completion and increased costs to the client. These consequences can be managed through careful choice of contract type and the use of good project management.

Guidance on construction risks that may arise during construction of scour protection can be found in *Construction risk in river and estuary engineering* (Morris and Simm, 2000).

6.3.4 Risk assessment techniques

Risk assessment techniques range from fully qualitative assessments based on engineering judgement alone, to fully quantitative assessments using probabilistic methods. In between are a range of semi-quantitative assessments.

The use of fully qualitative assessments, or engineering judgement, is invaluable and the knowledge and experience of hydraulic engineers should not be undervalued. Qualitative risk assessments can formalise the process of engineering judgement and are often used to screen or rank risks in order to identify which hazards to concentrate on. Qualitative assessments can include the listing of all identifiable hazards in a risk register. The risk register allows hazards to be explicitly identified, and their probability and consequences considered, so that actions can be taken to avoid or mitigate the risks. Any residual risk that remains after all actions have been taken should be documented so that it is clearly identified who is taking on that risk. The risk register therefore provides a mechanism for tracking risks, so that all parties involved understand how the risks have been eliminated or reduced, what risks remain and how they are shared between the parties.

The fully quantitative techniques involve a detailed analysis of each of the components making up a structure and its surroundings, to identify the events that could lead to failure. Two techniques routinely used in hazardous operations, such as in the offshore and aerospace industries, are fault-tree and event-tree analysis. Fault-tree analysis involves identifying a failure state (eg movement of a bridge pier) and working back to identify all the events leading to that failure state being reached. By assigning probabilities to each discrete event, the overall risk of the failure state being reached can be assessed. The event-tree analysis works in the opposite direction, by identifying the events which may occur as a result of the loads on a structure (eg removal of material at abutment toe due to current action) and assessing the consequences of those events.

Fault-tree analysis is more appropriate for mechanical or electrical systems where there are well defined interactions between components and statistical data on failure rates are available. It is less appropriate for hydraulic situations, where failure is commonly progressive. The application of fully quantitative techniques has been difficult in riverine conditions because the supporting statistical data are limited and may not be applicable to different situations. There is also a large degree of uncertainty associated with allocating probabilities to some failure events, and the cost of the analysis is often prohibitive.

Semi-quantitative techniques can combine the benefits of the above two techniques. An example of this type of assessment is the Railtrack scour risk assessment. This uses hydraulic data (eg channel dimensions) with structural data (eg pier dimensions) to estimate scour depths. A number (or priority rating) is determined based on a comparison of the calculated scour depth with the depth of the foundation, and taking account of the type of foundation and how much is known about the structure. The priority rating lies within a certain category from 1 to 6, which determines whether the bridge is at high or low scour risk. The process thus uses qualitative data about a structure to generate a quantitative assessment or ranking.

Another type of semi-quantitative technique is the multi-criterion approach. All the criteria that affect the decision making process are listed, such as maintenance cost, environmental impact and safety. Each option is assigned a score for each of the criteria. For example, the scoring system for environmental impact might be: 1 no impact, 2 low impact, 3 medium impact, 4 high/unknown impact. The criteria are assigned a weight according to their relative importance in the decision-making process. The weights are applied to the scores for the criteria for each option and summed to give an overall score for each option, from which the preferred option can then be selected. It is a useful technique for comparing different options where the decision making cannot be reduced to a simple monetary comparison of economics/finance, and is often used in large-scale planning/conceptual design.

6.4 COST-BENEFIT ANALYSIS

Cost-benefit analysis (CBA) is a tool for guiding decision making. Its main uses in scour at structures are to guide investment decisions about the scale and programme for scour protection improvements of assets (such as bridges), and to guide design decisions when comparing structural measures with alternative scour protection measures. Its main value is that options can be compared using a common unit of measurement, money. It is the valuation and conversion of the different kinds of non-monetary costs and benefits (such as environmental costs) into monetary units that can take time and effort in CBA. An example of the use of CBA for design decision making is given in Box 6.1.

The choice of superflood for design can be determined in terms of economics alone using cost-benefit analysis. However, it can be difficult to take account of the non-economic consequences of failure, such as loss of life. An example of the use of cost-benefit analysis to determine an appropriate superflood is shown in Box 6.2.

More detailed guidance can be found in *Cost-benefit analysis for engineers and planners* (Snell, 1997) and in *Flood and coastal defence: project appraisal guidance: economic appraisal* (MAFF, 1999).

Box 6.1 *Example of the use of cost-benefit analysis for making design decisions*

An existing bridge which carries a minor road across the River X is known to be at high risk from scour. The main choices faced by the asset owner are to do nothing and accept that the bridge may one day fail, to construct scour protection works around the bridge to reduce the risk of failure, or to abandon the bridge and build a new one nearby with deep foundations.

The cost of scour protection works is estimated to be £500 000. The cost of a new bridge is estimated to be £5 million. The costs associated with a bridge collapsing and being rebuilt are estimated to be £10 million (about twice the cost of a new bridge).

The damage to the bridge and its associated infrastructure that are expected to be caused by floods are estimated to be:

Return period (years)	**Exceedance probability (per year)**	**Damage per flood (£ × 1000)** Do nothing	Install scour protection	Build new bridge
10	0.1	0	0	0
20	0.05	100	20	0
50	0.02	1000	50	0
100	0.01	5000	1000	0
200	0.005	10 000	2000	20
1000	0.001	10 000	10 000	10 000

This relationship between damage and probability is shown in Figure 6.2. The area under each curve represents the total annual expected damage or average annual damage. The average annual damage (or area under the curve) for each option is:

Do nothing	£136 000
Scour protection	£48 000
New bridge	£30 000

If these annual costs are discounted at 6 per cent over a life of 50 years to a present value, the present value cost of damage is:

Do nothing	£2 150 000
Scour protection	£760 000
New bridge	£470 000

The difference between the present value cost of damage of the do-nothing option and the present value cost of damage of the other options represents the benefits of carrying out works. Thus the benefits of carrying out the scour protection works is £1 390 000 (£2 150 000 – £760 000) and the benefit of replacing the bridge is £1 680 000 (£2 150 000 – £ 470 000). This can be compared with the costs of the options. It can be seen that the cost of scour protection works (£500 000) is less than the benefits that it will yield (£1 390 000). The cost of the new bridge (£5 000 000) exceeds the benefits that it will yield (£1 680 000).

Dividing the benefits by the costs gives benefit-cost ratios for the two improvement options:

Option	**Benefit-cost ratio**
Scour protection	2.8
New bridge	0.3

The scour protection option therefore appears the most appropriate.

What have not been explicitly included in this simple example are the non-monetary costs, such as loss of life and environmental costs. In addition, the owner of the bridge may have limited resources to fund the improvements, but by carrying out similar CBAs on his other expenditure he can target his spending towards expenditure with high benefit–cost ratios.

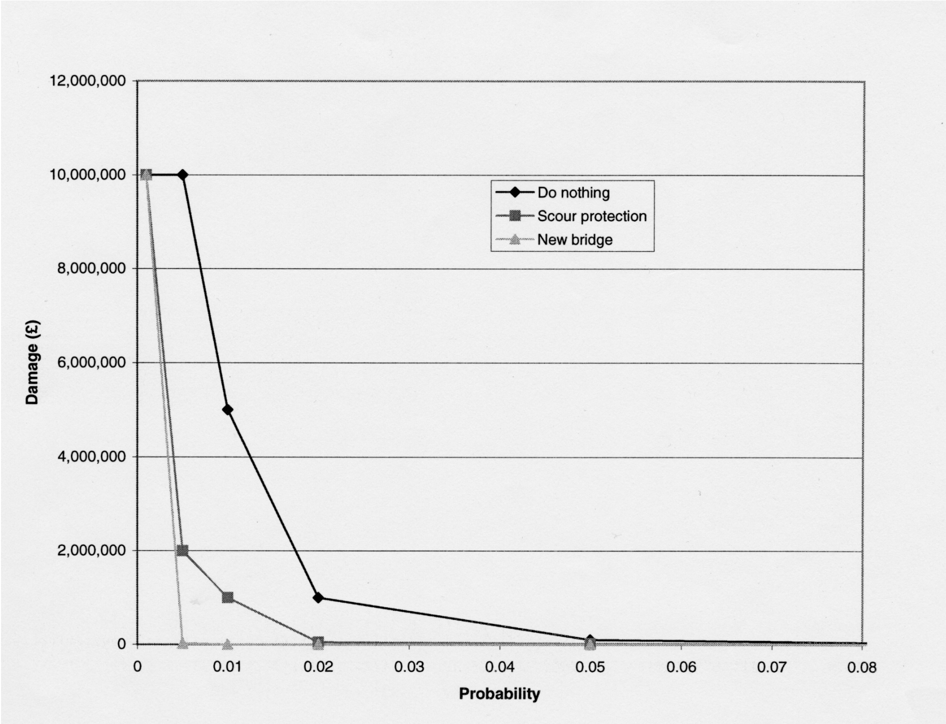


Figure 6.2 *Example of damage–probability relationship*

Box 6.2 *Example of the use of cost-benefit analysis to determine the superflood*

A bridge with a design life of 100 years is required across the River Severn. The cost of construction varies depending on the superflood it is designed to withstand, as deeper foundations are required for the more extreme floods:

Return period (years)	Cost of construction (£ million)
100	10.0
500	12.0
1000	13.0

Reconstruction costs following failure are estimated to be about two times the construction costs. (From general experience, typical reconstruction costs are about two to three times the construction cost.) Each year there is a probability of failure of 0.01 if designed for the 100-year superflood, 0.002 if designed for the 500-year superflood and 0.001 if designed for the 1000-year superflood. Combining the probability of failure with the cost of reconstruction and discounting to a present value over the 100-year design life (with a discount rate of 6 per cent) gives the following present value costs of reconstruction:

Return period (years)	Present value cost of reconstruction (£)
100	3 300 000
500	700 000
1000	33 000

Thus the additional construction cost of increasing the design standard from 100 to 500 years is £2.0 million, but it will yield benefits of £2.6 million (£3.3 million–£0.7 million). The additional cost of increasing the design standard from 500 years to 1000 years is £1.0 million, but will only yield benefits of £0.67 million (£0.7 million–£33 000). An appropriate return period for the superflood would therefore be 500 years.

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Appendix 1 Monitoring equipment

There has been considerable interest recently in the potential for utilising scour monitoring techniques to provide an early warning of danger, as an alternative to undertaking preventive works. There are several reasons for considering the use of scour monitoring equipment:

- scour holes generally refill, at least partly, during the recession of the flood, so that the extent of actual scour, the damage that may have resulted, and the associated risks, cannot be appreciated from a post-flood inspection
- the high cost of implementing remedial works on a precautionary basis at a large number of structures where there is no direct evidence of a scour problem but calculations indicate that there may be such a problem
- the opportunity to improve public safety more cheaply and quickly and at a larger number of structures than would be practicable if measures to reduce scour or protect structures against its effects were to be adopted.

The interest is probably greatest in the USA, because of the large stock of bridges. It is at bridges where the potential dangers to the public posed by scour and the benefits of monitoring are most obvious. The principles are, however, potentially applicable to all scour-susceptible structures.

Scour monitoring has two main functions. First, monitoring of an at-risk structure can warn of problems, so that the structure can be closed or repaired. Second, data collection can improve understanding of scour processes. While both roles can be carried out by the same installation, the primary function is likely to dictate the choice of device and the type of data collection.

An extensive research project was carried out between 1989 and 1996 under the US National Cooperative Highway Research Program (Lagasse *et al*, 1997), which comprised three phases:

1. the development of performance criteria and identification of suitable instruments
2. field evaluations of the most promising devices
3. additional field evaluations and refinement of sonar and sliding collar devices.

The results of the research are summarised here, to give the reader initial guidance on the devices that may be considered. For detailed guidance reference should be made to *NCHRP Report 396* (Lagasse *et al*, 1997) and *Reports 397A* and *397B* (Schall *et al*, 1997). In places the trade names of particular products have been quoted, to aid recognition of the devices and techniques used. The use of such trade names should not be taken to imply any endorsement by CIRIA of the products concerned.

A1.1 PERFORMANCE CRITERIA

The performance criteria formulated for the types of scour monitoring equipment that were considered suitable for further evaluation were as follows.

Mandatory criteria

- capable of installation on or near a bridge pier or abutment
- able to measure maximum scour depth within an accuracy of ± 0.3 m
- capable of obtaining readings from above the water surface
- suitable for use at a remote site
- operable during storm and flood conditions.

Desirable criteria

- capable of installation on most existing bridges or during construction of new bridges
- capable of operating in a range of flow conditions
- able to withstand ice and debris
- relatively low cost
- resistant to vandalism
- operable and maintainable by highway maintenance personnel.

None of the criteria explicitly specifies the reliability of the device, which has been found to be the most difficult aspect to control.

A1.2 EQUIPMENT AND TECHNIQUES

Historically, the equipment used to monitor scour comprised sounding rods for shallow flows and lead-weight sounding lines for deeper flows. By the mid-1950s devices such as sonic sounders were commercially available and became widely used in hydrographic surveys. Pioneering work on scour monitoring was done at the University of Iowa in the 1950s, where a "scour meter", mounted in the bed upstream of a bridge pier and operating on the "electrode-impedance" principle, was able to sense the water–sediment interface. The research project found examples of the recent or current use of sonic depth sounders, lead-weight lines, sounding rods and sliding collars. Yet, by 1990, "there were no accepted methods or off-the-shelf equipment for collecting scour data in the United States" (Lagasse *et al*, 1997).

From a literature search and review of available technologies, devices were grouped into:

- sounding rods
- buried or driven rods
- sonar
- other buried devices.

Sounding rods

In their simplest form, sounding rods may be deployed manually, but a development allowing unattended operation comprises a rod contained in a sleeve, the rod resting on the bed and free to move down as scour occurs, with its base remaining at the lowest scour level when the scour hole refills. The maximum scour depth can be determined by measuring the displacement of the top of the rod. Such devices are reported (Lagasse *et al*, 1997) to have been deployed as early as the 1920s.

Buried and driven rods

These devices mostly include a hollow tube and might alternatively be described as "tube-mounted sensors", but we will retain the same terminology as in the reports on the NCHRP studies, to avoid confusion. The devices are installed at the location where maximum scour is expected and those previously used include:

- the Scubamouse, installed on many bridges in New Zealand and consisting of a vertical tube, around which is placed a radioactive horseshoe-shaped collar, which initially rests on the bed and sinks as scour occurs, its final position being detected by a probe sent down the tube
- the Wallingford tell-tail device, installed on many older high-risk bridges in the UK since 1990, which uses a set of several omnidirectional motion sensors, mounted on flexible "tails" connected to a rod and buried in the bed at a range of depths and connected via cable to a logger
- the US Geological Survey conductance probe, first used in Arizona in the mid-1970s and more recently in Arkansas.

Sonic fathometers

Sonic fathometers, which use reflected acoustic waves to detect the interface between the water and the riverbed, were successfully used for scour monitoring at several sites in Oregon in the early 1990s, with the transducers mounted on brackets fixed to the piers, pointing slightly outwards to avoid the side of the pier causing interference.

Other buried devices

The category includes sensors or markers – which may be tethered or untethered – buried at various elevations, which are exposed by scour and displaced, either by floating or rolling. In their simplest form, the disappearance of a marker would be evidence of scour to below its level. Alternatively, a sensor may include a motion-activated switch hard-wired to a logger or a motion-activated wireless transmitter.

A1.3 LABORATORY TESTING

From an initial screening of the practicability of the devices, several were selected for laboratory testing (Lagasse *et al*, 1997). This took place at Colorado State University, using both indoor and outdoor flumes, the latter at a scale described as "near-prototype".

Sounding rods

Various devices featuring a sleeved rod were tested, with and without a base plate. It was found that the device worked best when mounted vertically, so would generally be unsuitable for use on the noses of inclined piers and abutments. For sand-bed rivers it was found that the base plate has to be large enough to avoid penetrating the bed.

Buried and driven rods

Three versions of this type of device were tested, deploying different forms of sensor, all mounted on a single support tube for the tests:

- piezoelectric film sensors (with exposure by scour detected from the electrical signal generated by flexure or vibration)
- mercury tip switches (which flip down and break a circuit when exposed by scour)
- magnetic sliding collar (with magnetic switches on the wall of the pipe to detect movement of the collar).

The tests indicated that the presence of the rod did not have an appreciable effect on the amount of scour that occurred. The sliding collar could experience problems with sticking or jamming, but these problems were overcome with careful design. There were some problems with the mounting, encasement, durability and reliability of the sensors and switches, but improvements were made and it was thought that these could be resolved with further refinements.

Sonic fathometers

Commercially available sonar equipment ranges from expensive research-quality devices to low-cost sounders, typically used for shipboard depth echo-sounding and fish finding. The laboratory trials concentrated on methods of deploying the low-cost devices. There were problems with loss of signal, which were generally attributed to air entrained in the flow by the inlet arrangements to the laboratory flume.

Other buried devices

The tethered devices tested in the laboratory were 25 mm oval fishing floats, connected to the bridge pier by fishing line, while the untethered devices were neoprene bottles of a suitable size to contain a transmitting device and batteries. The tests on dummy devices indicated that the techniques were potentially viable, but further work would be needed to develop suitable sensing methods and equipment.

A1.4 FIELD TRIALS

The field trials (Lagasse *et al*, 1997) were intended to build up realistic experience in the operation of the devices that appeared most promising in the laboratory tests, and were carried out in conjunction with the bodies that would ultimately be responsible for their routine installation and operation. The trials covered a wide range of site conditions, some of which were undertaken in co-operation with other research projects.

Sounding rod

The equipment tested was the Brisco Monitor, a patented device in which a cable is fixed to the top of the rod and movement is detected by a counter on the cable reel. Problems were experienced at the trial site, due to the sounding rod settling into the sand bed. The addition of a larger base-plate overcame that problem, but the subsequent scour at the site was sufficient to allow the rod to slip completely out of the guide pipe.

Magnetic sliding collar

Magnetic sliding-collar devices were installed at several bridges, for which a standard field prototype design was developed, including an “open architecture” collar and 50 mm stainless steel support tube in 1.5 m-long sections. Some devices provided automatic readout from magnetic switches, while others were designed for manual readout; in the latter type a probe containing a magnetic switch, connected via a graduated cable to a battery and buzzer, was lowered manually down the support pipe to detect the position of the collar.

The field trials were generally successful, the main problems being encountered in driving the support tubes. A suggested refinement would be to use a split-ring type of collar, which can be placed over the tubes from one side, so that a second collar can be readily installed if the first is buried by refilling of the scour hole.

Low-cost sonic fathometer

Five field trials were conducted under the NCHRP project, one of which was on a sloping abutment, and four more under other projects, using low-cost “fish-finder” sonar equipment. All the installations and trials were successful, apart from one transducer that was damaged by impact by a barge.

Piezoelectric film devices

Limited field trials were conducted at two sites on driven rod devices with external piezometric film sensors to detect exposure by scour. At one, it was found that the sensors were highly sensitive to vibrations of the support pipe caused by the flowing water and by traffic on the bridge, sometimes with little difference between the signals emitted by the buried and unburied sensors. At the other site, good results were obtained.

A1.5 APPRAISALS

Arising from the laboratory tests and field trials *NCHRP Report 396* (Lagasse *et al*, 1997) considered two of the devices to be "fully operational":

- the magnetic sliding collar (mounted on a driven tube)
- the low-cost sonic fathometer.

Separate documentation was prepared on these two devices, including supplier details, fabrication, installation and operation information, in *NCHRP Reports 397A* and *397B* (Schall *et al*, 1997).

General evaluation

The above "fully operational" devices and several of the other devices and techniques considered were rated according to their performance with respect to the mandatory and desirable criteria.

Sounding rods, as represented by the Brisco Monitor, were considered strong, durable, simple and suitable for measuring scour at vertical and near-vertical piers and abutments. The basic concept was considered sound and successful operation had been reported on coarse-bed streams, but "fundamental problems existed with the cable reel, pulse counter, and associated electronics" (Lagasse *et al*, 1997). The manufacture had reported that improvements had been made to the data storage and retrieval systems.

The magnetic sliding collar device was considered to meet or exceed all the mandatory performance criteria, to be "relatively inexpensive, easy to install and operate, and can be upgraded to provide an automated readout capability". However, it was noted that "the installation methods and the design of the device in terms of withstanding installation forces and surviving operational conditions, require careful consideration for each site".

Both sounding rods and sliding collar devices are designed to detect the maximum scour, so are suited to safety monitoring of bridges, perhaps including direct warnings to the highway police and motorists, but not to circumstances where it is desirable to track the entire scour episode, such as scour measurement for analysis and research purposes. Both devices have to be "reset" to monitor the next event if the scour hole has refilled, the rods by being forcibly raised or dug out, the sliding collar normally by sacrificing the buried collar and installing a new one on the bed.

Low-cost sonic fathometers were found to be reliable and durable under a wide range of conditions and able to meet all the mandatory criteria and most of the desirable criteria, although it was noted that debris, ice, entrained air and possibly high sediment loads would present problems. In the marine environment, the growth of marine organisms on the transducer can be a problem.

The investigations of driven rod devices with piezoelectric film and other types of sensors indicated that these are well suited to measuring scour, but that further design development was needed for the device to withstand installation forces, limiting use at present to ephemeral streams and overbank areas where installation could take place in the dry.

Sonic fathometers and piezoelectric film devices both have the advantage of being able to record multiple scour and refilling episodes, so are useful, if used in conjunction with water depth and velocity data, for increasing understanding of scour and improving predictive formulae.

Costs

Equipment cost data given in *NCHRP Report 396* indicated a range of US$2500–8000 per site for the various types of device evaluated, the magnetic sliding collar device with manual readout being the cheapest and the patented sliding rod device the most expensive.

Installation costs were estimated to range between US$2500 and US$7000, with a great deal of uncertainty associated with the remoteness of the site and differences in complexity between the sites. The cheapest installation related to the sonic fathometers installed under NCHRP, due to the simple installation technique that was developed. Information on operation and maintenance costs was considered insufficient to give realistic guidance.

It was noted that the various devices were at different stages of research and development, had different capabilities and degrees of functionality, so that the absolute and relative costs might be expected to change in the future.

A1.6 OTHER ISSUES

NCHRP Report 396 (Lagasse *et al*, 1997) gives detailed guidance on several other issues, including:

- suitability of different devices for monitoring, warning and data acquisition
- concepts for installation on sloping abutments
- wireless (underwater) data transmission
- data logging
- telemetry, including microwave, radio, cellular telephones, landlines and satellite
- ground truth instrumentation (independent confirmation of monitoring data).

Appendix 2 Case studies

This appendix contains a number of case studies, drawn from the experience of the authors' organisations, various bodies that were consulted during the course of the study and from published literature. Table A2.1 summarises the case studies and the sources of information used to compile them. Where a date is given in parentheses, further details appear in the main reference list for the manual.

Table A2.1 *Summary of case studies included in Appendix 2*

Case study	Nature of case study	Source
Storebælt west bridge	Design of scour protection to bridge piers	Hebsgaard *et al* (1994)
Cattawade road bridge	Investigation of scour around bridge support piles, design of new piers and scour protection	WS Atkins, HR Wallingford
Cil-Cewydd rail bridge	Scour appraisal and protection measures	WS Atkins
Bransford rail bridge	Scour of embankment downstream of bridge and remedial works	Jeremy Benn Associates, Railtrack
Glanrhyd rail bridge	Collapse of bridge deck following scour failure of pier	Railway Inspectorate (1990), Railtrack
Wraysbury rail bridge	Scour failure of bridge as result of nearby gravel extraction and stream diversions	Railtrack
Bromborough outfall	Scour outside a cofferdam, aggravated by piping	Amec Civil Engineering Ltd
Bulls road bridge	Pier failure from local scour and deck collapse	Melville and Coleman (2000)
Bealey road bridge	Pier settlement due to scour	Coleman *et al* (2000)
Oreti River road bridge	Downstream weir to protect foundations threatened by general degradation	Melville and Coleman (2000)
Schoharie Creek road bridge	Collapse of two piers and three bridge spans due to scour	Jackson *et al* (1991), www.ntsb.gov/ Publictn/ 1988/HAR8802.htm Ayres Associates
Hatchie River road bridge	Collapse of two piers and three bridge spans due to channel migration	Jackson *et al* (1991), www.ntsb.gov/Publictn/ 1990/HAR9001.htm Ayres Associates
Kaoping road bridge	Collapse following scour failure of pier, attributed to gravel extraction	*New Civil Engineer*, 7 September 2000

A2.1 STOREBÆLT (GREAT BELT) WESTERN BRIDGE (DENMARK)

Figure A2.1 *The combined road and railway bridge across the western channel of the Storebælt between Fyn and Sprogø (photograph courtesy of Great Belt A/S)*

The fixed link across the 18 km Storebælt between the Danish islands of Fyn (Funen) and Sjælland (Zealand), via the small island of Sprogø, comprises four elements:

- a combined road and railway bridge across the western channel between Fyn and Sprogø
- land reclamation to increase the size of Sprogø about threefold
- a railway tunnel under the eastern channel from Sprogø to Sjælland
- a suspension road bridge over the eastern channel, which is the main shipping route to the Baltic.

The scour protection for the 64 piers supporting the western bridge, which is 6.6 km long with a clearance of 18 m above sea level, was designed using numerical analysis and physical modelling (Hebsgaard *et al*, 1994). The pier structures, which are located in water depths up to about 30 m, each include two stems, supporting the road and the railway bridges. The plinth beneath the stems is about 6 m wide by 30 m long and is supported on a caisson (17 m × 30 m), which is founded about 3.5 m below mean seabed level. The soils at the bridge are generally erodible and, if subjected to excessive erosion, could lead to undermining and failure of the pier foundations.

The design hydrographic conditions were determined from a combination of field measurements and numerical modelling. Both northerly and southerly tidal currents and wave directions were considered. Wave measurements and numerical analyses indicated that the piers would be exposed to significant wave heights of typically between 3 m and 4 m along the length of the western bridge, at an annual exceedance probability of 1 per cent (return period 100 years). The design currents, based on a numerical simulation using the MIKE 21 hydrodynamic model, calibrated with current measurements at two locations close to the western channel, reach a maximum of about 2.0 m/s, for both northerly and southerly flow, also at an annual exceedance probability of 1 per cent.

For a return period of 100 000 years, the wave heights are estimated to be 30–35 per cent higher and the flow velocities 10–15 per cent greater than the 100-year values.

A numerical model was used to identify the locations along the bridge at which the greatest shear stresses induced by waves and currents would occur, from which three piers, representing shallow, medium and deep water conditions were selected for further studies by physical model testing. The model tests, at scales of between 1:50 and 1:30, comprised two phases:

- Phase 1 investigated the shear stress amplification pattern around the piers for a range of angles of wave and current attack and was used to estimate the proposed extent and size of the stone scour protection
- Phase 2 comprised stability tests under the most critical wave/current direction established in Phase 1, with the proposed scour protection installed.

The Phase 2 tests included two or three stone sizes for the main armour at each of the piers tested, to verify the stone size required. The size of the main armour layer at the piers varies along the length on the bridge, with a minimum of W_{50} = 100 kg and a maximum of W_{50} = 240 kg. It extends to 10 m outside the caisson and is generally 1 m thick, increasing to 2 m thick at the northern and southern ends, to provide additional sacrificial material to armour the scour holes expected in the seabed beyond the scour protection. A maximum scour depth of about 3 m was found under design wave and current conditions in the model tests, but this was expected to take at least 100 years to occur in practice. The main armour is supported on a 0.5 m underlayer, extending a further 5 m beyond the armour and thickened to 1.0 m opposite the ends of the piers.

A2.2 CATTAWADE BRIDGE (UK)

This road bridge, on the Essex–Suffolk border, carries the A137 over the South Channel branch of the River Stour and is located about 20 m upstream of the Cattawade tidal barrage. The bridge was suffering from scour of the bed around several of the bridge support piles, which the owner believed had been caused or aggravated by operation of a tidal barrage 20 m seaward of the bridge.

Remedial measures were planned including:

- construction of two large piers to replace the arrays of small piles
- laying scour protection under the bridge and across the riverbed upstream and downstream.

WS Atkins Consultants Ltd, in association with HR Wallingford, undertook a study to investigate the causes of the scour and assess the performance of the proposed new pier arrangement and scour protection measures. The study involved the use of a 1:20-scale mobile-bed physical model of the bridge and barrage structures and a 90 m reach of the river.

The model tests demonstrated that the proposed new pier arrangements would perform satisfactorily. The proposed scour protection would prevent significant scour during flood tide conditions, but more extensive measures were necessary to control scour under ebb tide conditions.

Construction of the remedial works started in 1997 and are complete.

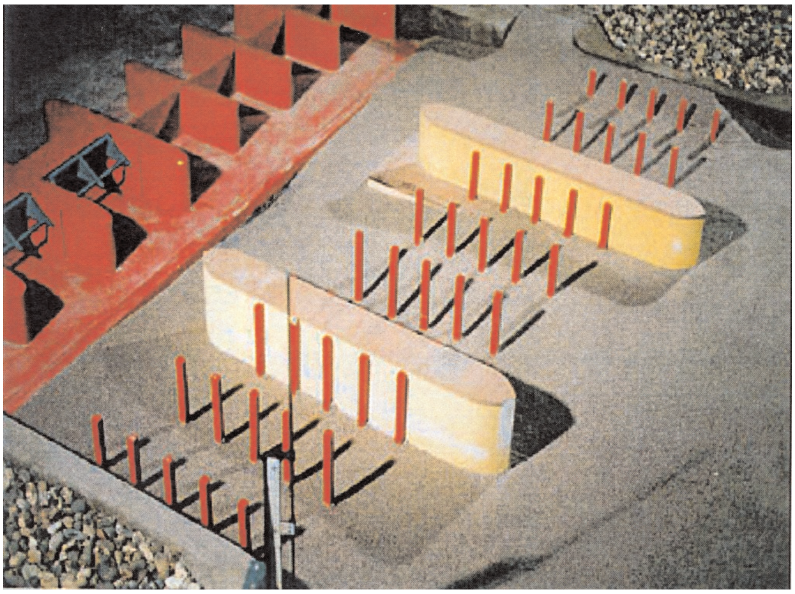

Figure A2.2 *Hydraulic model of Cattawade Bridge (photograph courtesy of HR Wallingford)*

A2.3 CIL-CEWYDD RAILWAY BRIDGE (UK)

Figure A2.3 *Cil-Cewydd railway bridge (photograph courtesy of WS Atkins Ltd)*

A scour appraisal was carried out at this railway bridge, which crosses the River Severn obliquely, and protection measures were designed and installed as part of a national programme of improvement and scour preventive measures at railway bridges.

The appraisal included an assessment of the hydrological conditions at the bridge and a detailed bathymetric survey, which were used to build a MIKE21 hydraulic model of the site. This was used to simulate flow velocities and depths for the 100-year flood event, which were then used for the detailed design of the scour protection measures.

It was believed that previous measures to protect the piers with sheet piling had aggravated the scour, because of the increased effective width of the piers. The chosen remedial measures comprised stone-filled gabion mattresses, which were placed across the river bed, around the piers and at the abutments. The flow velocities were used to determine the appropriate weight and thickness of the gabions.

A2.4 BRANSFORD RAILWAY BRIDGE (UK)

Bransford railway bridge crosses the River Teme 3 km west of Worcester. The bridge originally comprised six arches, two of which were across the main river channel, with the other four forming flood relief arches. Following flood damage, the two main spans were replaced in the 1920s by a single through-span. In the 1980s, works were undertaken at the bridge to remove the remains of the original central pier and also to provide scour protection to the flood arches in the form of concrete inverts.

Within a few years of completion of the works, the western river bank downstream of the bridge started to suffer erosion and, by the early 1990s, the river channel was threatening to migrate towards the railway embankment. A modelling study, using a two-dimensional computer model *SED2D-WES* developed by the US Army Corps of Engineers, concluded that while the sandy river banks are susceptible to undercutting as part of the normal geomorphological processes on the river, the construction of the bridge scour protection works may have contributed to the acceleration of the channel movement.

A feasibility study investigated several options for stabilising the river bank, including the use of riprap and "soft" protection measures. The recommended option was to use a bioengineering solution in combination with a rock armour toe. This option was chosen for its reduced cost, flexibility and also as a result of an environmental impact assessment (the River Teme at this point being a Site of Special Scientific Interest). The 600 mm nominal size rock armour toe was constructed in a trench along a new alignment judged from the modelling work to be an optimum one for reducing local flow velocities. A gravel bed was then created over the rock armour, but below normal river level, to act as a fish spawning area. Above this, the river bank was reinstated using imported fill in a series of terraces formed from live willow fascines and faggots (see Figure 5.28). Finally, additional planting took place of native species – iris and reed canary grass on the gravel bed, and ash and willow on the new bank.

Construction began in September 1996 and was completed within eight weeks. The bank vegetation was well established by late spring 1997 and survived two large floods on the Teme in 1998 and 2000.

Figure A2.4 *The protected embankment six months after construction (photograph courtesy of Jeremy Benn Associates)*

A2.5 GLANRHYD RAILWAY BRIDGE (UK)

At about 07.15 on 19 October 1987 a two-car diesel-multiple-unit train passed on to the bridge carrying the Central Wales Line across the River Towy. Following heavy rainfall and flooding along the Towy, the bridge deck had collapsed as a result of the scour failure of two of its supporting piers at some time during the previous night. The front carriage landed in the river, but the rear one remained on the collapsed deck. Of the 10 travellers on the train, four were drowned, including the driver.

The single-track bridge, which was opened in 1858, comprised five spans of deck timbers resting on pairs of wrought iron box girders. These were supported, at a skew angle of 46°, on masonry abutments and four intermediate masonry piers. In 1958, as a result of overstressing of the wrought iron girders, the superstructure was replaced with a steel structure, comprising welded plate main girders and prefabricated steel deck units. The bridge soffit was raised 840 mm, to give increased flood clearance, by casting concrete blocks on top of the existing abutments and piers. A public footbridge was cantilevered off the downstream main girders.

There was evidence that repairs to remedy scour at one of the piers had been made in 1929, but the records did not identify the pier concerned. Underwater inspections of the bridge had been undertaken at regular intervals and had revealed the need for periodic repairs, which were undertaken in 1970, 1976 and 1982. In 1982, following the discovery that the upstream end of pier 2 had been undermined, mass concrete fill, reinforced with steel rods, was placed at a depth of 6 feet below riverbed level, between the face of the pier and a concrete bagwork cofferdam.

The probable sequence of the collapse, which was thought likely to have occurred in a short time, was considered by the Deputy Chief Inspecting Officer of Railways (Railway Inspectorate, 1990) to be:

- undermining of the downstream end of pier 3, causing it to settle and break its back, after which the downstream end of the pier migrated into an adjacent zone of deeper scour
- the collapse of the downstream end of the main girders of the spans supported on pier 3, leading to the general collapse of the bridge superstructure
- the collapse of pier 2, which was located in about the centre of the main river channel, towards the upstream side.

It was considered that the 1982 repairs to pier 2 may, by thickening it, have increased the amount of scour. The recommendations that emerged from the Railway Inspectorate inquiry included:

- the implementation of an action plan to identify bridges susceptible to damage by river action
- improvements to bridge inspection and assessment procedures, including increased understanding by the staff involved of the effects of hydraulic action on bridge foundations
- improvements to operational procedures that apply at times of flooding, including the receipt of timely flood warning information.

The bridge was damaged beyond repair, so was demolished and replaced with a single-span steel girder bridge on new foundations. The line was reopened just over a year after the failure.

A2.6 WRAYSBURY RAILWAY BRIDGE (UK)

Bridge No 71, a short distance north-west of Wraysbury station on the Staines–Windsor line, collapsed at 00.20 on 10 May 1988, while the last train of the night was crossing. The rear bogie of the train was derailed, but the train came to a safe stop clear of the bridge.

The bridge comprised three 3 m spans of steel trough decking supported on low piers and abutments, carrying the two railway tracks over a small channel, which was normally dry, in the floodplain of the nearby Colne Brook.

Gravel abstraction was being undertaken alongside the railway line, to a maximum depth of about 8 m, with a clearance margin of 15 m to the railway boundary. Initially, this margin contained a stream diversion on the downstream side of the railway, intercepting the flows of three streams – including that passing through the bridge – which previously crossed the abstraction site.

After the collapse, it emerged that the gravel workings had flooded on two previous occasions, due to the bank of the diversion channel breaching into it. In an effort to avoid a repetition, the operator of the gravel pit had replaced the diversion channel with a new channel on the upstream side of the railway line, which collected the flows of the other two streams and passed them all through the one bridge.

On the night in question, heavy runoff resulted in the stream passing through bridge No 71 running full and either overtopping into the gravel workings or causing a seepage failure of the intervening ground, and in any event breaching into the gravel pit just downstream of the bridge. The increased discharges and flow velocities resulting from the escape of water into the gravel pit were sufficient to erode and steepen the bed of the channel through the bridge, undermining its piers and abutments and the adjacent railway embankment.

Bridge No 71 was replaced with a four-cell reinforced box culvert, which was completed about four months after the collapse.

A2.7 BROMBOROUGH OUTFALL (UK)

A temporary working area was required to construct the outfall structure for a water treatment works outfall within the Mersey estuary. A 12 m-diameter sheet-piled cofferdam was driven some distance offshore to provide the working area. The piles were driven through about 2.8 m of loose sand and through a layer of silty clay. It had been estimated that scour from tidal currents might be about 3 m.

After the piles were driven a scour hole developed rapidly around the cofferdam. Within about two weeks it had reached about the same level as the top of the silty clay layer and then it stabilised. When excavation was carried out within the cofferdam below the level of scour, the scour hole began to deepen, however.

It appeared that what was first thought to be a scour problem was in fact a piping problem, with the silty clay material being washed through the pile clutches of the cofferdam due to the difference in hydrostatic pressure across the cofferdam. The scour/piping hole was filled with dumped stone, which alleviated the problem.

A2.8 BULLS ROAD BRIDGE (NEW ZEALAND)

Figure A2.5 *Bulls road bridge failure (photograph courtesy of Bruce Melville)*

The two-lane structure on State Highway 1 across the Rangitikei River, which opened in 1949, comprised 17 spans of 27 m and two end spans of 20 m. The piers were constructed in reinforced concrete and each was supported on two rows of six vertical reinforced concrete (RC) driven piles, extending to about 9 m below the base of the pile cap at the pier which failed. The piles were reported to have been difficult to drive, with anecdotal evidence that some had fractured. The superstructure comprised steel girders with a composite RC deck, with alternate bays including a central suspended span supported by cantilever extensions of the adjacent spans.

The river had been braided at the time of construction. Substantial quantities of gravel were abstracted from the river bed over about a 5 km length centred on the bridge between 1959 and 1973, resulting in the number of channels reducing to one or two main channels, which were deeper than the original channels, meandering within about the limits of the original braided channel. River control works were developed between 1949 and 1973 to protect the meander bends upstream of the bridge. By the early 1970s, the river meanders, reinforced by the upstream bank protection works, led to the main channel deflecting off a terrace of relatively erosion-resistant material at the left bank just upstream of the bridge and passing under the bridge at an angle of about 55° to its centreline.

At 16.45 on 15 June 1973, one pier collapsed, together with the end of the bridge deck it supported and the adjacent suspended spans. An empty school bus being driven over the bridge at the time of collapse struck an adjacent pier during its descent into the river, the driver being injured but surviving the incident.

The lowest bed level at the bridge had fallen by about 3 m between 1945 and 1970 and by up to 2 m between 1970 and 1973. The maximum depth of scour measured after failure was about 12 m below the armoured bed level. The flows in the river shortly before the failure had been equivalent to about the mean annual flood, less than 20 per cent of the flood of record in 1897.

The failure was attributed to local pier scour, which had been exacerbated by the oblique flow, obstruction of the flow by a nearby old timber pile, against which debris had accumulated, and by the exposure of a readily erodible sand layer beneath the shingle stratum. Lateral forces from the oblique flow were also instrumental in the rotational collapse of the pier, with hinging at the bridge deck and between the piles and the pile cap. Significant factors were progressive degradation and morphological changes caused by the gravel extraction for many years.

A2.9 BEALEY ROAD BRIDGE (NEW ZEALAND)

Figure A2.6 *Bealey bridge failure (photograph from Coleman* et al*, 2000, by permission of the authors)*

In October 1998, Pier B of the single-lane road bridge across the Waimakarari River on State Highway 73 settled by 1.7 m, as a result of undermining by scour, with a corresponding drop in the bridge deck. However, the deck spans remained supported by the dropped pier, and a truck that was crossing the bridge at the time managed to complete its crossing.

The bridge, which had been designed in the 1930s, comprises 20 simply supported reinforced concrete spans of about 13 m, supported on slab-type piers founded on driven RC piles, with in most cases each pier having three piles at 1.3 m centres. The design pile length for the piers was 7.9 m, of which 7.1 m would be below the pile cap, but the test records indicate problems at Pier B, with the upstream and central piles driven to only 5.2 m and 6.6 m below the base of the pile cap. The right abutment, which supports the opposite end of one of the spans supported by Pier B, is founded on rock.

The Waimakarari River is a braided gravel-bed river, with a median bed material size estimated as 50–70 mm and a maximum stone size of 200–300 mm. At the time of design, there was a major channel at the right abutment and another about 25 per cent of the overall width from the left abutment. It appears that the channel at the right abutment has been reasonably stable over most of the history of the bridge and that there has been no progressive degradation at the site. In 1948, the upstream edge of Pier Q, located in the channel nearer to the left abutment, dropped by about 60 mm, after which it was underpinned by three additional piles at each end.

During the 1990s there was a significant movement of the braided channels, which resulted, by the time of the failure in 1998, in a highly skewed channel approaching the right abutment from the left, and merging with the right-bank channel before impacting on the piers. The flow at the time of the failure was estimated to be less than the mean annual flood magnitude, but was concentrated into the one braided channel approximately centred on Pier B.

Scour depths measured four days after the failure indicated a maximum scour of 2.7 m below the base of the pile caps, but this was assumed to under-represent the scour at the time of failure, due to refilling of the hole during the flood recession. It was surmised from the amount of settlement and depths of the piles that the total scoured depth (relative to a peak water level 0.9 m above the tops of the pile caps) would have been about 9.6 m. This is comparable to estimates of the scour of 11–14 m obtained by adding the calculated local scour either to the calculated bend scour or to the calculated confluence scour (Coleman *et al*, 2000).

Emergency remedial works included cutting a new channel under the bridge a few piers away, to divert some of the flow away from the failed pier, and the installation of a Bailey bridge over the dropped pier. The planned remedial works comprised jacking the pier back into position, underpinning it and repairing the damaged spans. Consideration was also to be given to preventive action at other piers.

The failure was attributed to braid migration at the bridge, which resulted in a confluence of channels immediately upstream of the failed pier and a highly skewed angle of approach. The case demonstrates that "combinations of the full range of possible scour components need to be considered when assessing bridge scour" (Coleman *et al*, 2000) and that, as a result of the variability in braided channel positions, each pier of the bridge could be subjected to a similar amount of scour in the future.

A2.10 ORETI RIVER ROAD BRIDGE (NEW ZEALAND)

Figure A2.7 *Oreti River road bridge (photograph courtesy of Stephen Coleman)*

The two-lane bridge, built in 1955, carries State Highway 99 across the Oreti River and comprises 20 spans of 12 m, with eight spans located over the main channel. The RC piers are founded on driven RC piles, founded in gravels, with some sands, silts and clays, each pier being designed with two rows of six 7.6 m piles, although the central four piles for each pier were only to be driven "if required". The superstructure is of composite RC beam and slab construction.

Degradation of the riverbed by between about 1 m and 3 m occurred between 1939 and 1974, exacerbated by gravel extraction over a 7 km reach upstream. It is reported that, owing to the gravel extraction, the size of the largest bed material has reduced from about 150 mm initially to 50 mm at present.

After scour occurred in 1975, it was noted that the four central piles in each group of 12 were absent from five of the piers in the main channel. A survey of the riverbed showed that the scoured bed level was 1–5 m below the underside level of the pile caps.

Two remedial measures were implemented in 1977–78:

- a protective rock mattress was placed beneath the bridge, with a top elevation of 1.7 m below the underside of the pile caps, a minimum thickness of 1.3 m and up to 4 m thick in places, a crest width of 16 m, an upstream slope of 1:1, and downstream slope of 1:10 (V:H)
- a rock weir was built approximately 60 m downstream of the bridge.

These measures prevented significant additional scour at the bridge, in spite of further degradation of the riverbed upstream due to continued gravel mining. Subsequently, notable floods, approaching twice the mean annual flood magnitude, occurred in 1978, 1980, 1984, 1987 and 1994.

A2.11 SCHOHARIE CREEK ROAD BRIDGE (USA)

Figure A2.8 *Schoharie Creek road bridge after collapse of second pier (photograph courtesy of Ayres Associates)*

In April 1987, two spans of a New York State Thruway bridge fell about 25 m into the rain-swollen Schoharie Creek, after the pier that partially supported the spans collapsed, resulting in four cars and one truck plunging into the rain-swollen creek, with 10 fatalities. Ninety minutes later a second pier and a third span collapsed.

Schoharie Creek is subject to sudden severe floods and there had been many cases of dams and bridges destroyed since the 1820s. The 1987 flood in Schoharie Creek was studied using channel survey data, highwater marks, computer modelling and physical models. A 1:50-scale physical model was used to determine velocities and scour, and to allow comparisons between the 1987 flood and a flood of similar magnitude in 1955, soon after the bridge was constructed. The model demonstrated horseshoe-shaped local scour around the collapsed piers, similar in shape and depth to that which had been found after the failure. A 1:15-scale model was also used to assess the stability of riprap around the piers. The studies "concluded that the cumulative effects of the floods between 1955 and 1987 primarily influenced the depth of the local scour and stability of the riprap around pier 3" (Jackson *et al*, 1991).

The National Transportation Safety Board found that the probable cause of the collapse was the failure of the bridge owner to maintain adequate riprap protection around the bridge piers, leading to erosion beneath the spread footings. The accident was aggravated by a lack of structural redundancy, and contributory causes included ambiguous construction plans and specifications and an inadequate inspection and supervision regime.

It was found that the bridge inspections in 1979, 1982–83 and 1986 had not properly evaluated the condition of the riprap at the piers, although as early as 1977 it was known that some of the riprap had been displaced. The plans for a rehabilitation contract let in 1980 originally called for riprap, but this was deleted before inviting tenders, apparently for budgetary reasons. No diving inspection had been conducted at any time prior to the failure.

A2.12 HATCHIE RIVER ROAD BRIDGE (USA)

Figure A2.9 *The aftermath of the Hatchie River bridge failure (photograph courtesy of Ayres Associates)*

In April 1989 a 26 m section of the northbound US Route 51 bridge over the Hatchie River, near Covington, Tennessee, fell about 6 m into the rain-swollen river after two piled trestle-type piers, which supported three spans, collapsed. Firstly, one of the piers (No 70) and the two adjacent spans collapsed, causing four cars and one truck to plunge into the river, after which the adjacent pier (No 71) and span collapsed onto the vehicles. All eight vehicle occupants perished in the collapse.

The Hatchie River had been continuously flowing at above its mean daily discharge for over four months preceding the failure. Although the 1989 flood season "did not have consistently high peak flows", there were sustained out-of-bank flows in every month (Jackson *et al*, 1991).

A two-lane bridge on Route 51, spanning a total of about 1200 m over the main channel and floodplains, was opened to traffic in 1936. In 1974 a second two-lane bridge was built to convey southbound traffic. Its length was only about 300 m, which was centred approximately on the main channel downstream of the 1936 (northbound) bridge. The approach embankments to the southbound bridge blocked much of the floodplain flow and resulted in the flow through both bridges being concentrated into the 300 m width. In the years up to 1975 the main channel had migrated about 11 m northward, but after the construction of the new southbound bridge, the rate of northward migration increased.

The main channel spans of the northbound bridge were supported on piers with a pile cap supported on 6 m-long precast concrete piles. The base level of the pile caps supporting the floodplain spans was some 4.3 m higher and supported on 6 m-long untreated timber piles. The northward migration of the main channel exposed the piles of the floodplain pier (No 70) next to the channel and its collapse from local scour was the cause of the failure.

A physical model study of one of the collapsed piers indicated that 0.9–1.2 m of local scour could be expected. When added to the scour resulting from lateral migration, a total scour of 2.7–3.0 m was to be expected.

Recommendations for the installation of scour protection at pier No 70 were made in 1985, but no priority was assigned and no action taken. When inspected in 1987, the effect of the migration of the main channel on the same pier was highlighted. The inspectors did not have any design or as-built plans to hand and were mistaken in the thickness of the pile cap, as a result of which it was estimated that 0.3 m depth of pile was exposed, whereas they were actually exposed by 0.9 m at the time. A note was made of the need for repair, but without a specific maintenance recommendation.

The National Transportation Safety Board found that the probable cause of the failure was migration of the main river channel, which the bridge owner had failed to evaluate and correct, and that the accident was aggravated by a lack of structural redundancy. Pier No 70 probably failed from detachment of the supporting timber piles from the pile cap, perhaps in combination with pile buckling, after which pier No 71 collapsed as a result of successive vehicle impacts.

The NTSB report made several important recommendations about the monitoring, inspection and maintenance of highway bridges.

A2.13 KAOPING ROAD BRIDGE (TAIWAN)

On 27 August 2000 the Kaoping motorway bridge in Taiwan collapsed during high flows generated by Typhoon Bilis, injuring 22 people. The bridge carried National Highway 24 between the counties of Kaohsiung and Pintung and was used by more than 60 000 vehicles per day. About 100 m of the bridge collapsed following failure of one of the bridge piers.

Local opinion was that illegal sand and gravel extraction upstream had contributed to the collapse. It was reported that the bed had been lowered by up to 8 m over the preceeding 20 years. Local scour had been a problem for the bridge for many years and the bridge had been repaired at least 10 times since 1988, with the last work completed in January 2000. There was speculation that earlier scour protection work had contributed to the failure by diverting flow towards an unprotected pier foundation. Repairs were expected to take at least eight months to complete.