VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 21

BD 97/12

THE ASSESSMENT OF SCOUR AND OTHER HYDRAULIC ACTIONS AT HIGHWAY STRUCTURES

SUMMARY

This standard outlines requirements for the assessment of scour and other hydraulic actions at highway structures crossing or adjacent to waterways. It provides processes to determine the level of risk associated with scour effects. It also includes processes to assess the robustness of structures in a flood, and references to measures for reducing risk. It supersedes BA 74/06.

INSTRUCTIONS FOR USE

1. This document supersedes BA 74/06 which is now withdrawn.

2. Remove existing contents page for Volume 3 and insert new contents page for Volume 3 dated May 2012.

3. Remove BA 74/06 and archive as appropriate.

4. Insert BD 97/12 in to Volume 3, Section 4, Part 21.

5. Archive this sheet as appropriate.

Note: A quarterly index with a full set of Volume Contents Pages is available separately from The Stationery Office Ltd.

THE HIGHWAYS AGENCY

TRANSPORT SCOTLAND

WELSH GOVERNMENT
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THE DEPARTMENT FOR REGIONAL DEVELOPMENT
NORTHERN IRELAND

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Contents

Chapter

1. INTRODUCTION

Background

1.1 Scour of foundations by the action of water is a major cause of bridge collapse, and it is important to manage the risks in an effective and consistent manner. A key part of that process is the assessment of structures to identify those that are most at risk and require further action.

1.2 This Standard replaces the Advice Note BA 74/06 “Assessment of Scour at Highway Bridges”.

Scope

1.3 This Standard is applicable to highway structures crossing or adjacent to watercourses and covers the assessment of highway structures for the effects of scour and other hydraulic actions. It provides methods for identifying structures most at risk of collapse due to a flood event. It also provides guidance on measures that can be used following the assessment to manage the risks of scour.

1.4 This standard has been prepared by the Overseeing Organisations specifically for use on roads for which they are responsible and may also be applicable by other highway authorities to other roads in accordance with GD 01 (DMRB 0.1.2).

Purpose

1.5 The purpose of this Standard is to provide a method to assess the risks associated with scour and other effects on highway structures during floods, and to provide some guidance on measures that can be used to eliminate or manage those risks. This Standard includes simple methods for identifying structures that have a low risk of being affected by scour, with the aim of minimising the cost associated with assessments.

1.6 This Standard has been written to be consistent with the principles and requirements of BD 79 “The Management of Substandard Structures” (DMRB 3.4.18).

1.7 More detailed guidance on hydraulic effects and scour at bridges may be found in “CIRIA C551, Manual on scour at bridges and other hydraulic structures” (May *et al.*, 2002).

Mandatory Sections

1.8 Sections of this document containing mandatory requirements are identified by being contained in boxes. These requirements must be complied with or a prior agreement to a Departure from Standard must be obtained from the Overseeing Organisation. The text outside boxes contains advice and explanation, which is commended to users for consideration.

Implementation (England, Scotland and Wales)

1.9 This **document** must be used forthwith on all projects for the assessment, design, construction, operation and maintenance of motorway and all-purpose trunk roads in **England, Scotland and Wales** except where procurement of works has reached a stage at which, in the opinion of the Overseeing Organisation, its use would result in significant additional expense or delay progress (in which case the decision must be recorded in accordance with the procedure required by the Overseeing Organisation).

Implementation (Northern Ireland)

1.10 This **document** must be used forthwith on all projects for the assessment, design, construction, operation and maintenance of **all roads in Northern Ireland** except where procurement of works has reached a stage at which, in the opinion of the Overseeing Organisation, its use would result in significant additional expense or delay progress (in which case the decision must be recorded in accordance with the procedure required by the Overseeing Organisation).

1.11 The list of abbreviations and definitions used throughout this standard can be found at Annex C.

2. ASSESSMENT PROCESSES

General

2.1 All structures crossing a waterway must be periodically assessed to determine whether measures are required to reduce the risk from scour and other hydraulic effects in accordance with the following:

(i) Structures that have not previously been assessed in accordance with this standard or BA 74 must be assessed within a period to be advised by the Overseeing Organisation.

(ii) Periodic reassessment must be carried out as specified by the Overseeing Organisation.

(iii) Where inspections carried out in accordance with BD 63 (DMRB 3.1.4) (including inspections after flood events) indicate significant changes to the characteristics of the watercourse or the integrity of the foundations, the structure must be assessed in accordance with this standard as soon as possible.

Scour Assessment

2.2 Assessment of a structure should be carried out in levels of increasing complexity, with the object of efficiently determining its adequacy. Level 1 Assessment comprises simple methods, including the use of engineering judgement, to identify structures that are not at risk from scour or where the risk is tolerably low. Provided a structure is shown to be adequate at Level 1 then the assessment is complete.

2.3 Where a Level 1 assessment does not show that a structure is adequate, then the assessment should progress to Level 2.

2.4 The Technical Approval Authority should be advised in advance of proposals to carry out level 2 assessments. Technical Approval in accordance with BD 2 (DMRB 1.1.1) is not required for assessment of scour depth and scour risk rating.

2.5 The scour assessment process is illustrated in the flowchart in Figure 2.1.

Assessment of Vulnerability to Other Hydraulic Effects

2.6 Guidance on the assessment of vulnerability to other hydraulic effects is given in Chapter 6.

2.7 Assessments that relate to structural integrity (the ability of the structure to resist hydraulic actions and debris impact actions) require Technical Approval in accordance with BD 2.

Immediate Risk Structures

2.8 The Overseeing Organisation must be informed immediately if, at any stage during inspection, assessment or whilst monitoring, a structure is considered to be at immediate risk of significant structural distress or collapse. Once confirmed and agreed with the Overseeing Organisation, immediate risk structures must be managed in accordance with BD 79.

Management of Scour Susceptible Structures

2.9 Following the assessment, the need for further measures to control the risk of scour or other flood-related instability should be assessed and recorded. The method for determining Scour Risk Rating is in Chapter 5. Measures for managing structures and reducing risk are contained in Chapter 7.

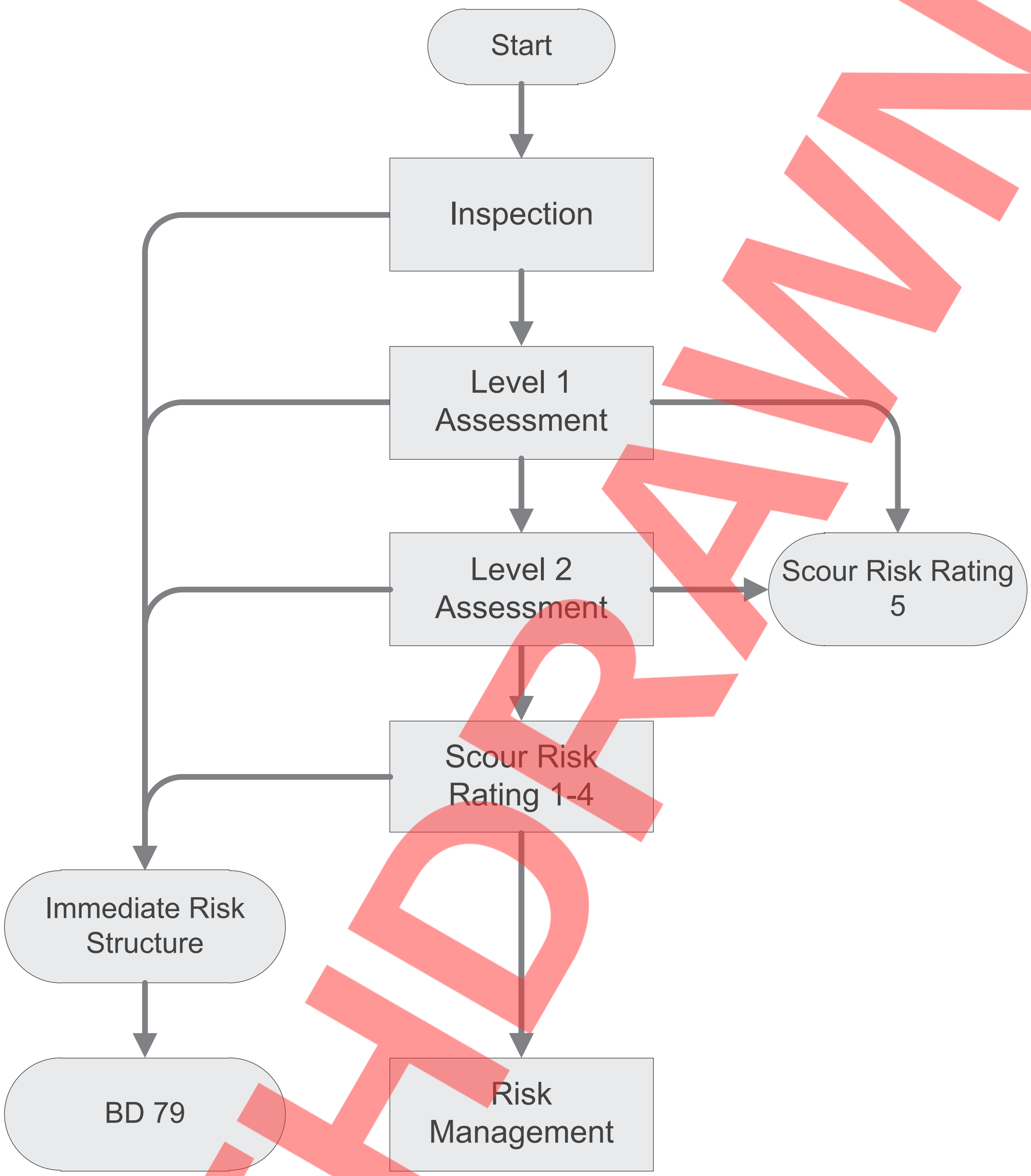


Figure 2.1 – Scour Assessment Process

Document Management

2.10 For each structure crossing a waterway an auditable record must be maintained to enable the assessment and management of the structure to be clearly documented. This record must include details of the decisions taken at each stage of the assessment process, evidence of the approval and implementation of any measures or actions, and documentation of the regular review of the structure. Records should follow the requirements of BD 62 (DMRB 3.2.1).

3. INSPECTION AND LEVEL 1 SCOUR ASSESSMENT

General

3.1 The first part of the assessment process comprises an inspection and a Level 1 Assessment. The Level 1 Assessment is a coarse screening method to identify those structures for which the risk of scour damage is tolerably low. It need not involve calculations or numerical analysis, and it may be based to some extent on the judgment of the Assessment Team, considering the information gathered through inspection and from existing records. The outcome of the assessment should be a recommendation either that the assessment should proceed to Level 2 or that the structure should be designated as Scour Risk Rating 5.

Inspection and Data Gathering

3.2 When carrying out an inspection the requirements and advice given in BD 63 with respect to health and safety must be complied with.

3.3 The purpose of the inspection is to gather information to be used in the scour assessment, including information regarding the structure and its foundations, the waterway and its bed, any protection measures or flood relief measures, and evidence of changes or erosion. Existing records should also be sought and compiled for consideration in the assessment. Note should be made of any changes to the watercourse near the structure location (upstream and downstream). The extents of the distance to be considered should be determined and agreed by the teams carrying out the inspection and the level 1 assessment, and should be no less than 30 times the average width of the channel in each direction.

3.4 Annex A contains a recommended form for recording inspection data that should be collected before proceeding with the Level 1 Assessment.

3.5 A full survey of the river bed is not required unless the nature of the river and river bed mean that the scour behaviour cannot be predicted using the simple inspection method, for example, if the river is made up of many channels or the river channel is known to move within the confines of its bank. Levels should be taken at both banks and within the channel to allow the average depth of the channel to be estimated, and measurements of the channel width should be taken to characterise the bank to bank dimension and the average width dimension between points of half average depth, as illustrated in Figure 3.1. These measurements should be obtained at sufficient locations to characterise the channel geometry for the conditions at the bridge and upstream of the structure. It may also be appropriate to consider the downstream geometry where this could have a significant impact on the assessment of flow conditions.

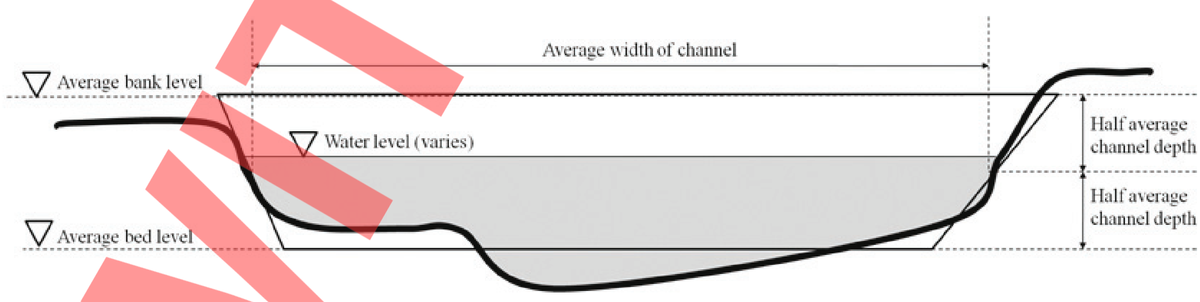


Figure 3.1 – Characterisation of Channel Geometry from Inspection Data

3.6 The depth of the foundation is a key parameter for determining its resistance to scour and the risk of scour-related collapse. In all cases information on the type and depth of foundations should be sought. However if information is not immediately available, then the Scour Risk Rating may be estimated using the calculated scour depth and the default ranges of foundation depths D_{F1} and D_{F2} in Table 3.1. This will give an estimated Scour Risk Rating which should be verified by further investigation of the foundation depth.

Values for foundation depth	Concrete substructure	Masonry substructure
D_{F1}	1m	0.3m
D_{F2}	3m	1m

Table 3.1 – Default Ranges of Foundation Depths

3.7 Photographs should be taken for reference in the subsequent assessment and to provide a comparative reference for future inspections. At least four photographs should be taken of every bridge site: two views of the bridge (one from upstream and one from downstream), and two views of the river channel (from the bridge looking upstream and downstream). In addition, photographs of particular relevant features should be taken. These might include, but need not be limited to, upstream bends, particularly if there are signs of erosion to the outer banks, signs of bank erosion or channel instability, upstream structures, such as other bridges that might affect the flow at the site or control the potential lateral movement of the river channel, and downstream structures such as weirs that control the river bed at the bridge site.

Level 1 Assessment

3.8 The Level 1 Assessment considers the risk of scour damage, the stability of the waterway and the robustness of the structure under flood conditions.

3.9 Aspects to be considered at Level 1 include the following:

(i) the possibility of scour developing in the bed of the river and undermining the structure foundations;

(ii) the possibility of erosion of the channel banks and movement of the channel causing greater risk of scour damage or damage to the approach embankments of the road;

(iii) the potential for debris blocking the flow;

(iv) the potential instability of the superstructure during a flood or once scouring has occurred

3.10 Figure 3.2 illustrates the basic considerations and decisions to be made by the Assessment Team at Level 1. Guidance is provided in 3.11 – 3.20.

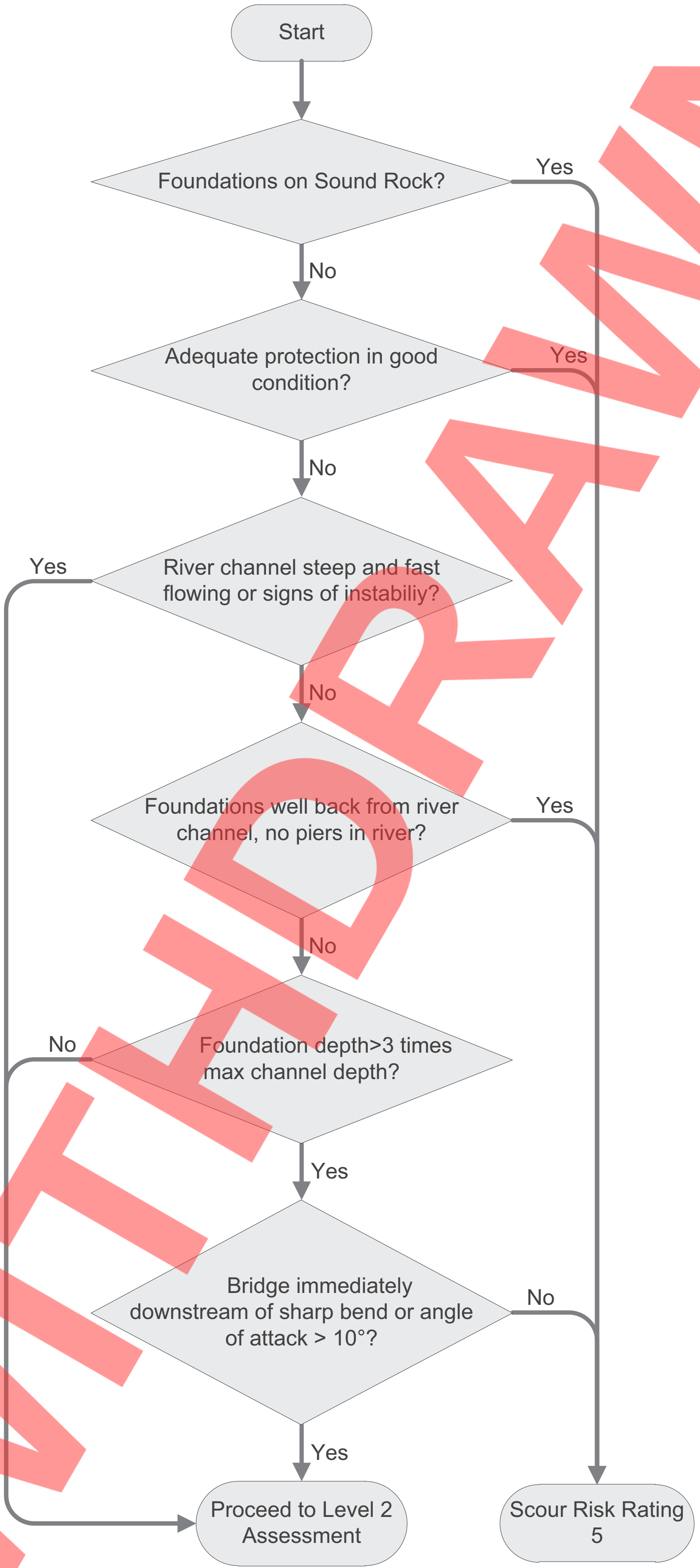


Figure 3.2 – Level 1 Assessment Decisions

3.11 If effective protection, such as described in 7.18 and 7.19, has been provided and the protection is in good condition, the risk from scour is likely to be low. The design calculations for the protection measures should be reviewed, if they are available.

3.12 When checking the depth of foundations relative to the maximum channel depth, the calculation should be based on the depth from the average bed level to the underside of a spread footing or pilecap. Although there is less risk of scour causing damage to a piled foundation, it is undesirable for the piles themselves to be exposed, as this may affect their bearing capacity or the lateral stability of the bridge.

3.13 When considering whether abutments are set well back from the river channel, the possibility of overbank flood conditions should be taken into account. Local scour can occur at abutments under these conditions, particularly if the bridge alignment is oblique to the main flow direction.

3.14 Examples of structures for which a Level 2 assessment will be necessary include the following:

(i) structures currently experiencing scour or those which have a history of scour problems as identified, for example, from inspection and maintenance records, but which do not have adequate scour protection;

(ii) structures that do not have adequate scour protection and which have design features that make them more likely to be vulnerable to scour, such as:

- piers and abutments founded on shallow spread footings in the river channel;
- bridges on unstable river channels;
- bridges on fast flowing steep channels;
- bridges on or immediately downstream of bends in the river;
- piers subject to an oblique angle of attack from the flow (note: this can be a particular problem if there is an upstream obstruction as, for example, another bridge with piers aligned at a different angle to the bridge under consideration causing the flow to be directed obliquely towards the bridge being assessed);
- abutments that protrude into the river channel;
- open spans of such lengths that the abutments or piers cause significant contraction of the river channel;
- relatively small bridge openings or bridges with debris screens or obstructions that could easily be blocked by floating debris.

3.15 Consideration should be given to the potential for changes to the plan geometry of the river channel. A channel may be considered as having only a small risk of lateral movement if any of the following apply:

(i) River confined within a valley with little or no floodplain.

(ii) No history of movement.

(iii) Adequate bank protection or training works are provided and there are no signs of deterioration.

3.16 The prediction of future movement of the river channel can involve a degree of uncertainty. If there are clear signs that the channel is unstable then the analysis must proceed to Level 2. Lateral erosion and movement of the watercourse tends to occur as a series of steps associated with the flood events and even a 20-year period of relative stability does not necessarily indicate that large movements would not occur in a major flood. However, if a river has shown no signs of lateral instability and has clearly been confined to its present channel for a considerable time then it may be reasonable to assume that the channel will remain stable. This assumption should be recorded and reviewed in future inspections.

3.17 The lack of potential for significant lateral movement does not, however, indicate a lack of potential for local bank erosion. Narrow valleys with a confined channel are often associated with steep fast flowing rivers and streams. Local bank erosion can be a major problem in such cases and only if the bridge abutments are founded on rock or are well clear of the watercourse under flood conditions may it be possible to conclude that the risk of such erosion leading to damage is small.

3.18 Conclusions on lateral stability should be made only after a site visit and visual observation.

(i) Indications of long-term stability can include:

- a straight reach upstream of the bridge;
- mature tree growth along the banks.

(ii) Indications of potential instability can include:

- a bend immediately upstream of the bridge (erosion is most severe on the outside bank of a bend and the bend will tend to become more pronounced over time);
- evidence of bank erosion;
- evidence of bank protection or training works upstream of the bridge, that may have been necessitated by erosion in the past, and in particular if those works show signs of distress themselves.

3.19 The presence of bank protection works does not necessarily indicate erosion problems. Works may have been provided to improve navigation or access along the bank, to protect adjacent property or to act as wharfing. Such protection can help to stabilise the channel approaching the bridge and needs to be taken into account when assessing the potential for scour at the bridge site.

3.20 Historical data should be treated with caution. Memory of flood events and associated problems tends to be limited to about 15 to 20 years, and such a period will probably not have included the more extreme events which the bridge must be capable of withstanding without damage.

Reporting

3.21 Reports should be produced containing the findings of the inspection and Level 1 assessment. The reports may include data recorded in the forms provided in Annex A.

4. LEVEL 2 SCOUR ASSESSMENT

General

4.1 The primary purpose of the Level 2 Assessment is to calculate the estimated scour depth corresponding to the Assessment Flow, and to compare this with the foundation level. The scour risk rating can then be determined in accordance with Chapter 5.

4.2 The Level 2 Assessment should follow the process outlined in Figure 4.1.

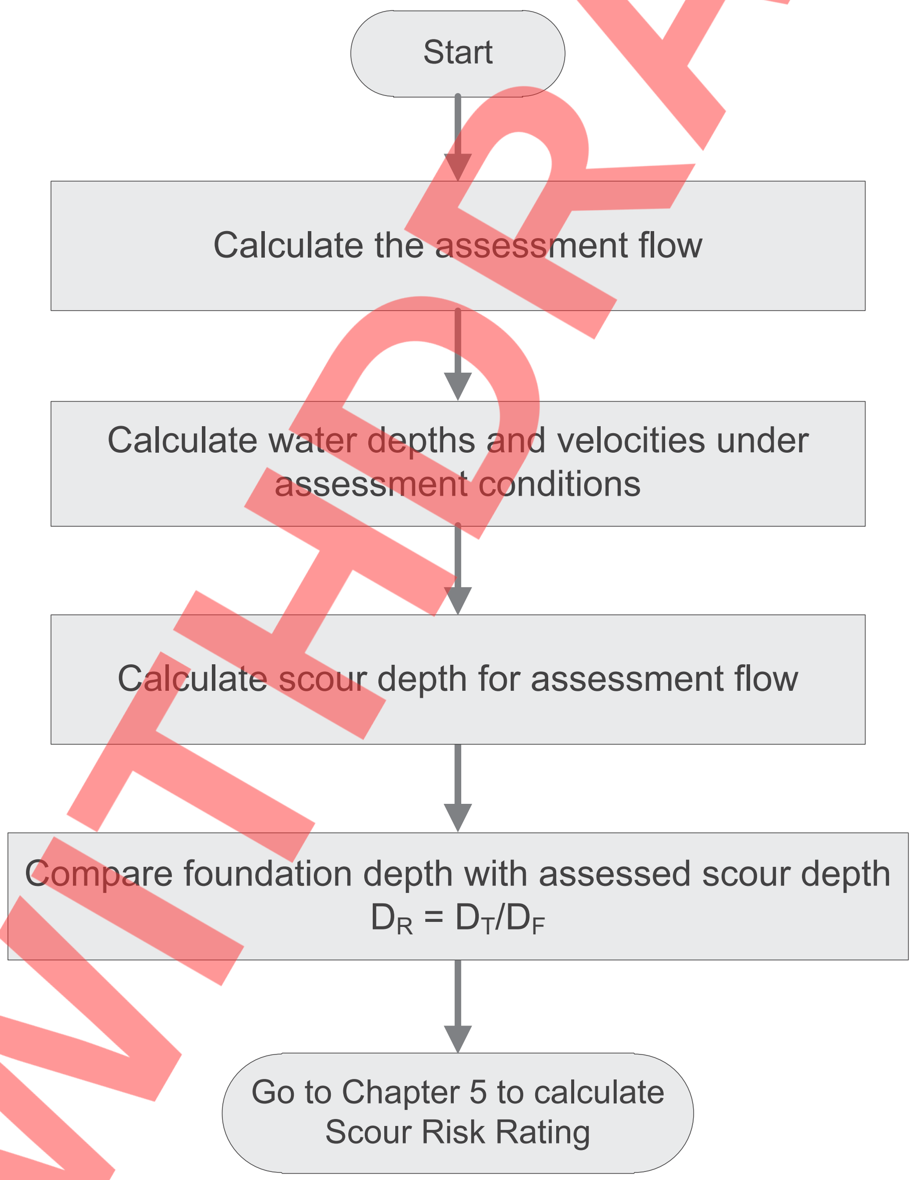


Figure 4.1 – Outline of Level 2 Assessment Process

Assessment Flow

> 4.3 The Assessment Flow must be determined for the bridge site based on a statistical analysis of the available flood data, combined with tidal data where appropriate.
>
> 4.4 The fluvial component of the Assessment Flow Q_F must be no less than the flow corresponding to a return period of 200 years (equivalent to a probability of 0.005 that this flow will be exceeded in any one year).

4.5 Data for estimating Q_F should be obtained from The Flood Estimation Handbook and associated software. Q_F may be derived from the index flood, also known as QMED, scaled by a dimensionless growth curve.

4.6 The index flood for a gauged catchment is the median of the annual maximum flood. Where sites are not gauged, the index flood may be estimated using the method in the Flood Estimation Handbook based on catchment descriptors and adjusted using one or more donor sites, which are gauged stations located in the vicinity of the site. Potential donor sites should be selected based on geographical closeness rather than 'hydrological similarity' as defined by catchment descriptors. Further guidance on the calculation procedure for estimating and adjusting QMED is provided in the Flood Estimation Handbook.

4.7 The index flood should be scaled by a growth curve derived from data extracted from a pooling group of gauged UK catchments. The method is given in the Flood Estimation Handbook. Sites for the pooling groups should be selected based on a measure of hydrological similarity derived from catchment area, annual average rainfall, soil type, flood attenuation, and urbanisation. The number of sites within the pooling group should ideally be sufficient to provide a total of 1,000 years of data (or five times the return period), but no less than 500 years. The recommended form of the growth curve to be used in the UK is the Generalised Logistic distribution, as described in the Flood Estimation Handbook.

4.8 An adjustment to Q_F to account for the effects of climate change is required. For design it is common to allow for an additional 20% on the design flood to account for potential climate change over the life of the structure (Planning Policy Statement 25). This addition should also be applied for assessment.

4.9 In certain special cases the Assessment Flow may be controlled by adjacent structures, such as dam outflows. In these situations the method for determining the Assessment Flow should be agreed with the Overseeing Organisation.

> 4.10 For waterways that are affected by tidal fluctuations, an allowance for tidal flows must be included in the calculation of the Assessment Flow.

4.11 Tidal periods and ranges at standard ports are published in the Admiralty Tide Tables. Values for other locations may be interpolated as appropriate. Within estuaries funnelling and surge effects can affect the tidal movements. Where there is any doubt about the suitability of the simplified method in 4.12 – 4.15, specialist advice should be obtained.

4.12 A simplified method for estimating the peak tidal flow is as follows:

$$Q_{Tide} = \frac{\pi V_{Tide}}{T_{Tide}}$$

Where

Q_{Tide} is the peak tidal flow,

V_{Tide} is the volume of the tidal prism, calculated from the area upstream of the bridge that is subject to inundation at high tide multiplied by the tidal range,

T_{Tide} is the tidal period.

4.13 Q_{Tide} should be calculated for the average tidal range, which is the average of the mean spring and neap tidal ranges. If the flow calculated in this way is less than 0.5 Q_F, then the Assessment Flow should be taken as:

$$Q_A = Q_F + Q_{Tide}$$

However, if the tidal flow based on the average tidal range is greater than 0.5 Q_F, then the maximum tidal flow $Q_{Tide,\max}$, should also be considered, based on the range between the Highest and Lowest Astronomical Tides, and the Assessment Flow should be taken as the greater of

$$Q_A = Q_F + Q_{Tide}$$

and

$$Q_A = \frac{Q_F}{3} + Q_{Tide,\max}$$

4.14 In an estuary the tidal rise during the incoming tide can be much more rapid than the ebb tide. If the seaward face of the piers or abutments appears to be vulnerable to scour during reverse flows on the flood cycle of the tide, this must be assessed.

4.15 Reverse flows may be allowed for by taking the Assessment Flow as no less than

$$\frac{4.5}{\pi} Q_{Tide,\max}$$

Calculation of Assessment Flow Depth and Velocity Upstream of Bridge Site

4.16 The depth and velocity upstream of the bridge site should generally be calculated from an hydraulic analysis based on the assessment flow and the characteristics of the waterway and the floodplains. However, where the flow through the bridge is governed by minimum energy considerations the depth and velocity at the bridge may be directly determined from 4.32 and 4.33.

4.17 In the absence of more detailed methods which would need to be agreed with the Technical Approval Authority, the simplified methods for calculating the depth and velocity upstream of the bridge site given in 4.18 – 4.23 may be used.

4.18 For waterways with flood embankments on both sides of the river, unless it can be shown that these would not be overtopped by the assessment flow, the maximum flood level upstream of the bridge y_u should be taken as 300mm above the top of the flood embankments and the velocity upstream of the bridge should be taken as

$$v_u = \frac{Q_A}{By_u}$$

where B is the average width of the channel.

4.19 Where flood embankments are not present, the depth and velocity upstream of the bridge should be calculated assuming uniform flow conditions, where the energy lost to drag at the boundary layer is equal to the change in potential energy, and the depth is uniform with a value of y_n as defined in 4.21. This simplified approach neglects the afflux effect caused by the bridge itself and will generally provide a conservative estimate of the velocity for the calculation of scour.

4.20 Where the normal depth for the assessment flow exceeds the bank levels then an adjustment should be made to account for the effect of floodplains, as described in 4.25.

4.21 The normal depth and velocity upstream of the bridge based on uniform flow conditions may be calculated based on a simplification of Manning's equation for wide channels:

$$y_n = \left(\frac{Q_A n}{Bs^{1/2}}\right)^{3/5}$$

$$v_n = \frac{Q_A}{By_n}$$

where y_n is the normal depth (in m), Q_A is the assessment flow (in m^3/s), B is the average width of the channel (in m), s is the slope of the channel as calculated in 4.23, n is Manning's coefficient as defined in 4.24, and v_n is the normal velocity corresponding to y_n.

4.22 If $y_n > \frac{B}{10}$ using the approach in 4.21 then the calculation may be rather conservative. In this case the following approach may be used:

(i) The value of A_n should be found such that Manning's equation is satisfied:

$$A_n = \left(\frac{Q_A P^{2/3} n}{s^{1/2}}\right)^{3/5}$$

where A_n is the area of the flow corresponding to the normal depth (in m^2), Q_A is the assessment flow (in m^3/s) P is the length of the wetted perimeter of the channel (in m), s is the slope of the channel as calculated in 4.23, and n is Manning's coefficient as defined in 4.24.

(ii) The normal depth of flow y_n should be determined based on the value of A_n and the approximate shape of the cross section, as illustrated in Figure 4.2. If the flow is not sufficient to overtop the banks then P and A_n will both depend on the depth and an iterative approach may be required to solve the equation. Where the depth exceeds the bank level then the cross section boundary should be assumed to extend vertically above the banks for the calculation of y_n.

(iii) The normal velocity corresponding to y_n should be determined as:

$$v_n = \frac{Q_A}{A_n}$$

4.23 Unless more detailed survey data are available, the longitudinal slope *s* should be estimated based on the height of contours on 1:25,000 or 1:10,000 OS maps and the total length along the channel between the contour positions. At least two contours should be used on either side of the bridge. Unless there is a marked change of slope close to and upstream of the bridge, an average slope should be calculated. If there is such a change of slope then the relevant gradient should be taken as that of the river reach immediately upstream of the bridge.

4.24 Values of Manning's coefficient *n* should be assumed to be as follows:

(i) n = 0. 035 for a reasonably straight channel, clear of obstructions and with only light bank vegetation;

(ii) n = 0. 060 for a channel with irregular banks or heavy brush and trees on the banks;

(iii) intermediate values may be adopted for channels between these two extremes. Further guidance is given in Open Channel Hydraulics, Chow and McGraw (1959).

4.25 If the level of the normal depth for the assessment flow as calculated in 4.21 does not exceed the level of the banks, then the upstream depth and velocity should be taken as the normal values:

$y_u = y_n$

and

$v_u = v_n$

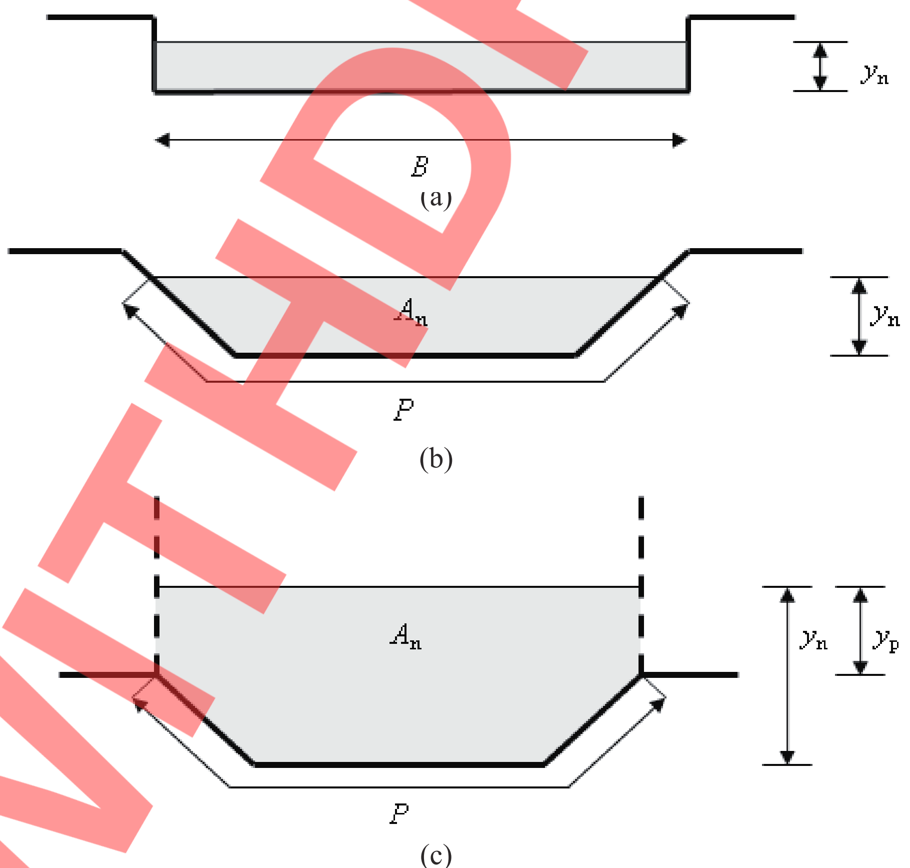


Figure 4.2 – Definition of Parameters for Determining the Normal Depth
(a) simplified approach for wide channels based on average channel width B
(b) general approach
(c) approach for overbank flow conditions

4.26 However if the level of the normal depth for the assessment flow as calculated in 4.21 exceeds the level of the banks, and flood embankments are not present, then an adjustment is necessary to account for the flow over the floodplains. The following simplified method may be used. The upstream depth and velocity should be taken as:

$$y_u = y_n - \alpha_y y_p$$

and

$$v_u = \alpha_v v_n$$

where y_n is the normal depth in the channel alone assuming vertical boundaries to the cross section as illustrated in Figure 4.2, y_p is the difference between the level of the normal depth in the channel and the floodplain level, as illustrated in Figure 4.2, and α_y and α_v are coefficients given in Table 4.1 that depend on the floodplain factor F_p as defined in 4.27.

Floodplain factor	α_y	α_v
$F_p \leq 1$	0.25	1.00
$1 < F_p \leq 4$	0.50	0.95
$F_p > 4$	0.70	0.90

Table 4.1 – Values of Coefficients to Account for Floodplain Flow

4.27 For overbank flow, the floodplain factor F_p should be determined based on the characteristics of the floodplain immediately upstream of the bridge site.

(i) The floodplain should be divided into three sectors on each bank, with each sector having a length parallel to the river of about ten channel widths, and the width perpendicular to the river should be an estimate of the anticipated extent of flooding. In addition to these six sectors, the river banks adjacent to the bridge on its upstream side should also be assessed in order to characterise the difficulty that any floodplain flow would have when re-entering the channel upstream of the bridge. Idealised floodplain sectors are illustrated in Figure 4.3. Each sector should be considered in turn and classified according to the obstruction it presents to any potential flow across it. Factors needing to be taken into account include the density and height of vegetation, the presence of any raised ground and the existence of any man-made obstructions. Each sector should be classified as very obstructed, obstructed, or open. Seasonal variations of vegetation should be considered and the most severe obstruction to flow taken into account. The classification of typical floodplain situations is illustrated pictorially in Figure 4.4.

(ii) The left and right bank floodplains should be rated according to the worst classification assigned to any of their three sectors. For example, if any of the left bank sectors are classified as very obstructed, then the left floodplain will be classified as very obstructed; a floodplain will only be classified as open if all three sectors have been classified as open. Once a classification has been obtained for each floodplain this should be compared with the classification of the river bank adjacent to the bridge to determine the overall effective width of each floodplain in accordance with Table 4.2.

(iii) The floodplain factor should be determined as:

$$F_p = \frac{W_{eff,L} + W_{eff,R}}{B}$$

where $W_{eff,L}$ and $W_{eff,R}$ are the effective widths of the left and right floodplains obtained from Table 4.2 and B is the average channel width upstream of the bridge.

River bank classification:	**Open**	**Partially obstructed**	**Very obstructed**
Floodplain classification:	Effective width of each floodplain (W= full width of the respective floodplain)		
Open	$W_{eff} = W$	$W_{eff} = 0.5W$	$W_{eff} = 0.5W$
Partially obstructed	$W_{eff} = 0.5W$	$W_{eff} = 0.5W$	$W_{eff} = 0$
Very obstructed	$W_{eff} = 0$	$W_{eff} = 0$	$W_{eff} = 0$

Table 4.2 – Effective Width of Floodplains

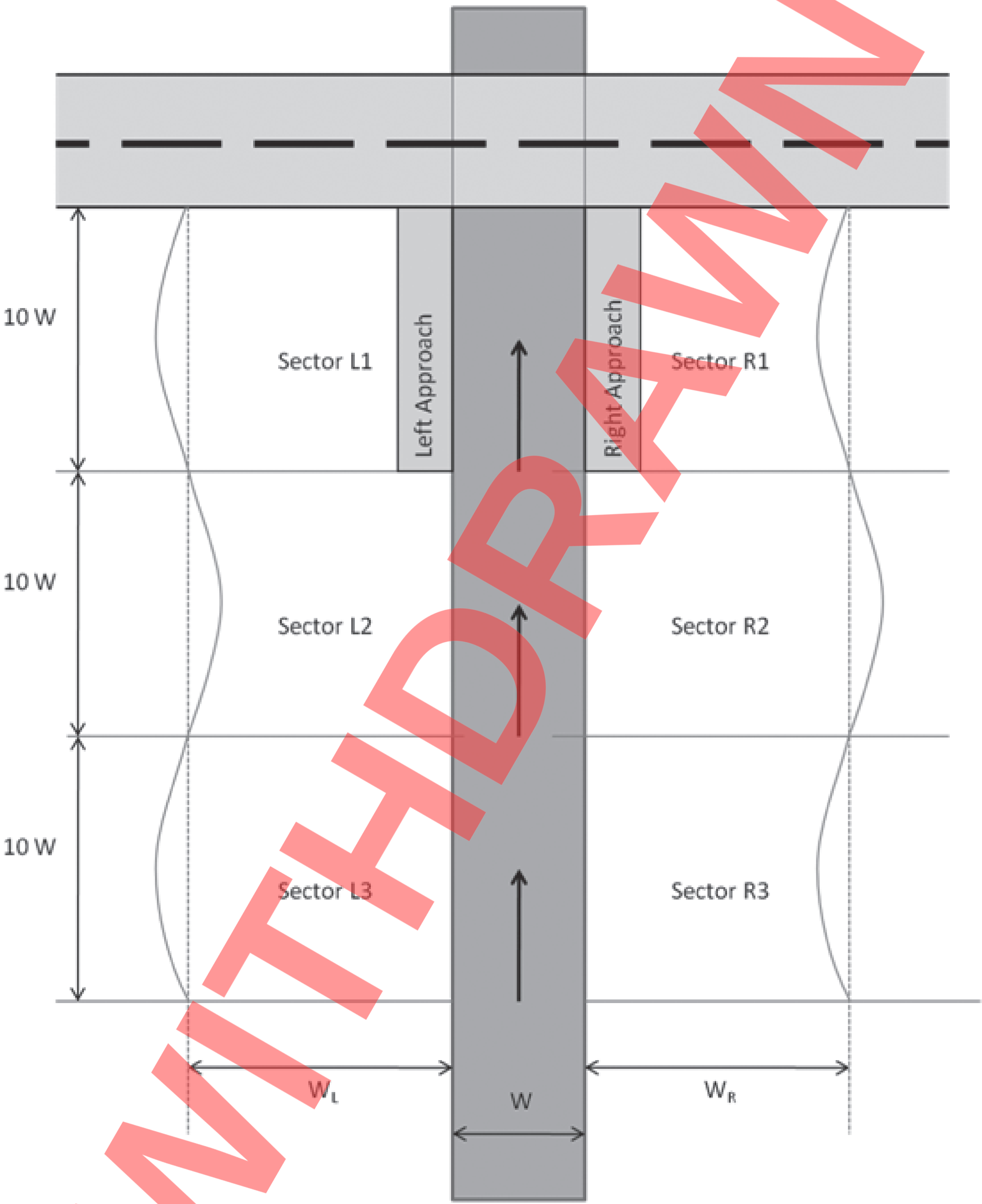


Figure 4.3 – Illustration of Idealised Floodplains

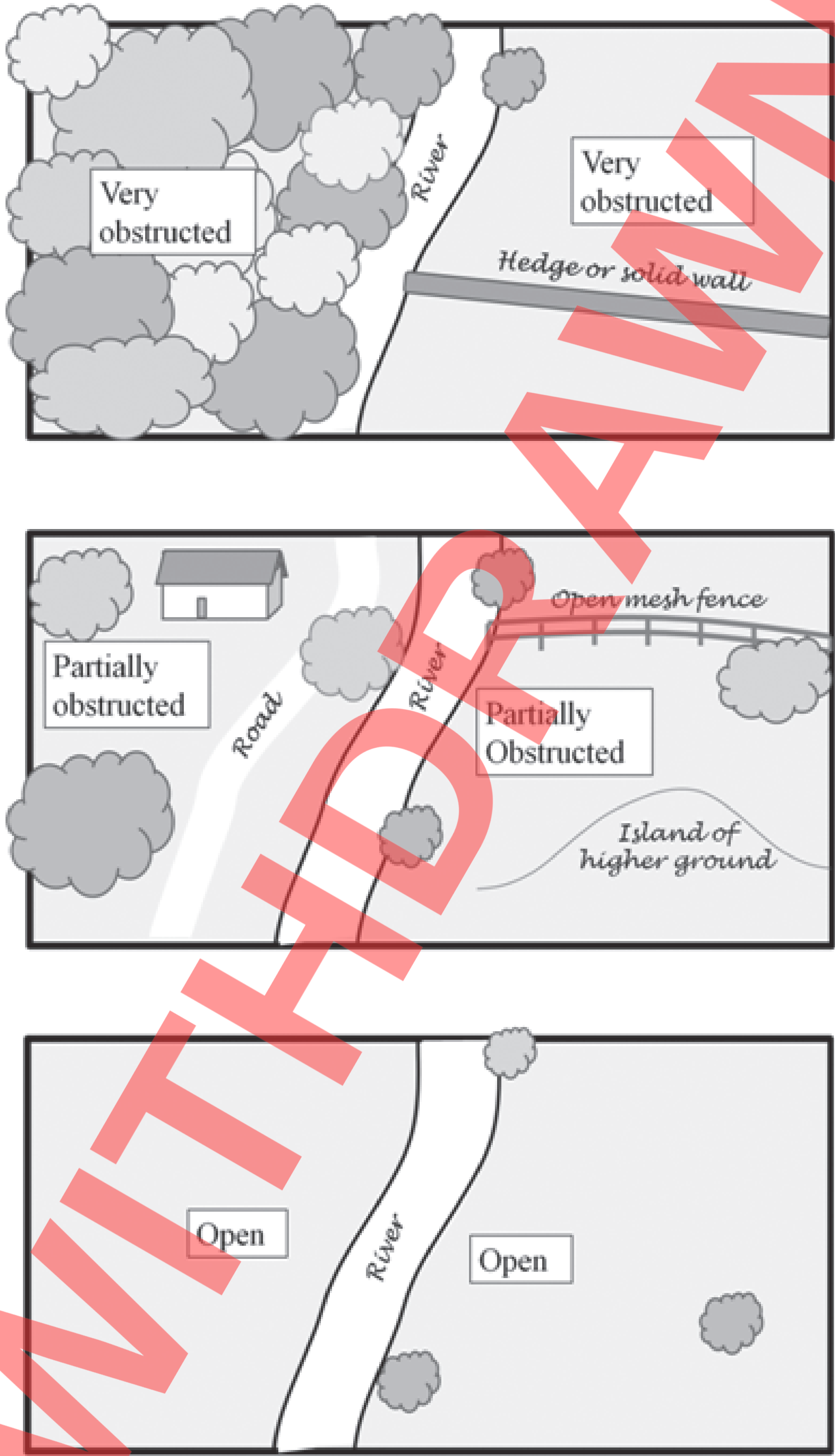


Figure 4.4 – Classification of Floodplains

Calculation of Depth and Velocity of Assessment Flow Through Bridge

4.28 The depth and velocity at the bridge site must be determined based on an analysis of the flow as it transitions from its upstream characteristics through the bridge opening. This analysis will depend on the energy available in the upstream flow and the characteristics of the bridge opening.

4.29 For the calculation of the velocity through the bridge opening it should be conservatively assumed that the total assessment flow will pass through the bridge opening and to neglect any flow over or around the structure.

4.30 Bernoulli's equation provides a relationship between the upstream flow and the water depth at the bridge site, assuming that there is no loss in energy, as illustrated in Figure 4.5:

$y_U + v_U^2/2g - y_B - (Q_A/y_B.B_B)^2/2g = 0$ for rectangular openings

$y_U + v_U^2/2g - y_B - (Q_A/A_{By})^2/2g = 0$ for non-rectangular openings

where:
g is the acceleration due to gravity
y_B is the depth of flow through the bridge
B_B is the average width of the bridge waterway.
A_{By} is the area of flow through the bridge opening, which is a function of y_B

4.31 There are 3 solutions for y_B, only two of which are positive. Of the two positive solutions the smaller value $y_{B,sup}$ represents supercritical flow (where the velocity exceeds the wave speed) and the larger value $y_{B,sub}$ represents subcritical flow (where the velocity is slower than the wave speed). Either of these conditions is possible, depending on the slope and the characteristics of the bridge opening.

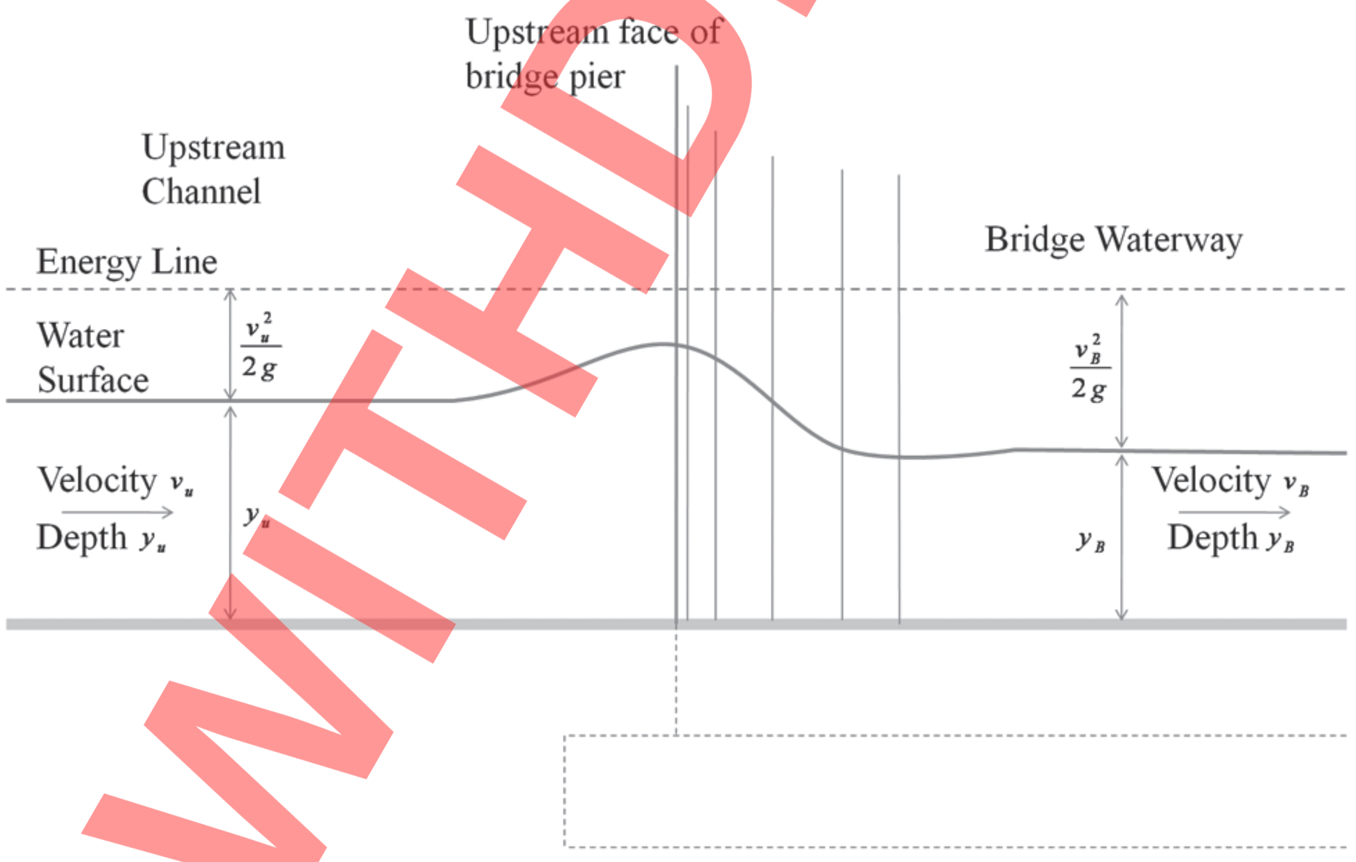


Figure 4.5 – Illustration of Bernoulli's Equation, Based on Conservation of Energy

4.32 The possibility of supercritical flow through the bridge should be considered. Supercritical flow is most likely over steep gradients.

(i) The critical slope should be calculated, using the following equation:

$$s_c = n^2\left(\frac{B_B{}^2 g^{10}}{Q_A{}^2}\right)^{\frac{1}{9}}$$

where s_c is the critical slope, n is Manning's coefficient, B_B is the width of the bridge waterway (in m), g is the acceleration due to gravity (in m/s^2) and Q_A is the assessment flow (in m^3/s).This value should be compared with the estimated slope value s as calculated in 4.23.

(ii) If $s > 0.9\ s_c$, or if it is considered likely that the local slope at the bridge would exceed s_c, or if there is any other particular reason that the flow could be supercritical, then the supercritical solution should be selected with $y_B = y_{B,sup}$ based on the smaller of the two positive solutions of Bernoulli's equation as in 4.31.

(iii) Otherwise, it is often reasonable to assume that the flow will be subcritical. However it is likely that there will be energy loss at the bridge constriction, which for subcritical flow would result in a reduced depth and a faster velocity. For the calculation of velocity at the bridge, the depth y_B should therefore be taken as no greater than the critical depth for minimum energy:

$$y_B \le y_C = \left(\frac{Q_A{}^2}{B_B{}^2 g}\right)^{\frac{1}{3}}$$

4.33 The area of flow through the bridge opening A_{By} should be calculated based on the depth y_B. This should be taken as no greater than the total area of the bridge opening. The mean velocity at the bridge should be calculated as:

$$v_B = \left(\frac{Q_A}{A_{By}}\right)$$

Maximum Depth of Water at the Upstream Face of the Structure

4.34 The maximum depth of water just upstream of the bridge y_{uf} should be compared with the height of the structure soffit above bed level, z. The following methods may be considered:

(i) The upstream depth y_u will provide a lower bound to the depth just upstream of the structure y_{uf}. If y_u calculated using the method in 4.25 exceeds soffit level, i.e. $y_u > z$, then it may be assumed that the water level just upstream of the structure will exceed the soffit level.

(ii) It may be assumed that the water level does not reach the level of the soffit if the specific head of the flow through the bridge does not exceed the soffit level, i.e.:

$$y_b + v_b{}^2/2g < z$$

(iii) If the conditions in (i) and (ii) are not satisfied then a calculation of the water height based on pressure flow conditions should be carried out. The depth y_{uf} may be determined iteratively using the following equation based on a sluice gate approach, which initially assumes that the water does exceed soffit level:

$$2g\left(y_{uf} - \frac{z}{2}\right) + \left(\frac{Q_A}{A_{uf}}\right)^2 - \left(\frac{2Q_A}{A_B}\right)^2 = 0$$

where A_{uf} is the area of flow just upstream from the structure, A_B is the area of the bridge opening and z is the height from the bed level to the soffit. If there are no positive solutions for y_{uf} greater than z then it may be assumed that the water will not reach the soffit.

4.35 For structures where the water level just upstream from the structure is expected to exceed the soffit level, the stability and robustness of the bridge should be considered as in Chapter 6. Debris impact should also be considered in cases where:

$$y_b + v_b{}^2/2g \ge z - 0.6$$

Calculation of Scour Depth

4.36 The depth of scour consistent with the calculated flow parameters must be determined. The effects of constriction scour and local scour must be obtained and added together.

4.37 Constriction scour is associated with the erosion of bed material caused by increased velocities through a constriction in the channel at the bridge location. Local scour is associated with the additional effects of piers or abutments disturbing the flow and causing vortices that erode the bed locally.

Calculation of Constriction Scour Depth

4.38 The average depth of constriction scour must be calculated that would result in the average velocity through the bridge opening dropping to a threshold value that would not result in further scouring of the bed.

4.39 The additional area of the flow consistent with the constriction scour should be calculated as

$$\Delta A = \frac{Q_A}{v_{B,c}} - A_{By}$$

where ΔA is the additional area of bridge opening, $v_{\mathrm{B,c}}$ is the mean threshold velocity that would not cause further scouring, as given in 4.41, and A_{By} is the area of flow through the bridge opening without constriction scour, based on y_{B} but no greater than the total area of the bridge opening.

4.40 The average depth of constriction scour below the original bed level $D_{\mathrm{c,ave}}$ should be calculated to provide the additional flow area across the width of the channel bed, as illustrated in Figure 4.6.

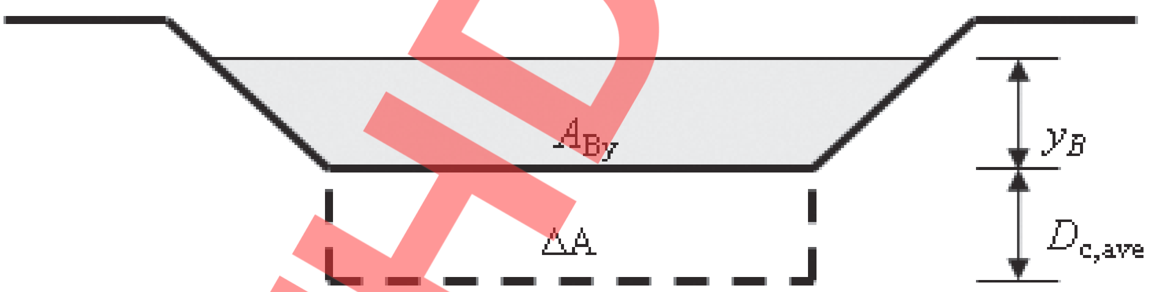


Figure 4.6 – Parameters for Calculating Constriction Scour

4.41 The threshold velocity $v_{\mathrm{B,c}}$ should generally be assumed to be equal to the competent mean velocity, which may be estimated based on Figure 4.7 for granular materials or Table 4.4 for cohesive materials. An iterative approach will generally be required because the competent mean velocity depends on the depth of flow, accounting for scour. The bed material grain size should be based on an estimate of the median grain size where this is practicable, and should be no less than the default values in Table 4.3.

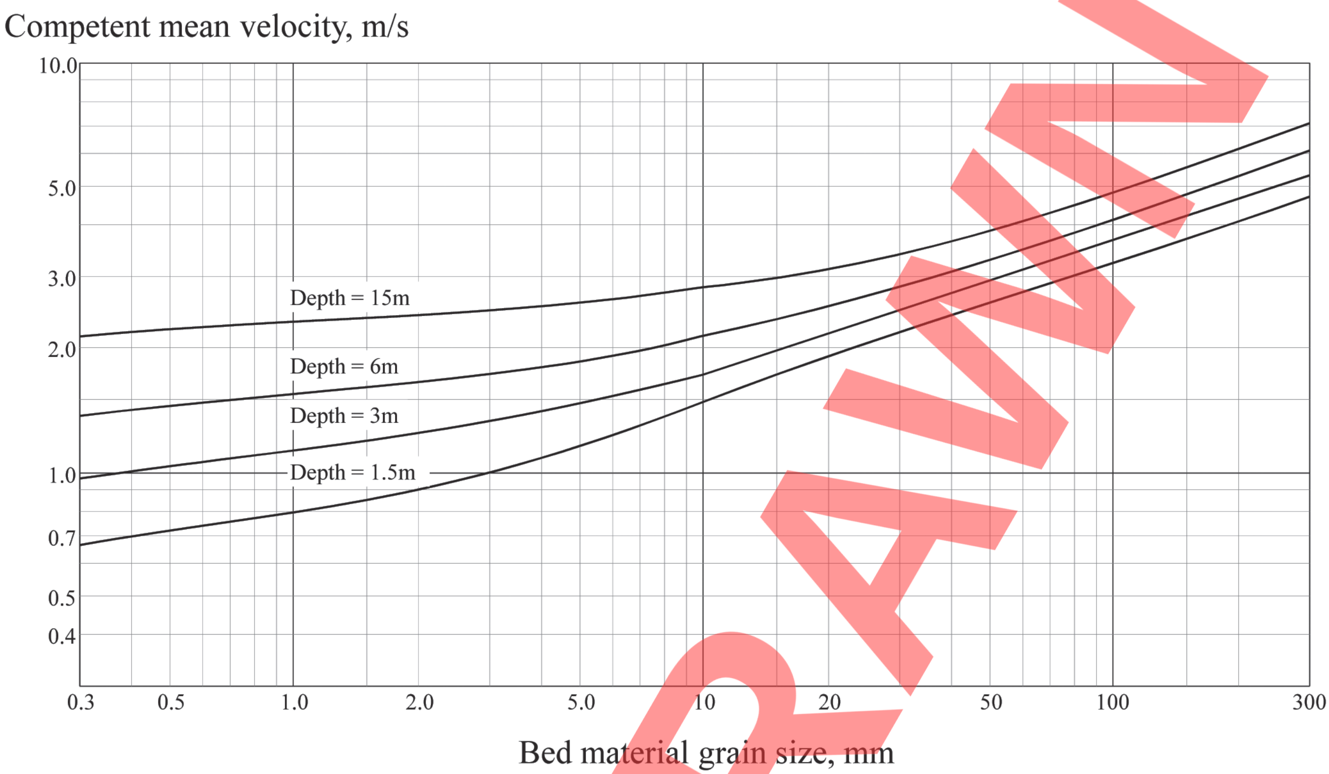


Figure 4.7 – Competent Mean Velocities
(based on Guide to Bridge Hydraulics (TAC))

Terrain	Typical bed material	Typical median grain size
Mountainous, steep	Boulders, cobbles, gravels, sands	10mm
Upland, moderately steep	Cobbles, gravels, sands	5mm
Hilly, moderate	Gravels, sands	2mm
Lowland, flat	Sands, silts and clays	0.5mm
Estuary	Sands and silts	0.1mm

Table 4.3 – Typical Bed Material Characteristics

Depth of flow (m)	Competent mean velocities (m/s)		
	Easily erodible material	Average values	Resistant material
1.5	0.6	1.0	1.8
3	0.65	1.2	2.0
6	0.7	1.3	2.3
15	0.8	1.5	2.6

Table 4.4 – Competent Mean Velocities for Cohesive Materials
(Reproduced from Guide to Bridge Hydraulics (TAC))

4.42 The maximum depth of constriction scour must be calculated based on the average constriction scour and the expected variation of constriction scour across the channel.

4.43 The depth of constriction scour should be taken as:

$$D_c = F_s D_{c,ave}$$

where the constriction scour distribution factor F_s should be taken from Table 4.5. A sharp bend is defined as one with a radius of curvature of less than 3 times the top width of the channel and where the change of direction is more than 45 degrees.

Location	Outside of bend	Centre of channel	Inside of bend
On or downstream of sharp bend	2.0	1.25	1
On or downstream of moderate bend	1.5	1.25	1
On straight reach	1.25	1.25	1.25

Table 4.5 – Constriction Scour Distribution Factor F_s

Calculation of Depth of Local Scour

4.44 The depth of local scour adjacent to piers or abutments must be determined.

4.45 If the depth of constriction scour is sufficient to expose an enlarged footing or pile cap then the local scour calculation must be based on the geometry of the enlarged section.

4.46 The effects of enlarged footings on local scour may be beneficial or adverse, depending on whether the footing is exposed by the constriction scour, as illustrated in Figure 4.8. Beneficial effects of footings on local scour should be neglected.

4.47 The depth of local scour for piers should be based on the maximum potential scour depth, which depends primarily on the geometry of the pier. The maximum local scour depth is given by:

$$D_{l,pier} = 1.5 W_p f_{PS} f_{PA} f_y$$

Where W_p is the width of the pier, f_{PS} is a shape factor, f_{PA} is a factor depending on the angle of attack of the flow and f_y a factor depending on the relative depth of the approach flow to the pier width.

4.48 The shape factor f_{PS} should be taken from Figure 4.9, except where the angle of attack exceeds 10°, when it should be taken as 1.0.

4.49 The angle of attack factor, f_{PA} should be calculated as:

$$f_{PA} = \left(\cos\alpha + \frac{L}{W_p} \sin\alpha \right)^{0.65}$$

Where L is the pier length, W_p is the pier width, and α is the angle between the flow direction and the pier centreline.

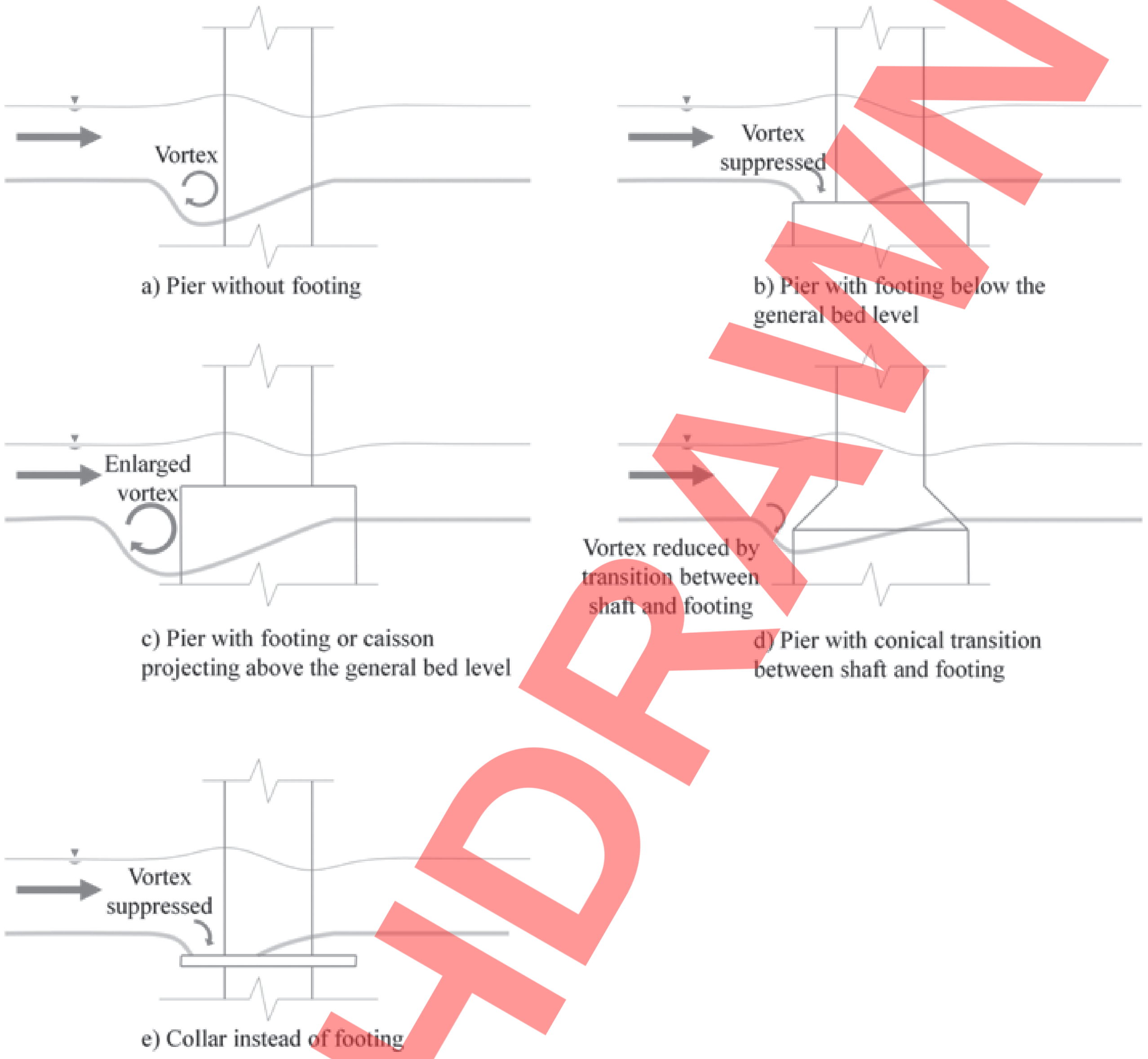


Figure 4.8 – Effect of Pier Footings on Local Scour (based on figure in 'Guide to Bridge Hydraulics', TAC)

4.50 The depth of flow factor f_y should be taken as:

(i) 1.0 where the depth at the pier including constriction scour y_{sp} exceeds 2.6 times the pier width, where $y_{sp} = y_B + D_C$.

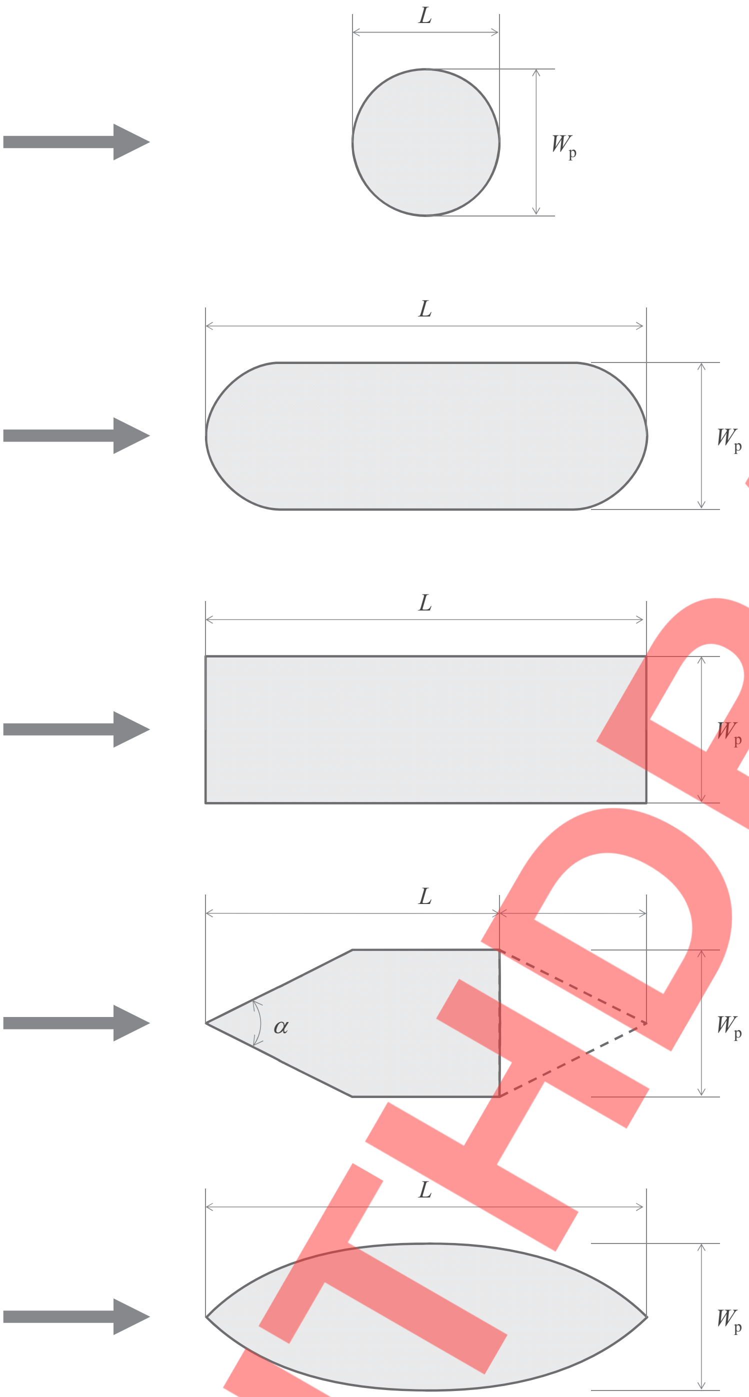
(ii) In other cases, $f_y = 0.78\left(\frac{y_{sp}}{W_p}\right)^{0.255}$

4.51 For piers comprising a line of columns then the local scour may be calculated based on the following:

(i) if the columns are in line with the flow then the local scour should be based on 1.15 times the depth for a single column

(ii) if the columns are not in line with the flow and the spacing exceeds 5 column diameters then the local scour should be based on 1.2 times the depth for a single column.

(iii) otherwise the line of columns may be approximated as a solid rectangular pier.



Plan shape of nose	**L/W_p**	**Shape Factor**
Circular	1.0	1.0
Rounded	All values	1.0
Rectangular	< 2	1.2
	2 - 6	1.4
	> 6	1.2
Triangular		
$\alpha = 60°$	All values	0.8
$\alpha = 90°$	All values	1.2
Lenticular	2	0.9
	3	0.8
	4	0.7

Figure 4.9 – Pier Shape Factors

4.52 For piers comprising other column group configurations the local scour may either be conservatively based on a single solid pier ignoring the spaces between columns, or a more detailed analysis should be carried out based on specialist advice. Further guidance is provided in CIRIA C551.

4.53 The depth of local scour for abutments or retaining walls should be determined based on the method for piers in 4.47 by considering the structure to be equivalent to half a pier. This assumption may be particularly conservative if there are long embankments onto the flood plain, in these cases reference should be made to the recommendations of 'Evaluating Scour At Bridges' (FHWA).

Comparison of Scour Depth with Foundation depth

4.54 The total scour depth at each pier and abutment is determined as the sum of the constriction scour depth and the local scour depth, and compared with the foundation depth. The ratio of calculated total scour depth to depth of foundations is the main indicator of risk. The basis for the Scour Risk Rating is therefore the relative scour depth:

$$D_R = D_T/D_F$$

where D_T is the total depth of scour and D_F is the depth to the underside of a spread footing or the underside of the pile cap in the case of a piled foundation.

Reporting

> 4.55 A report must be produced summarising the outcome of the Level 2 Assessment, including the calculated scour depth and the corresponding foundation depth for each foundation. For Scour Susceptible Structures the report must also include an assessment of risk and vulnerability. The recommended approach for assessing risk and vulnerability is given in Chapter 5.

4.56 The report may include the forms in Annex A which may be used to summarise and document the assessment findings and recommendations.

5. SCOUR RISK RATING

General

5.1 The objective of scour assessment is to allocate a Scour Risk Rating to each assessed structure on a scale of 1 to 5, with 1 being the highest risk. Structures allocated a risk rating of 1 must be designated as Immediate Risk Structures and managed in accordance with BD 79. Structures allocated risk levels of 1 to 4 must be designated as Scour Susceptible. The assessment results must be recorded in the Overseeing Organisation's management information system.

5.2 It is important to recognise that the calculated scour depth is a theoretical estimate of the potential scour depth. If this estimate suggests that the scour will extend below the foundation, it does not necessarily imply that the bridge is at high risk of failure. The calculation methods used to calculate scour depth are considered conservative. There may also be specific reasons why the depth of scour at a bridge may not be as great as the assessment suggests, including:

(i) The presence of an enlargement of the foundation that is not exposed by the constriction scour and hence is likely to restrict the depth of the local scour.

(ii) The location of the maximum depth of constriction scour might not coincide with the location of the maximum local scour.

(iii) Local scour development below a pile cap could be less than that calculated on the basis of the pile cap width.

(iv) The presence of earlier unrecorded protection works, or the presence of more erosion resistant layers of material below the bed, may inhibit the development of scour.

(v) The presence below the river bed of the original cofferdam, or other sheet piling works built for the construction of the works and left in place after completion of the bridge, may act as an enlargement to the pier and suppress the horseshoe vortex and resulting scour hole.

(vi) The hydraulic calculation is based on several conservative assumptions.

(vii) The foundations may be constructed on rock or on piles that have not been recorded.

5.3 Where the estimates of scour depth are very much greater than the foundation depth but the bridge has no history of problems, possible explanations including those in 5.2 should be investigated to achieve a more realistic indicator of risk.

5.4 Immediate Risk Structures are those structures that are considered to be at immediate risk of collapse, either based on qualitative observations of changes to the waterway or the structure, including the appearance of scour holes, debris build up at the bridge, or damage to the structure indicative of scour; monitoring data; or by calculation of the assessed risk. Immediate Risk Structures must be urgently identified to the Overseeing Organisation as described in 2.8.

Assessment of Scour Risk Rating

5.5 The approach adopted is to assess an approximate depth of scour that potentially could occur under the extreme flood condition and to compare that with the actual depth of the bridge foundation. This comparison itself provides the dominant parameter in any prioritisation.

5.6 However there are other parameters that may increase or reduce the likelihood of damage occurring. Some of those have been taken into account in the Stage 2 assessment but there are others that are more difficult to assess from any numerical analysis but which should be considered. These include the following parameters:

- a history of scour problems;
- the type of foundation and material on which the bridge is founded;
- the type of river;
- the importance of the bridge as indicated by vehicle traffic volume and other factors.

5.7 The parameters are combined to provide a Priority Factor. A resulting Scour Risk Rating, based on the magnitudes of the Relative Scour Depth and the Priority Factor, can then be assigned but this should be considered as indicative only and not as a definitive statement of relative risk.

Relative Scour Depth, D_R

5.8 The relative scour depth D_R is calculated in accordance with 4.54. If the scour depth does not exceed the depth to the underside of the foundation for each pier and abutment, then the structure should be designated as Scour Risk Rating 5. Otherwise it should be allocated a Scour Risk Rating in the range of 1 to 4 in accordance with 5.9 – 5.16.

Priority Factor, P_F

5.9 The priority factor, P_F, is defined as:

$$P_F = F . H . M . T_R . V$$

where F, H, M, T_R and V are factors assessed as described below.

Foundation type factor, F

5.10 The foundation depth as defined for D_R, makes no allowance for the depth of any piles. Where the foundation is constructed on piles the priority for further action is reduced as follows:

For a piled foundation	F = 0.75
For a spread footing	F = 1.0

History of scour problem factor, H

5.11 The history of scour problem factor, H is given by:

If the bridge has a history of scour problems then	H = 1.5
If the bridge has no history of problems then	H = 1.0

Foundation material factor, M

5.12 The foundation material factor, M is given by:

If there is no information on the foundation material or the material is granular (silts, sands, gravels etc)	M = 1.0
If there is some evidence that the bridge is founded in clay	M = 0.75
If there is strong evidence that the bridge is founded in clay or there is a reasonable possibility of rock under the foundations	M =0.5

Type of river factor, T_R

5.13 The potential for instability and scour is highest in steep mountain and upland watercourses. Hence the Type of river factor, T_R is given by:

If the terrain is mountainous	$T_R = 1.5$
If the terrain is upland	$T_R = 1.3$
If the terrain is hilly	$T_R = 1.2$
If the terrain is lowland or an estuary	$T_R = 1.0$

Importance factor, V

5.14 The greater the importance of the bridge and the greater the disruption caused by any interruption to its use, the higher the priority. Importance is typically related to traffic flow. Hence the Importance factor V is given by:

Type of road	**12 hour traffic flow**	**V**
Motorway/A road	≥ 30,000	1.0
Motorway/A road	10,000 – 29,999	0.9
A/B class road	1,000 – 9,999	0.8
B/other class road	< 1,000	0.7

5.15 There are some circumstances where the traffic flow alone does not fully reflect the importance of a bridge. Examples include:

- bridges with no suitable diversion route or the diversion route is very long;
- bridges on rural roads to ports serving island communities where there is no diversion route
- bridges that provide a link within a community where loss of the bridge would result in unacceptable community severance.

In such cases, the figures for V in 5.14 can be multiplied by an additional factor of up to 1.3.

Scour Risk Rating

5.16 The Scour Risk Rating is then assessed from Figure 5.1, based on the Priority Factor and the Relative Scour Depth (4.54). This graph shows five bands which define the risk rating (1 being the highest priority and 5 the lowest). Bridges falling in band 5 have either been eliminated at Stage 1, as having a very low risk of scour damage, or have been assessed in Stage 2 as having a depth of foundation greater than the estimated maximum depth of scour.

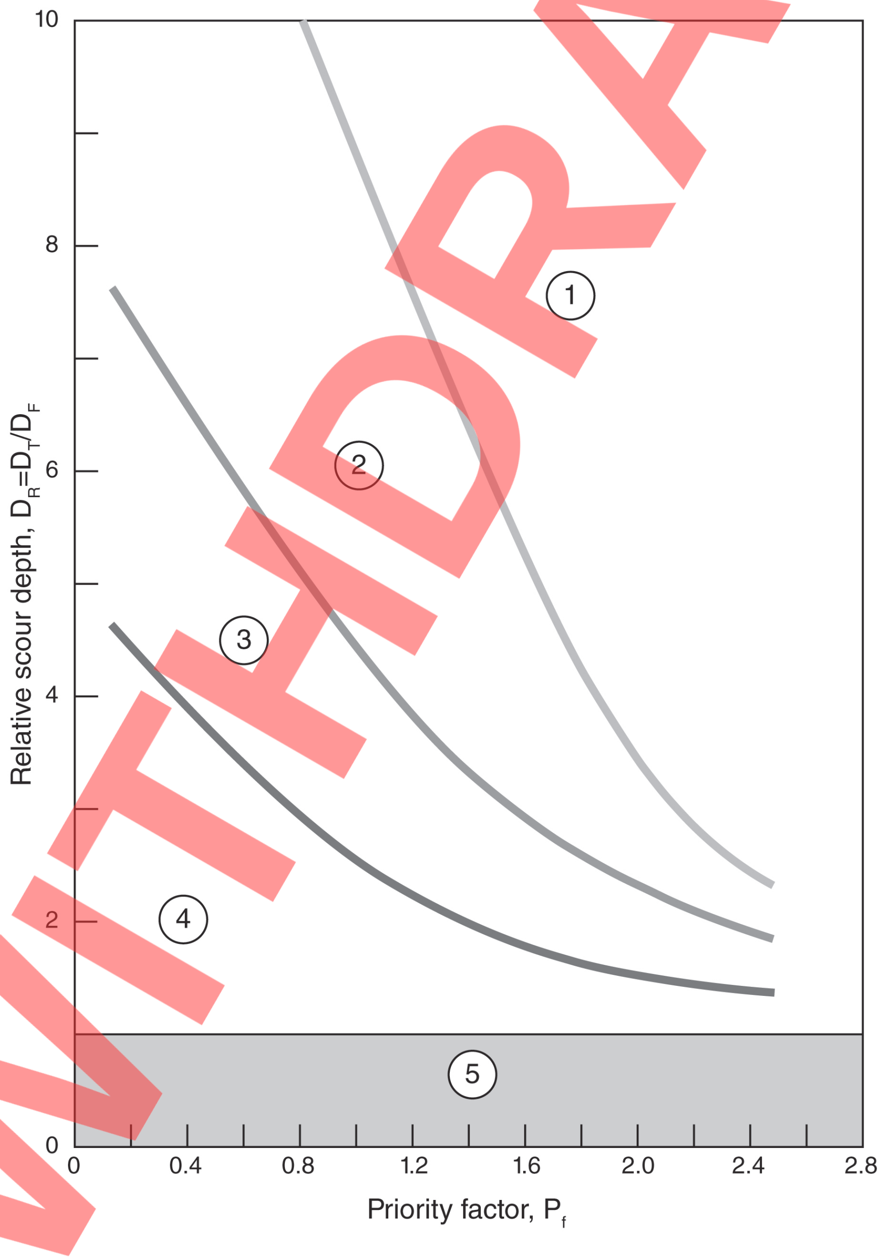


Figure 5.1 – Scour Risk Rating

6. ASSESSMENT OF VULNERABILITY TO OTHER FLOOD EFFECTS

6.1 Although scour is the most common hydraulic action that causes failure of highway structures, there are other actions that need to be considered.

6.2 BA 59 (DMRB 1.3.6) The Design of Highway Bridges for Hydraulic Action contains advice on designing against failure due to hydraulic forces on piers and bridge decks, and failure due to debris. Structures should be assessed for these aspects using BA 59 unless they have already been designed or assessed for these aspects in accordance with BA 59 and the design or assessment parameters have not changed.

6.3 For structures where the flood water is predicted to reach or exceed the deck soffit level, the following aspects should be considered:

(i) whether uplift on the bridge soffit would lead to a reduction in stability or load carrying capacity (e.g. masonry arches);

(ii) whether the bridge deck or parapets would be dislodged or destroyed by hydrodynamic actions on the deck.

6.4 For structures where the flood water is predicted to reach within 0.6m of the soffit level, the possibility of the structure becoming dislodged or destroyed by actions related to debris should be considered. Further advice is provided in BA 59.

6.5 The Assessment report should include a description of the assessment work carried out under this chapter and a statement of the findings for consideration by the Overseeing Organisation.

> 6.6 Any structures found by assessment to be unable to resist the applied actions must be managed in accordance with BD 79.

7. SCOUR RISK MANAGEMENT MEASURES

General

7.1 Measures to manage scour risk will vary according to the risk level obtained in the assessment and the nature of the deficiency. In view of the earlier mentioned conservatism in the scour depth calculation only bridges with highest Risk Ratings need be of serious concern. Bridges with ratings 3 and 4 are of less concern with regard to potential scour damage but that potential scour damage cannot be ignored. Table 6.1 summarises the recommended action.

Risk Rating	Actions
1 & 2	Carry out further investigations, determine and if necessary implement appropriate monitoring and scour protection measures as a high priority. Structures with a Risk Rating of 1 to be managed as Immediate Risk Structures in accordance with BD 79.
3 & 4	Carry out further investigations, determine and if necessary implement appropriate monitoring and scour protection measures when resources allow and after Risk Rating 1 and 2 structures have been dealt with. Re-inspections, both as part of regular bridge inspections and after major floods, should examine for signs of scour and bank erosion. If conditions at the bridge change then re-assessment should be carried out.
5	No action required other than routine inspections in accordance with BD 63.

Table 7.1 – Actions in Response to Scour Risk Rating

7.2 Management strategies for scour susceptible structures should be developed by the Maintaining Organisation. Management strategies will vary according to structure type and risk level but will typically comprise a selection of risk management measures including further investigation/refined assessment calculations, emergency planning, monitoring measures and scour protection measures. For some Scour Susceptible Structures with lower Scour Risk Ratings the management strategy may include monitoring on a long-term basis instead of installing Scour Protection measures. These measures are discussed in more detail below.

Further Investigation/Refined Assessment Calculations

7.3 For structures where the assessment has been carried out based on limited data, for example regarding the foundations or the bed material, then the possibility of further investigation work which could lead to a lower assessed risk should be considered. The potential benefit of this work in terms of a reduced risk should be considered within the context of:

(i) the cost of carrying out the investigation and reassessment;

(ii) the cost of providing protection measures.

7.4 The first step following the assessment should be an appraisal of the conclusions by a specialist river engineer. Before any significant expenditure on further studies and/or remedial works it needs to be confirmed that the simplifications inherent in this methodology are not leading to an excessively conservative conclusion.

7.5 The second step should be to review the theoretical conclusions in the light of the age and history of the bridge. There may be particular reasons why scour could not develop at a site to the extent predicted. For example, an old bridge with no history of problems may well be founded in a scour-resistant layer of material or even rock.

Example

7.6 One particular bridge, included in the sample set on which the methodology was tested, is a large multi-span structure on a major river. The bridge is located immediately downstream of an abrupt 90° bend, with angled approach flow across the face of the wide piers – all the circumstances which would be expected to lead to a major scour problem.

7.7 The theoretical depth of scour is some six times the depth to the pile cap and comparable to the depth of the piles themselves, yet the bridge has been in place for 70 years with no instances of major problems having been noted in the summary records held by the Overseeing Organisation. The fact that the river channel has remained stable at the bend is, however, indicative of erosion resistant soils and the construction drawings do indicate layers of clay beneath the gravel bed. It could also be seen when the bridge was examined at a time of low flow in the river that cofferdam works had been left in place following either construction of the original bridge or modifications to the bridge piers.

7.8 Those modifications, which involved reshaping the bridge pier noses, are perhaps indicative that scour problems were experienced at some stage despite the lack of any such note in the bridge records. Further searches as suggested in 5.3 might discover additional records which describe the modifications to the bridge, their purpose and their date, all of which would be helpful to a reviewing engineer. It may also be the case that the pile caps are fully exposed in quite moderate flows but that the scour does not develop to an extent that endangers the bridge. This could be either because of the cofferdam works or the layers of scour resistant material below the river bed.

7.9 In this case there may be no need of additional protection works but borehole confirmation of the foundation material and monitoring of the bed profile and the development of any scour may be appropriate. However, it would also be sensible to carry out a more detailed investigation of the remnants of the cofferdams and their condition. If the bridge is dependent upon these cofferdams to prevent damaging scour then they must be considered as an essential part of the bridge and be subject to the same inspection criteria.

7.10 In some cases further theoretical studies may be appropriate, possibly backed up by site measurements. For example, the theoretical approach outlined in this document for bridges in tidal locations is very simplified and will be over-conservative in some cases. Further analysis of the potential flows and velocities through such bridges may be warranted and could be cross-checked by site velocity measurements.

7.11 Generally, however, the relative lack of sensitivity of the local scour depth to the depth and velocity of flow make any further refinement of the flood flow unlikely to be worthwhile unless there were particular uncertainties with the estimation at a particular site. In most cases this will also apply to the hydraulic assessment of the conditions at the site. Where control structures affect the downstream levels or other bridges, or training works affect the approach flow, it may be worthwhile having further analysis carried out by hydraulic engineers. This could involve mathematical or even physical modelling but before such studies are embarked on, their costs need to be compared with the costs of providing protection.

Flood Emergency Plan

7.12 A flood emergency plan may be appropriate for some structures most at risk of scour-related collapse in the event of a flood of a particular magnitude. The plan should include the relevant trigger levels, associated actions (e.g. closure of the structure and diversion of traffic), details of relevant authorities to authorise and implement the measures, and the processes for reviewing the measures as appropriate.

Monitoring Measures

7.13 In the interim period before protection measures can be implemented, Scour Susceptible Structures should be monitored to identify changes to the structure or the watercourse, or to measure the development of scour where appropriate. Methods for managing the monitoring of structures are described in BD 79.

7.14 Scour monitoring techniques fall into the following broad categories:

(i) those that seek to measure the maximum scour levels that have occurred at the bridge site;

(ii) those that seek to measure the development of scour adjacent to the structure as it develops during a flood;

(iii) systems based on monitoring analogues (conditions that may correlate with the development of scour) such as flow velocities, water level, or weather warnings.

7.15 Retrospective measurement of scour depth can be difficult because scour holes tend to refill on the receding flood. Generally therefore the techniques rely on assessing the differences between material filling a scoured hole and the underlying bed material. This may be indicated by changes in the material grading, caused by the natural armouring effect that occurs at the bottom of a scour hole, or, if investigations are carried out shortly after a flood, by changes in the compaction of the material. There are a number of possible techniques, including:

(i) Test pits: These may be costly and the results may not be sufficiently sensitive to identify subtle changes in the bed material.

(ii) Borings: These also have limitations and may disturb the material such that the distinctions looked for are masked. They are not suitable in coarse material.

(iii) Ultrasonic or radar-based measurements.

7.16 There are technical limitations to retrospective measurement of scour and it can be difficult to economically obtain reliable evidence. Monitoring the development of scour holes during a flood can be more reliable if robust equipment is used. This can typically be carried out using:

(i) systems based on a simple weighted line or rod;

(ii) sensor-based systems;

(iii) ultrasonic or radar-based systems.

7.17 The monitoring regime should be developed with defined risk-based criteria and associated actions. Where the monitoring indicates an unacceptable risk then further measures should be provided to reduce the probability or the consequences of scour.

Scour Protection Measures

7.18 Scour protection can make a bridge less vulnerable to failure or damage by scour. There are many options to be considered, including:

(i) Flow Control Measures – to improve flow conditions at a structure so reducing the magnitude and effects of scour, e.g. streamlining of piers, streamlining the channel through the bridge waterway, river training, deflectors such as guide banks, or sacrificial piles.

(ii) Structural Measures – to withstand the predicted depths of scour, which in the case of remedial measures include underpinning foundations, reinforcement and extension of foundations, other options such as 'bagged' concrete, sheet piling, or concrete grout.

(iii) Bed Protection Measures – to limit the extent to which scour can occur, such as riprap, rock-filled gabion mattresses, concrete block revetments and similar 'rigid' systems, so called 'bio-technical' solutions to stabilise river banks.

7.19 Specialist input is generally required to determine the most appropriate option. Selected remedial measures to provide scour protection to a vulnerable pier or abutment are described below (for further information see BA 59 and CIRIA C551):

(i) **Stone (riprap) aprons**

Stone aprons may be placed around piers and abutments as a flexible 'falling apron' to prevent local scour development. The stone must be large enough to remain stable under the maximum velocities. Comprehensive design methods are available, which cover not only size and grading of stone riprap but also the extent and thickness of the protection and the possible need for underlying filter layers. Such designs will require specialist involvement with manufacturers of proprietary systems and water engineers. Where an apron is installed the stone should be placed in a pre-excavated position below the bed of the river so that velocities through the waterway are not increased by its presence.

(ii) **Paved invert**

The bed beneath the bridge may be paved with concrete placed in situ or pre-formed slabs, or one of the commercially available grouted mattresses. (Older bridges may have existing brick or masonry inverts). The protection may be local to the pier or abutment or across the full invert. The disadvantage of the use of concrete slabs is the difficulty of placing them without major diversion works. Grouted mattresses can be placed and filled underwater but care needs to be taken with the details, particularly at the upstream and downstream edges to ensure that they themselves cannot be undermined and damaged by scour at those points.

(iii) **Enlargements to piers**

Enlarging the base of a pier may limit the depth of local scour but it has to be done carefully as it can also increase the total scour. Any such enlargement needs to be below the level of general and constriction scour. Sheet piling can be used for the enlargements and will also protect the pier or abutment but its use may be limited by plant headroom considerations under the bridge and by river bed material.

(iv) **Other methods of protection**

There are other types of bed and bank protection, including gabions and gabion mattresses and proprietary systems of interlocking blocks, which might be utilised. Other approaches that have been tried include modification of pier nosing shape and provision of piles of smaller diameter than the width of the pier, upstream but on the alignment of the pier such that the pier is in the wake or shadow of the piles. Local scour then occurs predominantly around those additional piles and is a function of those pile dimensions rather than those of the pier.

7.20 Proposals for scour protection measures and other physical works should be referred to the TAA, who will determine whether the requirements of BD 2 apply and if appropriate agree the category of the proposals.

8. REFERENCES

8.1 Design Manual for Roads and Bridges

BD 79 – The Management of Sub-Standard Highway Structures

BD 62 – As Built, Operational and Maintenance Records for Highway Structures

BD 63 – Inspection of Highway Structures

9. BIBLIOGRAPHY

9.1 Design Manual for Roads and Bridges

GD 01 – Introduction to the Design Manual for Roads and Bridges

BD 2 – Technical Approval of Highway Structures

BA 59 – Design of Highway Bridges for Hydraulic Action

9.2 Other Documents

Manual on Scour at Bridges and Other Hydraulic Structures; CIRIA C551, Construction Industry Research and Information Association, 2002.

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Evaluating Scour at Bridges, Fourth edition, Hydraulic Engineering Circular No 18, Federal Highway Administration, US Dept of Transportation, May 2001.

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10. FURTHER READING

Hydrometric Register and Statistics 1996-2000; Centre for Ecology and Hydrology, Wallingford, and British Geological Service 2003 (4th edition).

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Design Method for Local Scour at Bridge Piers; Melville BW & Sutherland AJ, Journal of Hydraulic Engineering, ASCE, Vol 114, No. 10, 1988.

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Risk-Based Management Guidelines for Scour at Bridges with Unknown Foundations, Web only document 107, National Cooperative Highway Research Program, TRB, 2006.

Evaluation of Bridge Scour Research: Pier Scour Processes and Predictions, Web only document 175, National Cooperative Highway Research Program, TRB, 2011.

Stream Stability at Highway Structures, Third edition, Hydraulic Engineering Circular No 20, Federal Highway Administration, US Dept of Transportation, March 2001.

Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volumes 1 & 2, Third edition, Hydraulic Engineering Circular No 23, Federal Highway Administration, US Dept of Transportation, September 2009.

Evaluation of Bridge Scour Research: Abutment and Contraction Scour Processes and Predictions, Web only document 181, National Cooperative Highway Research Program, TRB, 2011.

11. ENQUIRIES

All technical enquiries or comments on this Standard should be sent in writing as appropriate to:

Chief Highway Engineer The Highways Agency 123 Buckingham Palace Road London SW1W 9HA	G CLARKE Chief Highway Engineer

Director, Trunk Road and Bus Operations Transport Scotland Buchanan House 58 Port Dundas Road Glasgow G4 0HF	R BRANNEN Director, Trunk Road and Bus Operations

Director of Transport Welsh Government Cathays Park Cardiff CF10 3NQ	F DUFFY Director of Transport

Director of Engineering The Department for Regional Development Roads Service Clarence Court 10-18 Adelaide Street Belfast BT2 8GB	R J M CAIRNS Director of Engineering

This document was notified in draft to the European Commission in accordance with Directive 98/34/EC, as amended by Directive 98/48/EC.

ANNEX A FORMS FOR SCOUR INSPECTION AND ASSESSMENT

Scour Inspection

Structure name and number: OS Grid Ref:

Details of Inspection

Inspected by:		Inspection Date:	
Weather:		Flow Conditions:	

Details of Structure

Record details of the following, identifying any estimated values.

Construction Type:		Road over structure:	
Foundation Type:		Waterway under structure:	
Construction Date:			

Items to be included with this inspection (minimum)

• Photographs of the structure and the channel from upstream and downstream of the structure	
• Completed inspection form	
• Plan and section drawings of structure	
• OS plan of bridge and site	

Notes

Scour Inspection

Structure name and number: Inspection Date:

General

Answers must be accompanied by further details in the notes section

Is there a bend in the river immediately upstream or under the structure?	
Does the river geometry agree with the OS plan of the site?	
Are there any confluences within 1km of the structure?	
Are there any islands or bars within the channel?	
Are there any control structures in the vicinity of the structure, e.g. weirs, sluice gates?	
Are there scour countermeasures in place (note form and condition)?	
Is there evidence of ongoing scour at the bridge site (note approximate depths and locations of any scour holes)?	
Is there evidence of general bed degradation or aggradation?	
Is there evidence of movement or settlement of the bridge structure?	
Is the structure fouled by debris or likely to become fouled in flood conditions? Action to remove fouling should be made an urgent recommendation.	
Are there signs of long-term bank stability?	
Is there erosion at the outside of river bends, e.g. undermining of the river bank?	
Are there adjacent flood relief structures?	
Is there evidence of previous flood levels?	

Scour Inspection Form

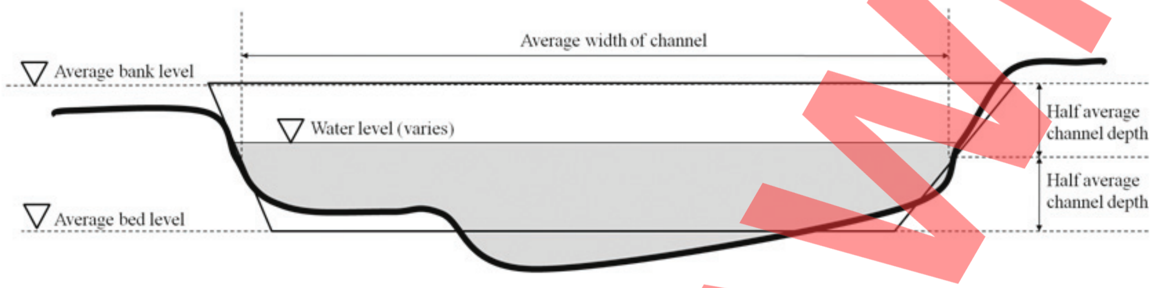
Structure name and number:	Inspection Date:

Notes:

Scour Inspection Form

Structure name and number:

Inspection Date:



Geometry at the structure

Bank to bank width of channel	
Height of soffit (or arch crown) above average bed level	
Average depth of channel (to average bank level)	
Average width of channel	
Average bank height above water level during inspection	
Average depth of water during inspection	
Max depth of water during inspection	
Sketch of structure including location of piers and abutments and approximate channel geometry.	

Pier width (if piers are not uniform, provide details for all)	
Pier length	
Pier nose shape	
Protrusion of abutments into channel (from bank)	
Angle of flow relative to piers	
Average bed material size	

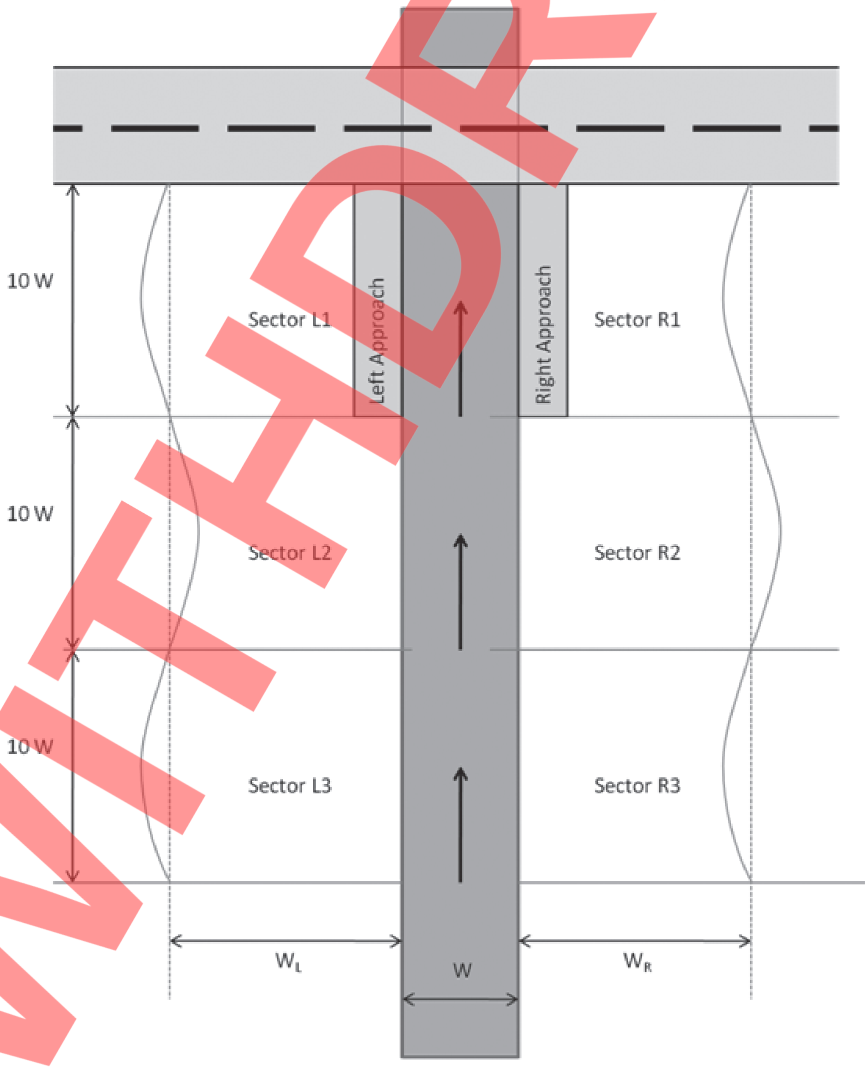
Scour Inspection Form

Structure name and number:	Inspection Date:

Geometry upstream of the structure

Average width of channel	
Average bank height above water level during inspection	
Width of left flood plain	
Width of right flood plain	

Sketch of flood plain characteristics
(indicate for each sector and approaches whether very obstructed, partially obstructed or open)

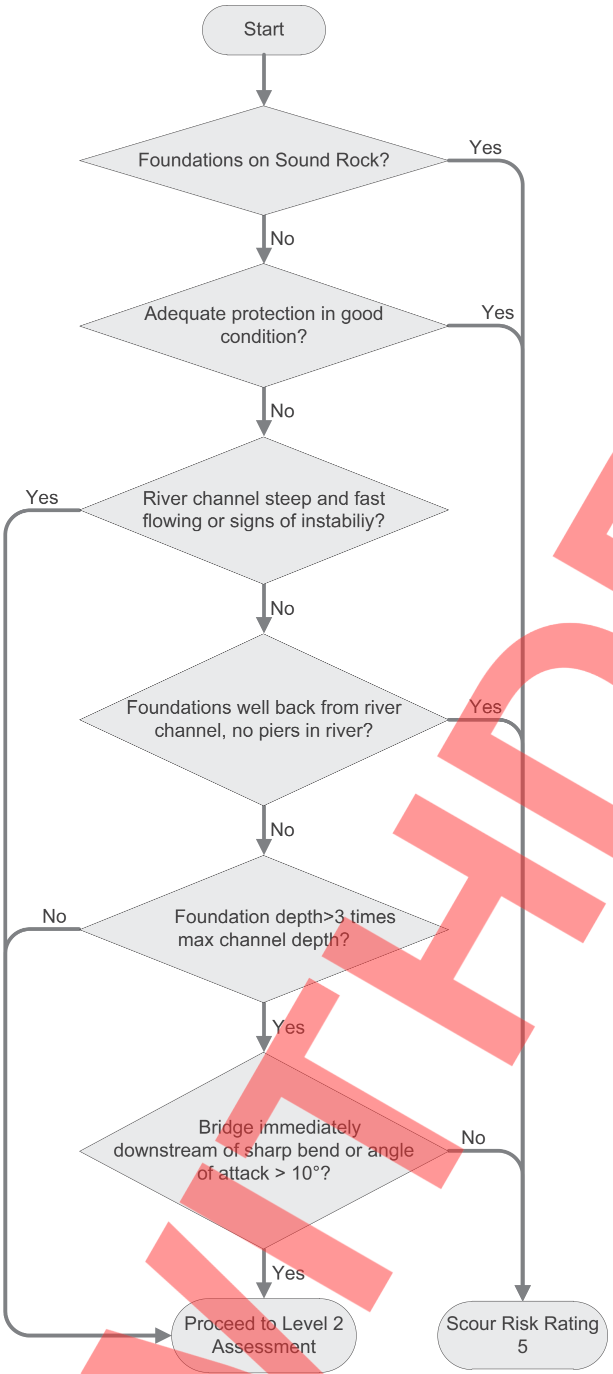


Notes

Level 1 Assessment Summary

Structure name and number: Assessment Date:

Notes



Level 1 Assessment outcome

Proceed to Level 2 ☐

Scour Risk Rating 5 ☐

Level 2 Assessment Summary

Structure name and number: Assessment Date:

Refer to Assessment Calculations ref:	
Value of Assessment Flow:	
Is it expected that the water level will reach the structure soffit in the assessment flood?	

Comment on the structure robustness in the event of a flood, including the vulnerability of the structure and its components to hydraulic actions and effects from debris, and recommended measures:

Support Location	Depth of Constriction Scour	Depth of Local Scour	Total Depth of Scour

Level 2 Assessment Summary

Structure name and number: Assessment Date:

Comment on the calculated scour depths. Are there any specific reasons why there may be uncertainty?

Foundation depth	
Maximum scour depth/foundation depth	
Foundation Type factor, F	
History of scour problem factor, H	
Foundation material factor, M	
Type of river factor, T_R	
Importance Factor, V	
Scour Risk Rating	

Notes on the calculated Scour Risk Rating:

Level 2 Assessment Summary

Structure name and number:	Assessment Date:

Recommended options for further action:

ANNEX B LIST OF TPB CONTRIBUTING ORGANISATIONS

Association of Directors of Environment, Economy, Planning and Transport (ADEPT)

Department for Regional Development – Northern Ireland

Highways Agency

Transport Scotland

Welsh Government

ANNEX C DEFINITIONS AND ABBREVIATIONS

Definitions

Afflux	The effect whereby water will back up behind an obstruction, such as a bridge, in the river channel. This acts to increase the depth upstream of the bridge beyond that which would occur if the bridge were not there. This may also be referred to as a backwater effect.
Assessment Flow	Flow rate to be used for the assessment, typically calculated based on a return period, and including fluvial and tidal components as appropriate.
Constriction Scour	Scour associated with the increased flow velocity at a constriction in the channel. The presence of a bridge across a waterway will tend to restrict the waterway opening. Even if the bridge opening is wider than the main channel, it is likely to create a restriction to flood flows when the water level is above bank level. This impedance of flood flows causes an increase in the velocity through the bridge opening and a greater depth of scour results. This type of scour in the bridge waterway is referred to as "constriction scour".
Local Scour	Scour due to effects of the bridge structure on the local flow patterns. It is most likely to develop around piers or abutments in the main river channel but may also occur around piers and abutments set back on a floodplain when there is overbank flow.
Maintaining Organisation	The organisation appointed by the Overseeing Organisation to manage highway assets on its behalf.
Scour Risk Rating	A whole number in the range 1 – 5 determined in accordance with Chapter 5 of this standard.
Scour Susceptible Structure	A structure allocated a risk rating in the range 1 to 4 in accordance with Chapter 5 of this standard.

A	Cross-sectional area of flow
A_n	Normal cross-sectional area of flow for uniform flow conditions
A_{By}	Cross-sectional area of flow through the bridge waterway
ΔA	Increase in cross section area of flow for constriction scour
B	Average width of channel
B_B	Average width of channel at bridge opening
D_s	Depth of scour below average level of river bed
D_F	Depth of foundation below average bed level
D_C	Depth of constriction scour below average bed level
$D_{C,ave}$	Average depth of constriction scour below average bed level
D_L	Depth of local scour below average bed level
D_R	Relative scour depth
D_T	Total depth of scour
f'_A	Abutment scour factor
f_{AA}	Abutment alignment factor
f_{AS}	Abutment shape factor
f_{PA}	Pier alignment factor

f_{PS}	Pier shape factor
f_T	Flood growth factor for period of T years
f_y	Flood depth factor
F	Foundation type factor
F_s	Scour distribution factor
F_P	Floodplain factor
g	Gravitational acceleration (9.81m/s^2)
H	History of scour problem factor
L	Length of pier
M	Foundation material factor
n	Manning's coefficient
P	Wetted perimeter of flow cross-section
P_F	Priority factor
Q	Flood flow
Q_A	Assessment Flow
Q_F	Fluvial component of Assessment Flow
Q_{Tide}	Peak flow due to tidal effects based on average tidal range
$Q_{Tide,max}$	Maximum tidal flow due to tidal effects
s	Longitudinal slope of river channel
s_c	Critical longitudinal slope of river channel
T_{Tide}	Tidal period
T_R	Factor denoting type of river
v	Mean velocity of flow
v_B	Mean velocity of flow through bridge opening
$v_{B,c}$	Mean threshold velocity below which scour does not occur
v_u	Mean upstream velocity
v_n	Mean normal velocity for uniform flow conditions
V	Importance factor
V_{Tide}	Volume of tidal prism
W	Floodplain width
W_{eff}	Effective floodplain width
W_P	Bridge pier width
y	Depth of flow
y_B	Depth of flow through bridge opening
$y_{B,sup}$	Depth of flow through bridge opening for supercritical flow conditions
$y_{B,sub}$	Depth of flow through bridge opening for subcritical flow conditions
y_c	Critical depth of flow for minimum energy conditions
y_n	Normal depth for uniform flow conditions
y_p	Height of normal flow above floodplain level
y_u	Upstream depth
α_y	Coefficient for floodplain effect on depth
α_v	Coefficient for floodplain effect on velocity
α	Angle of pier centreline to flow