Design Manual for Roads and Bridges

Highway Structures & Bridges
Design

CD 356
Design of highway structures for hydraulic action

(formerly BA 59/94)

Revision 1

Summary
This document provides information on the hydraulic aspects of the design of structures in or over rivers, estuaries and flood plains, including the studies required to support these design aspects.

Application by Overseeing Organisations
Any specific requirements for Overseeing Organisations alternative or supplementary to those given in this document are given in National Application Annexes to this document.

Feedback and Enquiries
Users of this document are encouraged to raise any enquiries and/or provide feedback on the content and usage of this document to the dedicated Highways England team. The email address for all enquiries and feedback is: Standards_Enquiries@highwaysengland.co.uk

This is a controlled document.

Contents

Release notes

Version	Date	Details of amendments
1	Mar 2020	Revision 1 (March 2020) Revision to update references only. Revision 0 (September 2019) CD 356 replaces BA 59/94. The full document has been re-written to make it compliant with the new Highways England drafting rules.

Foreword

Publishing information

This document is published by Highways England.

This document supersedes BA 59/94, which is withdrawn.

Contractual and legal considerations

This document forms part of the works specification. It does not purport to include all the necessary provisions of a contract. Users are responsible for applying all appropriate documents applicable to their contract.

Introduction

Background

Scour and flooding are a leading cause of bridge failures in the UK and worldwide. Examples include the flooding in December 2015 which caused collapse of bridges in Cumbria (ICE 1700009 [Ref 18.I]) and North Yorkshire. Other studies of failure records indicate the risk of structural failure during flooding (ICE JFE 1300013 [Ref 21.I]). A study of US bridge failures between 1980 and 2012 found that approximately 50% were attributable to scour and flooding, see: Technical Report MCEER-13-0008 [Ref 1.I]

With storms in the UK expected to become both more severe and more frequent as a consequence of climate change, there are potentially increased structural risks to highway bridges and other highway structures, due to severe weather events, such as:

1) scour in non-tidal rivers, tidal rivers and estuaries;
2) very large horizontal forces on bridge piers and uplift on bridge decks during extremely high river flow rates; and
3) ocean storm surges and coastal wave action.

The design of bridges across watercourses requires a multi-disciplinary approach, involving structural, geotechnical as well as specialist hydrological and hydraulics expertise.

The need to cross a watercourse can be the reason for construction of a highway bridge. It is essential that sufficient attention is paid to the prevention of failure due to scour and flooding when designing bridges over rivers, estuaries or flood plains. Hydraulic actions and scour need to be considered early in the design process.

Hydraulic actions need to be considered in the design of the structure. The structural Eurocodes do not define hydraulic actions in detail, therefore supplementary requirements applicable to the UK are provided.

The presence of the bridge and approaches can have an impact on the watercourse. Environmental legislation emphasises the importance of minimising the impact on the water environment.

A variety of approaches are available for modelling flows in the watercourse and for estimating scour. It is not always possible to offer a single, best method or equation for a particular calculation and there can be a wide range of answers produced by alternative methods. There is a need for a competent practitioner to select an appropriate methodology.

Assumptions made in the preparation of the document

The assumptions made in GG 101 [Ref 6.N] apply to this document.

Abbreviations

Abbreviations

Abbreviation	Definition
ALARP	As Low As Reasonably Practicable
BGS	British Geological Survey
CFD	Computational Fluid Dynamics
FEH	Flood Estimation Handbook
H++	The upper limit of climate projections that are considered plausible
HAT	Highest Astronomical Tide
HOST	Hydrology of Soil Types
LIDAR	Light Detecting and Ranging, a remote aerial survey technique
MF	Mean flow
OS	Ordnance Survey
PE	Potential evaporation
ReFH2	Revitalised Flood Hydrograph method (v2)
SAAR	Standard annual average rainfall
SEPA	Scottish Environmental Protection Agency
UKCP09	UK Climate Projections produced in 2009
UKCP18	UK Climate Projections due to be published in 2018

Symbols

Symbols

Symbol	Definition
A	Accidental action
A_b	Blocked area (portion of the cross-sectional area of flow through the opening which is blocked by debris)
A_d	Design value of an accidental action
A_F	Reference area
A_u	Unblocked area (portion of the cross-sectional area of flow through the opening which is not blocked by debris)
BR	Blockage Ratio
C_D	Drag coefficient
C_L	Lift coefficient
F_k	Characteristic value of drag (or lift) force
$\hat{k}$	Effective contact stiffness
m	Single-item debris mass
Q	Variable action
$Q_{combination}$	Combination value of a variable action
$Q_{frequent}$	Frequent value of a variable action
Q_k	Characteristic value of a variable action
$Q_{quasi\text{-}permanent}$	Quasi-permanent value of a variable action
U_{TC}	Critical threshold velocity for the surface bed material
V	Water reference velocity
Y_S	Equilibrium depth of local scour
ψ	Factor applied in combinations of actions
ψ_0	Factor applied to obtain the combination value of a variable action
ψ_1	Factor applied to obtain the frequent value of a variable action
ψ_2	Factor applied to obtain the quasi-permanent value of a variable action
γ_Q	Partial factor applied to a variable action
ρ	Water density

Terms and definitions

Terms

Term	Definition
Afflux	The increase in water level upstream of a bridge over that which would have occurred if the bridge were absent.
Aggradation	A natural increase in bed levels due to sediment deposition, tending to oppose the effects of natural scour and potentially occurring as a result of transport of natural scour products from upstream.
Angle of attack	Angle between the longitudinal axis of a pier and the direction of flow.
Annual probability of exceedance	The probability of a certain flood event being exceeded each and every year. NOTE: For example a flood event with an annual probability of exceedance of 0.5% has a 0.5% (or 1 in 20 0) of being exceeded each and every year. The equivalent return period is the inverse of the annual probability of exceedance. For this example, the equivalent return period is 200 years (the 1 in 200 year return period event).
Armouring	The process of progressive removal of finer sediment from the bed which leaves a layer of coarser sediment lying on the bed.
Backwater length	The distance upstream where water depth is raised above normal depth conditions due to a downstream hydraulic control (for subcritical flow conditions).
Baseflow index	A flow statistic which characterises the proportion of river runoff which is derived from stored sources. NOTE: It is a measure often used in flow estimation techniques.
Bend scour	Bend scour occurs at a bend in a watercourse, typically on the outside of the bend.
Channel pattern	The plan form of a river.
Check event	The largest event that should be used for additional design checks, taken as the larger of: 1) the design flood check event, including normal climate change allowance; and 2) the largest flood event for design, with a higher climate change allowance.
Confluence scour	Natural scour that occurs where watercourses meet.
Contraction scour	Contraction scour occurs in a confined section of a watercourse and results in a lowering of the bed level across the width of the watercourse.
Degradation	Reduction in bed levels with time.
Design flood check event	The largest flood event (fluvial, tidal or combined) that should be used for additional design checks.

Terms (continued)

Term	Definition
Design working life	Assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.
Design event	The event which gives the largest value of the parameter under consideration for the events up to: 1) the largest fluvial flood event for design; 2) the largest tidal event for design; 3) the largest combined fluvial and tidal event for design; 4) other, more frequent, event for assessing natural scour.
Dredging	The removal of sediment from the bed of the river. NOTE: This can reduce bed levels both upstream and downstream of the location of dredging.
Equilibrium depth	Under a give steady flow the depth of scour will increase until an equilibrium is reached. After that time no further increase in scour depth takes place
Flood plain constriction	Where embankments are used to cross a flood plain the flow over the flood plain may be constricted leading to increased flow velocities and scour.
Freeboard	Vertical distance between water surface and the soffit of a bridge or top of embankment.
Froude number	A non-dimensional flow parameter which expresses whether flow in an open channel is sub-critical, critical or super-critical.
General scour	Scour which affects the whole width of the river but is still confined to the reach adjacent to the structure. It is commonly caused by a channel constriction at the bridge site (such as by bridge approach embankments encroaching onto the floodplain and/or into the main channel) or a change in downstream control of the water surface elevation or flow
Hydrodynamic action	The force exerted on a pier or abutment by the flow of water around it. NOTE: This can include buoyancy forces. There are similarities with the drag and lift forces acting on an aerofoil.
Hydrostatic action	The force exerted by the pressure due to depth of water on an object in the water.
Invert protection	Protection of the river-bed under the bridge to prevent scour.
Impact loading	The loading applied to the bridge structure by the collision of debris carried by the flow or by a ship.

Terms (continued)

Term	Definition
Largest combined fluvial and tidal event for design	The largest combined event with a joint annual exceedance probability equivalent to the largest flood event for design.
Largest flood event for design	The largest flood event (fluvial or tidal) that should be used for the design.
Largest fluvial flood event for design	The largest fluvial flood event that should be used for the design with an annual exceedance probability equivalent to the largest flood event for design.
Largest tidal event for design	The largest tidal flood (surge) event that should be used for the design with an annual exceedance probability equivalent to the largest flood event for design.
Local scour	Scour caused by an acceleration of flow and its resulting vortices around an obstruction such as a pier or abutment. NOTE: Such scour only occurs in the immediate vicinity of the obstruction.
Lowest fluvial flow (Q95)	The lowest flow rate expected in a watercourse. NOTE 1: A Q95 flow is that which is exceeded 95% of the time. NOTE 2: For gauged sites the flow rate is typically derived from daily mean average flows. NOTE 3: The flow can also be estimated for ungauged catchments.
Mean flow	The average daily flow in a watercourse (calculated from gauge records as the mean average of the daily mean average flows). NOTE 1: The mean flow can also be estimated for ungauged catchments. NOTE 2: Typically, the annual mean flow is used although seasonal mean flows can be used where appropriate.
Mean spring tidal range	The range of water levels between the mean high spring tide and the mean low spring tide. NOTE: Spring tides are those in which the difference between high and low tide is greatest.
Natural scour	Long-term river bed elevation changes due to natural or man-induced causes within the reach of the river on which the structure is located.
Regime	Conditions under which a hydraulic process is occurring.
Relative flow depth	The ratio of obstruction size to flow depth. If this is large then local scour may be inhibited.
Rip-rap	Stone placed to prevent scour.

Terms (continued)

Term	Definition
Scour	The removal of sediment and hence reduction in river-bed level by flowing water. NOTE: It can be divided into components including: 1) natural scour (progressive aggradation / degradation); 2) general scour; 3) contraction scour; and 4) local scour.
Scour holes	The bed features caused by the reduction in bed level due to local scour.
Section mean velocity	The mean average flow velocity at a particular watercourse section.
Sub-critical flow	Flow conditions where the Froude number is greater than 1.
Surge event	The abnormal rise in sea water level during a storm over the sea, measured as the height of the sea water above the normal predicted astronomical tide. NOTE 1: The storm surge is caused primarily by a storm's very strong winds blowing over the surface of the sea, pushing sea water towards the coastline. NOTE 2: The height of the storm surge at any given location can depend on: 1) the angle of storm impact on the coast; 2) the intensity, size, and speed of the storm; and 3) the local sea bed profile.
Watercourse	A natural or artificial channel through which water flows, including a river, canal, estuary.

1. Scope

Aspects covered

1.1 This document shall be applicable to all new structures in or over rivers, estuaries and flood-plains.

NOTE 1 The assessment of existing structures for scour is covered by DMRB document BD 97 [Ref 11.N].

NOTE 2 Structures include bridges, piers, abutments, retaining walls, foundations, associated protection works, and new structural elements to existing structures, such as re-decking.

NOTE 3 Specific requirements for environmental, hydraulic, scour and structural designs of culverts are outside the scope of this document. See CD 529 [Ref 2.N] for culverts.

1.2 This document shall be applicable to permanent and temporary structures.

1.3 This document shall be applicable to the hydraulic aspects of the design and the studies required to support these design aspects.

NOTE 1 Hydraulic aspects of design include scour assessment, design of scour protection and hydraulic actions on structures.

NOTE 2 This document supplements the requirements for other aspects of structure design that are provided elsewhere in DMRB.

Implementation

1.4 This document shall be implemented forthwith on all schemes involving the design or construction of structures in or over rivers, estuaries and flood-plains on the Overseeing Organisations' motorway and all-purpose trunk roads according to the implementation requirements of GG 101 [Ref 6.N].

Use of GG 101

1.5 The requirements contained in GG 101 [Ref 6.N] shall be followed in respect of activities covered by this document.

2. Design process

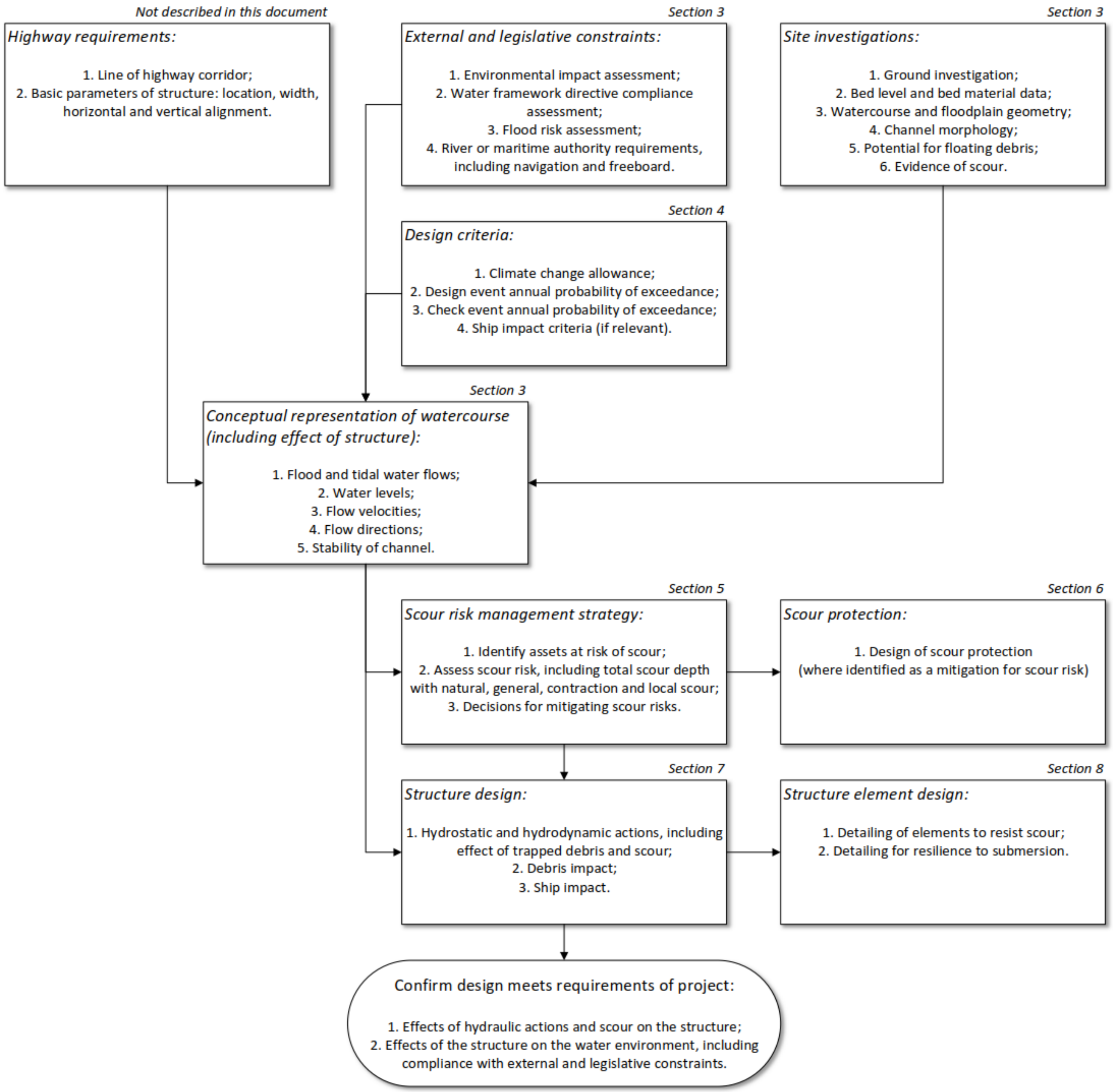
2.1 The design procedure shall include:

1) establishment of design principles (see Section 3);
2) determination of design criteria (see Section 4);
3) assessment of scour risk (see Section 5);
4) design of scour protection, where required (see Section 6);
5) calculation of hydraulic actions and checks of the structure under the effect of these actions (see Section 7);
6) design of specific elements of the structure (see Section 8).

NOTE 1 Design for hydraulic actions requires coordination between many different design and environmental disciplines.

NOTE 2 Figure 2.1N2 summarises the main considerations of the design procedure. Iteration between steps can be needed to achieve a satisfactory design, but for clarity, such iteration is not shown on the flowchart.

Figure 2.1N2 Summary of design procedure



2.1.1 Requirements for site investigation and geotechnical design should be specified to provide the information which will be needed to undertake the scour assessment and potentially the design of scour protection.

NOTE For example, information on bed material is needed to undertake the scour assessment, in addition to the usual geotechnical requirements to establish conditions for structural foundations.

3. Design principles

Design objectives

3.1 The hydraulic design of structures shall include:

1) an assessment of the hydraulic actions on the structure; and
2) an assessment of the effects of the structure on the water environment.

NOTE *Effects of the structure on the water environment include navigation, watercourse and coastal management and maintenance, water quality, flood risk, aquatic and riparian habitat and geomorphology including erosion and deposition processes. Specific requirements are provided later in this document.*

3.1.1 The assessment of the effects on the structure and the effects of the structure on the water environment should be commenced at the outset of the design process (for example, from the initial planning / concept design stage).

NOTE *The findings of the assessment of the structure and water environment can affect the location and form of the structure.*

3.2 The hydraulic design of structures shall have the following design objectives:

1) to reduce risks of structural failure and other adverse effects on the structure to as low as reasonably practicable (ALARP) under the effects of water flows and scour, for the design working life of the structure;
2) to minimise the adverse effects of constructing the structure on the existing water regime and to enhance this where realistic opportunities exist;
3) to maintain navigation access (where applicable);
4) to minimise adverse effects on flood risk.

3.2.1 The design objectives may be achieved through measures including the following:

1) allowing for natural channel movement, especially in mobile environments such as estuaries;
2) avoiding a need to realign the watercourse or channel through design of the road and structure alignment, unless this would improve the existing water environment;
3) minimising obstructions to flood flows from approach embankments, abutments and piers;
4) setting back abutments from the channel, preferably located outside the floodplain, and aligned with the natural flow direction;
5) using relief openings (to allow for flood flows and reduce afflux), guide banks and river training works to reduce adverse flow conditions at abutments (noting that over provision of flood relief openings can add excessive unnecessary cost to a scheme);
6) avoiding or reducing the number of piers across the channel;
7) aligning piers with the direction of flow and using streamlined pier shapes to decrease scour and reduce potential build up of debris;
8) positioning structure soffit levels above the general level of the approach roadways to reduce hydraulic forces acting on the structure in the event of overtopping;
9) using a smaller number of large openings rather than a larger number of smaller openings (for a given cross-sectional area of an opening, the greater the wetted perimeter, the greater the afflux);
10) considering the use of softer engineering for scour and erosion protection that allows for some continued channel movement, ecological habitat and water treatment, where appropriate and practical.

NOTE *The SEPA River crossings [Ref 6.I] guide for river crossings can be referred to for further guidance. LA 113 [Ref 10.N] also references additional sources of information.*

3.2.2 The design should both inform and be informed by relevant assessments for the scheme including:

1) environmental impact assessment LA 101 [Ref 5.N], including navigation;
2) water framework directive compliance assessment (see LA 113 [Ref 10.N]);
3) flood risk assessment (see LA 113 [Ref 10.N]).

3.2.3 Consultation should be undertaken with the relevant river or maritime authority, and other relevant authorities, commencing from the start of the design process.

NOTE Approvals and consents from relevant authorities can be required.

3.2.4 The design should aim to reduce the need for hard engineered solutions and consider alternative designs that avoid or reduce the need for protection.

3.3 The design shall achieve any specific required mitigation that is identified in the environmental impact assessment, water framework directive compliance assessment or flood risk assessment for the scheme.

Conceptual representation

3.4 The effect of the structure on the water environment shall be understood through development of an appropriate conceptual representation of the behaviour of the watercourse.

3.4.1 The zone of assessment covered by the conceptual representation should encompass the zone required for the scour assessment and for an understanding of the effect of the structure on the watercourse and of the hydraulic actions on the structure.

3.4.2 Appropriate conceptual representations may include any or all of the following approaches, depending on the complexity of the situation:

1) simple hand calculations and sketch plans;
2) 1- and/or 2-dimensional computational hydraulic models;
3) 3-dimensional computational fluid dynamics (CFD) or flow models;
4) sediment transport models;
5) physical hydraulic models.

NOTE 1 The simpler approaches (items 1-2 above) are likely to be adequate in normal cases where there is no undue complexity.

NOTE 2 Computational or physical modelling can be preferred in more complex situations, where simple hand calculations are unlikely to be accurate, or to refine the design. Examples of situations where computational or physical models can be more valuable include:

1) where flow patterns are complex such as at river confluences, large floodplains or urban environments;
2) where floodplains provide potential for significant storage and attenuation of flood flows;
3) in areas subject to both fluvial and tidal flows, where critical combinations can be difficult to define;
4) where there is a need to assess several different design scenarios and/or where conditions can change during an event, for example if a defence breach is a possibility;
5) where there is a need to demonstrate the effectiveness of proposed mitigation and protection features.

NOTE 3 In many cases 1- and/or 2-dimensional computational hydraulic models will be developed to inform the flood risk assessment for the scheme and can provide parameters to inform the structure design. These include information on flood extents and flow routes, water levels, afflux and flow velocities, which will generally be more accurate than obtained through simple calculations.

NOTE 4 A variety of computational hydraulic model packages exist that can provide more detailed assessment of flows around structures, sediment movement and scour. Computational hydraulic model packages can be useful where it is important to understand channel or sediment movement patterns either to inform the structure design or for environmental reasons; or where simpler models are not appropriate for the situation, for example at complex structures and large estuaries.

NOTE 5 *The more sophisticated modelling techniques can produce additional information which can assist the development of the design.*

NOTE 6 *Geomorphological studies can supplement the other approaches.*

NOTE 7 *Specialist geomorphologists can provide advice in respect of river processes and channel movement, particularly in especially mobile or complex environments.*

NOTE 8 *Physical models can also be used where preferred or where computational models are not sufficient to represent the channel or sediment movement processes. Physical models can however be more costly to develop and use than computational models, particularly if multiple scenarios need to be assessed.*

3.4.3 The conceptual representation of the behaviour of the watercourse should produce the following outputs for the applicable scenarios:

1) water levels;
2) flow velocities;
3) flow directions;
4) an assessment of the stability of the channel including natural scour (change over time including upstream and downstream of the proposed crossing).

3.5 The scenarios that shall be considered in the design are summarised in Table 3.5.

Table 3.5 Scenarios used for design

Scenario	Description	Typical values
Design event	The event which gives the largest value of the parameter under consideration for the events up to: 1) the largest fluvial flood event for design; 2) the largest tidal event for design; 3) the largest combined fluvial and tidal event for design; 4) other, more frequent, event for assessing natural scour.	1,2,3 - 0.5% annual exceedance probability, including normal climate change allowances; 4 - as determined from conceptual representation.
Check event	The event which gives the largest value of the parameter under consideration for the events up to the check event (more extreme than the design event). This can be a fluvial, tidal or combined event.	The larger of: 1) 0.1% annual exceedance probability event, including normal climate change allowances; 2) 0.5% annual exceedance probability event, including higher climate change allowances.
Low water level	As determined for the lowest fluvial flow and lowest astronomical tide	Q95 (fluvial flow)
Normal flow/level	Representative of normal conditions	Mean fluvial flow; Water levels within the mean spring tidal range.

NOTE *Requirements and guidance on the scenarios used for design are provided in Section 4.*

Site investigations

3.6 The information to be obtained from desk studies and site investigations shall include:

1) geotechnical data to inform the structure foundation design;
2) bed level and bed material data to inform the scour assessment;
3) watercourse and floodplain geometry;
4) an understanding of the channel morphology and stability, water levels and flows, and flood behaviour to inform the conceptual representation.

NOTE 1 Further guidance is provided in Appendix C to this document.

NOTE 2 Desk studies and site investigations can also determine:

1) expected type and size of floating debris (see Section 4);
2) evidence of scour (see Section 5).

Structure opening

3.7 The structure opening size(s) shall be determined.

NOTE 1 The structure opening size can be influenced by multiple factors including:

1) design flow capacity;
2) effect on flood risk;
3) sensitivity to flood events larger than the design event;
4) navigation requirements;
5) effects on ecology and the water environment (including scour);
6) susceptibility to blockage by debris;
7) design of structural elements.

NOTE 2 Determining the structure opening size can require an iterative process involving consultation with relevant authorities.

3.8 The minimum design flow capacity for the structure shall be agreed with the relevant authority and to meet any specific requirements identified in the environmental impact assessment, water framework directive compliance assessment or flood risk assessment for the scheme.

NOTE 1 The acceptable afflux at the structure will likely be dictated by potential for increased flooding upstream of the structure.

NOTE 2 The flow capacity of the structure can be dictated by other design factors. For example, a larger flow capacity can be provided to achieve navigation or to reduce scour and hydraulic loads on the structure.

NOTE 3 Flood flows do not always follow the alignment of the river channel and can require separate openings to be provided.

4. Design criteria

Design working life

4.1 The minimum design working life shall be as detailed in Table 4.1.

Table 4.1 Design working life

Element	Design life
Structures (including associated substructures)	As required in CD 350 [Ref 13.N]
Scour protection	The same as the applicable structure or lower as determined in the scour risk management strategy

4.1.1 Where a structure is temporary or has a design life less than that indicated in Table 4.1, then relaxations on the design criteria specified in this section may be agreed with the Overseeing Organisation and recorded in the Approval in Principle.

NOTE Relaxations can be applied, as examples, to:

1) *omit or reduce climate change allowance;*
2) *amend navigation requirements;*
3) *increase the value of annual probability of exceedance of the largest design event;*
4) *accept damage to the structure for some events, depending on the use of the structure.*

4.1.2 Where relaxations on design criteria are applied, the selection of appropriate relaxed design criteria and the structure design should be informed by the environmental impact assessment, water framework directive compliance assessment and flood risk assessment for the scheme.

NOTE Even if relaxations are applied for the structure, temporary or short-term adverse effects on the watercourse and surroundings can still need to be avoided or reduced.

Climate change

4.2 The assessment of design events and check events shall include appropriate climate change allowances for the lifetime of the structure.

4.2.1 The relevant authority should be consulted for advice on current climate change allowances.

NOTE Climate change research is ongoing and climate change allowances can change over time. At the time of writing current allowances are largely based on UKCP 09 Report [Ref 29.I] research but this has recently been replaced by UKCP18 [Ref 11.I]. Derived products, such as increases in peak river flows, can be published separately. The climate change scenarios used to derive published allowances can also change over time.

4.2.2 Climate change allowances applied to fluvial flows should be based on the 90th percentile estimate when these have been derived from a range of low to high emissions scenarios.

NOTE Allowances for Upper End increases in peak river flows for river basins in England are available on the Gov-UK site. FRA CCA [Ref 8.I]

4.2.3 Climate change (sea level rise) allowances applied to tide levels should be based on the 50th percentile when these have been derived for a high emissions scenario.

4.2.4 Where directed by the Overseeing Organisation, a higher climate change allowance should be applied as a check event.

NOTE For example, the H++ scenario in UKCP 09 Report [Ref 29.I] represents the upper limit of climate projections that are considered plausible and can be specified for critical infrastructure or overall resilience.

4.2.5 Climate change allowances should be applied to all parameters which are expected to have an influence on the design.

4.2.6 The climate change allowances for other parameters (that is, parameters other than changes in peak river flow and sea level rise) may be based on different emissions scenarios and/or probability percentile in consultation with the relevant authority.

NOTE 1 These other climate change allowances can include changes in rainfall intensity (which can be more applicable to small rivers), storm surge, wind speed and wave height in addition to peak river flows and sea level rise.

NOTE 2 Published climate change allowances for these other parameters are not necessarily updated in line with updates to peak river flow and sea level rise, and can be based on different underlying assumptions.

4.3 Where evidence supports an adjustment to the recommended climate change allowances given in this section this shall be agreed with the Overseeing Organisation, in consultation with the relevant authority.

Largest flood event for design

4.4 The largest flood event for design (that is, the lower limiting value of annual probability of exceedance) shall be taken as 0.5% annual probability of exceedance unless otherwise agreed with the Overseeing Organisation.

NOTE 1 The largest flood event for design can be different to that required in the flood risk assessment. The flood risk assessment can often need to consider larger events than the 0.5% annual probability event.

NOTE 2 Situations in which a more extreme flood event can be required by the Overseeing Organisation include:

1) *strategic crossings such as large estuary crossings;*
2) *structures where the diversion route is exceptionally lengthy;*
3) *routes where disruption would have extreme consequences, for example due to high traffic volumes;*
4) *disproportionate effects resulting from failure of the structure, such as increased flooding or failure of major utilities.*

NOTE 3 The return period of the flood event is distinct from the design working life of the structure and implies a certain probability of exceedance over the design working life of the structure. For example, a 0.5% annual probability fluvial event (200 year return period event) has an approximate 45% chance of occurring in a 120-year time period, compared with an approximate 11% chance for the 0.1% annual probability event (1000 year return period event). It is equally likely that a given flood event occurs in the first year of the life of the structure, compared with any other year.

Check event

4.5 The design flood check event (that is, the lower limiting value of annual probability of exceedance) shall be taken as the 0.1% annual probability of exceedance unless otherwise agreed with the Overseeing Organisation.

NOTE The check event is used to check for any unacceptable adverse effects on the structure for an extreme event.

4.6 Where a higher climate change allowance is established for the design, the check event shall be taken as the larger of:

1) the design flood check event above in clause 4.4, using the normal climate change allowance; or
2) the largest flood event for design, combined with the higher climate change allowance.

Design fluvial flows

4.7 The design calculations shall be based on a range of events, up to and including the largest flood event for design and the check event, to assess which events produce the worst effects considering different flow velocities and depths.

NOTE The reason for this is that in many rivers velocities can be high when flows are just within banks, and scour can be worse under these conditions than at higher flood discharge rates.

4.7.1 Fluvial flood flows should be estimated in accordance with current best practice, which includes:

1) FEH [Ref 7.I] statistical method and Science Report SC050050 [Ref 14.I];
2) ReFH2 method (ReFH 2.2: Technical Guidance [Ref 28.I]).

NOTE Guidance on applying these approaches can be updated frequently. The relevant authority can be consulted for guidance on current best practice.

4.8 Representative normal conditions shall be established.

4.8.1 Representative normal conditions may be taken as mean fluvial flow and water levels within the mean spring tidal range (where applicable).

NOTE 1 A broader understanding of representative normal conditions can be established by:

1) survey information;
2) analysis of river gauge data;
3) analysis of gauge data for similar catchments;
4) flow measurements at the site.

NOTE 2 Further guidance can be found in Appendix B.

Design tidal flows

4.9 The design calculations shall be based on a range of events, up to and including the largest flood event for design and check event, to assess which events produce the worst effects considering different flow velocities and depths.

NOTE 1 The largest tidal event can include astronomical and surge effects. Surge events can produce the highest water levels. However, astronomical tides can sometimes produce higher flows.

NOTE 2 In tidal situations the flow velocities can depend on the speed of incoming (flood) and outgoing (ebb) tides. The tidal flows can vary depending on the volume of the tidal prism, peak tide level and tidal range.

NOTE 3 The reduced atmospheric pressure in the eye of the storm, combined with high wind speed over the sea blowing towards the coast, could form large and fast propagating ocean storm surges which are able to inundate inland areas near the coast and cause severe damage to structures at or near to the coast. An example of such an extreme weather event occurred on Tues 04 / Wed 05 February 2014 at Dawlish on the south Devon coast.

4.9.1 Tidal flows may be estimated using the simplified method provided in Appendix B.

NOTE 1 The simplified method is unlikely to be suitable in large estuaries or where tidal flows are significant in comparison to fluvial flows at the site.

NOTE 2 Within estuaries, funnelling and surge effects can have a significant effect on tidal flows and levels which can have a significant effect on the accuracy of simple calculation methods. Hydraulic modelling can provide more accurate estimates to inform design.

4.9.2 Where there is any doubt over the suitability of the simplified method specialist advice should be obtained.

Combinations of fluvial and tidal flows

4.10 The assessment shall combine fluvial and tidal events where applicable.

NOTE 1 Depending on the structure location and structural element being assessed, the worst case conditions can occur for fluvial events, tidal events, or a combination of the two.

NOTE 2 It can be unduly onerous in the design of the structure to take the extreme events for both tide and fluvial flood simultaneously.

4.10.1 The assessment should include worst-case events for flows in upstream and downstream directions.

4.10.2 Where tidal flow effects are small in comparison to fluvial flow effects, or the overall flow is not significant in the design of the structure and associated works such as scour protection, then the following approach may be used:

1) for the downstream direction, add the maximum tidal flow to the maximum fluvial flow;
2) for the upstream direction, use the maximum upstream tidal flow.

NOTE Tidal flows can be considered small in comparison with fluvial flows if they lie well within the uncertainty range of the fluvial flow estimates. As a guide, tidal flows which do not exceed 10 - 20% of the fluvial flow will usually fall within the range of uncertainty.

4.10.3 Where tidal flow effects are significant compared to fluvial flow effects, and the overall flow is significant in the design of the structure and associated works, then a joint probability assessment should be undertaken to determine combinations of fluvial and tidal events with an annual probability of exceedance equivalent to the design event.

NOTE DEFRA report FD2308/TR2 [Ref 16.I] includes guidance on joint probability assessment for fluvial and tidal flood events.

Wave action

4.11 Appropriate parameters for the assessment of wave action on the structure shall be determined where applicable.

4.11.1 Where wave effects are significant the assessment approach should be agreed with the Overseeing Organisation.

NOTE 1 The assessment of wave action is outside the scope of this document.

NOTE 2 If wave effects are significant then other guidance can be referred to such as BS 6349-1-2 [Ref 7.N].

Design water level

4.12 The design water level(s) shall be determined for the scenario under assessment.

4.12.1 The design water level may be obtained using a hydraulic model or manually using appropriate hydraulic calculation methods.

NOTE 1 Appendix B provides further information on the estimation of water levels and flow depths which can apply in simple situations.

NOTE 2 Hydraulic models are likely to be preferred where an iterative calculation approach is necessary or in more complex situations. Examples of such situations include: changing flow conditions through the study reach, multiple flow routes or structure openings which need to be assessed, where the effects of storage and flow attenuation are significant, and where varying flows need to be considered.

4.13 Where applicable, tidal levels shall be established for the highest and lowest astronomical tides and for surge events equivalent to the design event and check event.

4.13.1 Astronomical tidal levels should be taken from the current Admiralty tide data, with appropriate corrections for the site location.

4.13.2 The relevant authority should be consulted regarding design tide levels and the estimation of tidal flows.

NOTE *Data about surge tide levels can be updated as new research is completed. The availability and source of this information can change over time as individual national environmental agencies or coastal authorities complete their own studies.*

4.13.3 The water level should be appropriate for the check being undertaken.

NOTE 1 *Water levels can vary rapidly through the structure due to energy losses. Water levels are often lower through the structure with higher levels upstream and downstream.*

NOTE 2 *Hydraulic modelling software packages can vary as to which water level is reported.*

NOTE 3 *Typically, calculated water levels will be for the approach to the structure, rather than beneath the structure. This is likely to be suitable for design in most cases, however further analysis can be necessary to calculate the water level acting on individual structural elements.*

4.14 Where a known water level at the site is higher than obtained for the design event or check event, then this water level shall be used in the design in the place of the calculated water level.

4.15 Where low water level conditions are significant in the performance of the structure, then the water level equivalent to the lowest fluvial flow (Q95) and the lowest astronomical tide, where applicable, shall be established.

NOTE 1 *Low water level conditions can be significant in the following cases:*

1) *exposure of scour protection;*
2) *where wave actions are significant and breaking waves occur in shallower water;*
3) *where the design relies on the presence of water to provide stability.*

NOTE 2 *Further guidance on the estimation of low flows can be found in Appendix B.*

Freeboard allowance

4.16 A minimum freeboard allowance of 600mm shall be provided above the maximum water level, either known or as calculated for the largest flood event for design.

4.16.1 Freeboard allowances should be calculated for the water levels immediately upstream or downstream of the structure (whichever results in the smallest freeboard), ignoring any draw-down effects on flows through the structure, and including an allowance for afflux caused by the structure.

4.16.2 Where wave effects are significant, then either:

1) the freeboard allowance should be increased; or
2) the structure should be designed for the load effects of waves on the structure.

4.17 Where a freeboard allowance greater than 600mm is proposed, then the freeboard allowance shall be agreed with the Overseeing Organisation and recorded in the Approval in Principle.

4.17.1 The relevant river, maritime and navigation authorities should be consulted to determine any greater requirements for freeboard allowances.

NOTE 1 *The requirements of the environmental impact assessment or flood risk assessment can often dictate the minimum freeboard allowances and soffit height.*

NOTE 2 *A larger freeboard can be required to manage the effects on the structure and/or the effects of the structure as described in relation to determining the structure opening size (see Section 3).*

Floating debris

4.18 The expected type and size of floating debris at the structure site shall be determined.

NOTE 1 *Debris can lodge and accumulate against structural elements situated in flood water flows. Debris can include vegetation such as trees and wood remains, but also household items and, in extreme events, vehicles. Debris can include vessels, for example, boats loosened from moorings.*

NOTE 2 *Floating debris can be carried by rivers for some distance during flood conditions. Debris can also be picked up from the wider floodplain.*

4.18.1 The investigation may be a simple desk-based assessment of OS maps, aerial and street-level imagery to review the size and characteristics of the river and catchment and the area through which it flows.

4.18.2 The investigation may include a site walkover to assess and verify the assumed debris loads.

4.19 The reduction in opening area through the structure due to trapped debris shall be included in the calculation of water level, flow, velocity and scour.

NOTE *Debris can cause a blockage in the opening area below the structure. The restriction in the opening area can result in an increase in water levels upstream of the structure and increased velocities through the structure opening. In turn this can lead to increased scour.*

4.20 The size of debris mat shall be determined in order to establish effects on the structure.

4.20.1 The size of debris mat should be based on a design log length.

4.20.2 The design log length should be taken as the lowest of:

1) the width of the channel upstream of the site;
2) 9m plus one quarter of the width of the channel upstream;
3) the maximum length of sturdy logs.

4.20.3 The maximum length of sturdy logs in the UK may be taken as 24m.

4.20.4 The design log length may alternatively be determined by field observations.

4.21 A minimum single-item debris mass of m=3000 kg shall be used to check for debris impact on the structure.

4.21.1 A greater value of single-item debris mass may be required depending on the results of the investigation.

Ice

4.22 The effects of ice shall not be included in design unless specified by the Overseeing Organisation for the particular project.

4.23 Where the effects of ice are to be assessed, the appropriate design criteria shall be agreed with the Overseeing Organisation.

NOTE *It is unusual for ice loading to be a significant problem in the UK.*

Ship impact

4.24 Where applicable, the ship impact design criteria shall be agreed with the Overseeing Organisation, after discussions with the relevant navigation authority, and assessment of appropriate design criteria.

4.24.1 The ship impact design criteria should include:

1) the mass, speed and type of critical ship;
2) flow conditions and water levels;
3) design approach to be followed.

4.25 Where applicable, the water levels, flows and speeds shall be checked separately for the case with a vessel passing through the structure opening under normal flow conditions.

NOTE 1 *Where a vessel travels against the flow and causes significant blockage of the opening, then there can be significant increases in water velocity and depth as the water flows around the vessel, which can increase the forces on the structure and the risk of scour.*

NOTE 2 The forces on the structure and risk of scour due to passage of ships can be less onerous than during the flood event. Passage of ships is not considered to coexist with the flood event.

Dredging

4.26 Where applicable the effect of dredging on bed levels and flows to be included for in the design shall be agreed with the Overseeing Organisation, after discussions with the relevant navigation authority.

5. Scour assessment

General

5.1 An assessment of the risk of scour shall be undertaken for all proposed structural elements that are:

1) in a watercourse;
2) near the banks of a watercourse.

NOTE 1 In or near the banks of a watercourse includes structures that rely on support or protection from the river bank, and those structures that are on land that could be subject to frequent inundation.

NOTE 2 Scour can include natural, general, contraction and local scour.

NOTE 3 Structural elements can include:

1) foundations including piers and abutments,
2) associated structures such as wing walls and revetments, and
3) flood relief openings.

NOTE 4 The assessment of existing structures for scour is covered by BD 97 [Ref 11.N].

5.1.1 The assessment of the risk of scour should be undertaken using the conceptual river model developed using guidance given in Section 3.

5.2 Where the risk of scour is identified, the magnitude and extent of scour shall be quantified.

Scour risk management strategy

5.3 A scour risk management strategy shall be prepared.

NOTE 1 The Overseeing Organisation can include the strategy in technical assurance processes.

NOTE 2 The scour risk management strategy is a document aimed at introducing the consideration of risk assessment and mitigation for scour within the design process.

NOTE 3 The scour risk management strategy is initially a design decisions document rather than an operational management document. However, it can be updated during the life of the structure to reflect management and maintenance changes.

NOTE 4 Production of the scour risk management strategy can support and simplify the future maintenance of the structure and the application of BD 97 [Ref 11.N] and CIRIA C742 [Ref 19.I].

NOTE 5 Guidance on preparation of a scour risk management strategy can be obtained from the CIRIA C742 [Ref 19.I].

5.3.1 The scour risk management strategy should define plans to anticipate, assess and mitigate the consequences of scour.

NOTE Anticipation is about identifying assets at risk of scour. Assessment is about quantifying the risk. Mitigation is about developing strategies to reduce the risk.

5.3.2 The strategy should identify structural elements that could be at risk of scour and scour hazards that could arise due to changes to the structure or watercourse.

5.3.3 The strategy should include an assessment of:

1) the importance of the structure;
2) consequences of failure, including the risk to public safety and infrastructure;
3) ease of monitoring, inspection and repair;
4) where a less onerous design event is selected for the design of training, scour protection or impact protection works, the implications of the chosen design event; and

5) the environmental impact of the recommended scour mitigation measure.

5.3.4 The strategy should include the outcome of the scour risk assessment, including:

1) a description of the methodology used to undertake the scour risk assessment;
2) the scour parameters to be used in the structural design;
3) justification for the selection of these parameters;
4) the uncertainty associated with the parameters.

5.3.5 The strategy may be used as a means of recording a low risk of scour and the reasons why no further action is needed.

5.3.6 The strategy should set out the proposed method for mitigating against the effects of scour at each structural element, including the justification for the choice.

5.3.7 The choice of scour mitigation method may be one of the following:

1) design for the predicted scour depth, such as by deepening the foundation;
2) include measures to reduce scour:
    a) relief openings within the embankment;
    b) abutments set back from the channel;
    c) structural alignment with the natural flow direction;
    d) streamlining of pier and abutment shapes to reduce local turbulence;
    e) use of inclined spill-through abutments rather than vertical wall abutments;
3) include scour protection measures;
4) exceptionally, operational measures such as planned interventions or more frequent inspections.

5.3.8 The strategy should identify vulnerable structural elements which need to have scour protection.

5.3.9 The strategy may be used as a means of recording residual risks of scour and any operational requirements to mitigate them.

NOTE Operational measures are non preferred and are considered to be a last resort for new designs.

Methodology for scour assessment

5.4 The design event used for the assessment of natural scour shall be defined as part of development of the conceptual model in order to allow for the characteristics of the river.

NOTE 1 Natural scour depends on the overall characteristics of the river rather than on effects local to the structure.

NOTE 2 Natural scour tends to be governed by more frequent events and can be better assessed using higher annual probability events, that is, a different event compared to those used for general and local scour.

5.5 The scour assessment shall include an evaluation of the risk of natural, general, contraction and local scour.

5.5.1 The scour assessment methodology should be appropriate to the scale, complexity and consequences of failure of the structure.

NOTE 1 Many scour assessment methodologies, particularly simplistic approaches, can have limitations on use.

NOTE 2 Different approaches to scour assessment can produce significantly different results for the same case.

5.5.2 Where numerical or physical modelling is used for scour assessment, then specialist advice should be obtained.

5.5.3 The scour assessment should be verified using evidence of scour from field measurements and historic evidence, unless the effort to obtain the evidence is disproportionate to the effect on the design.

NOTE Evidence of scour can be obtained, as examples, from:

1) *historic maps indicating changes to the river over time;*
2) *river surveys;*
3) *historic evidence at an existing structure in the vicinity of the new structure;*
4) *monitoring of scour at large temporary works such as cofferdams.*

5.5.4 Historic and desk-study evidence should be reviewed for indications of irregularities.

NOTE Irregularities such as obstructions or small promontories can influence flow conditions or bed levels at the structure site.

5.6 The assessment of general, contraction and local scour shall be carried out at a range of events up to the design event and check event as defined in Section 3.

5.6.1 The design event and check event for scour at a structural element should be taken as the event which is determined to give the highest velocity at that structural element.

NOTE Scour is typically governed by velocity rather than depth. The velocity can vary across the width of the watercourse, hence can be different for different structural elements. Different design events can be critical for different structural elements, depending on the nature of the channel flow and floodplain during different return period events.

5.6.2 A less onerous design event than the event chosen for the structure may be selected for the design of river training, scour and impact protection works, provided that:

1) the principal structural elements, such as the structure foundation, piers and abutments, are not compromised;
2) the selection of a less onerous design event is described in the scour risk management strategy.

5.7 The zone of assessment shall be defined for the scour assessment.

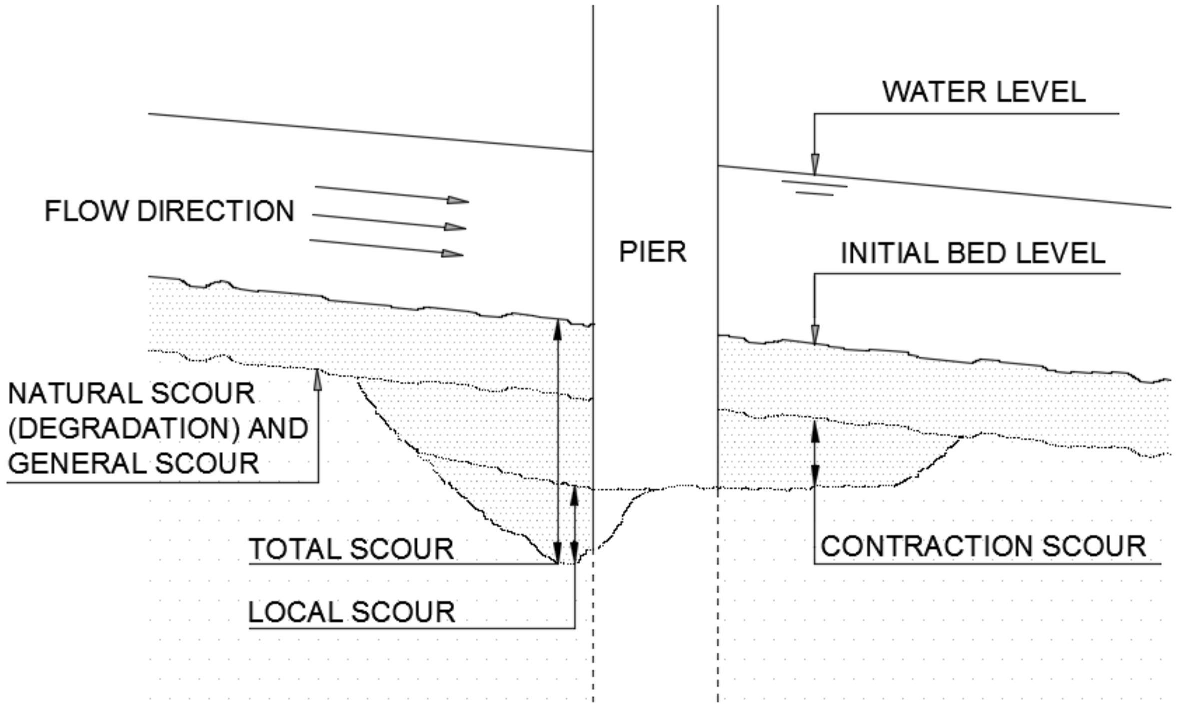
5.7.1 The zone of assessment should be appropriate for the scour assessment methodology used.

5.7.2 The scour assessment should include the wider implications of the structure on the up and downstream reaches of the river including the likelihood of scour or sediment deposition.

5.8 Where the risk of natural, general, contraction, or local scour is identified, and the magnitude and extent of total scour is required to inform the proposed scour mitigation measures, the magnitude and extent of total scour shall be quantified at each structural element.

5.8.1 Local and contraction scour should be superimposed on general and natural scour in order to determine total scour as shown below unless there is clear justification why these effects are not coincident.

Figure 5.8.1 Relationship between natural, general, contraction, local and total scour



5.8.2 The long-term equilibrium depth for scour may conservatively be used in design.

NOTE 1 *Scour develops over time and the equilibrium depth represents the long-term case which is the greatest value and hence the most onerous.*

NOTE 2 *The rate of scour is different for various bed materials and the time to reach the equilibrium scour depth can be:*

1) *for non-cohesive materials in hours;*
2) *for cohesive or cemented bed materials in days;*
3) *for glacial tills, sandstones and shales in months;*
4) *for limestones in years;*
5) *for dense rock formations in centuries.*

5.8.3 Where the duration of the design event for scour is short compared with the duration taken to develop the full scour depth, then a more detailed evaluation of scour development over time may be undertaken.

NOTE *Scenarios where an evaluation of scour development over time can be appropriate include tidal locations, scour in cohesive sediment and rivers subject to flash-floods.*

Natural scour

5.9 The risk of natural scour shall be assessed.

NOTE *Natural scour can be caused, for example, by degradation, bend scour, confluence scour and channel migration. Aggradation can increase the bed levels by sediment deposition and hence counteract the effects of natural scour.*

5.9.1 The risk of natural scour should be assessed with the conceptual model of the river, developed as described in Section 3.

5.9.2 The assessment of the risk of natural scour should account for whether the watercourse is stable or in an unstable state.

NOTE An unstable river is one that is undergoing natural change.

5.9.3 The natural scour expected over the design life of the structure may be quantified as part of a geomorphological study of the river.

General scour

5.10 The risk of general scour shall be assessed.

NOTE General scour can occur during high flows and relates to an overall change in bed levels within the watercourse, irrespective of the presence of a structure.

5.10.1 The risk of general scour should be assessed with the conceptual model of the watercourse, developed as described in Section 3.

5.11 Where there is a risk of general scour, the depth and extent shall be quantified.

NOTE Guidance on quantifying the risk of general scour is provided in Appendix A.

Contraction scour

5.12 The risk of contraction scour shall be assessed.

NOTE Contraction scour occurs in a confined section of a watercourse and results in a lowering of the bed level across the width of the watercourse.

5.12.1 The output of the assessment of contraction scour should be the depth of scour and likely extent.

NOTE 1 There are many available methods for quantifying contraction scour.

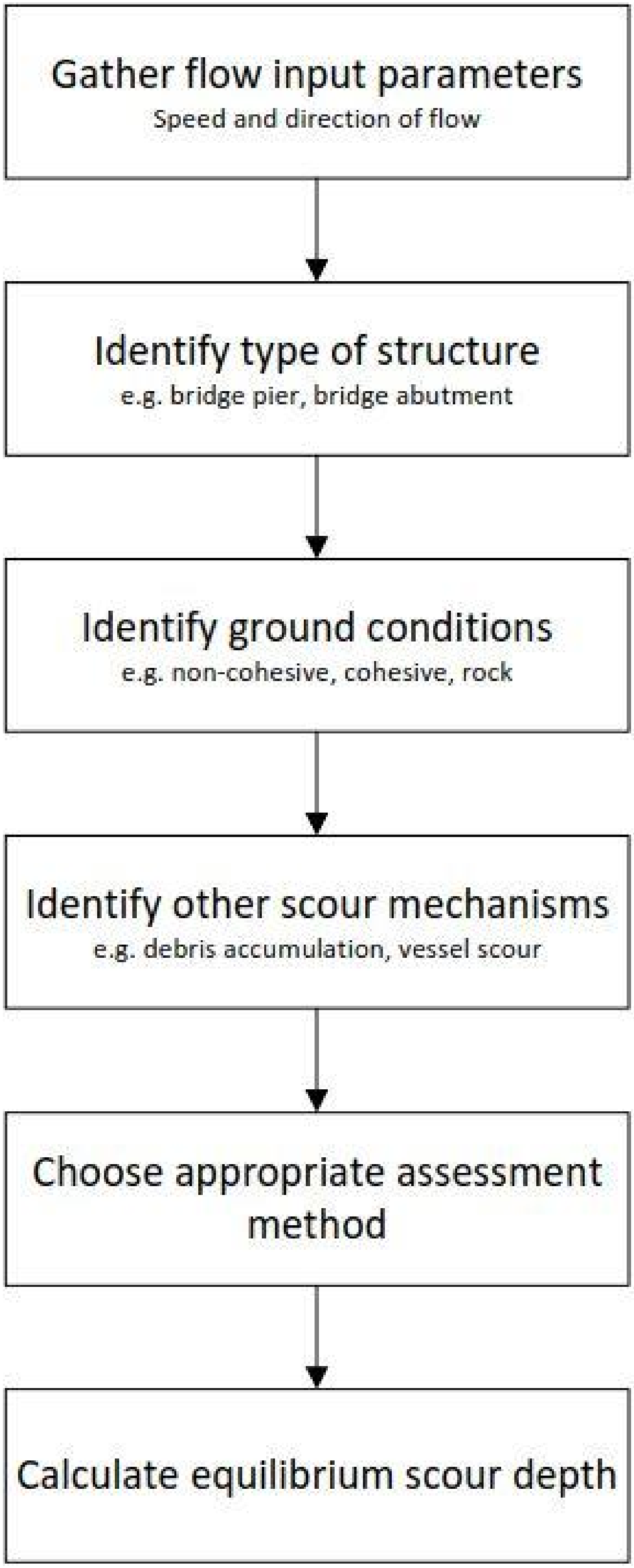
NOTE 2 An example methodology suitable for simple cases is given in Appendix C.

Local scour

5.13 The risk of local scour shall be assessed.

NOTE Local scour can be caused by short-term scour at a structural element during the largest flood event for design, the largest tidal event for design and combinations of flood and tidal events where relevant.

Figure 5.13N Steps to calculate local scour



5.14 The risk of other mechanisms that can vary the depth or distribution of local scour or its evolution over time shall be assessed.

NOTE *Other mechanisms which can cause variations in local scour are:*

1) *ice accumulation;*
2) *debris accumulation;*
3) *vessel scour.*

Bridge piers

5.15 Where there is risk or evidence of local scour at a bridge pier, the depth of local scour shall be quantified.

5.15.1 The assessment of local scour depth due to flow at bridge piers should be determined with consideration of the following parameters:

1) pier shape, size and structural arrangement;
2) pier orientation to flow;
3) flow characteristics including water depth and velocity;
4) geotechnical conditions.

NOTE *To estimate the scour where debris accumulation is a problem, groups of columns can be designed as though they were solid piers.*

5.15.2 The assessment of local scour depth should take into account any changes in foundation shape and size that occur as the scour hole is formed.

NOTE *The exposure of buried structural elements such as pile-caps that are larger than the pier can increase the risk of scour.*

5.15.3 The output of the local scour depth assessment at bridge piers should include:

1) maximum scour depth at the face of the pier;
2) extent and shape of scour hole at the pier.

5.15.4 The method for assessing scour should be appropriate for the geotechnical conditions at the site.

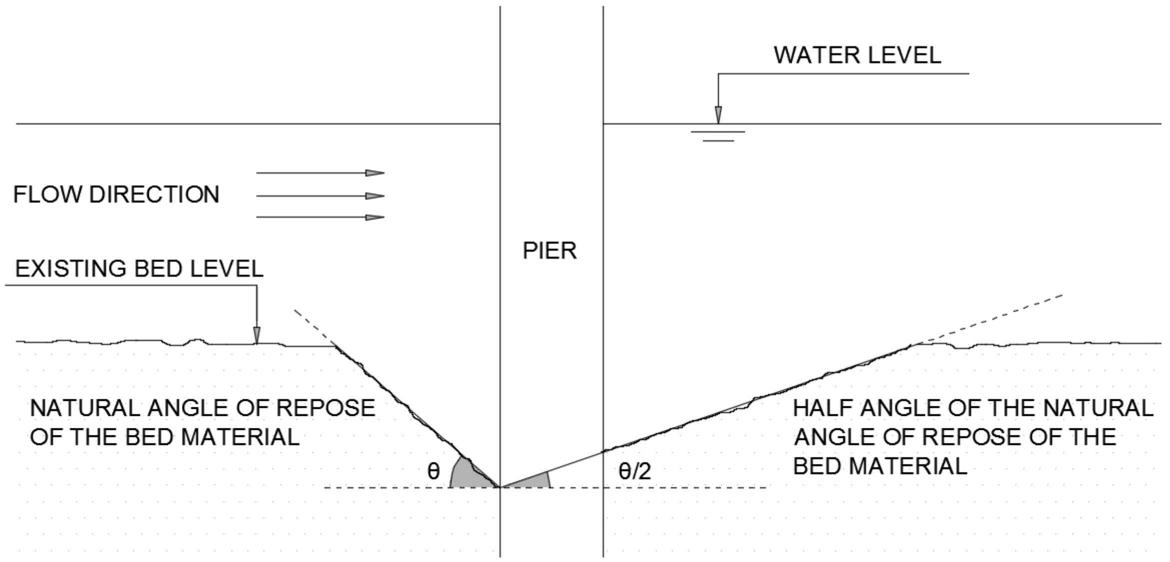
5.15.5 In non-cohesive soils, the following procedure may be used to calculate the depth of local scour, following the procedure given in the CIRIA C742 [Ref 19.I]:

1) determine the depth-averaged critical threshold velocity for the surface bed material, U_{TC} ;
2) for the design event, determine whether live-bed or clear water scour will occur;
3) calculate the equilibrium depth of local scour, Y_s .

5.15.6 In non-cohesive soils, the dimensions of the scour hole may be obtained by assuming:

1) the upstream slope of the scour hole is equal to the natural angle of repose of the bed material;
2) the downstream slope is equal to half the angle of repose;
3) where the pier is circular or elliptical, the equilibrium scour depth occurs at the midpoint of the upstream face of the structure;
4) where the pier is polygonal, the equilibrium scour depth occurs at the corner furthest upstream;
5) where the pier is rectangular, the equilibrium scour depth occurs at the midpoint of the upstream face where this is perpendicular to the flow or the corner furthest upstream where it is not.

Figure 5.15.6 Typical upstream and downstream slopes of a scour hole in non-cohesive soil



5.15.7 In cohesive soils and weathered rock, an estimate of equilibrium scour depth may be obtained using the method for non-cohesive soils, as outlined in Appendix A.

NOTE 1 The results of these methods are approximate. Scour in cohesive materials typically progresses more slowly than in non-cohesive materials, so it possible that the equilibrium scour depth is not reached during a flood or even over the life of the bridge.

NOTE 2 The shape of a scour hole in cohesive soil will differ to the shape in non-cohesive soil. The location of maximum scour depth will vary depending on soil properties (cohesiveness and degree of saturation) and shape. Typically scour in cohesive soils is more pronounced at the sides of a pier than it is for scour in non-cohesive soils.

5.15.8 Where a more refined estimate of scour depth is needed or the size of the scour hole needs to be established, then specialist guidance should be sought for scour in cohesive soils and weathered rock.

NOTE Guidance for determining the depth of scour in cohesive soils and weathered rock can be obtained from the Manual HEC 18 - FHWA-HIF-12-003 [Ref 10.I].

5.15.9 The risk of overlapping local scour should be considered in the scour assessment.

NOTE Local scour at pier, e.g. groups of piles, can overlap one another in some instances, which can increase total scour.

Bridge abutments and training walls

5.16 Where the risk of local scour at a bridge abutment has been identified, the scour depth shall be quantified.

5.16.1 The assessment of local scour depth at bridge abutments should be determined with consideration of the following parameters:

1) abutment length;
2) abutment shape;
3) geotechnical conditions;
4) flow characteristics including water level and velocity.

5.16.2 The methods for assessing local scour depth at bridge abutments may be used to determine the local scour depth at training walls.

5.16.3 The output of the local scour depth assessment at bridge abutments should include:

1) maximum scour depth at the face of the abutment;
2) extent and location of scour holes.

NOTE Guidance on the extent and location of scour holes can be obtained from CIRIA C742 [Ref 19.I].

5.16.4 In non-cohesive soils, the following procedure may be used to calculate the equilibrium depth of local scour, following the procedure given in the CIRIA C742 [Ref 19.I], considering the abutment as a half-pier:

1) determine the depth-averaged critical threshold velocity for the surface bed material U_{TC} ;
2) for the design event, determine whether live bed or clear water scour will occur;
3) calculate the equilibrium depth of local scour, Y_s .

5.16.5 Local scour in cohesive bed material and rock should be determined as for bridge piers.

5.16.6 The risk of overlapping scour should be considered in the scour assessment.

NOTE Local scour at an abutment can overlap with local scour at a pier, depending on proximity. Overlapping local scour can increase total scour.

Local scour at other structures

5.17 Local scour due to flow or other mechanisms shall be evaluated at all other structures associated with the highway structures, including training and protection works, pipeline crossings, rigid aprons and temporary structures.

NOTE Local scour assessment methods given in the CIRIA C742 [Ref 19.I] can be used to calculate scour depth at such structures.

Scour caused by other mechanisms

5.18 The risk of scour caused by mechanisms shall be assessed.

NOTE Other causes of scour can include:

1) vessel scour;
2) debris flow.

5.19 Where additional causes of scour are found to be likely, the resulting scour depth for each cause shall be quantified and added to total scour due to fluvial and tidal action unless it can be clearly justified why this is unnecessary.

5.19.1 Vessel scour may be quantified using guidance given by BAW CoP 2010 [Ref 3.I].

5.19.2 Where scour due to debris flow (very high sediment concentration) is significant, then it should be quantified using specialist guidance.

NOTE Debris flows are known as 'mud flows' when the sediment is fine material. These events can occur due to failures in saturated soils in steep river banks after periods of prolonged rainfall, although in the UK the volumes involved are rarely sufficient to cause a problem.

6. Scour protection design

Principles

6.1 Where the scour risk management strategy specifies scour protection as the mitigation method for scour, then scour protection shall be designed in accordance with this section.

NOTE 1 The following methods can be used to protect against possible scour:

1) *rigid methods:*
   a) *paved inverts;*
   b) *enlargements or plinths to piers and abutments;*
   c) *sheet piling;*
   d) *gabion and grouted mattresses;*
2) *flexible methods:*
   a) *stone mattresses (rip rap);*
   b) *grout bags;*
   c) *articulated concrete mattresses;*
   d) *grass mats;*
   e) *geotextiles;*
   f) *willow mats and similar products.*

NOTE 2 'Flexible' scour protection, which accommodates bed movement, can be more environmentally beneficial than rigid methods.

6.2 The scour protection shall be designed for the design events defined for scour protection in the scour risk management strategy.

NOTE The scour risk management strategy can define a different probability of exceedance for the design of the scour protection compared to the the probability of exceedance for the design of the overall structure.

6.3 The scour protection shall be designed to prevent degradation of the scour protection under the effects of:

1) debris accumulation;
2) vessel scour, where this is a risk.

NOTE The methods for calculation of the effects of these scour mechanisms on scour protection are available in the references given for scour assessment in Section 5.

6.4 Scour protection shall protect the at-risk structural elements from the effects of scour.

NOTE The scour risk management strategy (see Section 5) identifies vulnerable structural elements which need to have scour protection.

6.4.1 The extent of scour protection should be sufficient to prevent lowering of the bed level that would have an effect on foundation stability.

NOTE 1 Some bed lowering is acceptable if it is proven that there is no effect on foundation stability.

NOTE 2 Guidance on the minimum extent of scour protection is given in CIRIA C742 [Ref 19.I].

6.4.2 The extent of scour protection should be sufficient to prevent degradation of the river bed at the structure element due to the effects of:

1) natural scour;
2) general scour;
3) contraction scour;
4) local scour;

5) debris accumulation;
6) vessel scour.

6.4.3 The scour protection should be designed to prevent loss of the bed material beneath itself.

NOTE Bed material can be lost through the scour protection if there are insufficient measures for retention.

6.4.4 The scour protection should be designed to prevent undermining and failure at its edges.

6.5 The choice and alignment of scour protection shall be designed to encourage smooth flow transitions around the structural elements and scour protection itself.

6.6 The elevation of the scour protection design shall allow for natural, general and contraction scour.

6.6.1 The top of scour protection should be placed below the level of natural, general and contraction scour. This includes bed-forms, such as dunes, which can be naturally present on the river bed.

6.6.2 Where it is not possible to place scour protection below the level of natural, general and contraction scour, the design of scour protection should allow for the predicted bed movement with the scour protection in place.

Scour protection methods

6.7 The choice of scour protection method(s) shall be stated in the scour risk management strategy.

6.7.1 The choice of scour protection method for a structural element should be based on an assessment of the following site specific factors:

1) flow during the design event(s);
2) environmental considerations that may limit certain forms of construction;
3) natural scour, especially channel stability, which may promote the need for flexible options;
4) access constraints, including if construction is in the dry;
5) safety of construction and maintenance;
6) availability of materials; and
7) cost of construction and maintenance.

NOTE 1 Stone mattresses (rip rap) are a commonly used form of scour protection. The suitability of this solution for a particular project depends on consideration of the factors listed above.

NOTE 2 Design considerations for stone mattresses are given below. Design guidance for alternative scour protection methods is given in CIRIA C742 [Ref 19.I] and in the clauses below.

6.7.2 The requirements for scour protection should be included in the contract-specific requirements for the structure.

Stone mattresses (rip rap)

6.8 Stone mattresses shall be designed to be statically stable for the design events specified for scour protection.

NOTE Some degree of movement can be allowed under larger events if defined within the scour risk management strategy.

6.8.1 The size of armour stone should be chosen based on an assessment of the following parameters:

1) stability under the flow velocity and conditions (degree of turbulence) at the proposed location of the scour protection;
2) the location of the scour protection (bed or bank);
3) the properties of the armour stone, especially density and shape.

NOTE 1 Guidance on the calculation of the main armour material size for a stone mattress at a pier or an abutment is available in CIRIA C742 [Ref 19.I].

NOTE 2 Standard gradings are given in BS EN 13383 [Ref 26.I].

6.9 The stone mattress system shall be designed to retain the riverbed material beneath it.

6.9.1 Where direct retention of the underlying bed material by the main armour is not possible, granular filter layers and/or geotextile filters should be introduced.

6.10 Stone mattresses shall be designed to resist edge scour.

6.10.1 The edges of stone mattresses should be embedded or incorporate a falling toe in order to resist the effects of edge scour.

6.11 The material chosen for the main armour layer shall be durable, resistant to breakage and resistant to freeze/thaw effects where this material will be exposed at low water levels (including river banks).

NOTE Guidance on suitable specifications are given in BS EN 13383 [Ref 26.I].

Other forms of scour protection

6.12 Rigid scour protection methods such as reinforced concrete plinths and sheet piling shall be designed in accordance with CD 350 [Ref 13.N] supplemented by the overall requirements of Section 6.

6.13 Where flexible methods are proprietary products, then the design shall be undertaken in accordance with the manufacturer's requirements and recommendations.

6.14 Where flexible methods are not proprietary products, then the design shall be undertaken in accordance with methods appropriate for the protection method.

6.14.1 The design of flexible methods may be based on case studies.

7. Structure design

Structural and geotechnical basis of design

7.1 Structures subject to hydraulic actions shall be designed in accordance with CD 350 [Ref 13.N] supplemented by the requirements of this section.

NOTE CD 350 [Ref 13.N] requires that Eurocodes are used for the design of new highway structures.The Eurocodes do not adequately define hydraulic actions on structures for UK conditions.

7.2 Structures subject to hydraulic actions shall be designed for the following actions which supplement those specified in Eurocodes:

1) water actions:
   a) hydrostatic forces;
   b) hydrodynamic forces: drag and lift on submerged structural elements;
   c) hydrodynamic forces: drag on trapped debris;
2) impact forces due to debris;
3) where instructed, impact forces due to ship traffic.

7.3 Other permanent actions and other variable actions such as traffic, wind, temperature, shall be applied in combinations in accordance with BS EN 1990 [Ref 4.N].

Classification of actions

7.4 Water actions shall be applied as a variable action, Q, other than for the extreme high water design situation.

NOTE Different values of the water action, Q, are needed in the different design situations.

7.4.1 Water actions should be applied as the leading or accompanying variable action in combination with other variable actions such as traffic, wind, and temperature, in order to generate the most onerous load effects on each structural element.

7.5 Water actions corresponding to the check event at the extreme high water design situation shall be applied as an accidental action, A.

7.6 Where a quasi-permanent value of water action is used, then the water action shall be assessed directly based on the mean annual flow and the mean tidal level.

7.7 Impact forces due to debris impact and ship impact shall be applied as an accidental action, A.

Design values of actions

7.8 Partial factors shall be applied in accordance with Table 7.8.

Table 7.8 Design values of actions

Ultimate limit state	EQU (Set A)	STR/GEO (Set B)	STR/GEO (Set C)
Supplementary to UK National Annex to BS EN 1990 [Ref 4.N]	Table NA.A2.4(A)	Table NA.A2.4(B)	Table NA.A2.4(C)
Water actions	$\gamma_Q = 1.50$	$\gamma_Q = 1.50$	$\gamma_Q = 1.30$

Representative values of actions

7.9 Representative values of the variable actions shall be applied in combinations of actions in accordance with BS EN 1990 [Ref 4.N].

7.9.1 Values of ψ factors should be taken in accordance with Table 7.9.1.

Table 7.9.1 Values of psi factors

	ψ_0	ψ_1	ψ_2
Water actions	0.8 (or directly determine the action as $Q_{combination}$)	0.8 (or directly determine the action as $Q_{frequent}$)	ψ_2 is not defined, use $Q_{quasi\text{-}permanent}$(see Note)
NOTE: The quasi-permanent value of water action is directly assessed, in accordance with the clauses above about classification of actions. The mean value of water action cannot be computed using a ψ factor from the design event water action.			

7.9.2 Values of $Q_{combination}$ and $Q_{frequent}$ may be directly determined by considering a water event with an appropriate probability of exceedance, instead of factoring the characteristic value Q_k by ψ_0 or ψ_1 .

NOTE Direct determination of the values of actions involves additional calculations since more scenarios need to be considered using different probabilities of exceedance. However, direct determination is likely to result in lower values of actions, since the values of 0.8 given in Table 7.9.1 are intended to be conservative but easy-to-apply values.

7.9.3 Where values of $Q_{combination}$ and $Q_{frequent}$ are directly determined, then appropriate probabilities of exceedence for the water event should be agreed with the Overseeing Organisation.

Design situations

7.10 The following design situations shall be verified:

1) high water levels with scour and trapped debris;
2) low water levels with scour;
3) extreme high water levels (check event) with associated scour;
4) debris impact at high flow velocity;
5) where instructed, ship impact;
6) construction and repair.

NOTE 1 The same design situations apply to temporary structures, for which a reduced return period design event can be specified in accordance with Section 3.

NOTE 2 A summary of the design situations is provided in Table 7.10N2. Full details are provided in the requirements and advice below.

Table 7.10N2 Summary of design situations

Design situation	Classification of design situation	Water as variable action Q_k	Accidental action A_d	Scour	Trapped debris	Limit state checks
High water level	Persistent	Water level and flow based on design event	N/A	Scour applicable to design event including trapped debris	Yes	ULS and SLS
Low water level	Persistent	Omit or determine directly based on low water level	N/A	As for high water level	No	ULS and SLS
Extreme high water level	Accidental	Treat water as accidental action	Water level and flow based on the check event	Scour applicable to check event	No	ULS
Debris impact	Accidental	Level and flow based on design event (maximum velocity). Apply as leading variable action using the frequent value	Debris impact force	As for high water level	No	ULS
Ship impact	Accidental	Normal water level/flow. Apply as leading variable action.	Ship impact force	As for high water level	No	ULS
Construction / repair	Transient	Water level and flow based on design event (option for reduced return period)	N/A	Dependent on the obstruction to flow present in the design situation	Yes (option to justify omission)	ULS and SLS

Figure 7.10N2a High water level design situation

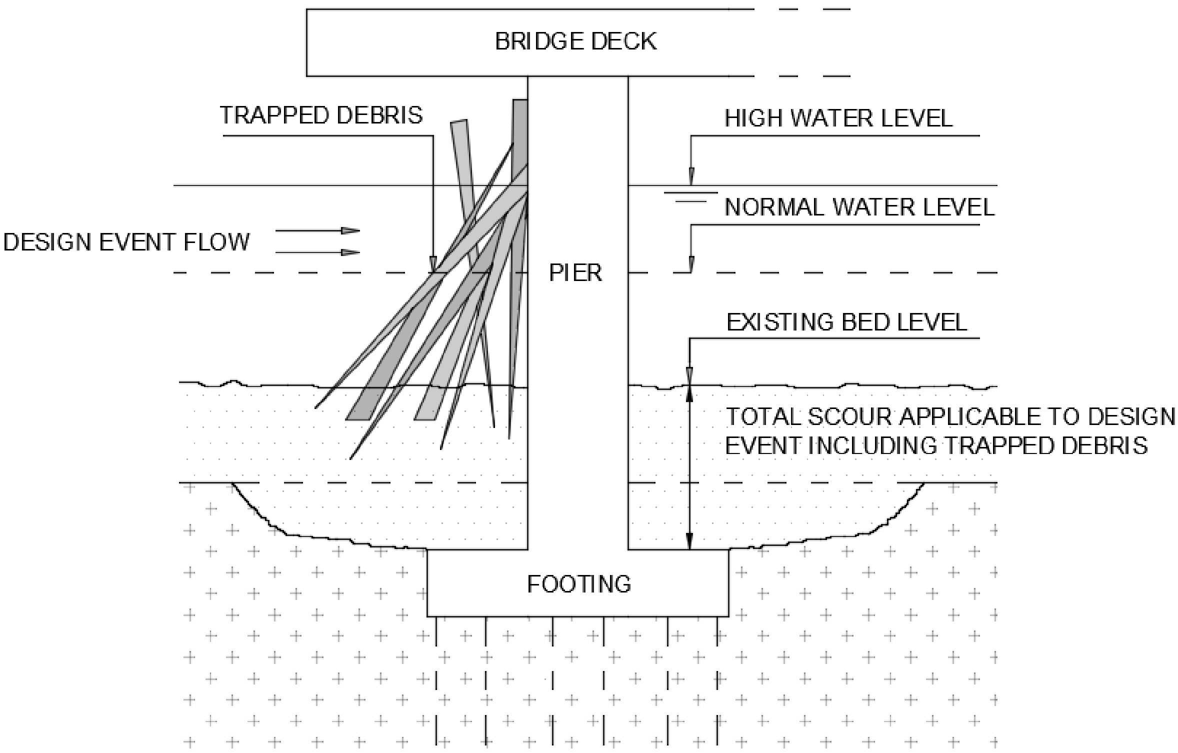


Figure 7.10N2b Low water level design situation

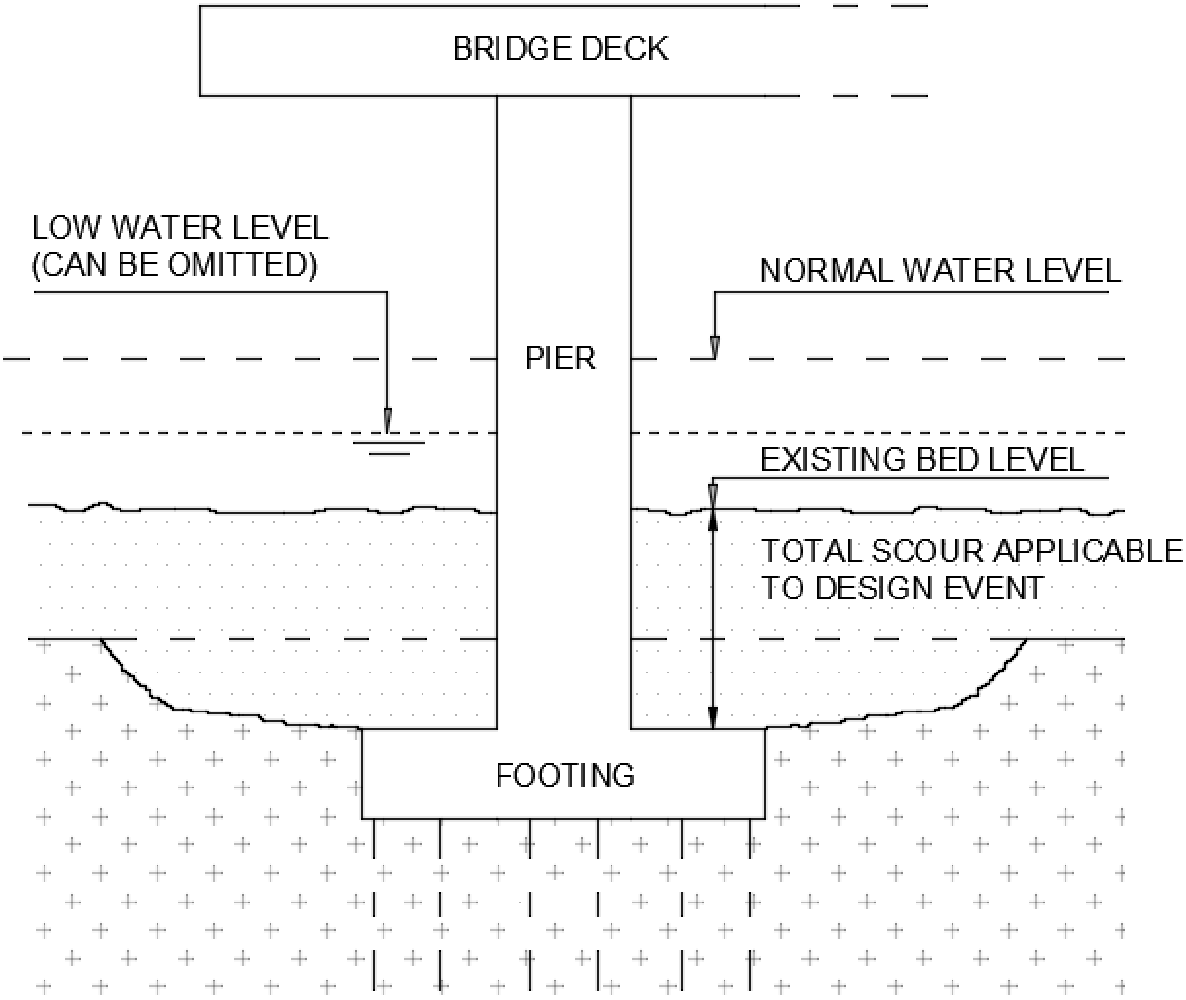


Figure 7.10N2c Extreme high water level design situation

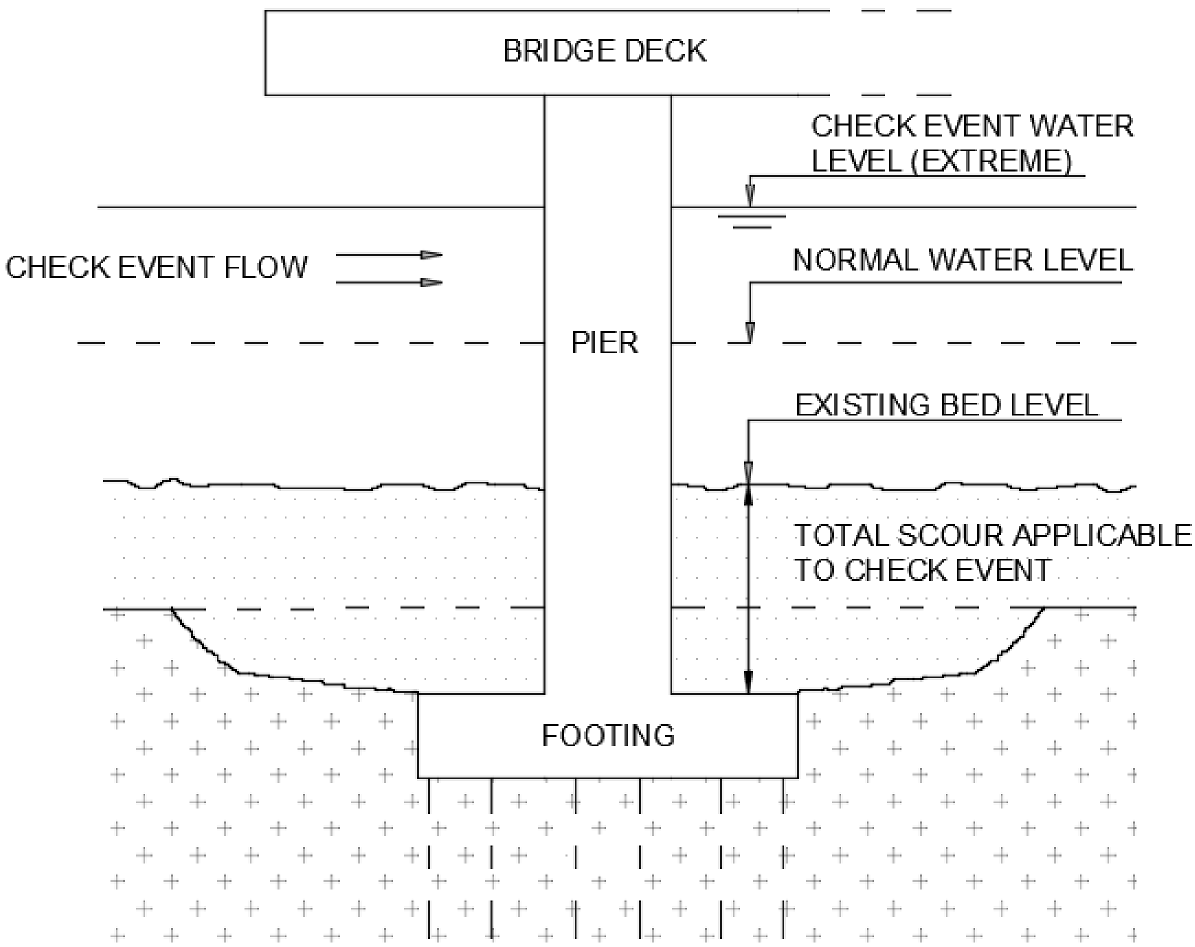


Figure 7.10N2d Debris impact design situation

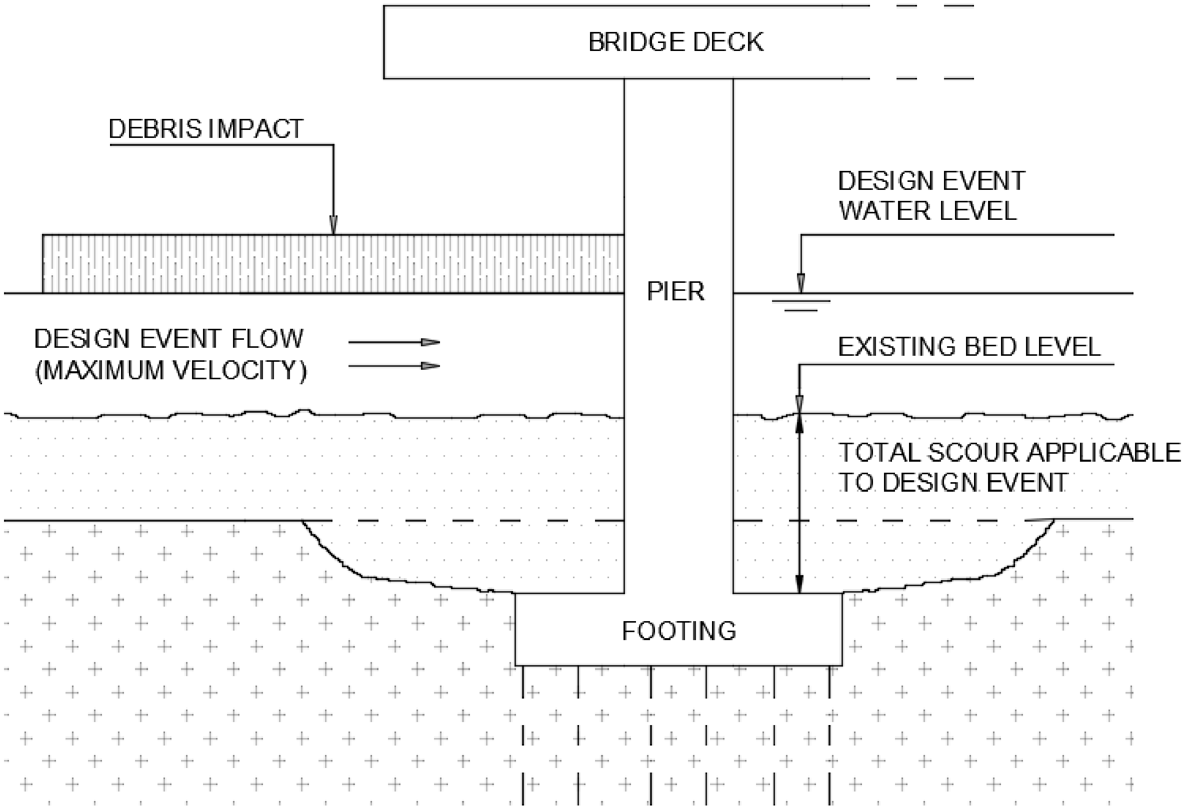


Figure 7.10N2e Ship impact design situation

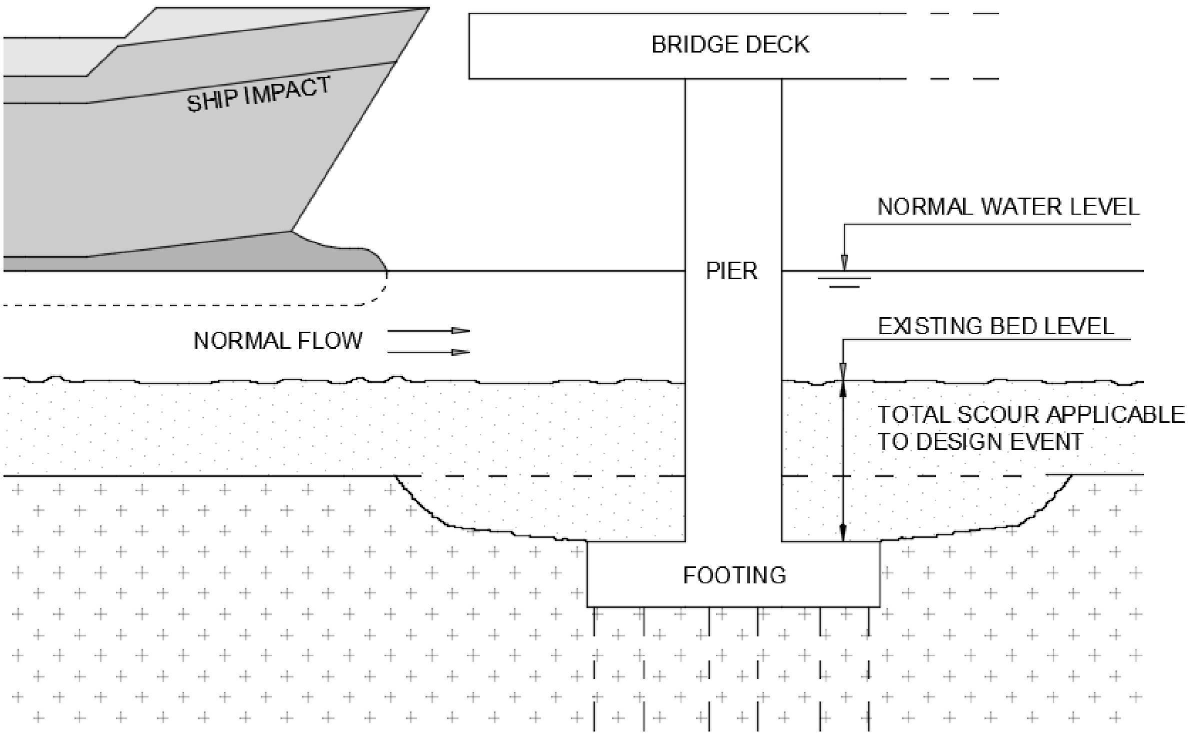
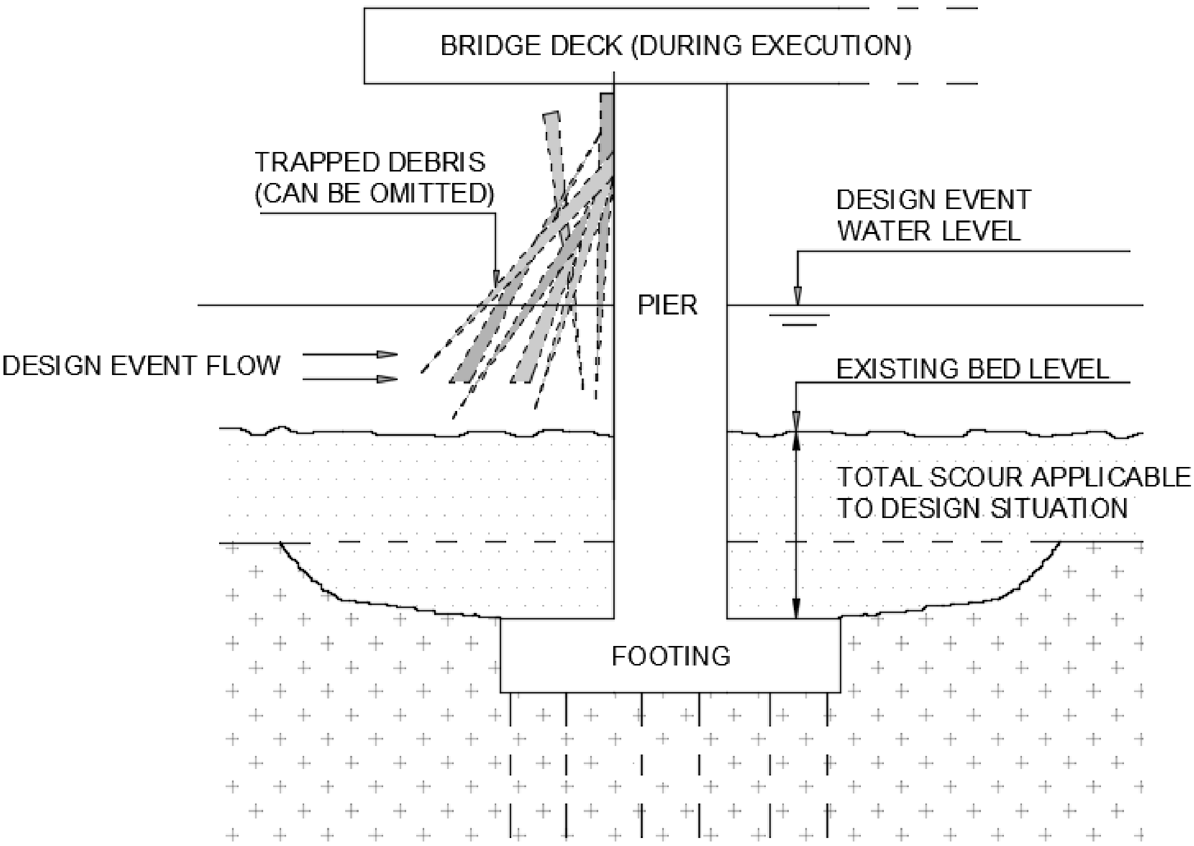


Figure 7.10N2f Construction or repair design situation



Effect of scour protection

7.11 The effect of scour protection on the scour depth shall be determined in the scour risk management strategy.

7.11.1 Where scour protection is provided and is designed to the same return period and design life as the structure, then the maximum scour depth may be taken as the depth of the scour protection.

7.11.2 Where scour protection is provided, and either a reduced return period is used for the design of the protection, or the design life of the protection is less than the design life of the structure, then:

1) an assessment should be undertaken of the scour depth in the event that the scour protection fails;
2) the design situations should include the scour depth associated with failure of the scour protection.

High water level design situation

7.12 High water level shall be treated as a persistent design situation.

7.13 The water action, Q_k, at the high water level design situation shall be calculated based on the water level and water flow velocity for the design event.

7.14 A scour depth applicable to the design event and including adjustment for the obstruction due to trapped debris shall be included in the design situation.

7.15 Trapped debris shall be assumed to be present simultaneously at any two adjacent piers or any one span.

Low water level design situation

7.16 Low water level shall be treated as a persistent design situation.

NOTE *The low water level design situation is unlikely to govern the design. It is included to avoid a design falsely considering only the maximum water level and force and neglecting the potential for lower forces or water levels.*

7.17 The water action, Q_k, at the low water level design situation shall either:

1) be directly calculated based on a low water level; or
2) be omitted (take $Q_k = 0$).

7.18 Where a partial factor γ_Q or a ψ factor is applied to water actions at the low water level design situation, then the reciprocal of each factor shall be applied to the water action.

NOTE *The reciprocal is applied in order to factor the variation from the mean level. In the low water level design situation it is assumed that a lower water level is more onerous.*

7.19 The scour depth shall be taken as the same as for the high water level design situation.

NOTE *Scour can be generated during a flood event but can remain present once water levels reduce.*

7.20 Forces due to trapped debris shall not be included in the low water design situation.

Extreme high water level design situation

7.21 Extreme high water level shall be treated as an accidental design situation.

NOTE 1 *The extreme high water level design situation is included to check the sensitivity of the structure to a higher return period event with increased water levels and flows.*

NOTE 2 *Where the water level increases significantly at the extreme high water level design situation, it can be beneficial to raise the soffit level of the structure to avoid having to design the superstructure for drag forces.*

7.22 The water action, A_d, at the extreme high water level design situation shall be calculated based on the water level and water flow velocity for the check event.

7.23 A scour depth applicable to the check event shall be included in the design situation.

7.24 Forces due to trapped debris shall not be included.

Debris impact design situation

7.25 Debris impact shall be treated as an accidental design situation.

NOTE *The debris impact design situation is intended to check for robustness of local elements.*

7.26 Drag forces due to trapped debris shall not be applied coincident with the debris impact force.

NOTE *Trapped debris can act as a buffer and hence can provide protection to the structure from an impact force.*

7.27 The water action, Q_k, shall be calculated based on the water level and water flow velocity for the design event.

NOTE 1 *The most onerous impact force is likely to be generated by the maximum flow velocity. In some circumstances the maximum flow velocity can be generated where the flow is just within the banks, rather than at a higher water level where the water covers a wider flood plain.*

NOTE 2 *The value of water action, Q_k, can be different to that used at the high water level design situation, since trapped debris is not included. Using the same value of water action is conservative, if it is desired to avoid recalculation of the value of the water action.*

7.28 The water action shall be applied as the leading variable action using the ψ_1 factor.

NOTE *For physical reasons, the debris impact design situation can only apply with water as the leading variable action. Other variable actions, such as traffic, need not be applied as the leading variable action in this design situation.*

7.29 The scour depth shall be taken as the same as for the high water level design situation.

Ship impact design situation

7.30 Ship impact shall be treated as an accidental design situation.

7.31 Where the water action is applied as the leading variable action, then the water action at the ship impact design situation shall be applied as the frequent value, either:

1) using the ψ_1 factor as a factor on the same value of Q_k as for the debris impact design situation; or
2) using a directly assessed value of $Q_{frequent}$ corresponding to the water levels and flows corresponding to the ship impact design criteria, as defined in Section 3.

NOTE The water action can also be applied as an accompanying variable action using the quasi-permanent value.

7.32 The scour depth shall be taken as the same as for the high water level design situation.

NOTE Scour can be generated during a flood event but can remain present once water levels reduce.

7.33 Forces due to trapped debris shall not be included in this design situation.

Construction / repair design situation

7.34 Construction / repair shall be treated as a transient design situation.

7.35 The water action, Q_k, shall be calculated based on the water level and water flow velocity for a return period appropriate for the duration of construction or repair.

7.35.1 The same water levels and flows as for the high water level design situation may conservatively be applied to construction and repair situations.

7.35.2 A reduced return period may be selected.

NOTE Guidance on the relationship between duration of a design situation and return period is provided in BS EN 1991-1-6 [Ref 1.N].

7.36 A scour depth shall be included in the design situation.

7.36.1 Where the construction or repair operation introduces an obstruction in the watercourse, then the scour depth should be calculated based on the restricted flow width.

NOTE Obstructions in the watercourse can include, for example, a cofferdam or other temporary dewatering arrangement.

7.36.2 Where there is no additional obstruction in the watercourse, then the scour depth may be taken as the same as for the high water level design situation.

7.36.3 A reduced scour depth may be used in the construction / repair design situation subject to justification in the scour risk management strategy.

7.37 Trapped debris shall be assumed to be present, unless physical reasons render this impossible.

Hydrostatic actions

7.38 Hydrostatic pressure of water shall be applied to structural elements in water.

7.38.1 Specific checks should be carried out where structural elements are subject to unequal hydrostatic forces, including:

1) where there is a significant difference in water level across the element;
2) abutments and walls exposed to water on one side only.

Hydrodynamic actions

Drag and lift

7.39 Drag forces acting in the direction of flow of the water shall be applied to the submerged parts of a structure.

NOTE Drag can act:

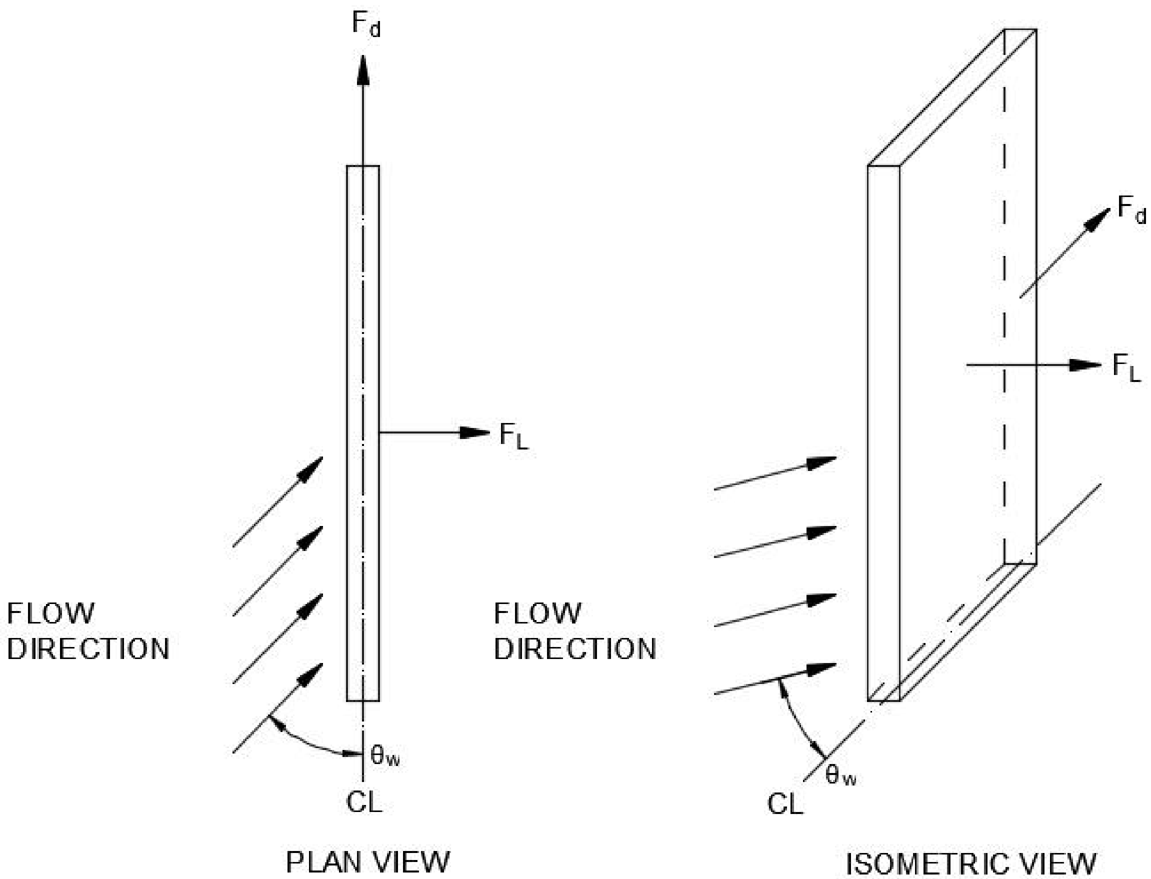
1) *on piers and abutments;*
2) *on superstructure if submerged;*
3) *on debris trapped against piers or superstructure.*

7.40 Lift forces acting normal to the direction of flow of the water shall be applied to the submerged parts of a structure.

NOTE Lift can act:

1) *transversely on piers which are at an angle to the flow;*
2) *vertically on superstructure if submerged.*

Figure 7.40N Drag and lift forces on a pier



7.41 The characteristic values of drag and lift forces shall be calculated from the equation 7.41:

Equation 7.41 Generic drag and lift equation

$$F_k = A_F \cdot C_D \cdot \left(\rho \cdot \frac{V^2}{2}\right)$$

where:

F_k	is the characteristic value of drag (or lift) force (N)
C_D	is the drag coefficient (dimensionless)
C_L (substituting C_L for C_D)	is the lift coefficient (dimensionless)
ρ	is the water density (kg/m^3)
V	is the water reference velocity (m/s)
A_F	is the reference area (m^2)

7.41.1 The drag coefficient and water reference velocity should be selected to be compatible with each other.

NOTE *Many sources provide drag coefficients which are calibrated against the mean speed of the water velocity on the approach to the structural element, averaged over the depth.*

7.41.2 Where drag coefficients are obtained from the literature or from test results, then the applicability of the coefficients to the case being studied should be verified.

NOTE *Coefficients for the same shape of element can vary considerably depending on the Reynolds and Froude numbers.*

7.41.3 The water density ρ may be taken as:

1) 1000 kg/m^3 for fresh water; and
2) 1025 kg/m^3 for salt water.

7.41.4 Where the flow includes significant quantities of sediment, then an appropriate increased value of water density ρ should be selected.

Drag on trapped debris

7.42 The trapped debris drag force shall be applied to all substructure elements (including piers and abutments) within the plan extent of water.

7.43 Where the water level at the design event exceeds the soffit level, then drag forces due to trapped debris shall be applied to the superstructure.

NOTE *If the minimum freeboard allowance set out in Section 3 has been provided, then the water level at the design event will not exceed the soffit level and hence drag forces due to debris trapped by the superstructure do not need to be applied.*

7.44 The trapped debris drag force shall be applied as a point load at the most onerous location on the relevant element of the structure.

7.45 The drag force on trapped debris shall be calculated using Equation 7.41.

7.45.1 The extent of debris accumulation should be determined from the Table 7.45.1, based on the design log length established in Section 3.

Table 7.45.1 Extent of debris accumulation

Accumulation type	Debris width	Debris height
Pier and abutment	Design log length	Flow depth (up to 3 m), debris extending from water surface downwards
Superstructure	Span length	Vertical height of superstructure plus 1.2m below superstructure
Low opening (design log length greater than height between soffit and bed level)	Width of gap	Height of gap (height between soffit and bed level)
Narrow opening (design log length greater than width between supports)	Width of gap	Smaller of height of gap (see above) or flow depth

7.45.2 The reference area A_F for trapped debris should be taken as the wetted area of the accumulated debris normal to the approach flow.

7.45.3 The drag coefficient C_D for water acting on trapped debris may be taken conservatively as 1.9.

7.45.4 A more refined value for the drag coefficient C_D may be obtained using the method presented in CIRIA C742 [Ref 19.I], taking into account the blockage ratio and Froude number.

7.45.5 The water reference velocity, V, should be taken as the mean velocity in the restricted section, including for the increase in velocity due to the restriction caused by the trapped debris.

7.45.6 Where the blockage ratio is less than 30%, then the mean velocity in the restricted section may be approximated by the mean approach velocity.

7.45.7 Where required, the blockage ratio, BR, should be calculated using equation 7.45.7:

Equation 7.45.7 Blockage ratio (BR)

$$BR = \frac{A_b}{A_b + A_u}$$

where:

A_b	is the portion of the cross-sectional area of flow through the opening which is blocked by debris
A_u	is the portion of the cross-sectional area of flow through the opening which is not blocked by debris

NOTE 1 $0 \leq BR \leq 1$ *with BR=0 for no debris and BR=1 for full obstruction.*

NOTE 2 *At high values of blockage ratio, the flow can reduce through the structure but the water level upstream can increase, leading to an increase in differential hydrostatic pressure across the structure and potential overtopping of the structure or approaches.*

Drag and lift on piers

7.46 Drag and lift forces on piers shall be calculated using Equation 7.41.

7.46.1 For piers the drag coefficient C_D should be determined from Table 7.46.1.

Table 7.46.1 Drag coefficient CD for piers

Pier shape	Drag coefficient C_D
Square ended pier	1.44
Semi-circular ended pier	0.7

NOTE *The values of drag coefficient for piers shown in the figures in CIRIA C742 [Ref 19.I] are given based on a reference area for the length, rather than the width, of the pier. This is not particularly clear in the figure or the text and the values have the potential to appear misleadingly low.*

7.46.2 The reference area A_F used for drag should be taken as the projected area of the pier in the direction of flow.

7.46.3 For piers the lift coefficient C_L should be determined from Table 7.46.3.

Table 7.46.3 Lift coefficient for piers

Angle between direction of the flow and the centreline of the pier	Lift coefficient C_L for piers
Circular pier	Apply drag in the direction of flow
$<$ 30 degrees	0.9
$\geq$ 30 degrees	1.0

7.46.4 The reference area A_F used for lift in conjunction with the coefficients in Table 7.46.3 should be taken as the area of the pier submergence times the length of a rectangular pier, or the diameter of a cylindrical pier.

7.46.5 Drag and lift coefficients and forces may be determined by other methods including:

1) appropriate literature;
2) model testing;
3) computational fluid dynamics models.

7.46.6 The possibility of variation in flow angle in relation to pier orientation should be included in the determination of drag and lift coefficients.

NOTE 1 *The coefficients provided in Table 7.46.3 include for variation in the flow angle.*

NOTE 2 *Lift forces on plate-type piers can increase rapidly as the angle of incidence increases from zero.*

7.46.7 The possibility for increased forces due to vortex shedding should be included in the determination of drag and lift coefficients.

NOTE *Vortex shedding can be significant especially for small diameter piers.*

Drag and lift on superstructures

7.47 Where the water level exceeds the soffit level, then drag and lift forces on the superstructure shall be applied.

NOTE *If the minimum freeboard allowance set out in Section 3 has been provided, then the water level at the design event will not exceed the soffit level and hence drag and lift forces on the superstructure do not need to be applied at the design event. A check is required at the extreme high water level design situation.*

7.47.1 Drag and lift forces on the superstructure should be calculated taking account of:

1) whether the superstructure is partially or fully submerged;
2) relative depth of the projected superstructure area compared with the depth from the superstructure to the river bed;

3) superelevation of the deck;
4) shape of the cross section.

NOTE Guidance on calculating drag and lift forces on the superstructure is provided in:

1) Australian bridge design standard AS5100.2 [Ref 2.I];
2) Hydrodynamic Forces on Inundated Bridge Decks, Publication no. FHWA-HRT-09-028 [Ref 13.I];
3) American bridge design standard AASHTO LRFD [Ref 4.I].

7.47.2 Forces on the superstructure should include calculation of the following components:

1) horizontal drag;
2) vertical lift due to water flow;
3) where the structure is partially submerged, vertical lift due to hydrostatic pressure;
4) moment component, accounting for centroid of application of drag and lift forces.

NOTE The vertical lift can act upwards or downwards depending on the configuration of the structure.

Impact force from debris

7.48 The debris impact force shall be applied to all substructure elements (including piers and abutments) within the plan extent of water.

7.49 Where the water level exceeds the soffit level, then the debris impact force shall be applied to the superstructure.

NOTE If the minimum freeboard allowance set out in Section 3 has been provided, then the water level at the design event will not exceed the soffit level and hence the debris impact force on the superstructure does not need to be applied.

7.50 The debris impact force shall be applied as a point load at the most onerous location on the relevant element of the structure.

7.51 The design value of the debris impact force shall be derived using Equation 7.51:

Equation 7.51 Debris impact force

$$A_d = V\sqrt{\hat{k}m}$$

where:

A_d	is the impact force (N)
V	is the debris velocity (m/s)
$\hat{k}$	is the effective contact stiffness (N/m)
m	is the single-item debris mass (kg)

7.51.1 The debris velocity, V, should be taken as the mean approach flow velocity.

7.51.2 The design event should be selected in order to generate the maximum velocity.

7.51.3 The effective contact stiffness, $\hat{k}$, should be taken as $2.4 \cdot 10^6$ N/m.

NOTE This value is based on an impact between a log and a rigid structure.

7.52 The single-item debris mass, m, shall be established in accordance with Section 3.

Actions due to ice

7.53 Where the Overseeing Organisation has specified that ice loading is included for the particular project, then specific actions due to ice, design situations and combinations shall be agreed with the Overseeing Organisation.

Actions due to ship impact

7.54 Where the Overseeing Organisation has specified that ship impact is included for the particular project, then specific actions due to ship impact shall be agreed with the Overseeing Organisation.

7.54.1 The actions due to ship impact may be based on BS EN 1991-1-7 [Ref 3.N] using the ship impact design criteria defined in Section 3.

8. Structure elements

Bridge piers

General

8.1 The shape, size and arrangement of piers shall be determined.

NOTE Constraints and effects which can influence the selection of a shape of pier include:

1) *general aesthetic considerations (see CD 351 [Ref 12.N]);*
2) *structural resistance;*
3) *resistance to flow, including streamlining and alignment with flow direction;*
4) *the depth of the local scour;*
5) *the built-up of debris;*
6) *blockage of the watercourse area.*

8.1.1 Where practically feasible, bridge piers should be designed to be located outside rather than within the watercourse.

8.2 Where ship impact is included for a particular project, then the impact shall be resisted either:

1) by the bridge piers; or
2) by protection measures.

NOTE Protection measures against a ship impact can include:

1) *safety barriers;*
2) *protective bollards;*
3) *arresting cables;*
4) *protective buffers.*

Bridge foundations

Stability of channel

8.3 Where there is a likelihood of channel movement over the life of the structure, then the foundation design shall accommodate foreseeable variations.

NOTE Channel movement can include:

1) *movement of a watercourse into a floodplain;*
2) *change in position of main flow channel(s) moves within a watercourse or estuary.*

8.3.1 Foundations within the extent of potential channel movement should be positioned at the same level as the foundations currently within or adjacent to the channel.

8.3.2 The stability check of the foundations should be performed after assessing the possible changes of the channel pattern with time.

8.3.3 Sensitivity to transition cases, such as a pier half in a channel, should be checked.

Foundation design assumptions

8.4 Bridge foundations shall be designed on the basis that all bed material within the scour hole above the total scour depth will have been removed and is not available to provide bearing or lateral support.

8.4.1 Conservative results may be obtained by assuming a prismatic shaped scour hole.

8.4.2 Benefit may be taken from the remaining material assuming a cone-shaped scour hole.

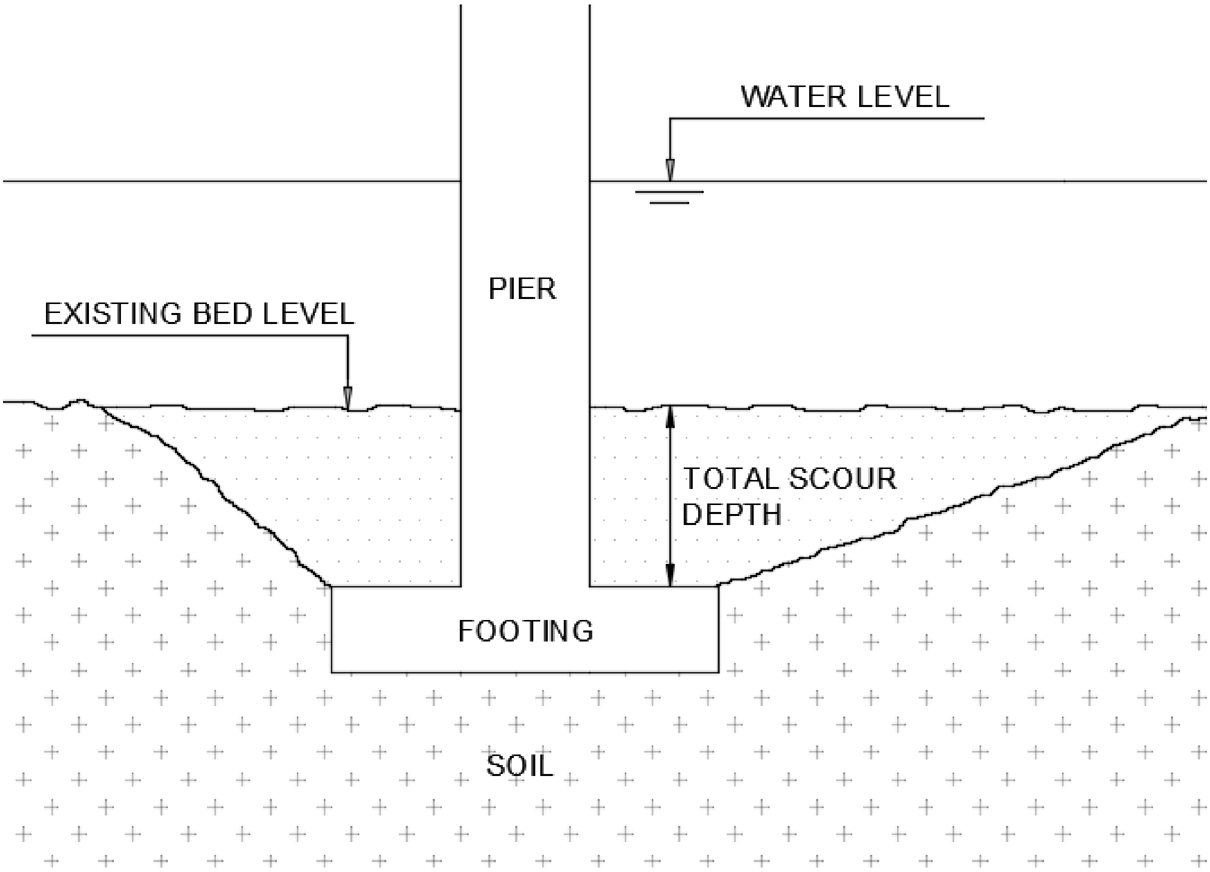
Spread footings on soil

8.5 The footing shall not be undermined by scour.

8.5.1 The top of the footing should be located at least below the total scour depth.

NOTE Placing the top of the footing above the scour depth, can result in a greater obstruction to the flow and lead to an increased scour depth due to the increased flow velocity.

Figure 8.5.1N Spread footing on soil



8.5.2 Risk of undermining may be reduced by positioning the bottom of the footing not less than 2 metres below the existing bed level and below the total scour depth.

8.6 The footing shall be checked for the effects of differential scour.

NOTE 1 Scour holes tend to form eccentrically around the structural element, leading to a risk of differential scour.

NOTE 2 Shallow foundations can be more susceptible to settlement, lateral movement, or overturning due to the development of the lateral earth pressures from differential scour.

8.6.1 Spread footings on soil should be checked against circular slip failure.

NOTE Circular slip failure can occur along a formed concave surface due to differential scour.

Spread footings on rock

8.7 Where the underlying rock is scour-resistant, then the form and proposed construction method of the spread footing shall maintain the integrity of the supporting rock.

8.8 Embedments or keys which require blasting shall not be specified.

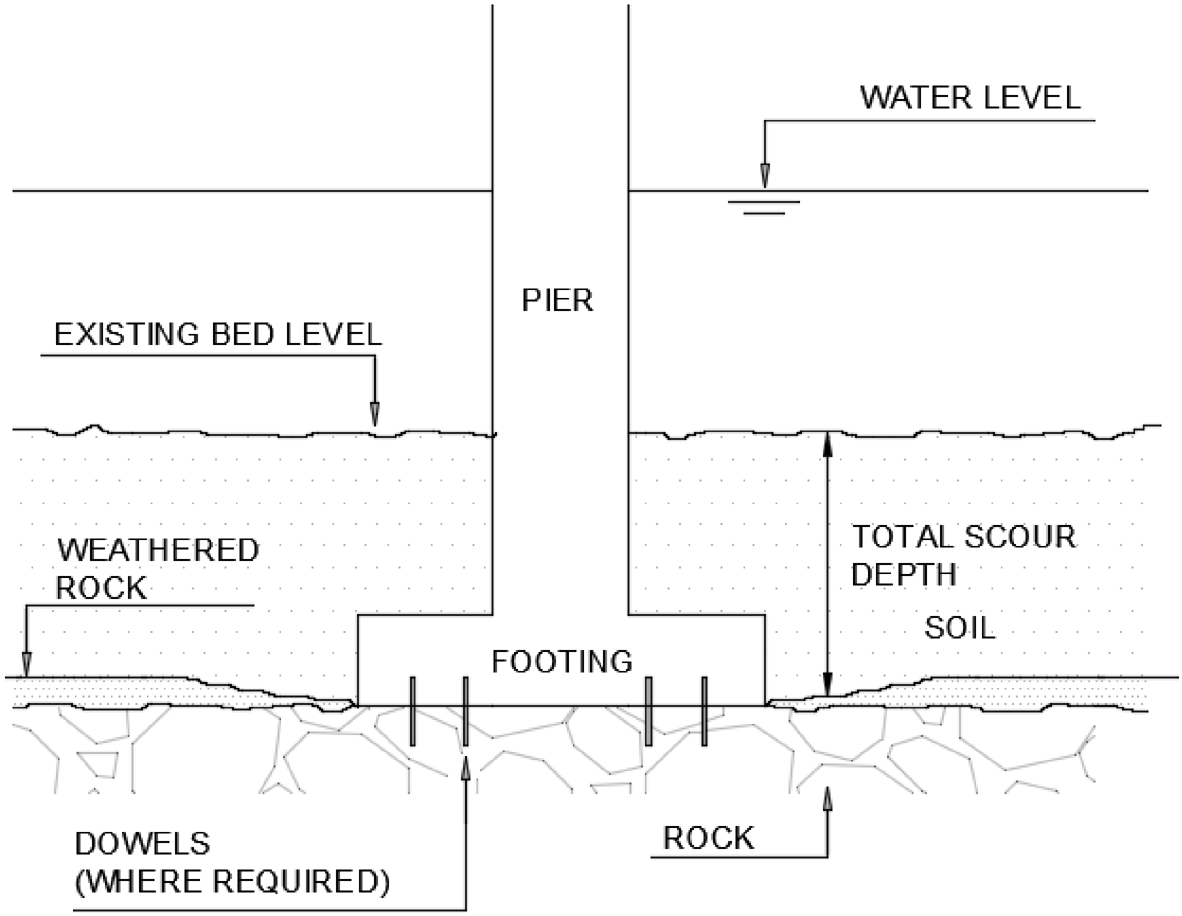
NOTE 1 *The formation of the surface finishing of the rock where small embedments or keys are provided at the bottom of the foundation, can require local excavation with the use of blasting.*

NOTE 2 *Blasting can damage the rock surface and result in over-break and fracturing of the sound rock below the bearing elevation.*

NOTE 3 *Overbreak and fracturing can result the reduction of the scour resistance within the zone of these defects.*

8.8.1 Steel anchors or dowels may be drilled and grouted into the founding rock to provide lateral restraint for footings on smooth rock surfaces.

Figure 8.8.1 Spread footing on rock



Spread footings on weathered or potentially erodible rock

8.9 The potential for degradation and/or erosion of weathered or potentially erodible rock formations under the effects of scour shall be established.

NOTE 1 *The strength and condition of weathered or potentially erodible rock formations can be affected by scour over the life of the structure.*

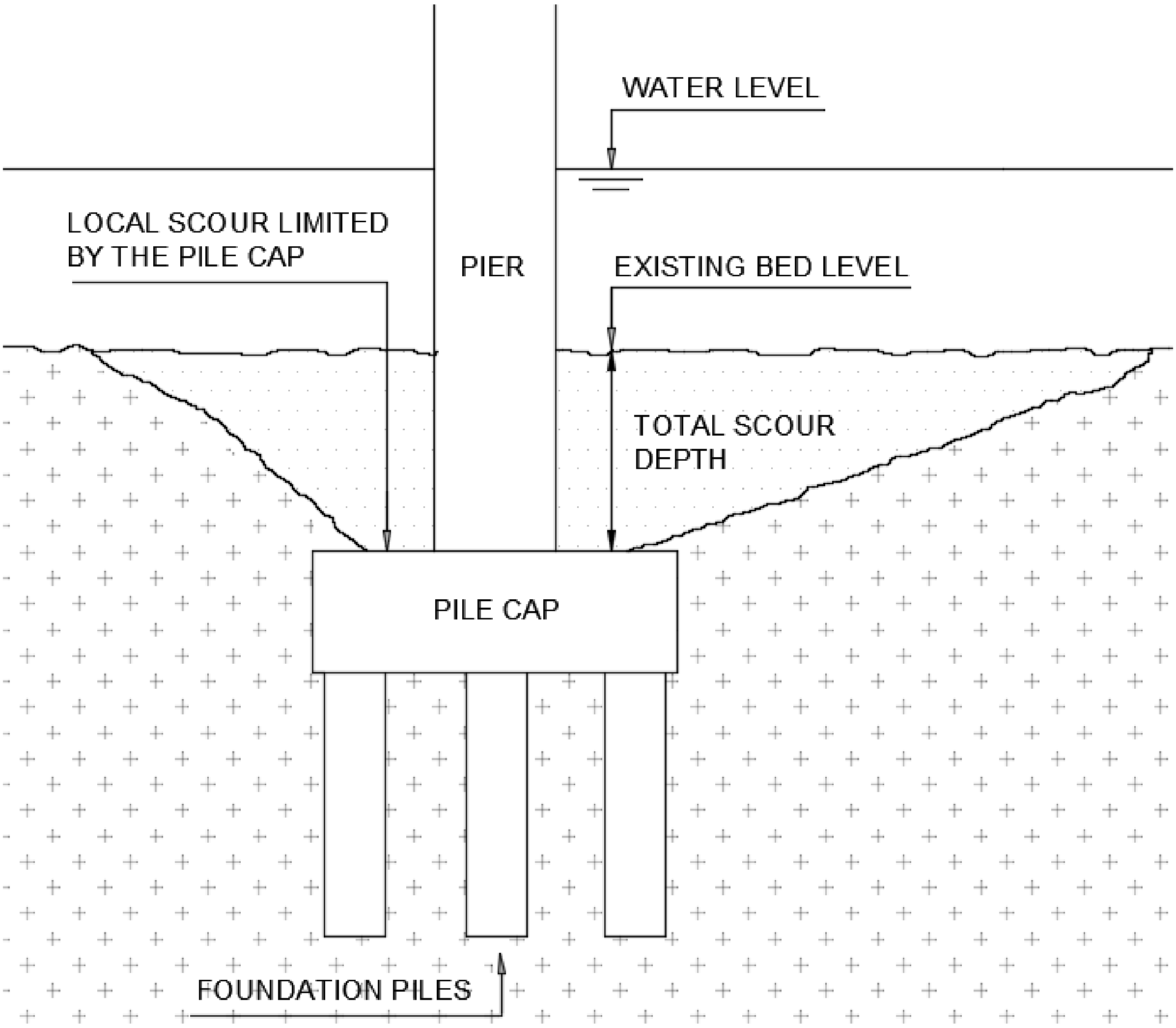
NOTE 2 *Land-based foundations on weathered rock could be subject to scour in future due to channel movement.*

8.10 Where excavation into eroded rock is required, this shall be made with care to minimise the extent of overbreak beneath the footing level.

Piled foundations

8.11 Piled foundations shall be resistant to the effects of scour.

Figure 8.11 Piled foundation



NOTE A lesser amount of long piles instead of a larger amount of short piles can provide a greater factor of safety against pile failure due to scour, given the inherent uncertainties in the scour calculation.

8.11.1 The top surface of the pile cap may be placed below the total scour depth to minimise the local scour due to the obstruction.

NOTE Where the pile cap is situated below the scour depth, a downward current that can result in local scour at the pier can be interrupted.

8.12 Where the depth of total scour exceeds the depth of pile cap and the piles are exposed, then the piles shall be checked as columns with reduced lateral restraint appropriate to the remaining bed level.

NOTE An increased unrestrained length of a pile due to exposure from scour will result in increased bending moments within the piles.

8.13 Where the piles are exposed due to scour, the pile capacity shall be checked with the reduced skin friction due to removal of surrounding bed material.

Bridge abutments

General

8.14 The extent and nature of scour protection, if any, at bridge abutments shall be determined.

NOTE Scour protection measures can negate the need to compute the abutment scour. There are fewer methods available in the literature for calculating scour at abutments compared with piers.

8.14.1 Where significant built-up due to debris is anticipated, bridge abutments should be placed back from the channel banks.

8.15 Where spill-through abutments are provided, then the abutment and embankment material shall be designed to prevent loss of material particularly at the toe of the embankment.

NOTE Scour at spill-through abutments can be less than at vertical abutments, but loss of spill-through material can occur.

Bridge superstructures

General

8.16 Bridge superstructures shall be designed for the main component of the minimum forces for robustness given for bridges over roads in the National Annex NA to BS EN 1991-1-7 [Ref 14.N], acting in the direction of the water flow.

8.17 Where the superstructure is subject to water actions, then the superstructure shall be designed to resist the water actions.

NOTE 1 Increasing the freeboard can avoid the need to design the superstructure to resist water actions.

NOTE 2 The use of a streamlined cross-section for the superstructure can be used to reduce the resistance to the flow and hence the water actions.

8.18 Load path(s) shall be provided to transfer forces applied to the superstructure to the foundations.

8.18.1 Restraint should be provided to prevent the superstructure becoming dislodged from its supports.

NOTE 1 The design forces calculated where bearings are provided can be subject to uncertainty. Bearings can be subject to forces higher than calculated such as in exceptional flood events.

NOTE 2 The provision of shear keys adjacent to the bearings is encouraged to provide restraint against large lateral displacements.

8.19 Where the watercourse is used by ship traffic, then the navigation requirements shall be provided.

NOTE Navigation requirements can include minimum vertical and horizontal clearances (see structure opening size in section 3), navigation lights, signage, and impact protection.

8.20 The likelihood of submergence of the superstructure shall be established.

NOTE Submergence could occur during the design event, the check event, or where the structure is close to the water and there is uncertainty on the water levels at the design or check event.

8.20.1 Where the superstructure could become submerged, then the bridge deck should be designed with resilience to the water unless the costs of doing so are disproportionate to the likelihood of this cause.

NOTE Resilience measures can include:

1) *an intended replacement strategy for services which could be affected by water;*
2) *use of stainless steel or galvanised steel for elements or components which could become submerged.*

8.21 The superstructure shall allow for drainage of all elements, including:

1) voids in the structure;

2) service bays;
3) deck.

9. Normative references

The following documents, in whole or in part, are normative references for this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Ref 1.N	BSI. BS EN 1991-1-6, 'Actions on structures. General actions. Actions during execution.'
Ref 2.N	Highways England. CD 529, 'Design of outfall and culvert details'
Ref 3.N	BSI. BS EN 1991-1-7, 'Eurocode 1 - Actions on structures - Part 1-7 General actions - Accidental actions'
Ref 4.N	BSI. BS EN 1990, 'Eurocode: Basis of structural design'
Ref 5.N	Highways England. LA 101, 'Introduction to environmental assessment'
Ref 6.N	Highways England. GG 101, 'Introduction to the Design Manual for Roads and Bridges'
Ref 7.N	BSI. Committee CB/502/-/PI. BS 6349-1-2, 'Maritime works. General. Code of practice for assessment of actions (May 2016)'
Ref 8.N	BSI. BS 3680-10C, 'Measurement of liquid flow in open channels. Sediment transport. Guide to methods of sampling of sand-bed and cohesive-bed materials''
Ref 9.N	BSI. BS 3680-10E, 'Measurement of liquid flow in open channels. Sediment transport. Sampling and analysis of gravel bed material'
Ref 10.N	Highways England. LA 113, 'Road drainage and the water environment'
Ref 11.N	Highways England. BD 97, 'The Assessment of Scour and Other Hydraulic Actions at Highway Structures'
Ref 12.N	Highways England. CD 351, 'The design and appearance of highway structures'
Ref 13.N	Highways England. CD 350, 'The design of highway structures'
Ref 14.N	BSI. NA to BS EN 1991-1-7, 'UK National Annex to Eurocode 1 - Actions on structures - Part 1-7 General actions - Accidental actions'

10. Informative references

The following documents are informative references for this document and provide supporting information.

Ref 1.I	MCEER. Lee, Mohan et al. Technical Report MCEER-13-0008, 'A study of U.S. bridge failures (1980-2012)'
Ref 2.I	Standards Australia. Committee BD-090 - Bridge Design. AS5100.2, 'Australian Standard - Bridge design - Part 2: Design loads'
Ref 3.I	Bundesanstatt fuer Wasserbau (Karlsruhe Germany). Abromeit, U. et al.. BAW CoP 2010, 'BAW Code of Practice, "Principles for the Design of Bottom and Bank Protection for Inland Waterways (GBB)" issue 2010.'
Ref 4.I	American Association of State Highway and Transportation Officials. AASHTO LRFD, 'Design Specifications'
Ref 5.I	Macmillan. EM Wilson. Wilson 1990, 'Engineering Hydrology'
Ref 6.I	SEPA / Natural Scotland. SEPA River crossings, 'Engineering in the water environment good practice guide: River crossings'
Ref 7.I	Wallingford HydroSolutions. FEH, 'Flood Estimation Handbook'
Ref 8.I	Gov.UK. Environment Agency. FRA CCA, 'Flood risk assessments: climate change allowances'
Ref 9.I	British Geological Society. BGS, 'Geoshore Index'
Ref 10.I	US Department of Transportation, Federal Highways Administration. Arneson, LA et al. HEC 18 - FHWA-HIF-12-003, 'HEC (Hydraulic Engineering Circular) No. 18 'Evaluating Scour at Bridges' 5th edition, 2012'
Ref 11.I	UK Met Office. UKCP18 , 'https://www.metoffice.gov.uk'
Ref 12.I	US Federal Highway Administration. USBPR, 'Hydraulics of Bridge Waterways'
Ref 13.I	US Department of Transportation - Federal Highway Administrartion. Kerenyi, Sofu, Guo. FHWA-HRT-09-028, 'Hydrodynamic Forces on Inundated Bridge Decks'
Ref 14.I	Environment Agency. Defra / Environment Agency. SC050050, 'Improving the FEH statistical procedures for flood frequency estimation'
Ref 15.I	HR Wallingford. IH 126, 'Insitutue of Hydrology IH126 report'
Ref 16.I	Dept. of Environment, Food & Rural Affairs (DEFRA), Flood Management Div., with Environment Agency. Hawkes, Peter J (HR Wallingford). FD2308/TR2, 'Joint probability: Dependence mapping and best practice - R&D Technical Report March 2006'
Ref 17.I	Cranfield University. NSRI, CEH, James Hutton Institute. LandIS, 'LandIS Land Information System'
Ref 18.I	ICE proceedings, journal of Forensic Engineering. Mathews and Hardman. ICE 1700009, 'Lessons learnt from the December 2015 flood event in Cumbria, UK'
Ref 19.I	CIRIA. Kirby, Roca, Kitchen et al. CIRIA C742, 'Manual on scour at bridges and other hydraulic structures, 2nd edition'
Ref 20.I	Ven Te Chow. Chow 1959, 'Open Channel Hydraulics'
Ref 21.I	ICE proceedings, journal of Forensic Engineering. Benn, J. JFE 1300013, 'Railway bridge failure during flooding in the UK and Ireland'

Ref 22.I	Institutue of Hydrology - National Environment Research Council. Gustard, A et al. IH 108, 'Report No. 108 'Low flow estimation in the United Kingdom''
Ref 23.I	HR Wallingford. HRW SR 182, 'Report SR182. Afflux at Arch Bridges'
Ref 24.I	Environment Agency - Gov UK. LIT531, 'Research and analysis: Coastal flood boundary conditions for UK mainland and islands|: design sea levels'
Ref 25.I	Centre for Ecology & Hydrology - Natural Environment Research Council. Wallingford Hydro Solutions Ltd. ReFH 2.2, 'Revitalised Flood Hydrograph Model (ReFH 2.2): Technical Guidance (2016)'
Ref 26.I	BSI. Committee B/502. BS EN 13383, 'Rock. Armourstone. BS EN 13383 Parts 1 & 2'
Ref 27.I	Soil Survey of England and Wales. Soils, 'Soils of England and Wales'
Ref 28.I	Wallingford HydroSolutions Ltd 2016. Centre for Ecology & Hydrology. ReFH 2.2: Technical Guidance, 'The Revitalised Flood Hydrograph Model'
Ref 29.I	Met Office. Jenkins, G. J., Murphy, J. M., Sexton, D. M. H., Lowe, J. A., Jones, P. and Kilsby, C. G.. UKCP 09 Report, 'UK Climate Projections: Briefing report.'
Ref 30.I	Centre for Ecology & Hydrology (CEH). National river flow archive , 'www.nrfa.ceh.ac.uk'

Appendix A. Sample methods for scour assessment

A1 General scour

General scour is defined as the short-term scour that will occur during the design flood event. A number of methods are available. The type of calculation will be depend on the level of complexity of the problem:

1) Morphological modelling - developed through the use of numerical models;
2) Extension of regime theory - typically a 'hand' calculation;
3) Threshold methods - these assume that scour will proceed until threshold conditions (see definition of the depth-averaged critical threshold velocity for non-cohesive surface bed material, U_{TC} , below).

The details of these methods are available in CIRIA C742 [Ref 19.I].

A2 Contraction scour

An approximation of the depth of Contraction Scour may be obtained by modelling the geometry upstream and under the bridge as long rectangular contractions and estimating an equilibrium scour depth on this basis. A method to estimate contraction scour given in CIRIA C742 [Ref 19.I] may be calculated using the method below

A3 Local scour at bridge piers in non-cohesive soils

Local scour at bridge piers can be calculated using the following steps:

1) Determine the depth-averaged critical threshold velocity for the surface bed material. This may be done using a modified Shields (1936) equation outlined in CIRIA C742 [Ref 19.I].

Equation A.1 Depth-averaged critical threshold velocity

$$\frac{U_{TC}}{U.} = -\sqrt{32}\log_{10}\left(\frac{k_s}{12y_o} + 0.22\frac{\nu}{y_oU.}\right)$$

where:

U_{TC}	is the critical threshold velocity (m/s)
U	is the shear velocity of the flow (m/s)
k_s	is the effective roughness of the sediment bed = d for uniformly graded sediment or approximately 2 x the mean particle size (d50) for graded material
y_o	is the local flow depth at the point being considered (m)
ν	is the kinematic viscosity of water (=1.4 x 10^{-6} m^2/s at 15^oC)

2) For the design flow, determine whether live bed or clear water scour will occur.

Live bed or clear water scour influences the behaviour of the scour hole. In live bed scour, the scour hole increases to an equilibrium scour depth, where the rate of material entering the hole equals the rate of material existing the hole. This is often accompanied by bedforms which cause the scour depth to fluctuate about the equilibrium depth. Clear water scour is a gradual increase to a maximum (equilibrium) scour depth. This step is useful for understanding the speed at which the maximum (equilibrium) scour depth will develop.

3) Calculate the equilibrium scour depth, Ys

Equation A.2 Determination of depth of local scour

$$\frac{Y_s}{B_s} = \Phi_{Shape}.\Phi_{Depth}.\Phi_{Velocity}.\Phi_{Ang}$$

where:

B_s	is the width of the structure measured normal to the longitudinal axis of the flow
ϕ	are factors which are dependent on the flow conditions and geometry of the pier. Guidance on appropriate values can be found in CIRIA C742 [Ref 19.I] .

The depth of local scour, Ys, may be calculated using the following equation originally developed by Breusers et al (1977) and modified by subsequent researchers, as described in CIRIA C742 [Ref 19.I]:

Appendix B. Methods for deriving input parameters

B1 Fluvial flows

B1.1 Flood flows

In the UK, the Flood Estimation Handbook methods are usually preferred for the estimation of flood flows, as detailed in the FEH [Ref 7.I] and subsequent updates. In accordance with current best practice the FEH Statistical method SC050050 [Ref 14.I] and/or ReFH 2.2 [Ref 25.I] methods should generally be adopted. A detailed description of the application of these methods is outside the scope of this document. Appropriate specialist advice should be sought and the current best practice guidance available from the relevant authorities followed as to the application of these methods. Alternative flood flow estimation methods may be more applicable for some catchments such as highly permeable catchments or very heavily urbanised catchments. Appropriate specialist advice should be sought in these instances.

B1.2 Normal flow conditions

Flow rates representative of normal conditions may be established by:

1) estimation of flow rate from survey data, defining the cross-sectional shape and the longitudinal gradient, and known (or assumed) water level. A simplified approach to calculate discharge is described in Section 4;
2) analysis of records from flow gauging stations upstream or downstream of the site. The record may need to be adjusted for changes in flows between the site and the gauge location. The average of daily mean flow for the period of record can be used to approximate the mean flow. Where applicable for design, seasonal mean flows can also be estimated;
3) analysis of records from flow gauging stations in adjacent catchments or those with similar characteristics. Appropriate adjustments to reflect differences between the study site and the gauged site will be required, for example to reflect differences in catchment area;
4) estimation of mean flow using established methods for ungauged catchments (clause B1.2.1). These are typically based on a relationship between rainfall and evaporation estimates;
5) direct flow measurements at the site using current meters and level gauges or other flow gauging techniques. Guidance on how to measure flow rates is given in a number of British and European Standards.

In most instances it is likely that flood flows will provide the worst case conditions for design and therefore, in the absence of better data, method 1 is likely to be appropriate to inform other design checks provided the water level(s) used in the assessment are representative of normal or design flow conditions. For larger or higher risk schemes, where suitable flow records are not available, it is recommended that more than one method is used to check that estimates obtained are consistent and realistic.

Analysis of gauge data should include an assessment of changes to a river catchment or gauge site that may affect the flow record over time.

Gauge records covering short periods of time should be used with caution as the record period may not be typical of long-term conditions, although this is likely to be less of a problem for estimation of "normal flows" than for extreme high or low flows. Comparison of the record with that for a nearby gauge with a longer record may help to determine if the recorded flows are representative of long term behaviour.

Details of existing flow gauges, and flow data, can be found on the National River Flow Archive National river flow archive [Ref 30.I]. The relevant authority may need to be contacted for additional data or information regarding a gauge. There may also be river level gauges in the vicinity of the site that could be used in the assessment, provided a suitable flow-level relationship can be determined.

B1.2.1 Estimation of Mean Flow for ungauged catchments

The IH 108 [Ref 22.I] report includes a regression equation to estimate mean flow for ungauged catchments as detailed below. It should be noted however that the report was published in 1992, and uses gauged records and parameters published earlier than this date. The resultant estimate may not therefore reflect current conditions and climatic trends and more up to date methods included in commercial software may be preferred. However, it is likely to be sufficient to inform initial estimates, secondary design checks or for comparison with other estimates. Further information on the basis of the approach can be found in the report. A more sophisticated catchment water balance approach is also presented in the report.

Equation B.1 IH108 Mean flow for ungauged catchment

$$\text{MF} = 2.70 \times 10^{-7} \times \text{AREA}^{1.02} \times \text{SAAR}^{1.82} \times \text{PE}^{-0.284}$$

where:

MF	is the mean flow (m^3/s)
AREA	is the catchment area (km^2)
SAAR	is the standard annual average rainfall 1941-70 (mm), available from various sources
PE	is potential evaporation (mm), available from the MET office

B1.3 Low flow conditions

Minimum flow rates (e.g. Q95) may be estimated from analysis of gauge data where suitable records exist. Records of daily mean flow can be used to derive a 'flow duration curve' at a site, from which flows for different exceedance probabilities can be estimated. The records can be heavily influenced by the period of record, and seasonality, and this should be considered in the assessment.

As described in section clause B1.2, records for gauges upstream or downstream of the site, or in adjacent or similar catchments may be used to estimate flows at the site. The importance of artificial influences such as abstractions and discharges should be assessed as these can have a significant impact on low flows in certain rivers.

The IH 108 [Ref 22.I] report includes methods for estimating Q95 and deriving a flow duration curve for ungauged catchments. The method requires an assessment of the appropriate HOST (Hydrology of Soil Types) classification for the catchment and provides estimates of Q95 as a proportion of mean flow. Flow estimates for alternative probabilities can also be derived. HOST classification data can be obtained from the Land Information System web service LandIS [Ref 17.I] or from published soil maps Soils [Ref 27.I] used in conjunction with the HOST report IH 126 [Ref 15.I]. Similar methods are also described in Wilson 1990 [Ref 5.I] (using Baseflow index for the catchment). The Baseflow index can be obtained from gauged data or the FEH web service FEH [Ref 7.I] Commercial software is available which can also provide low flow estimates.

B1.4 Tidal flows

Where structures are subject to tidal flows the design flows can be made of three components:

1) a fluvial flow corresponding to the design event in the upstream catchment;
2) a tidal component produced by astronomical tides;
3) 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 distance of the site from the upstream tidal limit.

Where a scheme is subject to significant tidal flows it is usually necessary to carry out a detailed study and hydraulic modelling to quantify the respective flows and to assess the probabilities of fluvial and tidal events occurring simultaneously. Specialist advice should be sought in such instances. Defra report FD2308/TR2 [Ref 16.I] provides guidance on the assessment of joint probabilities of fluvial and tidal events.

In many instances, particularly where the tidal component is less important, the highest flows and worst case conditions will occur for a fluvial flow coinciding with an outgoing (ebb) tide. However there may still be a requirement to assess incoming tidal flows, particularly if it is necessary to complete separate design checks for the downstream (seaward) side of the bridge structure.

Tidal surge events will typically result in the highest tide levels however the tidal range, and therefore the maximum flow rate, may be larger for astronomical tides. It is therefore recommended that a range of scenarios are assessed to determine the worst case conditions.

B1.5 Simplified approach to estimate tidal flows

A simplified method for estimating tidal flows is provided here. It is recommended that this is checked for both a surge event corresponding with the largest tidal flood event for design (and the check event) and for the highest astronomical tide (HAT).

Within estuaries, funnelling and surge effects can have a significant effect on tidal flows. This means that tide levels obtained for ports or coastal locations may not be representative of the bridge site. There is no known simple method of assessing these affects as it is highly dependent on the shape and size of the estuary. These effects are likely to be significant where there are more rapid changes in the size or depth of the estuary.

Where there is any doubt in the suitability of the simplified method it is recommended that specialist advice is sought and an appropriate detailed study completed.

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

Equation B.2 Peak tidal flow estimation

$$Q_{\text{tide}} = \frac{\pi V_{\text{tide}}}{T_{\text{tide}}}$$

where:

Q_{tide}	is the peak tidal flow (m^3/s)
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 (m^3)
T_{tide}	is the tidal period (s)

B1.5.1 Case 1 - Highest astronomical tide

Tidal periods and ranges at standard ports are published in the Admiralty tide data and values for other locations may be interpolated as appropriate.

$Q_{\text{tide(HAT)}}$ should be calculated for the tidal range between the highest and lowest astronomical tides, using the highest astronomical tide level to determine the area of tidal inundation upstream of the bridge.

B1.5.2 Case 2 - Largest tidal flood event for design / Check event

The surge event tidal range may be calculated by superimposing the appropriate surge shape onto the astronomical tidal curve, and fitting the resultant curve to the appropriate surge flood level. It is recommended the astronomical tidal curve is based on the mean high water spring, with the peak of the surge event coinciding with the peak of the mean high water spring tide. Various commercial tidal prediction software exists to produce the astronomical tidal curve.

Tidal periods and ranges are published in the Admiralty tide data. The relevant authority should be contacted to obtain current estimates of flood levels for a range of tidal surge events, alongside recommended surge shapes.

At the time of writing coastal flood levels at 2m intervals around the coast of England, Wales and Scotland, alongside standard surge shapes, are freely available on Gov-UK datashare LIT531 [Ref 24.I]. However these may be replaced in time. The advice of the relevant authority should be sought to confirm the best data to use at the time of the assessment.

$Q_{\text{tide(surge)}}$ should be calculated for the largest tidal range throughout the duration of the surge event, using the corresponding high tide level to determine the area of tidal inundation upstream of the bridge.

B1.5.3 Incoming peak tidal flow estimation

In an estuary the tidal rise during the incoming tide can be much more rapid than the ebb tide. Peak incoming tidal flows should therefore be taken as:

Equation B.3 Peak incoming tidal flow

$$Q_{\text{tide\_max}} = \frac{4.5}{\pi} Q_{\text{tide}}$$

where:

$Q_{\text{tide\_max}}$	is the peak incoming tidal flow (m^3/s)
Q_{tide}	is the peak tidal flow (m^3/s)

B2 Water levels and flow depths

As described in CIRIA C742 [Ref 19.I] design water levels may be estimated through:

1. analysis of historical data on water levels, if these are available from readings at gauge boards or gauging stations close to the site;

2. use of tide data and predicted tidal surge levels in estuary or coastal areas;

3. hydraulic calculations or hydraulic models using flow estimates in conjunction with maps and survey data defining the cross section geometry and longitudinal gradient of the watercourse;

4. anecdotal / informal information provided by local people on typical water levels or observed water levels for past floods (for example trash marks or heights reached on the walls of buildings);

5. assessment of water levels at the site as shown on topographical or channel surveys, noting these will only represent a single point in time and may not therefore be representative of design conditions.

It is important to check and compare data for any information obtained. Information may be based on different topographical surveys or datums. In particular tide levels will often be related to chart datum and will need to be adjusted to the national datum used for topographical surveys at the site. Anecdotal information may not always be reliable.

It is unlikely that design fluvial flood conditions will have been recorded at the site and therefore method 3 is usually necessary to estimate water levels for design flood conditions.

B2.1 Calculation of water levels or discharge in rivers and channels

A method to estimate water levels at the site (for a given flow rate) or flow rate (for a given water level) is provided here, based on the Manning's flow equation.

This method will usually provide reasonable estimates for a reasonably straight and uniform reach of channel. The method assumes normal depth flow conditions apply, with flow rates and water levels controlled by the conveyance capacity and frictional resistance of the channel and no other significant hydraulic influences. Examples of where the approach may not be suitable include:

1. where channel geometry, gradient, or other characteristics (such as vegetation cover or bed material) vary significantly through the study reach;

2. where there is another hydraulic control in the vicinity (either upstream or downstream) such as a weir, culvert or other form of flow constriction;

3. close to a junction with a major tributary;

4. where storage and attenuation of flows may be significant. In these cases it may still be applicable to apply the Manning's equation at the site, but an assessment of the flow rate to be used will need to take account of these effects;

5. at sites with a large or complex floodplain, where the assumption of uni-directional flow and constant water level across the section is not valid.

It may be necessary to use backwater flow calculations, alternative hydraulic calculation methods and/or hydraulic models in these instances. It is recommended that specialist advice is sought to determine the most appropriate method. Hydraulic models will often be developed for the purposes of flood risk assessment in any case and these can be used to provide estimates at the site.

Theoretically, the Manning's equation can be applied or the assessment of flood flows, normal flow conditions and low flow conditions. However it should be noted that during low flow conditions minor variations in the channel, low flow channel meanders and bed material will have more of an influence on flow and normal depth flow conditions may not apply. For these flows, estimates of water levels and depths may be better made by close examination of the channel at the site.

The Manning's equation approach detailed in this section does not take account of any constriction to flow created by the proposed structure(s). Reference should be made to sub-section B5 for assessment of these effects.

B2.1.1 Application of Manning's equation

The Manning's equation is as follows:

Equation B.4 Manning's equation

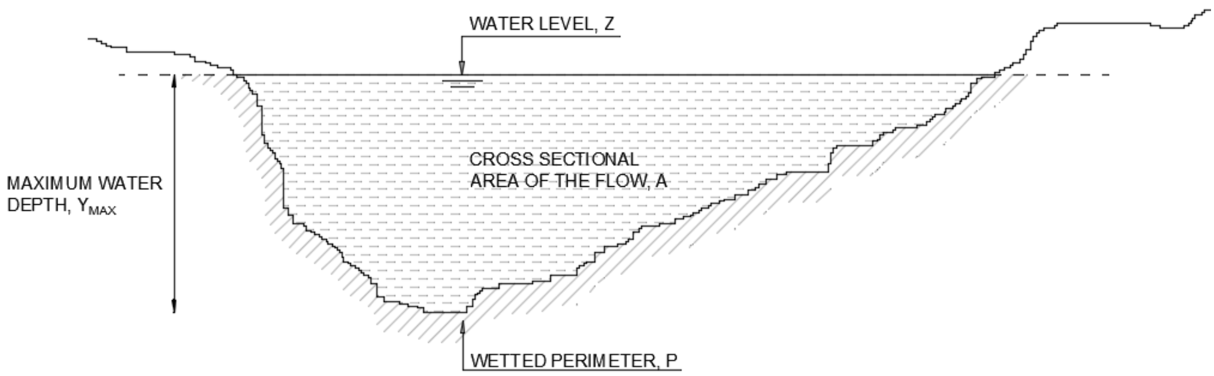
$$Q = \frac{1}{n}\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}}S_e^{\frac{1}{2}}$$

where:

Q	is the flow rate (m^3/s)
n	is the Manning's roughness coefficient
A	is the cross-sectional area of flow (m^2) normal to the direction of flow (note that in flood conditions this may not follow the channel alignment)
P	is the wetted perimeter (m) corresponding to the water level Z (m above datum), see Figure B.1
S_e	is the energy gradient through the channel reach. In normal flow conditions this can be assumed to be approximately equal to the longitudinal bed gradient

Estimates of Manning's roughness coefficient may be obtained from standard textbooks (e.g. Chow 1959 [Ref 20.I]). The calculation of water level, Z, or maximum water depth across the section, Y_{max}, for a known flow rate can be completed by rearranging equation B4. An iterative process may be required to solve the equation.

Figure B.1 Typical channel cross section

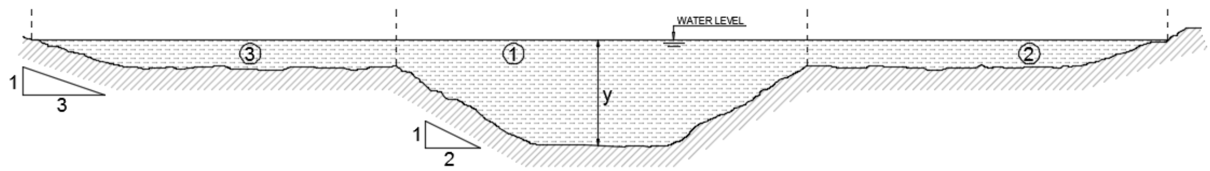


B2.1.2 Compound channels and varying channel cross section

For multi-stage channels or where out of bank flow occurs it may be necessary to determine how the flow is distributed between the main channel and shallower areas or floodplains.

Figure B.2 shows a typical compound channel. It is assumed that flow direction in the shallower or floodplain areas remains parallel to the main channel flows and the water level is the same across the full width.

Figure B.2 Compound Channel



The total flow can be calculated as:

Equation B.5 Total flow for compound channels

$$Q_T = Q_1 + Q_2 + Q_3$$

with the flow for each section calculated using the Manning's equation, for example:

Equation B.6 Manning's equation for compound channels

$$Q_1 = \frac{1}{n_1} \frac{A_1^{\frac{5}{3}}}{P_1^{\frac{5}{3}}} S_{e1}^{\frac{1}{2}}$$

where:

n_1	is the Manning's roughness coefficient for channel section 1
A_1	is the flow area up to the water level in section 1 (between the dashed lines)
P_1	is the wetted perimeter for section 1. It is recommended the water boundaries indicated by the dashed lines are included in the calculation for section 1 only, to account for drag resistance between the main channel and slower moving flows in sections 2 and 3
S_{e1}	is the energy gradient, typically assumed to be approximately equal to the bed gradient. For meandering channels the path length followed by floodplain flows is shorter than for the main channel, and therefore the gradient may differ. However it is generally recommended to adopt the same gradient value for all sections, based on the main channel length, to account for additional energy losses in floodplains caused by the meanders

The same approach can be applied to varying channel sections where variations in flow depth and velocity may be experienced across the section. The channel cross section should be divided into a suitable number of sections, of reasonably uniform flow depth and velocity, and the Manning's equation applied to each section. In most cases the drag force between sections can be ignored (so do not include the water boundary in the wetted perimeter), unless more significant changes in velocity between sections are expected such as might occur in multi-stage channels. In this instance the water boundary should be included in the wetted perimeter for the faster flowing section.

B2.1.3 Estimation of water levels in tidal situations

In estuaries or tidally influenced rivers the estimation of water levels may need to take account of tidal flows and/or downstream tide levels. The following approach can be followed for a simple assessment.

It is recommended the water level is calculated for two alternative scenarios:

1. The Manning's equation may be applied for the combined fluvial and maximum tidal flow rate, assuming the fluvial flow coincides with the outgoing (ebb) tide.

2. The fluvial flow rate, assuming a constant downstream tide level. Unless another tide level is assessed to be appropriate it is recommended this is based on the largest tidal event for design or check event. The tide level should be compared with the channel section at the site and the area below this level excluded from the flow area used in the calculations.

It is stressed that this may present an overly conservative scenario and neither method will properly account for the complex flow regime where tidal and fluvial flows interact. Where the tidal influence is significant specialist advice should be sought and a detailed assessment completed.

B3 Estimation of flow velocity

Flow velocities may be estimated from measured data at the site but as described in section B1 it is unlikely these are available for design flood conditions. However they can still be useful to inform an assessment of how flow velocity varies across the channel. Measurements taken in the main channel at or near bank-full conditions may also provide an indication of flow velocities during flood conditions as these often do not increase substantially.

Where channels are reasonably uniform a single estimate of velocity may be suitable for scour assessment and input to the design. In most cases separate flow velocities will need to be calculated for any out of bank flows. The section mean velocity is given by equation B.7.

Equation B.7 Section mean velocity

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

where:

V	is the average velocity (m/s)
Q	is the flow rate (m^3/s), either across the whole section or for the sub-section as appropriate
A	is the flow area (m^2), either across the whole section or the sub-section as appropriate

Where the flow velocity varies across the section then for design purposes it is likely to be necessary to estimate the local velocity U at the relevant structural element. For a simple assessment this can be achieved by:

1. applying equation B7 to the relevant sub-section of the channel section; and/or
2. applying a multiplication factor to the section mean velocity value.

For method 2 the multiplication factors may be derived from field measurements relating the local velocity to the section mean value, provided sufficient measurements are taken across the width of the channel.

Alternatively CIRIA C742 [Ref 19.I] provides guidance on assumed factors, as summarised in Table B.1.

Table B.1 Local velocity factors

Location	Recommended factor	Comment
Maximum across the section	1.25	Applicable to fairly straight lengths of channel, higher values may occur around bends or at constrictions
Toe of revetment	0.7	Straight channels
Toe of revetment	1.35	Outside of bends

Isolated structures, such as bridge piers, can cause a local acceleration of flow. The velocity around the structure can be taken as twice that immediately upstream, i.e. $U_s = 2U$. However depending on the scour depth calculation method used this acceleration should already be included in that calculation and the upstream velocity would typically be used as the input value to the scour depth calculation.

Where there is uncertainty it is recommended that both methods are assessed and an appropriate range of values considered for design.

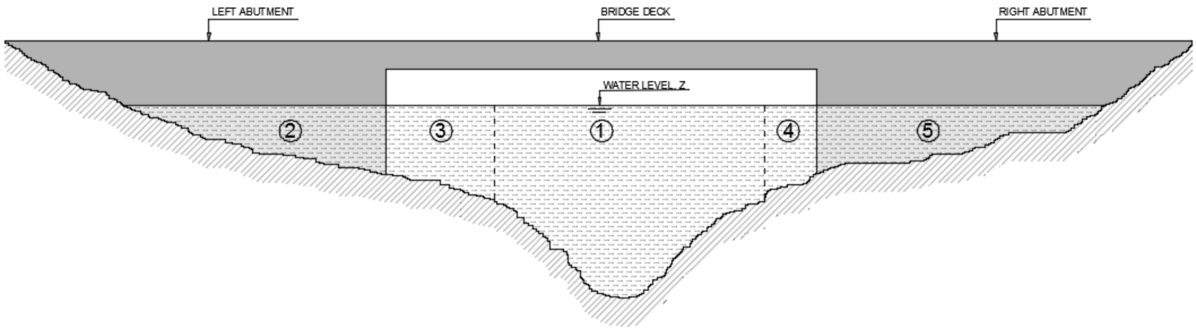
The flow velocities obtained as above are depth-averaged values. In most instances this depth-average value is appropriate for design however values for a specific depth can be obtained. CIRIA C742 [Ref 19.I] contains further guidance. If necessary, more sophisticated 2-D and 3-D hydraulic models can be used to obtain more accurate flow velocities.

B4 Assessment of the effects of the structure

Where the structure does not span the full width of the floodplain the constriction may cause an increase in water level upstream of the structure and increased flow velocities through the structure.

For a simple assessment, the flow depth, water level and average flow velocity through the structure may be calculated by applying the Manning's equation to the bridge section, assuming that all flow is channelled through the structure opening(s). Figure B.3 illustrates the approach. The floodplain flows (sections 2 and 5) are assumed to be diverted through sections 3, 1 and 4. For a simple assessment no account is taken of any effect of the structure in attenuating flows.

Figure B.3 Manning's application to proposed structure



Where there are multiple openings an iterative approach may be needed to determine the flow distribution between each opening that gives a constant water level across the section.

The size of the structure opening should be adjusted to account for blockage scenarios as required. If the calculated water level reaches the bridge soffit level the hydraulic flow regime would change and the Manning's approach is no longer valid. Appropriate specialist advice should be sought regarding alternative assessment methods. It should be noted that this method does not explicitly account for additional energy losses at the structure and associated afflux as described in section B4.1.

B4.1 Estimation of afflux

Constriction of flow causes a loss of energy which results in an increase in water level upstream of the structure. Afflux is the difference between the upstream water level without the structure and with the structure in place. The upstream water level used for design should generally include the afflux.

The US Bureau of Public Roads method (USBPR [Ref 12.I]) is widely used to calculate afflux at bridge structures and is suitable for bridges with flat decks. Other methods are available if required which are more suitable for other types of structure such as arch bridges (HRW SR 182 [Ref 23.I]).

The calculation of afflux can be reasonably complex involving an iterative process. The calculation methods are generally embedded in hydraulic models which may be developed for the purposes of flood risk assessment in any case. It is recommended that appropriate specialist advice is obtained as regards the approach to afflux estimation.

Appendix C. Site investigation

C1 Introduction

The objectives of the site investigation are as follows:

1) to record the present condition of the river to inform the conceptual representation and site-specific design criteria;
2) to identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation;
3) to obtain site data to use in the design.

The investigation(s) should be commenced at a sufficiently early stage to inform the initial planning/concept design stage and to coordinate with other surveys and site investigations, for example topographical / channel surveys and ground investigations.

The investigation needs to recognise and understand the relationship between the bridge, the watercourse and the floodplain. Typically, a bridge spans the main channel of a watercourse and perhaps a section of the floodplain. The highway approaches to the bridge are typically on embankments which can obstruct flow along the floodplain. Any out of channel flood flow will therefore return to the watercourse at the bridge site or overtop the roadways. Zones of turbulence are established where out of bank flow returns to the main channel and again where flow expands back onto the floodplain downstream of the bridge and scour may occur at the approach embankment and around the bridge abutments. In addition, piers and abutments may present obstacles to flows in the main channel, creating conditions for local scour caused by turbulence around the foundations.

C2 Scope of investigations

Every bridge site subject to hydraulic action should be reviewed in order to establish its potential vulnerability to damage by hydraulic actions and the potential effect of the structure on the water environment.

The scope of the investigations should cover the following aspects. Appropriate specialist advice should be sought where necessary:

1) channel stability and morphological conditions;
2) flood behaviour;
3) debris potential;
4) bed material;
5) sub-surface material and properties;
6) geometry of the watercourse and floodplain in the vicinity of the site;
7) range of water levels, flows and velocities expected at the site.

C3 Sources of information

Table C.1 provides guidance on data sets that can inform the investigation, particularly initial desk studies.

Table C.1 Sources of information

Dataset	Typical use	Comment
Old Ordnance Survey maps	Channel stability/ morphology	Scale of at least 1:10,000 preferred for plotting changes to channel plan form for most UK rivers
Aerial photographs	Channel stability/ morphology	

Table C.1 Sources of information (continued)

Dataset	**Typical use**	**Comment**
River habitat survey reports	Channel stability/ morphology	These reports can include observations of bed and bank material, channel form and signs of erosion or channel movement
Satellite imagery	Channel stability/ morphology, watercourse geometry, debris potential	
Current Ordnance Survey maps	Watercourse geometry, debris potential	1:1250 or 1:2500 scale maps can provide an initial indication of channel size when checked against other data sets
Digital terrain data	Watercourse geometry, flood behaviour	LIDAR data is freely available for most watercourses in the UK from the various UK government data sharing web portals. LIDAR can provide an initial indication of channel size and gradient, flood plain extent and flood flow routes
Flood maps	Flood behaviour, debris potential	Flood maps are typically available online and can be used to assess flood flow routes and extents. Some maps can provide enhanced information such as depths and speed of floodwaters.
Existing channel surveys/ topographical surveys	Watercourse geometry, water levels, bed material.	Channel and/or topographic surveys may be already available at a site from previous studies, such as for flood mapping, or earlier phases of the scheme. Channel surveys can provide an indication of bed and bank material
Hydrological and/or hydraulic models and assessments	Watercourse geometry flood behaviour, water levels, flows and velocities	These may be already available at a site from previous studies, such as for flood mapping, or be developed specifically for the scheme, for example to support a Flood Risk Assessment
Flow and level gauge records	Water levels & flows	River level gauges and gauge boards can supplement the flow gauges included in the National River Flow Archive National river flow archive [Ref 30.I]

Table C.1 Sources of information (continued)

Dataset	Typical use	Comment
Anecdotal observations, flood records and flood level markers	Flood behaviour, water levels & flows	Local reports, newspaper articles, notable flood markers can provide information although they may not reflect design conditions. Local residents can provide information on the typical watercourse regime. Anecdotal records and observations should be used with caution as they may be unverified
Geology and soil maps and borehole records	Channel stability/ morphology, flood behaviour, bed material, sub-surface material properties.	BGS maps BGS [Ref 9.I] and borehole records can provide an initial indication of the material at the site. The presence of river alluvium deposits can indicate potential floodplain extents and channel movement
Site walkover and field survey	Channel stability/ morphology, flood behaviour, watercourse geometry, bed material, debris potential, water levels and flows	Refer to guidance in sub-section C3.1
Topographic and channel survey	Watercourse geometry water levels	Refer to guidance in sub-section C3.2
Ground investigations	Bed material, sub-surface material properties	Refer to guidance in section C3.3

C3.1 Site walkovers and field surveys

Site walkovers and field surveys can be valuable to supplement desktop studies and inform the conceptual representation of the watercourse at the site. The investigation should be performed by staff with experience of the design of new watercourse crossings. There can be significant overlap with the requirements of field surveys to support Water Framework Directive, environmental impact and flood risk assessments for the scheme and therefore joint surveys may be worthwhile. The proposed scheme may provide opportunities to improve the existing water environment.

It is beneficial for the engineer to have an understanding of the proposed scheme, design criteria and requirements, potential specification and construction methods. However it should be noted that the purpose of the investigation is to inform the design and should take place before the design is too far progressed.

The following sections provide guidance on factors to be observed and considered when evaluating a proposed bridge site. Observations should be made upstream and downstream as well as at the site itself.

Banks:

1) Stable: Natural vegetation, trees, existing bank stabilisation measures such as riprap, paving, gabions, channel stabilisation measures such as dikes and groynes;

2) Unstable: Undermining of banks, evidence of lateral movement, damage to river stabilisation works etc.

Main Channel:

1) Type of channel: straight, meandering or braided; presence of low-flow channels; single or multi-stage channels;
2) Channel gradient: Steep, shallow, varying?;
3) Orientation of the main channel relative to proposed structure openings;
4) Existence of islands, bars, debris, cattle guards, fences that may affect flows;
5) Aggrading or degrading bed;
6) Bed material: General observations of the nature of the bed material, grain size, cohesive / non-cohesive etc; depth of soft bed material - a survey staff or pole driven through the soft bed material can determine hard and soft bed levels and help to identify areas of deposition or hidden scour pockets;
7) Evidence of watercourse movement with respect to the bridge site, or movement of flow channel(s) within the main watercourse channel;
8) Geometry: initial estimates can be obtained from site observations and measurements;
9) Evidence of scour or erosion.

Floodplain:

1) Size of floodplain;
2) Floodplain flow patterns - will flow overtop the road or return to the main channel? once out of channel will flood flows follow the watercourse alignment or follow a different route?;
3) Downstream: is the channel clear and open so that contracted flow at the site can return smoothly to the floodplain, or is it restricted and blocked by dikes, developments, trees, debris or other obstructions
4) Size of any flood relief openings;
5) Extent of floodplain development and any obstruction of flow towards the bridge and its approaches;
6) Evidence of scour or erosion.

Debris:

1) Extent of debris in upstream channel;
2) Type of debris that may be carried in flood: bankside and floodplain vegetation, presence of urban debris;
3) Existing structures that may prevent debris reaching the site.

Other features:

1) Existence of upstream tributaries, bridges, dams, weirs or other features that may affect flow conditions at the site;
2) Presence of drainage outfalls etc in the vicinity of the site;
3) Downstream dams, structures, or confluence with larger watercourses which may cause variable water levels downstream of the proposed structure. This may create conditions for higher velocity through the bridge and/or higher water levels at the bridge;
4) Water level influenced by tides. This may create conditions for higher velocity through the bridge and/or higher water levels at the bridge;
5) Evidence of engineering works, such as dredging, which could affect flows and bed levels at the bridge;
6) Environmental constraints that may influence the design.

Field measurements of velocity at the site can be useful, particularly where flow velocity varies considerably across the width of the channel. The values can be used to help define the relationship between local and section mean velocities. Velocities in the main channel of a river with floodplains tend not to increase substantially in flood conditions therefore measurements taken at or near to bank-full can provide an indication of velocities in flood conditions.

C3.2 Topographic and channel survey

A cross section survey of the channel can be undertaken to inform the assessment and design. This should include at least five cross sections: one at each face, at the centre line and upstream and downstream of the contraction and expansion reaches. The channel survey should be at sufficient resolution to provide an accurate profile of the channel bed. Standard topographic surveys which simply show top and bottom of bank levels, and may not extend into the centre of the channel, are unlikely to be sufficient. Hard and soft bed levels should be identified on the survey.

The spacing of channel sections should be sufficient to identify changes in channel geometry and gradient. For most UK rivers a spacing of 50m in the vicinity of the site is appropriate for the purposes of hydraulic modelling, although closer spacing, or a full bathymetric survey, may be needed for greater accuracy at the crossing location.

Where backwater calculations or hydraulic models are developed the survey should extend a sufficient distance upstream and downstream of the site to ensure that boundary assumptions do not unduly affect the results at the site. As a general rule the downstream extent should be greater than the backwater length, which can be estimated from equation C.1.

Equation C.1 Backwater Length

$$L = 0.7\frac{D}{S}$$

where:

L	is the backwater length
D	is the flow depth
S	is the channel gradient

C3.3 Bed material and ground investigation

Geotechnical data will be required at the site to inform the foundation design and the scour assessment. The requirements of the scour assessment and design should be considered in the specification of ground investigations at the site. The bed material should be characterised to inform the scour assessment and protection design. CIRIA C742 [Ref 19.I] provides further advice.

Where the bed material is mainly non-cohesive (for example alluvial and gravel rivers) the sediment characteristics are normally obtained by sieve samples and grading. Samples should be taken from several locations to identify any significant spatial variations. The dry density or specific gravity of the particles is also required. The assessment of scour depth typically uses the mean particle size d_{50} .

The investigation should include assessment and analysis of the bed material under the top armour layer, as this coarser material which can develop may be washed away in a flood, leaving more erodible material behind. Bed samples should be taken from a depth around 0.5m below the surface to be representative of the underlying material. Initial investigations, reliant on visual observations, should look for signs of armouring and not assume the surface layer is representative of the underlying material.

Where material is believed to vary with depth samples should extend below the maximum expected scour depth. It should be noted that locations which are significant for scour assessment may not coincide with the sample locations needed for foundation design; additional samples may be required.

Where the bed material comprises coarse gravels or boulders, direct measurement rather than sieve sampling, may be needed. Care should be taken to measure a representative sample of sizes. Further guidance is provided in BS 3680-10E [Ref 9.N].

In estuaries or lower lying rivers the bed material often comprises a mixture of cohesive and non-cohesive material. The resistance of such materials to scour can vary significantly with depth and the level of consolidation which has occurred over time. Bed samples should be analysed for their density and moisture content and settling velocity of smaller particles rather than sieve sampling may be used to characterise the sediment and erosion potential. Further guidance is provided in BS 3680-10C [Ref 8.N].