VOLUME 1 HIGHWAY STRUCTURES: DESIGN (SUBSTRUCTURES AND SPECIAL STRUCTURES), MATERIALS

SECTION 3 SPECIAL STRUCTURES

PART 6

BA 59/94

THE DESIGN OF HIGHWAY BRIDGES FOR HYDRAULIC ACTION

SUMMARY

This Advice Note supplements the extant requirements of the Overseeing Organisations in respect of the hydraulic aspects of bridge design.

INSTRUCTIONS FOR USE

This is a new document to be incorporated into the Manual.

1. Insert BA 59/94 into Volume 1 Section 3.

2. Archive this sheet as appropriate

Note: The new contents page for Volume 1 dated May 1994 is available with BD 60 (DMRB 1.3.5).

THE HIGHWAYS AGENCY BA 59/94

THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF
THE ENVIRONMENT FOR NORTHERN IRELAND

The Design of Highway Bridges for Hydraulic Action

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VOLUME 1	HIGHWAY STRUCTURES: APPROVAL PROCEDURES AND GENERAL DESIGN
SECTION 3	GENERAL DESIGN

PART 6

BA 59/94

THE DESIGN OF HIGHWAY BRIDGES FOR HYDRAULIC ACTION

Contents

1. INTRODUCTION

General

1.1 The most common cause of highway bridge failure is due to adverse hydraulic action. It is therefore essential that sufficient attention is paid to the prevention of such failure when designing new bridges over rivers, estuaries or flood-plains. A bridge's vulnerability to damage, or loss, as a result of hydraulic action, scour in particular, needs to be minimised. The purpose of this Advice Note is to provide appropriate guidance for engineers undertaking the design of bridges across watercourses. It is intended to aid in the determination of the hydraulic conditions which may occur and to quantify their associated effects upon the bridge structure which need to be adequately resisted. It will also assist the engineer in making decisions regarding prudent design safeguards.

1.2 It is emphasised that the design of bridges across watercourses requires a multi-disciplinary approach, involving structural, geotechnical as well as specialist hydrological and hydraulics expertise.

1.3 This Advice Note is intended to provide the best and most up to date advice, currently available, for the design of bridges to resist hydraulic actions. However, it is not yet always possible to offer a single, best method or equation for a particular calculation. The Advice Note does provide information and guidance on commonly used methods and equations for particular aspects of hydraulic design - allowing comparison and sensitivity checking of any results obtained. Wherever this approach may be considered insufficient, or the particular circumstances do not fit well with any assumptions made, then the designer is advised to seek specialist advice.

Design Principles

1.4 The two overall objectives for the hydraulic design of bridges are:

i. The effect of constructing the bridge on the existing water regime should be kept to the minimum.

ii. The structural design of the bridge should aim to prevent failure under the various types of hydraulic actions described in Chapter 2.

1.5 *To achieve the first objective of 1.4,* it is essential, at the planning stage, that close consultation is carried out with the National Rivers Authority (NRA), (or other applicable authority), and other relevant bodies as required by the Overseeing Organisation. In Scotland, the River Purification Board, Water Authority and land owners should be consulted. The design should satisfy the afflux flow capacity and navigational requirements, as necessary, (see 3.2 to 3.5). The acceptability of afflux (eg Steps 5 and 9 of Appendix A) will be determined in consultation with the NRA etc, as the issue of concern is likely to be any increase in risk of flooding upstream of the proposed bridge. The design should take into account possible effects on the stability of the river due to channel changes etc and also on the stability of any present or proposed adjacent structures downstream of the site.

1.6 *To achieve the second objective of 1.4,* it is necessary to calculate potential scour depths and various hydrodynamic forces, in order to check the adequacy of the structural design. It is also necessary to provide adequate freeboard for the passage of vessels, if relevant, and floating debris. Where necessary, the structure needs to be designed to resist ship collision forces as well as ice flow forces.

Design Process

1.7 An example procedure for preliminary design is shown in the flow diagram of Appendix A. More detailed advice for the steps involved is to be found in "Hydraulic Factors in Bridge Design" by Farraday and Charlton, [12]. It is necessary to carry out site investigations and collect various data before design can proceed. Guidance for this is contained in Appendix E.

Scope

1.8 This Advice Note is applicable to all proposed bridges over rivers, estuaries and flood-plains. It also applies equally to temporary crossings except that reduced return periods may be used in the calculations.

1.9 Advice contained herein is limited to the hydraulics aspect of the design of a bridge and is intended only to supplement the extant requirements of the Overseeing Organisation regarding overall design.

Implementation

1.10 This Advice Note should be used forthwith for all schemes currently being prepared and within the scope of 1.8, provided that, in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay. Design Organisations should confirm its application to particular schemes with the Overseeing Organisation.

Definitions

1.11 For definition of the hydraulics terminology used in this Advice Note refer to Chapter 5, Glossary of Terms.

2. TYPES OF FAILURE DUE TO HYDRAULIC ACTION

Failure Due to Scour

2.1 Most rivers beds and banks consist of, more or less, mobile material. During a flood, the bed level may fall as bed material is transported away by the moving water. Construction of bridge foundations within a river can result in additional lowering of the bed level at the bridge site. This extra erosion, or scour, has two possible causes:

i. From a general increase in flow velocity due to the constriction of the channel (general scour).

ii. From a local disturbance of the flow due to the bridge piers or abutments (local scour).

Where both types of scour occur, the total depth of scour is the sum of general and local scour. Additional scour may be caused, on navigable waterways, by the action of vessels causing rapid displacement of water and high local flow rates.

2.2 A bridge constructed on spread foundations will be at serious risk from scour when the scour level reaches the level of the base of a footing. When a substructure member is subject to lateral loads, which are partially or wholly resisted by resultant soil pressure, then the foundation may be at risk before scour reaches the footing level. Such lateral forces may be increased by hydrodynamic effects. The bearing capacity of a foundation may also be reduced due to a loss of overburden caused by scour.

2.3 Scour adjacent to a piled foundation may result in a loss of skin friction and reduction in load bearing capacity of the piles, even if they have been only partially exposed. Additional bending stresses may also be induced in the piles due to hydrodynamic forces and the loss of lateral restraint.

2.4 Overtopping of the approach roadways and turbulent flow adjacent to approach embankments can lead to erosion and scour of the side slopes and toes of the embankments. This may lead to instability of the approach embankments and possible loss of the road. Loss of fill material around and behind any wing walls can lead to instability and failure of the wing walls.

2.5 Many features of the bridge and river affect the depth of scour. However, the complex nature of the problem means that accurate prediction of scour is generally not possible. It is possible to identify the most important features and to predict how scour may develop due to each feature. Combining the effects from such significant features will allow an assessment to be made of the expected severity of scour.

2.6 Local scour at a bridge pier is normally greatest near the upstream nose of the pier. However, due to local geometrical effects and the nature of the flow or sediment, there may be cases where the local scour is greatest in other areas adjacent to the pier. This is particularly true if the direction of flow deviates significantly from the pier alignment.

2.7 The depth of the foundations is important in determining the risk to a bridge from a given degree of scour. Deep foundations subject to severe scour may be safer than a shallow footing subject to moderate scour.

2.8 There are a significant number of bridges where the flow is affected by tidal action. Depending upon the location of the bridge the predominant discharge can vary between fluvial and tidal. In all cases the flow will be in at least two different directions at different times and flow patterns may vary significantly. A pier which may be well aligned and subject to little scour during one part of the tidal cycle may be poorly aligned and subject to scour during another part.

Failure Due to Bank Erosion

2.9 Most natural rivers tend to change their course with time. One mechanism by which this occurs is bank erosion. A pier or abutment located on a flood plain or in an estuary may be placed at risk if the main channel moves sufficiently close to cause loss of support or undermining. Bank erosion may occur very slowly or be very rapid. It will normally be most rapid during floods. The rate of bank erosion depends partly on the character of the river - a river with a steep longitudinal gradient and high flow velocities will, in general, be more active and prone to bank erosion than a river with a fairly flat slope and lower velocities.

Failure Due to Hydraulic Forces on Piers

2.10 Water flowing past a bridge pier exerts a force on the pier. This force can be resolved into two components; one along the direction of flow, which is referred to as the drag force and one normal to the direction of flow, which is referred to as the lift force.

2.11 The forces involved depend somewhat upon the depth of flow and the length of the pier but there is a significant dependence upon the flow velocity. If the flow is aligned with the pier, the lift force is normally zero, but as the angle of attack increases the lift force (for non-circular piers) increases rapidly. The ability of a pier to withstand drag and lift forces will depend upon the construction of the bridge and its foundation details. This ability may be reduced during a flood if significant scour takes place around the base of a pier.

Failure Due to Hydraulic Forces on Bridge Decks

2.12 If the water level reaches above the soffit level of a bridge, or the springing in the case of an arch, the flowing water will exert a force on the bridge deck or arch intrados. The drag on the deck may be calculated in a similar way to drag on a pier, and similarly, is very dependent on the flow velocity. Such a force applied to the deck of the bridge is potentially dangerous due to:

i. The large overturning moment produced about the pier foundations.

ii. The increase in horizontal shearing force at the deck/pier or barrel/springing i nterface.

If it is known that historic flood levels have approached the proposed bridge deck height, it may be appropriate to carry out a site-specific study to assess future flood levels, flow velocities, hydraulic forces and the resistance of the bridge to these forces. Simply supported bridge decks may be lifted from their supports. Uplift on the soffit of masonry/brick arch may reduce the compression force in the arch and promote collapse.

Failure Due to Debris

2.13 Build up of trash and debris against bridge components can significantly affect the hydraulic performance of bridges. Difficulties are normally associated with small single span bridges which tend to be more easily blocked than large multi-span structures. For single span bridges the blockage can be extensive, reaching up to 90% of the bridge opening. This may result in large increases in water level upstream and associated flooding. Debris may partially restrict the flow leading to significant scour around piers or abutments - threatening the safety of the structure. The presence, in the catchment above a bridge, of large afforested areas with shallow roots on steep slopes produces a known risk of flood debris blockage and cascade type failure of a sequence of bridges down a watercourse.

2.14 Debris which is caught against or between piers can result in enhanced hydraulic forces by increasing the effective pier width. Floating debris which collides with piers can cause dynamic loading. The extent of these forces is not easily predicted and both will usually be most severe when the river is in flood.

Failure Due to Ice Forces

2.15 The critical mode of ice action is most likely to be the impact of large sheets of ice with piers or piles as ice break up occurs. In the case of vertical piers the colliding ice either crushes around the contact perimeter or splits in the case of small sheets.

Failure Due to Ship Collision

2.16 Ships and smaller craft may collide with bridge piers or superstructures in periods of adverse weather or when there has been a loss of control of the ship eg a loss of steering. The size, speed and type of critical ship to be designed for should be decided, in agreement with the Overseeing Organisation, after discussions with the relevant navigation authority, as should the anticipated worst-case river currents and flood levels. Some guidance on this specialist topic is available from references [24] and [29]. For major esturial crossings over busy shipping waters specialist advice should be sought. Where fendering is required to protect piers from ship collision guidance may be obtained from BS 6349: Part 4: Design of fendering and mooring systems.

3. DESIGN

General

3.1 It is recommended that the overall design should follow the general procedure shown in the flow diagram of Appendix A. This will involve the calculation of afflux, depth of scour, various hydraulic loads and the choice of appropriate scour protection measures. The return periods used in the calculations should be as recommended in 3.8. The following sections, together with the relevant Appendices B, C and D, contain current guidance available for these aspects.

Determination of Afflux

3.2 Bridges can cause a significant impediment to the river or tidal flow. The increase in water level upstream of a bridge over that which would have occurred if the bridge were absent is commonly termed the afflux. The afflux at a bridge is difficult to calculate accurately. For a given cross-sectional area of an opening, the greater the wetted perimeter the greater is the afflux. Therefore, it should be considered at the planning stage that a smaller number of large openings is preferable to a larger number of small openings.

3.3 A number of methods are available for calculating afflux. The most widely used is the US Bureau of Public Roads (USBPR) method, [45]. This is applicable to bridges with vertical piers and horizontal soffits. The method is described in Appendix C, which also refers to a method for estimating afflux at arch bridges. Further, detailed information on methods for estimating afflux may be found in HR Wallingford Ltd report EX 2695 "The effects of highway construction on flood plains", (October 1993: Draft), [19].

3.4 To control the afflux at a bridge crossing, particularly where long embankments across the flood plain are required, it may be necessary to provide additional flood openings. In simple cases, methods of calculating afflux such as the USBPR method can be used to determine the length of openings required. The calculations will indicate the overall length of openings required to achieve a certain afflux but the appropriate location for these openings will depend upon the local geometry. Information on the magnitude and direction of flow over the flood plain under existing conditions may be of value in selecting appropriate locations. It should be noted that over-provision of flood relief openings can add excessive unnecessary cost to a scheme and should be avoided.

3.5 Methods of afflux calculations are of limited accuracy and are not applicable in complex situations. In these situations or where greater accuracy is required, physical or mathematical models may be used. Such models can be used to size the openings and determine their appropriate locations. The savings resulting from a model study usually exceed the cost of the model. In particular, model studies can help to avoid gross over-provision of bridge opening for minimal improvements in afflux levels.

Design for Structural Stability

3.6 In order to be satisfied that the structure is adequate against hydraulic action, the structural design needs to be carried out in the following steps:

i. Calculate the total potential scour depth and check that the structural design is adequate with that depth of scour (See 3.9 to 3.27).

ii. Incorporate appropriate scour protection measures in the design (See 3.28 to 3.37).

iii. Calculate the loads on the structure (see 3.39 to 3.45) and its foundations (see 3.46 to 3.48) and check for structural adequacy.

Design Flows

3.7 The Flood Studies Report (NERC, 1975), and its associated Supplementary Reports, [34], provide methods for determining flows with a specified return period for both gauged and ungauged catchments. Some sites have data from gauging records held by the NRA regions and if these data are available it is recommended that they are used. In Scotland the River Purification Board should be consulted. An alternative source of data is the Surface Water Archive held by the Institute of Hydrology. This incorporates data from over 1200 gauging stations throughout the UK.

3.8 Calculations should be based on a range of flood return periods of up to 200 years in order to assess which events produce the worst effects from considering different flow velocities and depths. 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 flooding discharge rates.

Estimating Effects of General and Local Scour

3.9 The following procedure, broken down into ten stages, may be used to estimate the effects of scour at a bridge:

i. Obtain all relevant data.

ii. Select critical return period(s) and calculate design discharge.

iii. Draw cross-sections at the proposed bridge site showing proposed foundation depths. Additional cross-sections should also be taken in the neighbourhood of the bridge site, eg within approximately 5 river widths upstream and downstream of the bridge site. These areas should be inspected for signs of scour or irregularities which might influence flow conditions or bed levels at the bridge site.

iv. Decide whether long term bed level variation (eg. progressive degradation) needs to be allowed for. If so, assess any long term changes and superimpose their values on measured bed levels. Levels may change, among other reasons, due to mining subsidence. For bridges in tidal reaches, MAFF advice should be sought regarding expected rates of sea level rise. In Scotland, the Scottish Office Environment Department should be consulted.

v. Calculate design water levels and velocities. Establish or estimate direction of flow trajectories in relation to alignment of bridge piers - flow trajectories may be significantly different at various flood conditions than at normal flow conditions.

vi. Calculate hydraulic parameters eg. Froude numbers and floodplain/main channel discharge split.

vii. Calculate general scour depths. Redistribute general scour to the most critical bed profile, taking into account the layout of the bridge crossing and its foundation details.

viii. If measurements of bed level at the bridge are available, compare them with the calculated bed levels of (vii).

ix. Calculate local scour at each potentially vulnerable foundation, including abutments. Superimpose local scour upon general scour, assuming the top width of local scour holes, measured from the pier face, to be from approximately 1.0 to 2.8 times the local scour depth, [50]. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the total scour may increase.

x. Interpret scour depths in the light of potential effects upon the structural strength and stability of the foundations.

Calculation of Scour Potential (general comment)

3.10 Assessment of scour potential should be carried out with the help of specialist expertise. The designer should be aware of the limitations of any methods used and reference made to the appropriate texts contained in the References and Bibliography (Chapter 4).

3.11 Where possible, and appropriate, evidence of scour from field measurements should be used to verify calculated scour depths.

3.12 Example scour calculations are given in Appendix B, in order to illustrate available methods and the data required.

Progressive Degradation and Aggradation

3.13 Progressive degradation results from modification of the `stable regime' conditions to which a river has become adjusted. This may be as a result of alterations to water or sediment flows in the river. The result of progressive degradation at a bridge site will be a lowering of bed level, which may place the foundations at risk. Progressive degradation can affect a long reach of river, over many kilometres and over a period of many years.

3.14 Degradation will normally increase the risk to bridge structures from scour, however, in some cases, aggradation may occur - this will cause increased water levels but will probably reduce the risk from scour.

3.15 It is not possible to give guidelines on how any particular change will affect conditions at a bridge. The following list identifies some river works which could affect conditions at a bridge site, particularly if carried out after the bridge was constructed:

- Construction of flood embankments
- Construction of flood detention basins
- Channel improvement schemes such as dredging, weed clearance, re-alignment

- Reservoir impoundment
- Construction or removal of weirs
- Changes in water abstraction patterns
- Schemes for transfer of water between river basins
- Land drainage schemes.

The effects of many of these works can be predicted using numerical sediment transport models, or by careful extrapolation of present trends. Bed levels and water levels can be predicted for many years into the future. It is advised in the FHWA (1988), [46], recommendations that this type of study is carried out to predict bed levels (in the absence of general or local scour) in cases where long term change is thought to be likely are followed. If the channel is expected to degrade, then the estimate of long term bed elevation should be used to calculate general and local scour depth levels.

General Scour

3.16 General scour (also known as contraction scour or constriction scour) occurs in a confined section of a river and results in a lowering of the bed level across the width of the river. In contrast to local scour, few formulae are available to calculate general scour. Formulae are either based on measured dimensions of river channels, or on analysis of channel constrictions based on sediment transport relationships.

3.17 The most widely used methods for predicting general scour are based on sediment transporting capacities of unconstricted and constricted reaches. Neill (1973), [33], gives a method based on the ratio between discharge intensity (ie flow per unit width) in the constricted reach and discharge intensity at bank-full flow of a typical unconstricted reach. Complications are introduced by flood plain flows, but FHWA, [50], gives methods for the most common cases of general scour. These include contraction of the river channel either with or without floodplain flow, and general scour at a relief bridge on a floodplain.

3.18 The above methods for general scour calculate mean depths. Factors are recommended to account for the non-uniformity of general scour (Farraday and Charlton 1983), [12]. The factors range from 1.25 for straight reaches to 2.0 for a right angled abrupt turn. The location in the river of the maximum scour depth may be difficult to estimate. For channel bends, the maximum depth will normally occur towards the outside of the bend.

3.19 In complex cases where the above methods may not be applicable, or, in cases where more reliable and accurate estimates are required, numerical or physical modelling is recommended.

3.20 Examples of calculation of general scour are given in Appendix B.

Local Scour

3.21 Many formulae are available for predicting local scour at piers. The formulae differ in the variables which are included, but there is general agreement that local scour is strongly dependent on pier width, and can also depend on other factors such as the depth and velocity of flow, alignment of pier to the flow, bed material properties and the geometry of the pier.

3.22 When applied to a particular bridge, predictions given be different formulae can vary widely. Appendix B includes example calculations using three of the more commonly used methods. Methods are available for predicting local scour at abutments (eg FHWA, 1991), [50], however, if applied to typical UK cases, these methods tend to predict large depths of scour which are probably unrealistic in many cases.

Total (Combined) Scour

3.23 Once estimates of the different types of scour have been obtained it is necessary to combine them to produce estimates of total scour at each part of the bridge site.

3.24 The most common method of combining the estimates is to assume that local and general scour are independent. Estimates of local scour can then be added to estimates of general scour to obtain the total scour at the pier or abutment. It is assumed that changes to hydraulic properties from general scour do not affect local scour, and vice-versa.

3.25 A more refined method is to first calculate general scour and then to use the hydraulic properties of the general-scoured cross section to calculate local scour. This method normally gives a lower estimate of total scour. The method will be more accurate, particularly for cases of large depths of general scour, but is more time-consuming to apply than the simple method given in (3.24).

3.26 It must be stressed that calculations of scour depth are only approximate. Parameter values used in the calculations should be verified, if possible, by measurements from similar structures in similar locations. Alternatively, the use of physical models may be justified, particularly for more complex cases where existing prediction methods are not valid.

3.27 For some cases, estimates from more than one prediction method can be calculated. Estimates may vary widely, and a credible `design' value must be selected from the range of estimated values produced. The use of sensitivity analysis for the parameters used may be of value in reaching such decisions.

Scour Protection Measures for River Bed and Banks

3.28 It may not be economical to design the elements of a bridge to withstand the maximum possible scour. An alternative is to carry out scour protection works to prevent or reduce scour of the bed and banks. The following methods are commonly used to protect against possible scour:

- Paved inverts
- Enlargements or plinths to piers and abutments
- Stone or grout bags
- Sheet piling
- Gabion and grouted mattresses

3.29 *Paved invert on the river bed below the bridge* can be an effective measure against both general and local scour. Paved inverts are commonly constructed from concrete, masonry or brick. This form of protection has the advantage of forming a hard invert which resists further scour and can be easily inspected for signs of deterioration. The following should be noted:

i. The paved invert should protect the bed in the region of vulnerable piers and abutments and should extend a sufficient distance upstream and downstream of the structure. For short spans it may be economical to place a single paved invert across the width of the river, whereas larger spans may require separate paved inverts to protect each pier or abutment.

ii. Both the upstream and the downstream edges of the paved invert should be toed-in to the river bed for a sufficient depth, typically about 1.5 metres in the UK, to prevent scour from undermining the paving.

iii. The paved invert should not project above the bed, for this would encourage local scour at the upstream edge of the paving. This is particularly important during floods. The design of the invert should take account of the fall in bed level during floods.

iv. Measures should be carried out to prevent scour occurring at the upstream and downstream edges of the invert.

v. Any reduced bed friction across the paved invert results in increased velocity of the water. This can cause a standing wave, with increased turbulence, downstream of the bridge. The result may be scour downstream of the bridge which will then migrate back towards the bridge (see ii, above).

3.30 *Enlargements or plinths added to piers and abutments* to protect the foundations from local scour should be placed with the top level of the enlargement below river bed level. Again, allowance should be made for the possibility of a fall in bed levels during a flood. If the enlargement projects above river bed level, scour may be made worse in two ways:

i. The increase in width of the structure tends to cause an increase in the depth of local scour adjacent to the structure.

ii. The overall flow area of the crossing is reduced so that general scour at the bridge site will tend to increase.

A shaped cut-water plinth above river bed level may be effective in reducing local scour. The reduction in local scour will be effected by reducing turbulence due to improving the shape of the obstruction and/or realigning the obstruction in the direction of the flood flow. General scour will tend to increase since the flow area of the crossing will be reduced. The treatment of the edge of the plinth needs careful consideration to prevent scour from occurring beneath it. The overall effect may, however, be an improvement in some cases.

3.31 The construction of enlargements or plinths may require local areas to be dewatered, either totally or in sections. There could be little advantage over the construction of a full paved invert other than a possible saving in cost.

3.32 *Stone or grout bags* may be placed locally around piers and abutments, with the aim of increasing the resistance of the river bed to scour. The bed should be pre-excavated and the stone or grout bags carefully placed in the excavation to lie slightly below the general bed level. If the material is dumped on the bed or banks, particularly at a constricted crossing, the flow area may be further reduced and there could be an increase in both local and general scour depths.

3.33 Stone placement protection, to be effective, requires the use of carefully selected stone in terms of material properties, size and shape. Ideally stone should be abrasion resistant and of sufficient weight to prevent it being moved by the flow. The size (weight) of the stone should be roughly proportional to the sixth power of the flow velocity.

3.34 *Sheet piling* is effective for river bank protection. Sheet piling can also be used as a method of scour protection for piers or abutments, particularly those with shallow foundations. It increases, however, the effective width of the pier or abutment and hence may increase the depth of scour. This increase in scour depth should be allowed for when designing the sheet piling.

3.35 *Gabion and grouted mattresses* may be placed locally around piers and abutments or across the full width of the invert to increase the resistance of the river bed to scour. The river bed should be pre-excavated so that the mattresses lie below bed level. Mattresses may also be used to protect the river banks.

3.36 The mesh frame of gabion mattresses should be sufficiently durable to hold the contained stone in place. Therefore, smaller size stone can be used than discussed in 3.32 and 3.33. The durability of all the materials used and their structural interlocking should be considered.

3.37 *Other measures* which can be undertaken to prevent future scour are:

i. Channel Improvements

ii. Construction of a weir downstream to maintain higher bed levels at the bridge

iii. Increase the size of the waterway opening

Specific Design Considerations

3.38 *Freeboard* should be such that bridge soffit levels at flood spans are 600mm above the design flood level (or maximum known flood level on minor watercourses) in order to allow floating debris to pass freely through the structure. In determining the freeboard, allowance should also be made for afflux.

3.39 *Loading* **-** the following loads, in addition to those specified in BD 37 (DMRB 1.3), should be considered in the design:

i. Hydrodynamic forces (see 3.41)

ii. Ice forces, if appropriate (see 3.42)

iii. Debris forces (see 3.43 to 3.44)

iv. Ship collision forces, if appropriate (see 3.45).

3.40 The loads calculated using the methods recommended in this Advice Note should be considered, unless otherwise specified, as nominal loads. Unless otherwise specified the design checks should be carried out both at the ultimate limit state (ULS) and the serviceability limit state (SIS) using the combination rules and the partial factors of safety (γ_{fL}) given in Table 3.1. This table is an extension of BD 37 (DMRB 1.3), Table 1. The hydrodynamic and debris forces should be considered in Load Combination 2, however, for these checks, no wind loading should be assumed to be present. The ice and ship collision forces should be considered separately in Load Combination 4, but no other accidental load should be considered to act together with them.

3.41 *Hydrodynamic forces* **-** the method given in Appendix D should be used to calculate the hydrodynamic loading. For eg abutment structures, with water loading on one side only, the hydrostatic water forces should be added to the calculated hydrodynamic forces and the same partial factors used, as given in Table 3/1.

3.42 *Ice forces* **-** it is unusual for ice loading to be a significant problem in the UK. If ice loading is required for a bridge design then the method given in Appendix D is recommended. Design checks using this loading are required only at the ultimate limit state (ULS).

3.43 *Debris forces* **-** the type of debris occurring in a river will depend upon the size and characteristics of the river and the area through which it flows. An investigation should be carried out to determine the type and size of floating debris to be expected at the bridge site.

3.44 A minimum allowance shall be made for a debris collision force equivalent to that exerted by a 3 tonne log travelling at the stream velocity calculated for the peak design event and arrested within distances of 150mm for slender column type piers and 75mm for massive, non-yielding type piers.

Load	**Limit State**	**γ_{fL} to be considered in combination [2]**	**[4]**
Hydrodynamic forces on bridge supports			
During erection	ULS SLS	1.10 1.00	
With dead load plus superimposed dead load only and for members primarily resisting water forces	ULS SLS	1.40 1.00	
With dead load plus superimposed dead load plus other appropriate Combination 2 loads, excepting wind load	ULS SLS	1.10 1.00	
Relieving effect of water (when relevant)	ULS SLS	1.00 1.00	
Flood debris collision forces on bridge supports			
During erection	ULS SLS	1.40 1.15	
With dead load and superimposed dead load only	ULS SLS	1.50 1.20	
With dead load and superimposed dead load plus other appropriate Combination 2 loads, excepting wind load	ULS SLS	1.50 1.20	
Ice forces			
<u>not</u> to be combined with other accidental loads or live loads	ULS		1.5
Ship collision forces			
<u>not</u> to be combined with other accidental loads or live loads	ULS		1.0

ULS: ultimate limit state
SLS: serviceability limit state

TABLE 3/1: Hydraulic Load Combinations

3.45 *Ship collision* - if ship collision needs to be considered then specialist advice may need to be sought (see 2.16). Design checks using this loading are required only at the ultimate limit state (ULS). A ship collision force should be derived as the worst credible force that could be considered to act. Such a derived force may be considered to be the ultimate force for the circumstances pertaining to a particular bridge site. The average force acting during the short time of impact may be applied either statically or dynamically.

Foundations

3.46 Different bed materials scour at different rates. Loose granular soils have a low resistance to scour. Ultimately scour in cohesive or cemented soils will be as deep as scour in sand-bed streams. Scour will reach its maximum depth in sand and gravel bed materials within hours; cohesive bed materials in days; glacial tills, sandstones and shales in months; limestones in years and dense granites in centuries. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

3.47 Bridge foundation analysis should be carried out on the basis that all stream-bed material within the scour prism above the total scour depth will have been removed and is not available for bearing or lateral support. In the case of a piled foundation, the piling should be designed for reduced lateral restraint and for column action due to the increase in unsupported pile length after scour. In areas where the local scour is confined to the proximity of the pile cap, the lateral ground stresses on the pile length which remains embedded may not be significantly reduced from the pre-local scour conditions. The depth of local scour and volume of soil removed from the pile group should be considered when computing the pile embedment required to sustain vertical load.

3.48 Some specific points to note for different types of foundations are:

a) Spread footings on soil

i. Position the top of the footing below the calculated scour depth

ii, Ensure that the bottom of the footing is at least 2 metres below the present stream bed level.

iii, Ensure that "circular" slip-failure of the foundation will not occur.

b) Spread footings on rock highly resistant to scour

Position the bottom of a footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Footing embedment or keying should be avoided since blasting to achieve keying frequently damages the underlying rock structure and makes it more susceptible to scour. If footings on smooth, massive rock surfaces require lateral restraint, then steel dowels should be drilled and grouted into the founding rock,

c) Spread footings on erodible rock

Weathered or other potentially erodible rock formations needs to be carefully assessed for potential scour. An engineering geologist familiar with the area's geology should be consulted to determine the design and construction criteria to be used to estimate the support reliably available for the spread footing, The decision should be based on an analysis of rock cores including rock quality designations (RQDs) and local geological information, as well as hydraulic data and the structure's design life.

An important consideration may be the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated and the footing base placed below that depth.

Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize over-break beneath the footing level. Any loose rock pieces should be removed and the zone filled with concrete.

The footing should be poured monolithically, in contact with the sides of the excavation, for the full designed footing thickness in order to minimize water intrusion below footing level. The excavation above the top of the footing should be filled with riprap, sized to withstand flood flow velocities for the critical design event.

d) Piled foundations

For pile designs subject to scour, consideration should be given to using a lesser number of long piles to develop bearing resistance, as compared to a greater number of shorter piles. This approach will provide a greater factor of safety against pile failure caused by scour.

Placing the top of the pile cap at a depth, below existing river bed level, equal to the estimated general scour depth will minimize obstruction to flood flows and its resulting local scour. Lower settings will be required for pile-supported footings when the piles could be damaged by erosion and/or corrosion from exposure to river flow. Consideration should also be given to any reduction in pile capacity which would occur from a loss of lateral restraint or skin friction, due to scour.

Bridge Superstructures

3.49 Bridge superstructure soffit levels should be positioned above the general level of the approach roadways wherever practicable. In the event of overtopping of approach embankments this provides for a reduction of any hydraulic forces acting on the bridge. This is particularly important for bridges over rivers or streams carrying large amounts of debris which could clog the waterway of the bridge.

3.50 Bridge superstructures should be securely anchored to the substructure if the deck could become buoyant, or, floating debris or ice is probable. Where overtopping is likely, the superstructure cross-section should be shaped to minimise resistance to the flow.

Bridge Piers

3.51 Bridge pier foundations on floodplains should be positioned at the same depth as the pier foundations in the stream channel if there is any likelihood that the channel will shift its location onto the floodplain over the life of the bridge.

3.52 Align piers, as far as is practical, in the direction of flood and tidal flows. Assess the hydraulic advantages of different pier shapes, particularly where there are complex flow patterns during floods.

3.53 Streamline pier shapes to decrease scour and minimise potential for the build-up of debris.

3.54 Evaluate the hazard from debris build-up when considering the use of multiple pile bents in stream channels. Where debris build-up is a problem, then design the bent as though it were a solid pier for the purposes of scour estimation. Consider the use of other pier types where clogging of the waterway area could be a major problem.

Bridge Abutments

3.55 Available equations do not satisfactorily predict scour depths for abutments. It is recommended therefore that riprap and/or guide banks are considered for abutment protection. Correctly designed and constructed, the suggested protective measures can negate the need to compute abutment scour.

3.56 Relief openings, guide banks and river training works should be used, where necessary, to minimize the effects of adverse flow conditions at abutments.

3.57 Scour at spill-through abutments is about half of that for vertical wall abutments, however, consideration needs to be given to the loss of spill-through embankment material due to scour.

Bridges on Estuaries

3.58 An important hydraulic factor is the magnitude of the discharge under the bridge. Depending upon the bridge location the discharge can be predominantly fluvial, predominantly tidal, or, both tidal and fluvial components have significance. The tidal discharge depends upon the upstream tidal area and the tidal range at the bridge site.

3.59 Estuarial river beds are frequently, but not always, characterised by fine sediments consisting of silts and clays. When first deposited such sediments may have very low densities. In extreme cases the sediments may `flow' under gravity. Such sediments are easily re-eroded and even when remaining in position will not provide a significant support for foundations. For these situations care must also be taken in interpreting information from echo sounders and similar devices.

3.60 Within an estuary there is usually a pattern of low-flow channels. Depending upon the nature of the estuary and its sediments this pattern may change from with time. On a particular day one pier could be in a deep channel and another located on a sand-bank, the situation could well reverse within a few weeks or months. In such circumstances, unless there is strong evidence that the channel pattern is fixed, the foundations of all the piers should be sufficient to withstand the same conditions as where the flow is presently the deepest and fastest.

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4.1 The principal references in the bibliography are printed in bold. The brackets [ ] contain the reference number. This number has been used in the main text and appendices for the principal references.

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5. GLOSSARY OF TERMS

Afflux (Literally, the flow towards a point). The increase in water level upstream of a bridge over that which would have occurred if the bridge were absent is commonly termed the afflux.

Aggradation Increase in bed levels with time.

Angle of attack Angle between the longitudinal axis of a pier and the direction of flow.

Armouring The process of progressive removal of finer sediment from the bed which leaves a layer of coarser sediment lying on the bed.

Channel pattern The plan form of a river.

- *lateral shift* The horizontal movement of river channels in plan.

- *alignment* The horizontal plan form of a river, usually relative to a structure.

- *constriction* A reduction in the size of a channel. This frequently occurs at bridge crossings and may cause general scour.

Degradation Reduction in bed levels with time.

Dredging The removal of sediment from the bed of the river. This may reduce bed levels both upstream and downstream of the location of dredging.

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 soffit of 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.

Head erosion The tendency of a steep or vertical section of channel to migrate upstream.

Hydrodynamic loading The force exerted on a pier or abutment by the flow of water around it. This may include buoyancy forces. There are similarities with the drag and lift forces acting on an aerofoil.

Hydrofoils Vanes located in the flow in order to inhibit vortex action and to reduce scour.

Impact loading The loading applied to the bridge structure by the collision of debris carried by the flow.

Invert protection Protection of the river bed under the bridge to prevent scour.

Pier thinness Length to breadth ratio of a pier cross section.

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.

Retained water levels Structures such as weirs may be placed in a river to maintain water levels upstream. Alterations to retained water levels may affect the hydraulic conditions at a bridge.

Rip-rap Stone placed to prevent scour.

Scour Scour is the removal of sediment and hence reduction in river bed level by flowing water. It may be divided into **three components**, general scour, local scour, and natural scour (progressive aggradation/degradation).

- *general [1]* Scour which affects the whole width of the river but is still confined to the reach adjacent to the bridge. It is commonly caused by a channel constriction at the bridge site (eg 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.

- *local [2]* Scour caused by an acceleration of flow and its resulting vortices, around an obstruction such as a pier or abutment (see horseshoe vortex). Such scour only occurs in the immediate vicinity of the obstruction.

- *natural [3]* *(progressive aggradation and degradation)* These are long-term river bed elevation changes due to natural or man-induced causes within the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from other sections of a river reach, whereas degradation involves the lowering or scouring of the bed of a river.

- *due to bends* Flow around a bend causes greater scour at the outside of the bend.

- *clear water* Scour at flows for which the surrounding bed sediment is not moving (see sediment transport).

- *live bed* Scour at flows for which the surrounding sediment is moving. Some authorities claim that maximum scour depths are achieved at the transition from clear water to live bed scour.

- *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.

Scour holes The bed features caused by the reduction in bed level due to local scour.

- *refilling of* The depth of scour increases with flow. During the recession of a flood scour holes may be totally or partially re-filled with sediment.

Sediment

- *non uniform* Sediment in which a large range of sizes are present.

- *cohesionless* Sediment of sufficient size that electro-static forces between particles can be ignored, typically d>0.04mm.

Sediment transport, initiation of motion There is normally a critical shear stress below which sediment does not move. Thus for low flow, little or no sediment transport takes place. Once critical conditions are exceeded then sediment transport increases rapidly with shear stress. The critical shear stress varies with sediment size.

Shape factor Factor to account for the effect of pier shape on local scour.

Shear velocity A measure of the shear stress between flowing water and the bed of the river.

Sub-critical flow Flow with a Froude number less than 1.

Tailwater level The downstream water level which partly determines flow conditions at the bridge.

Uplift forces Buoyancy forces exerted by water on the bridge due to the partial or total submergence of the deck.

Vortex A mass of rotating or swirling fluid.

- *horseshoe* Flow around a pier generates a vortex whose axis has a horseshoe shape.

- *trailing* Flow around a pier normally sheds vortices downstream, frequently referred to as a Karman vortex street.

Water table The level below which the ground is saturated with water.

6. ENQUIRIES

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

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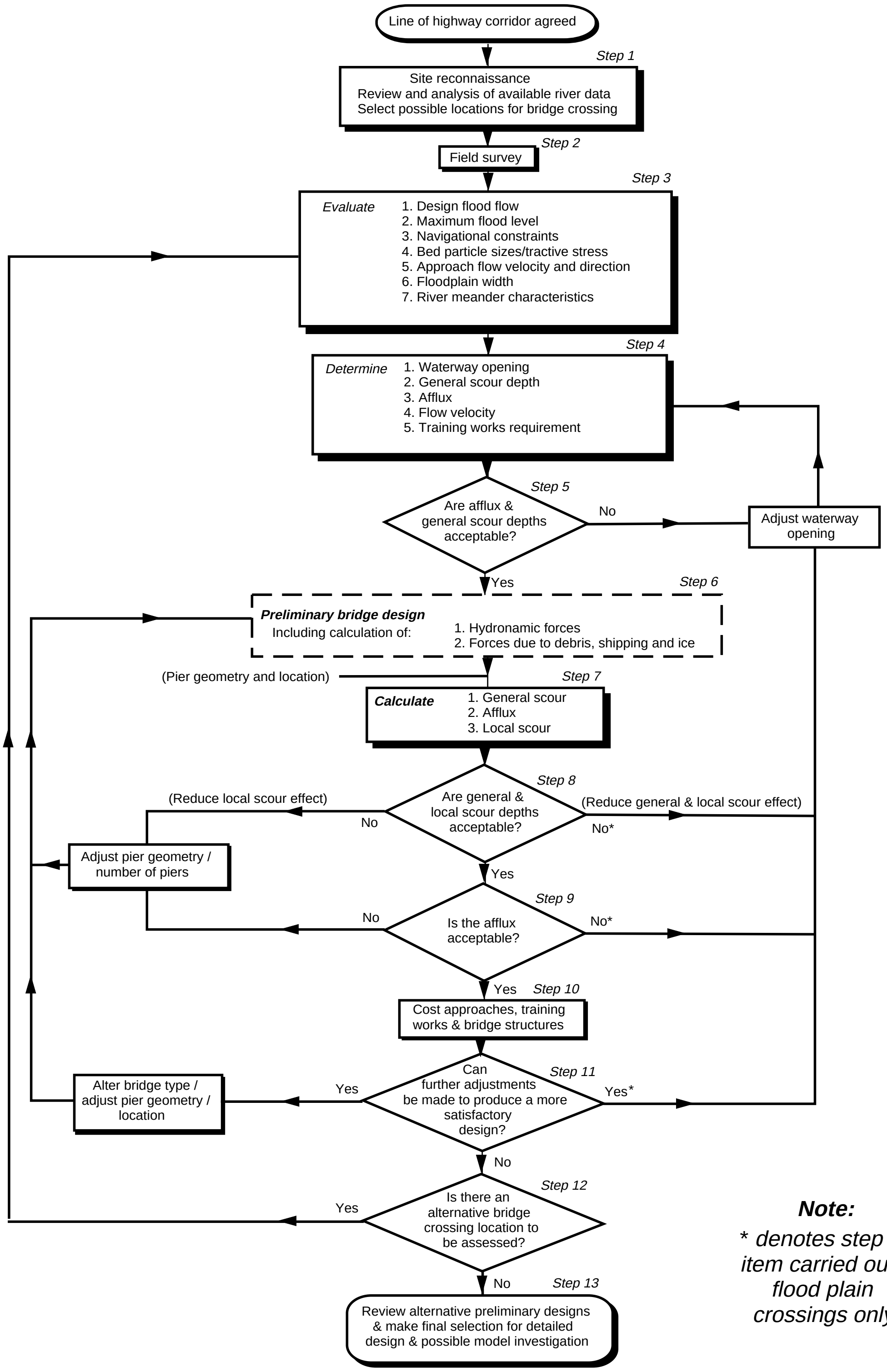
EXAMPLE PRELIMINARY DESIGN PROCEDURE



Note:
** denotes step or item carried out for flood plain crossings only*

Fig.A1 Example procedure for preliminary hydraulic design of bridges (after Farraday & Charlton, 1983, [12])

APPENDIX B: EXAMPLES OF SCOUR CALCULATION

B.1 Appendix B illustrates the types of method which may be used, and the type of data which is required for the calculation of scour depths at a bridge crossing. A comprehensive description of calculation for scour, in all circumstances, is beyond the scope of this Advice Note. Such calculations should be carried out by staff with specialist knowledge of the subject.

General procedure

B.2 For the calculation of scour depths the following six stage procedure generally applies:

i Obtain hydrological data, in the form of discharges.

ii Assess long term change to bed levels.

iii Calculate flow conditions (eg design water level and velocities) in the absence of the bridge. This may be carried out using uniform flow formulae such as Manning's equation or the Colebrook-White formula, or by backwater calculation. Unless it is certain that the flow at the site is uniform, a backwater calculation should be carried out. Other factors such as variation in roughness around the wetted channel perimeter should be considered.

iv Calculate general scour. Two cases are considered here, for illustrative purposes:

v Calculate local scour

vi Calculate combined scour

General scour

Example (A) Channel constriction without flood plain flow (general scour)

B.3 This is the simplest case of general (constriction) scour, where all of the flow is contained within the river banks. See Figure B1 for the dimensions and values used for the example bridge site.

B.4 Using the method given by FHWA, [50],

$$\frac{\boldsymbol{y_U} + \boldsymbol{d_g}}{\boldsymbol{y_U}} = \left(\frac{W_U}{W_B} \right)^{k_1}$$

Equation B1

Where $\boldsymbol{y_u}$ is the average depth in the channel, upstream of the contracted section, $\boldsymbol{d_g}$ is the depth of general scour, $\boldsymbol{W_U}$ and $\boldsymbol{W_B}$ are the channel widths upstream of and within the contracted section respectively (see Figure B1). The parameter k_1 depends on the mode of sediment transport. The value of k_1 ranges from 0.59, if the sediment is mostly transported as bed load, to 0.69 if the sediment is mostly transported in suspension. The value of k_1 is given as a function of the shear stress on the river bed and the fall velocity of the sediment, and may be difficult to determine for real rivers. For this example a value of 0.64 is assumed, corresponding to the case of "some suspended bed material discharge".

With the exception of the effect of k_1, the contraction scour calculated from the above equation is independent of flow velocity. The depth, however, must be known, and therefore the water level should be calculated from hydraulic studies. From Equation B1, the depth of general (constriction) scour d_g is given by:

$$\frac{4.0 + d_g}{4.0} = \left(\frac{20.0}{15.0}\right)^{0.64}$$

which gives an **average depth for general scour of 0.81m.**

B.5 Several approximations are made by the above approach. The formula calculates the equilibrium scour which would occur towards the downstream end of a long contraction. In reality it is likely that the contraction will be due to bridge abutments and piers, and will not be of sufficient length for the full depth of scour indicated by Equation B1 to develop. The method neglects the effect of armouring, whereby the average bed material size increases in time due to preferential erosion of finer material. Armouring may significantly reduce scour depths in rivers with bed sediments possessing wide grading bands.

The calculated general scour depth is the average across the width of the channel. In general, scour will not be distributed uniformly across the channel width. Factors to account for this effect are given by Farraday and Charlton, [12], who suggest that the maximum scoured flow depth can be estimated from the average scoured depth by using a scale factor. The value of the factor depends on whether the bridge crossing is located on a straight reach of the river or on a bend. Factors range from 1.25 for a straight reach to 2.0 for a right-angled abrupt turn. Applying such factors to the example gives maximum scoured flow depths in the contraction of 1.25 x (4.0+0.81) and 2.0 x (4.0+0.81), or **6.0m** and **9.6m** respectively. These values indicate maximum scoured depths of 2.0m and 5.6m for straight channel and abrupt bends respectively. The location of the maximum scour will normally be towards the outside of a bend.

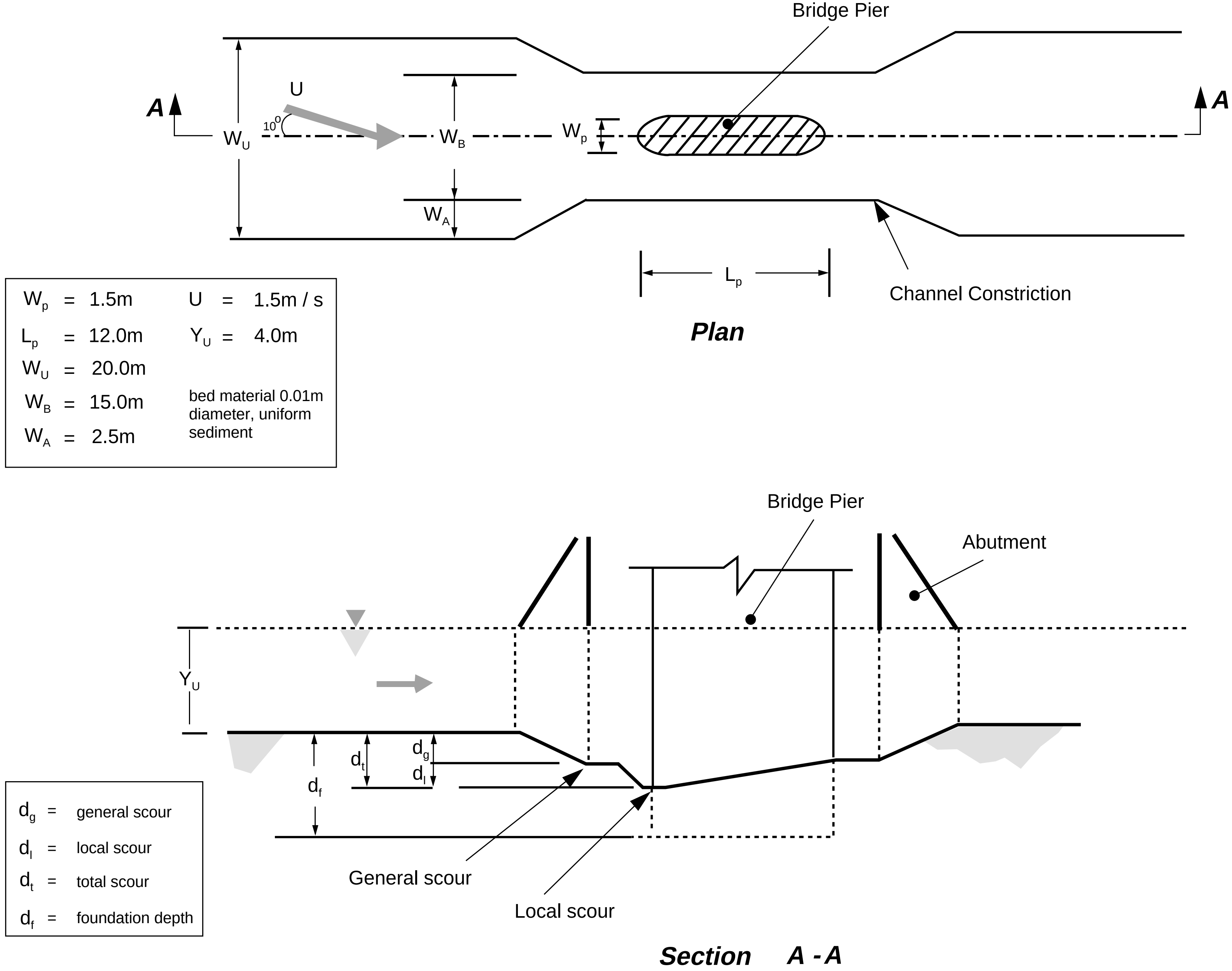


Figure B1 : Example Bridge Showing Scour Depths

Example (B) Channel constriction with flow over the flood plain, and abutments set back from the edge of the channel (general scour)

B.6 This is a more complex case. The bridge approach embankments constrict the flood plain flow, forcing a portion of the flood plain flow back into the main channel and increasing the general scour. This implies that this case cannot be accurately analysed without a knowledge of the distribution of flow between the channel and flood plain. Specialist studies may be required to establish the actual flow conditions.

If the flow distribution is known, the following equation can be used to estimate general scour in the main channel beneath the bridge:

$$\frac{y_U + d_g}{y_U} = \left(\frac{Q_{mcB}}{Q_{mcU}}\right)^{\frac{6}{7}} \left(\frac{W_U}{W_B}\right)^{k_1}$$

Equation B2

Q_{mcU} is the flow in the main channel upstream of the bridge and Q_{mcB} is the flow in the main channel at the bridge. For example, assume that hydraulic studies have indicated that Q_{mcU} is 120m^3/s and Q_{mcB} is 150m^3/s. The depth of general scour is calculated for the example from:

$$\frac{4.0 + d_g}{4.0} = \left(\frac{150}{120}\right)^{\frac{6}{7}} \left(\frac{20}{15}\right)^{0.64} ,$$

This gives an **average depth for general scour of 1.82m** and an average scoured flow depth of 5.82m. As Example (A), the average scour depth can be factored to obtain an estimate of the maximum scour depth at the bridge site. For a bridge located on a moderate bend, a factor of 1.5 could be applied to the average scoured flow depth, giving a depth of 8.73m and a maximum depth of scour of 4.73m.

Local scour

Example C Local scour at piers

B.7 This example shows, from comparing several formulae, how local scour at a pier could be calculated. It is based on the assumption that local scour and general scour are independent ie that local scour can be calculated based on hydraulic parameters which do not take account of the effect of general scour. An alternative approach is to re-calculate the hydraulic parameters (such as velocity and flow depth) based on the general scoured bed levels. The latter approach is more complex, but could be more accurate, particularly in cases where there is significant general scour. The examples are based on an isolated mid-channel pier (see Figure B1) with the following parameters:

Pier width W_p = 1.5m
Pier length L_p = 12m
Length/width ratio of pier = 8
Shape upstream nose of pier is semicircular
Angle between long axis of pier and flow direction = 10º
Approach flow velocity **U** = 1.5m/s
Flow depth just upstream of pier y_U = 4.0m
Bed material = 0.01m diameter, uniform sediment.

[C1] Neill (1973), [33]

B.8 The basic design equation for local scour for well-aligned piers is:

$$\begin{bmatrix} d_l = 1.5W_p & \text{for } y_U < 5W_p \\ d_l = 2.2W_p & \text{for } y_U \geq 5W_p \end{bmatrix}$$

Equation B3

This equation gives, for the example, a value d_l (depth of local scour) of 2.25m. Correcting factors to account for pier skewness as functions of angle of flow incidence and length to width ratio of the pier may be applied when necessary. The factor relevant for this example is 2.0, giving a **4.5m** depth of local scour.

[C2] Melville and Sutherland (1988), [29]

$$d_l = 2.4 W_p \cdot f_u(U, U_c, U_a) \cdot f_y\left(\frac{y_u}{W_p}\right) \cdot f_d\left(\frac{d}{W_p}\right) \cdot f_s(shape) \cdot f_\alpha\left(\alpha, \frac{L}{W_p}\right)$$

Equation B4

B.9 For this method, the basic scour depth $2.4W_p$ term is modified by various factors:

Function f_u takes account of the erosion resistance of the bed, allowing for the flow velocity U; the critical velocity for the bed material U_c and armouring effects U_a. The value of f_u depends on the size and grading of the bed material. If this information is not available, a conservative estimate of scour may be obtained if a value of 1.0 is assumed for f_u. For the example, the sediment size is sufficiently coarse to limit the depth of scour. The value of the function f_u is calculated to be 0.78.

Function f_y is included for reducing estimates of scour where the flow depth is shallow compared with the pier width. A conservative estimate will be obtained if $\boldsymbol{f_y}$ is taken to be 1.0. For the example, the ratio of the flow depth to the pier width is 2.67, and the value of $\boldsymbol{f_y}$(2.67) is, in fact, 1.0. Only when the flow depth reduces to below 2.5$\boldsymbol{W_p}$**,** approximately, does the function f_y have a significant effect in reducing estimates for local scour.

Function $\boldsymbol{f_d}$ is included to take account of the effects of sediment size. This factor is generally 1.0, except where the sediment size *d* is larger than (y_U/25). For this example the function would only affect estimates of local scour if the median sediment diameter was greater than 0.16m diameter. In the absence of further information, the conservative approach is to assume that f_d is 1.0.

Function $\boldsymbol{f_s}$ takes account of the shape of the upstream nose of the pier. Rounded or streamlined piers normally result in less scour than rectangular nosed piers. The factor for a circular nose is 1.0. The function $\boldsymbol{f_\alpha}$ takes account of the angle of flow incidence and the pier length to width ratio. It is evaluated from the same source as that used by Neill (1973), [33]. The value in this case is therefore 2.0 (see B.8)

By combining the factors, a value of $\boldsymbol{d_l}$ (local scour depth) is calculated as,

$$\begin{aligned} \boldsymbol{d_l} &= \boldsymbol{2.4 \times 1.5 \times 0.78 \times 1.0 \times 1.0 \times 1.0 \times 2.0} \\ &= \boldsymbol{5.6\,m} \end{aligned}$$

[C3] Colorado State University (CSU) equation, [50]

$$\boldsymbol{d_l} = y_U . 2.0\, k_1 k_2 \left(\frac{W_p}{y_U} \right)^{0.65} Fr^{\,0.34}$$

Equation B5

B.10 This equation is recommended by FHWA, the United States Federal Highway Authority. The Froude number, *Fr*, of the flow, given by,

$$\boldsymbol{Fr} = \frac{U}{\sqrt{g y_u}}$$

Equation B6

The factor k_1 is to account for pier nose shape, and k_2 accounts for angle of flow incidence.

For the example, therefore,

$$\begin{aligned} \boldsymbol{Fr} &= \frac{\boldsymbol{1.5}}{\sqrt{\boldsymbol{9.81 \times 4.0}}} \\ &= \boldsymbol{0.24} \end{aligned}$$

and $\boldsymbol{d_l}$ (local scour depth) is given by,

$$\boldsymbol{d_l = 4 \times 2.0 \times 1.0 \times 2.0 \times \left(\frac{1.5}{4.0}\right)^{0.65} \times 0.24^{0.34}}$$

$$\boldsymbol{= 5.2\,m}$$

Summary of Example C results for local scour depth calculations at a pier

B.11 The predictions given by three methods of local scour calculation for piers are:

Method	Local scour depth
Neill	4.5m
Melville and Sutherland	5.6m
CSU equation	5.2m

Example D Local scour at abutments

B.12 Methods for calculating scour at abutments are notoriously inaccurate, and results should be assessed by using engineering judgement. If necessary, to provide improved estimates, physical modelling should be carried out. The example calculations demonstrate two methods.

[D1] Pier - abutment analogy

B.13 This method is only applicable if an abutment projects into the river channel, and if the river bank is approximately vertical. For these cases, flow around the projecting abutment may resemble flow around one half of a hypothetical pier within the channel. The formulae for local scour at piers can be used, with an effective width $\boldsymbol{W_p}$ of twice the length of the projection of the abutment. For example, if the abutment projects a distance of 1.2m from the river bank into the river channel, its effective width $\boldsymbol{W_p}$ is 2.4m. Local scour may be estimated using the Neill formula (Equation B3), for example, as,

$$d_l = 1.5\,W_p$$

$$= 3.6m$$

B.14 This method has the benefit of simplicity, but may not be very accurate. The river bank will influence flows such that flows around the abutment projection are not the same as flows around one half of a pier, also the method may not be applicable where the river bank is not vertical. Where the abutment has wing walls, these may improve the flow past the projection, causing significantly less local scour than at an abutment which projected abruptly into the flow.

[D2] FHWA method [50]

B.15 This method may be used to predict local scour at abutments on flood plains or in the main river channel. It has been developed from laboratory data and is also of limited accuracy.

$$\boldsymbol{d_l} = y_a\left[2.27\,k_1 k_2 \left(\frac{a'}{y_a}\right)^{0.43} Fr^{0.61} + 1\right]$$

Equation B7

where $\boldsymbol{y_a}$ is the depth of flow at the abutment, $\boldsymbol{k_1}$ and $\boldsymbol{k_2}$ are coefficients to account for abutment shape and angle of the embankment to the flow respectively. a' is the effective length of the abutment normal to the flow:

$$\mathbf{a}' = \frac{A_e}{y_a}$$

Equation B8

where A_e is the area of flow which is obstructed by the abutment. **Fr,** the Froude number, is defined by Equation B6.

Abutment in main channel, no flood plain flow

B.16 For an abutment of the example bridge with A_e of 4.8m^2, d_l (local scour depth) is given by,

$$d_l = 4.0\left[2.27 \times 1.0 \times 1.0 \times \left(\frac{1.2}{4.0}\right)^{0.43} \times 0.24^{0.61} + 1\right]$$
$$= 6.3m$$

Abutment on flood plain

B.17 This section considers the case of an abutment founded on the flood plain and set back from the river bank. If the abutment is near to the river bank and founded on shallow foundations, a small amount of bank erosion could undermine the abutment or cause instability of the foundation. This may not be due to scour at the abutment, but to erosion of the river channel. No formulae are available to predict the latter effect. The risk of bank erosion and undermining of an abutment, located on the flood plain, should be assessed, taking into account the stability of the river and any history of bank erosion. Appropriate bank protection measures should be provided where this type of failure is considered to be a risk.

B.18 Local scour at an abutment on a flood plain is likely to be less severe than at an abutment in the main river channel. Flow velocities and depths are likely to be lower, and soils may be more resistant to erosion. Equation B7 can be used, in which case the Froude number, flow depth and velocity refer to flow conditions on the flood plain rather than in the main channel. However, the method will probably overestimate local scour depths. In the absence of suitable prediction methods it is suggested that engineering judgement, specialist studies or physical modelling should be used.

Combined scour.

B.19 At each bridge foundation subject to scouring action, the total scour d_t should be estimated by combining the general and local scour estimates:

$$\mathbf{d_t} = d_g + d_l$$

Equation B9

B.20 The design bed elevation $\mathbf{z_d}$ is calculated by subtracting the total scour $\mathbf{d_t}$ from the "natural" bed elevation $\mathbf{z_o}$. Surveys of bed levels should be used to establish the pre-construction bed levels at the bridge site. These levels may be modified to take account of long term aggradation or degradation to obtain the bed levels $\mathbf{z_o}$**.**

B.21 Improved estimates of scour depth may be obtained if site measurements of scour are available from nearby existing bridges of similar construction to the proposed bridge, particularly if scour has been monitored during flood periods.

B.22 It is recommended that bed levels and foundation levels are plotted onto several cross sections of the bridge site. General scour levels can then be adjusted to take account of its non-uniformity across the section. Local scour depths may then be superimposed on the general scoured bed profiles. The width of local scour holes may be assumed to be as given in 3.9 (ix).

APPENDIX C: METHODS FOR ESTIMATING AFFLUX

US Bureau of Public Roads (USBPR) method

C.1 This method, [45], is widely used, and is suitable for bridges with flat decks as opposed to arch bridges.

C.2 In the absence of any flow bypassing the bridge site the afflux caused by the bridge is given by,

$$\Delta h = kH_{ref} + H_u - H_d$$

Equation C1

where,

H_{ref} is the reference velocity head
H_u is the velocity head upstream of the structure
H_d is the velocity head downstream of the structure

In general, if the flow velocity is U, then the velocity head H is defined as,

$$H = \frac{U^2}{2g}$$

Equation C2

C.3 The principal effects of the geometry of the site are contained in the overall backwater coefficient k. The USBPR manual defines k by,

$$k = k_b + \Delta k_p + \Delta k_e + \Delta k_s$$

Equation C3

where,

k_b is the base coefficient which depends on the magnitude of the constriction of the flood plain flow by embankments on either side of the bridge.
Δk_p is the pier coefficient which depends on the proportion of the river channel blocked by piers and also their shape.
Δk_e and Δk_s account for the eccentricity and skew of the flood plain crossing, respectively.

Eccentricity is important if the width of the floodplain, on one side of the river, is more than about 6 times the width of the floodplain on the other side of the river. The effects of any skew of the flood plain crossing depend on the angle that the embankment makes with the general direction of flood plain flow and also the alignment of the ends of the flood embankments to that flow direction.

Afflux at arch bridges

C.4 A method has been recently developed specifically for estimating afflux at arch bridges, [5] and [18]. Single span and multispan arch bridges and other bridge types were investigated, using model experiments and field data. These studies were limited in respect of the upstream channel being of essentially uniform depth and velocity distribution.

Based on this work, an empirical method has been derived for determining the afflux at single or multispan arch bridges. The data required are the bridge geometry, water depth and flow velocity downstream of the bridge.

The same method applies to single and multispan arch bridges provided that the multispan arch bridges can be considered to be essentially a single, uniform, unit separated only by typical pier widths. The method is not designed for application to multispan arch bridges with different soffit heights or arches on floodplains, and only a limited range of arch shapes were tested in the laboratory.

APPENDIX D: CALCULATION OF HYDRODYNAMIC, ICE AND DEBRIS LOADING

Hydrodynamic forces on piers

D.1 Hydrodynamic forces from the action of flowing water past the submerged parts of a bridge act in addition to hydrostatic forces.

D.2 The Indian Road Congress, [20] and American Association of State Highway and Transportation Officials (AASHTO), [2] recommend the following equation for the hydrodynamic flow pressure **P** (kN/m^2),

$$\mathbf{P = 0.51KU^2}$$

Equation D1

where **K** is dependent upon the shape of the pier and **U** is the velocity of the current at the point where the pressure intensity is being calculated. **U** is assumed to vary linearly from zero at the point of deepest scour to a maximum at the free water surface. The method is not suitable for cases where the current strikes the pier at an angle (rather than head on) - for these conditions the drag and lift coefficients (see D.3) increase rapidly with angle of current and the hydrodynamic forces may be seriously underestimated.
Recommended values for **K** are as follows:

		K
Square ended piers		1.5
Circular piers		0.66
Pier with triangular	(< 30^o)	0.5
cutwater	(30^o to 60^o)	0.5 to 0.7
angle	(>60^o to 90^o)	0.7 to 0.9

D.3 A method proposed by Apelt and Isaacs, [1], has separate equations for the component of the force in the direction of flow and that normal to the flow. The component parallel to the flow is termed "drag" and that normal to the flow as "lift". Drag forces act on the pier in the direction of the flow and lift forces act upon the "exposed" longitudinal face, normal to the flow, in the direction of the pier centre. The equations are suitable for use for cases where the current strikes the pier at an angle (as well as head on).

D.4 In a study undertaken in 1966, values determined from the Apelt and Isaacs equations were compared with those derived from a model study of the River Exe bridge piers, Exeter, UK. The forces derived from the model study were found to be smaller than those derived from the Apelt and Isaacs equations - the apparent conservativeness of the equations may be due to the difference in the pier length to width ratio of the River Exe bridge to the ratios available from the Apelt and Isaacs charts (see D.5).

D.5 The following equations were proposed by Apelt and Isaacs:

Drag force (kN)

$$F_D = \frac{C_D \rho U_o^2 y_o L}{2000}$$

Equation D2

Lift force (kN)

$$F_L = \frac{C_L \rho U_o^2 y_o L}{2000}$$

Equation D3

where, U_o - approach flow velocity (m/s)
y_o - depth upstream of pier (m)
L - length of pier (or pier diameter for single cylindrical pier), (m)
ρ - water density (Kg/m^3)
C_D - drag coefficient
C_L - lift coefficient

The drag and lift coefficients for various shapes of pier and angles of approach flow are available in chart form. The forces are assumed to act at the mid depth of the flow and through the pier centroid. This was confirmed by the model study. In the case of rectangular piers, coefficients are given for piers with a length/width ratio of 6.52 only. (The latter values were used, by default, for the River Exe bridge study where the length to width ratio was 17.5).

D.6 The following values were used in the River Exe bridge study:

U_o 2.7m/s
y_o 6.35m
L 22.75m
W_p 1.3m
Angle of flow = 19°

which upon substitution in Equations D2 and D3 gave,

F_D = 0.4x1000x2.7x2.7x6.35x22.75/2000 = 210kN

F_L = 0.8x1000x2.7x2.7x6.35x22.75/2000 = 420kN

Resolving the forces parallel and normal to the pier axes, the forces were 62kN and 465kN, respectively, compared with 40kN and 330kN from the model.

D.7 The values predicted by the Indian Road Congress and AASHTO method for the River Exe bridge study (see D.4) were **14kN** and **86kN** only. To achieve a comparison with paragraph D.6 results, the flow was resolved into 2 components, normal and parallel to the pier to obtain these values - this is incorrect in engineering terms.

Hydrodynamic forces on submerged bridge superstructures

D.8 The US Federal Highways Authority (FHWA, 1991), [51], recommend the following formula for calculating the drag force on a submerged or partially submerged bridge deck,

$$F_d = C_d \rho H \frac{U^2}{2000}$$

Equation D4

where, F_d = drag force per unit length (kN/m)
C_d = coefficient of drag, 2.0 to 2.2 is suggested
ρ = density of water (kg/m^3)

H = depth of submergence (m)
U = velocity of flow (m/s)

Ice forces

D.9 A method is given in the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges, [2], where $\mathbf{F_H}$ the horizontal ice force on pier is given by,

$$\mathbf{F_H = C_n.p.t.W_p.(C_w)}$$

Equation D5

$\mathbf{C_n}$ = coefficient for nose inclination
(from appropriate table in specification)

$\mathbf{p}$ = effective ice strength (as advised in specification)

$\mathbf{t}$ = thickness of ice

$\mathbf{W_p}$ = width of pier or diameter of circular pier at the level of ice action

$\mathbf{C_w}$ = coefficient for $\mathbf{W_p/t}$ ratio.

The units for Equation D5 are to be consistent eg (kN,m)

Example E Ice force on a bridge pier

D.10 The ice force, acting in the direction of ice flow, on an example bridge pier is calculated thus:

Inclination of nose to vertical, (0° - 15°)
$\mathbf{C_n}$ = 1.00
Thickness of ice, 0.3m
Width of pier, 1.5m
Effective ice strength, 700 kN/m^2
$\mathbf{C_w}$ = 0.80 (for $\mathbf{W_p/t}$ = 1.5/0.3 = 5)

$$\mathbf{F_H} = 1.0 \times 700 \times 1.5 \times 0.3 \times 0.8 = \mathbf{252kN}$$

It is assumed that the pier is of substantial mass and dimensions. No guidance is given on what constitutes this. Where the direction of ice flow and the orientation of the pier are not coincident then the total force is to be resolved into its components.

D.11 The specification includes guidance on selecting values of effective ice strength, and values can also be modified by the use of a coefficient for variations in the relationship between pier width, pile diameter and ice thickness.

D.12 If the pier or piles are slender then consideration should be given to the dynamic effect of ice collisions and specialised advice sought.

D.13 Where piers became frozen into ice sheets on large bodies of water special consideration may need to be given to the effect on the pier of thermal movement within the ice sheet.

Debris forces

D.14 The National Association of Australian State Road Authorities (NAASRO), Highway Bridge Design Specification recommends that the designer should allow for a force equivalent to that exerted by a 2 tonne log travelling at the normal stream velocity and arrested within distances of 150mm and 75mm for column type and solid type concrete piers respectively. In the UK, 3 tonne logs travelling at 10mph (4.47m/sec) have been reported in upland areas. If such a log is arrested in 75mm then the force exerted may be estimated from the kinetic energy. The kinetic energy of a moving body must be fully absorbed to bring it to a stop. The magnitude of an arresting force depends on the distance the colliding body moves, during collision, before coming to rest.

Kinetic energy = $\mathbf{mv^2/2}$

Energy absorbed = $\mathbf{F \times d}$

The average force on impact is given by,

$$\mathbf{F \times d = mv^2/2}$$

F = : average collision force (kN)
d = : distance before coming to rest (m)
m = : mass of moving body (tonnes)
v = : velocity of moving body (m/s)

During the time taken for the body to come to rest this force has been erroneously suggested to vary from 2F to zero. The actual maximum forces are not readily calculable by energy methods and exist only for an instant. It is usual, therefore, to design for the average force (**F**).

D.15 For the 3 tonne log of paragraph D.14 the average collision force (**F**) is given by,

$$\mathbf{F = mv^2/2d}$$

Equation D6

ie $\mathbf{F} = \dfrac{3 \times 4.47^2}{2 \times 0.075} = \mathbf{400kN}$

D.16 For the case of the additional hydrodynamic force due to debris restricting the flow, the Australian specification recommends calculating the hydrodynamic force exerted on a minimum depth of 1.2m of debris. The length of debris to be applied to a pier should be half of the sum of the adjacent spans up to a maximum of 21 metres.

D.17 The formula proposed by Farraday and Charlton, [12], for the calculation of hydrodynamic pressure due to trapped debris is,

$$\mathbf{P = 0.517\, U_o^2}$$

Equation D7

where,

P = pressure (kN/m^2)
$\mathbf{U_o}$ = approach flow velocity (m/s)

APPENDIX E: INVESTIGATION OF A PROPOSED BRIDGE SITE

Introduction

E.1 There are two main objectives to be accomplished in investigating bridge sites for potential hydraulic problems:

i. To accurately record the present condition of the river

ii. To identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation.

E.2 In order to accomplish the objectives of E.1, the investigating engineer needs to recognise and understand the inter-relationship between the bridge, the stream, and the flood plain. Typically, a bridge spans the main channel of a stream and perhaps a section of the flood plain. The highway approaches to the bridge are typically on embankments which obstruct flow along the flood plain. Any over-bank or floodplain flow must, therefore, return to the stream at the bridge site or overtop the approach roadways. Where the over-bank flow is forced to return to the main channel at the bridge, zones of turbulence are established and scour may occur at the approach embankment and around the bridge abutments. In addition, piers and abutments may present obstacles to flood flows in the main channel creating conditions for local scour caused by turbulence around the foundations. After flowing through the bridge any flood water flow will expand back to the flood plain creating additional zones of turbulence and scour.

Scope of investigation

E.3 Every bridge site subject to hydraulic action should be reviewed in order to establish its potential vulnerability to damage by hydraulic action.

Collection of data

E.4 Data for a bridge site should be collected and retained in a separate bridge file.

E.5 The following information relating to the hydraulics of the bridge should be obtained:

i. The present geometry of the river in the vicinity of the bridge site, including its normal channel and flood channel cross section upstream and downstream of the bridge site, and the plan geometry of the river channel.

ii. The stability of the river channel. Old Ordnance Survey maps, Admiralty charts and aerial photographs may be useful in this respect. A scale of at least 1:10,000 is to be preferred for plotting changes to channel plan form for most UK rivers. Useful information sources are:

Ordnance Survey, Southampton, who can supply maps to 1" scale (or, for recent editions, 1:50,000 scale) for a numbered series dating back to 1897. 1:25,000 sheets are also available.

The National Map Centre (Caxton Street, London SW1) can supply latest editions of large scale plans. These are generally 1:2,500 scale for rural areas and 1:1,250 scale for urban areas. 6" to the mile (approx 1:10,000) maps are also available from the latest series.

The British Map Library, part of the British Library at Great Russell Street, keeps archives of a number of series, including 25" to the mile, dating back to 1871, and 6" to the mile, dating back to 1882. Maps may be viewed and traced at the library, and a copying service is available. In Scotland, this service is provided by the Map Library of the National Library of Scotland at Salisbury Place, Edinburgh.

Aerial photographs dating back to about 1940 are available from RAF Broughton and Cambridge University.

Satellite photographs may be useful in determining the width of the flood plain, channel stability and the extent of flooding. In general it is considered that the linear scale of these maps is too small, in relation to the size of the rivers in the UK, to be useful. They have however proved useful in the assessment of major rivers overseas.

iii. The range of water levels, particularly high water, and the frequency of occurrence. This data may be collected during or immediately following floods. The presence of nearby flow gauging stations should be determined. This information is often available from the local National Rivers Authority office. In Scotland, the River Purification Board may be consulted. Gauging stations on the river will provide information on the range of discharge and, in particular, on flood discharges.

iv. The general geology of the site. General geological data is available from the British Geological Survey at Kegworth. In Scotland, the British Geological Society at Edinburgh may be consulted. They also retain some borehole data. Other data may be available from local authorities who have undertaken site investigation work in the area. Where rock is present founding material it should not be assumed that there will not be a problem with scour. Advice should be sought on the susceptibility of the rock to scour. The following data should be collected:

a. The depth of the rock and its variation over the site.

b. The extent and character of the weathered zone.

c. The structure of the rock, including bedding planes, faults, and fissures which affect its erodability.

v. The type and grading of the bed material and the material on which the structure is to be founded.

Investigation procedure

E.6 An investigation should be performed by an engineer with design experience of bridges across waterways.

E.7 The investigating engineer should be familiar with the proposed approach to be adopted for the bridge design including: assumptions and criteria to be used in the design (for structural form, materials, geology, hydrology, wind speeds etc); factors of safety required; contract documents (including specifications and the proposed construction method).

E.9 The following sections present factors to be observed and considered when evaluating a proposed bridge site for scour and the overall scour potential and hydrodynamic loading:

Upstream Conditions

i. **Banks**

Stable: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilisation measures such as dikes and groynes.

Unstable: Undermining of banks, evidence of lateral movement, damage to river stabilization work, etc.

ii. **Main Channel**

Type of channel: straight, meandering or braided.

- Orientation of the main channel relative to proposed bridge openings.

- Existence of islands, bars, debris, cattle guards, fences that may affect flow.

- Aggrading or degrading stream bed.

- Evidence of movement of channel with respect to the bridge site.

iii. **Floodplain**

- Evidence of significant flow on flood plain.

- Flood plain flow patterns - will flow overtop road or return to main channel?

- Hydraulic adequacy of any proposed flood relief bridge spans (if relief bridge spans are obstructed, they will affect flow patterns at the bridge main channel).

- Extent of flood plain development and any obstruction to flow towards the bridge and its approaches.

iv. **Debris**

- Extent of debris in upstream channel.

v. **Other Features**

- Existence of upstream tributaries, bridges, dams, weirs or other features, that may effect flow conditions at the bridge site. Any proposed, or likely, future plans for land development or changes in the number or size of drainage outfalls etc.

Downstream Conditions

I. **Banks**

Stable: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilization measures such as dikes and groynes.

Unstable: Undermining of banks, evidence of lateral movement, damage to river stabilization works, etc.

ii. **Main Channel**

- Clear and open, or meandering or braided with bends, islands, bars, cattle guards, and fences that retard and obstruct flow.

- Aggrading or degrading stream bed.

- Evidence of downstream movement of channel with respect to the bridge site.

iii. **Flood Plain**

- Clear and open so that contracted flow at bridge site will return smoothly to flood plain, or restricted and blocked by dikes, developments, trees, debris, or other obstructions.

- Evidence of scour or erosion.

iv. **Other Features**

- Downstream dams or confluence with larger stream which may cause variable tailwater levels. This may create conditions for higher velocity flow through the bridge.

- Water level influenced by tides. This may create conditions for higher velocity flow through the bridge.

- Evidence of engineering works, such as dredging, which could affect flows and bed levels at the bridge.