Defra/Environment Agency
Flood and Coastal Defence R&D Programme

Afflux at bridges and culverts

Review of current knowledge and practice

R&D Technical Report W5A-061/TR1
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AFFLUX AT BRIDGES AND CULVERTS
Review of current knowledge and practice

R&D Technical Report W5A-061/TR1

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Research Contractor:
JBA Consulting – Engineers & Scientists
Statement of Use
This Technical Report contains the results of the first phase of a study to improve the estimation of afflux at river structures in high flows. The information in this document will be used in developing improved software and guidance for flood defence and land drainage practitioners, and is made available for reference and use.

Keywords
Afflux, backwater, blockage, bridges, culverts, channel structures.

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EXECUTIVE SUMMARY

This report sets out the results of a Scoping Study into the hydraulic performance of bridges and other structures, including effects of blockages, at high flows. It reviews current knowledge and practice. A separate document was produced which identifies the options for further research and development to improve current practice. This includes the need to

- develop robust algorithms for afflux and blockage risk,
- collect good quality field data,
- provide specific tools for users (including a robust afflux estimation system),
- consider any further gains and benefits from research.

This review of current knowledge and practice contains useful information as it stands and is published to make this available for practitioners pending the production of improved tools (due in late 2005).

The approach taken to the Scoping Study has been a mixture of consultation with key practitioners and academics, review of available literature and condensing of existing knowledge within and outside of the project team. Also specialist review papers have been commissioned from leading industry practitioners, and are published in a separate Technical Report.

Key findings

- There is confusion as to the definition of afflux and how it differs from headloss. For any flow, afflux is defined as the maximum difference in water level, at a location upstream of the structure, if the structure were removed.

- There is a general lack of confidence of users when estimating afflux. Users are unsure as to which is the ‘best formula’ to use in particular situations and many have no ‘feel’ for how much afflux to expect at high flows.

- Existing guidance is poor, either because it involves using several information sources, or because it is overly complex or too design-orientated.

- The most typical structures analysed are existing arched bridges and arched culverts.

- The most critical locations are considered to be in urban areas or reaches with formal flood defences, particularly where the structures may be subject to blockage. The flow conditions of most interest are bank-full or when structures are overtopped.

- The most typical tools available for estimating afflux are hand calculation or a 1-D river model such as MIKE-11, ISIS or HEC-RAS.

- Some of the most important users of afflux information are development control officers within the Environment Agency, Internal Drainage Boards and Local Authorities. Few of these users have access to river models and are unlikely to be able to make use of them even of they were available. Developers (and their consultants) are also important users of afflux data and estimation tools.
The implementation of existing afflux formulae in river modelling software is poor. It is generally not possible to readily compare different formula and it is not made clear to the user the relative importance of the various input variables as regards their affect on afflux. The significance of the opening ratio, angle of approach and tailwater depth in particular are not highlighted. The significance of cross-section spacing on the afflux calculations is not readily apparent to the user.

The available datasets on afflux are largely from laboratory studies and are poorly documented. There is little awareness or agreement of how afflux should be measured in the field – probably because it is rarely done. The currently available data is predominantly for bridges and is extremely variable in quality. Field data is required for bridges and culverts where overtopping occurs and for structures blocked with floating debris in order to confirm the adequacy of existing estimation methods. To obtain better field data will require a specifically targeted effort. All datasets could be improved by being clearly linked to blockage ratio and tailwater depth.

Blockage is considered an important issue, particularly with regard to trash screens and culverts, and this adds to the uncertainty of afflux calculation. There is currently no consistency on when or where blockage should be a consideration in an analysis. There is no guidance on how blockage should be best addressed in flood risk mapping.

There is a reasonable degree of confidence among users and experts in the SW Region ‘Blockage Risk’ model, which was examined under this study.

Professionals estimating afflux or blockage in the UK typically have a minimal background in hydraulics and are unlikely to have used hand methods for afflux estimation. This contrasts sharply with US and other European practice, where professionals are more likely to have specific background experience.

There are limitations to using physical/laboratory models to estimate afflux. These limitations are more pronounced for blockage. Physical models will still require prototype (field) data for validation.

**Afflux estimation and decision making in a risk-based framework**

Defra, the Environment Agency (EA) and other operating authorities manage the risk of flooding. They have recognised the value of considering the performance of systems of defences, within risk-based methods of planning, design and management. Key concepts are performance (achieving a desired outcome) and risk (the chances and consequences of failing to do so).

Afflux and blockage at structures are important components of flood risk, and thereby influence the performance of structures, and hence of defence systems. Conveyance, which is also a topic of Defra/Environment Agency research, plays a related role in determining reach performance. In both cases, uncertainty is an implicit and, increasingly, explicit part of management and decision making.

Future practice regarding the hydraulic performance of structures and reaches should therefore seek to adopt a consistent language and framework for addressing risk, performance
and uncertainty. This will be guided, wherever appropriate, by the risk-based framework to which Defra and operating authorities are moving. This follows the general approach set out in a recent R&D report on ‘Risk, Performance and Uncertainty in Flood and Coastal Defence’ (Defra/EA R&D Technical Report FD2302/TR1), which has provided a review of concepts and methods. It is agreed that a consistent approach should be adopted to dealing with risk and uncertainty across the different flood management guidance tools and techniques now being developed or used.

Natural variability underlies most flood risk analysis, and contributes to the uncertainty about afflux and blockage, not least through randomness in the frequency and magnitude of flood flows.

Knowledge uncertainty about afflux includes uncertainty about process models, resulting in the existence of many different methods for estimation, and uncertainty about data, especially the lack of ‘benchmark’ information on measured afflux. This contrasts for instance with conveyance estimation where there is greater agreement on the use of a formula (Manning) and related data (such as VT Chow or the new Conveyance Estimation System).

Process uncertainty associated with the different afflux estimation methods is not necessarily as significant as it first appears. There are clearly differences in afflux estimates depending on the choice of a particular set of equations and the parameters within them – but these generally do not lead to variations of more than 10-150mm for in-bank flow. Where process uncertainty is most marked is when flow reaches bank-full, or water levels reach or exceed the bridge/culvert soffit. In these situations current practice is almost universally to use an orifice or weir representation. Whether this is a valid approach to assessing the effects of structures on water levels at high flows is a key issue, and one that requires further investigation.

This study has shown that much of the uncertainty about afflux associated with data can be attributed to the location of the cross-sections used in calculations, particularly where river models are used. This can be addressed by providing clearer guidance and training, improving software packages, and ensuring that existing guidance such as the EA’s National Survey Specification and Flood Risk Mapping Guidelines are updated accordingly.

**Recommendations for improving current practice**

The uncertainties identified above are reflected largely in the confusion generated by having several different methods for afflux estimation and the lack of ‘benchmark’ information on measured afflux. The research programme developed within this scoping study has been designed to advance on this position. Key recommendations are:

- Setting out the available methods with a clear understanding of the limits of their validity and known ‘pros’ and ‘cons’.
- Providing a tool that allows rapid comparison of the valid methods and provides a clear visualisation of the process.
- Providing reference examples of structures stating the afflux and the features of the structure that have most influence on the afflux.
- Future development of afflux estimation tools needs to be integrated with the development of (a) conveyance estimation, and (b) overall performance-based flood risk management systems.
Bridges and culverts come in a multitude of sizes, shapes and interact with the river flow in numerous ways. It will never be possible to derive universal approaches that will fit all these situations exactly. A reasonable aim should be to develop procedures that will adequately address the most common structures/scenarios but will also clearly identify the ‘special’ cases, which require the use of specialist approaches such as physical modelling or 2-D or 3-D computer modelling.

Blockage

Blockage is a material consideration when assessing the effects of a bridge or a culvert at high flow and adopting a risk-based approach to flood management. It should not be an ‘add-on’ and it will be helpful to introduce the discipline of always considering blockage. The key questions relating to blockage are:

1. What material or objects might be available to cause a blockage?
2. What is the risk of blockage and the uncertainty associated with this?

The answer to the first question is already well known historically (but could change in the future). In the UK, blockage is usually caused by floating debris from natural and anthropogenic sources that collects on the piers and abutments and at the soffit of bridges and culverts. Blockages from sediment accumulations, ice, and large obstructions (everything from caravans being washed downstream to whole trees) are much rarer.

The risk of blockage is much more difficult to quantify and research into the subject is severely hampered by the difficulty of obtaining useful data. The risk of a particular structure blocking (which is a key question for new structure design) is a subtly different issue to the additional flooding risk blockage may present along a whole watercourse. This latter point focuses the question to ‘what is the additional flooding risk that blockage may present?’ The question needs to be addressed within the analysis of the overall system, and should be included in flood management decision-making, flood risk maps and assessments. Issues relating to the management of fluvial defence systems are currently being assessed through an R&D Scoping Study on Performance Based Asset Management Systems (PAMS).

Future research on afflux at bridges and other structures

There is a significant opportunity for Defra and the EA to establish best practice in the consideration of the effects of bridges and culverts at high flows relatively quickly and at low cost. A programme of Targeted Research has been identified which would take 18 months to implement. The research would be highly cost beneficial to flood defence operating authorities. A further programme of Strategic Research over a three to four year time scale is proposed to address inadequacies in understanding and hydraulic theory. This research is also cost beneficial and would be suitable for collaborative programmes with academia.

Best interim guidance

Until the research is completed, Appendix 4 of this document provides details of best interim guidance for the estimation of afflux and blockage.
### ABBREVIATIONS

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<th>Abbreviation</th>
<th>Full Form</th>
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<tr>
<td>ADA</td>
<td>Association of Drainage Authorities</td>
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<tr>
<td>EA</td>
<td>The Environment Agency</td>
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<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
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<tr>
<td>BIS ‘A’</td>
<td>Best Interim System, Class A</td>
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<tr>
<td>CEH</td>
<td>UK Centre for Ecology and Hydrology (formerly IH, Institute of Hydrology)</td>
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<tr>
<td>CFMP</td>
<td>Catchment Flood Management Plan</td>
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<tr>
<td>CFR</td>
<td>US Code of Federal Regulations</td>
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<tr>
<td>CIRIA</td>
<td>Construction Industry Research and Information Association</td>
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<tr>
<td>CIS</td>
<td>Commonwealth of Independent States</td>
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<tr>
<td>CIWEM</td>
<td>Chartered Institution of Water and Environmental Management</td>
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<tr>
<td>CoSLA</td>
<td>Confederation of Scottish Local Authorities</td>
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<tr>
<td>Defra</td>
<td>Department for Environment, Food &amp; Rural Affairs (formerly MAFF)</td>
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<tr>
<td>DTLR</td>
<td>Department of Transport, Local Government and the Regions</td>
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<tr>
<td>IDB</td>
<td>Internal Drainage Board</td>
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<tr>
<td>FEMA</td>
<td>US Federal Emergency Management Administration</td>
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<tr>
<td>FHWA</td>
<td>US Federal Highways Administration</td>
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<tr>
<td>FRMA</td>
<td>Flood Risk Mapping Framework Agreement</td>
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<tr>
<td>GIS</td>
<td>Geographic Information System</td>
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<tr>
<td>HA</td>
<td>UK Highways Agency</td>
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<tr>
<td>HEC</td>
<td>Hydrologic Engineering Center, Davis, California</td>
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<tr>
<td>HEC-RAS</td>
<td>HEC River Analysis System</td>
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<tr>
<td>HRW</td>
<td>HR Wallingford</td>
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<tr>
<td>ICE</td>
<td>Institution of Civil Engineers</td>
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<tr>
<td>IFM</td>
<td>Indicative Flood Map</td>
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<tr>
<td>ISIS</td>
<td>HRW/Halcrow river modelling programme</td>
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<tr>
<td>JBA</td>
<td>JBA Consulting - Engineers &amp; Scientists</td>
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<tr>
<td>LEAP</td>
<td>Local Environment Agency Plan</td>
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<tr>
<td>MIKE 11</td>
<td>DHI (Danish Hydraulic Institute) river modelling programme</td>
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<tr>
<td>MM</td>
<td>Mott MacDonald Consulting Engineers</td>
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<tr>
<td>NCPMS</td>
<td>National Capital Programme Management Service</td>
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<tr>
<td>NSF</td>
<td>National Science Foundation</td>
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<tr>
<td>NEECA</td>
<td>National Engineering &amp; Environmental Consultancy Agreement</td>
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<tr>
<td>SEPA</td>
<td>Scottish Environment Protection Agency</td>
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<tr>
<td>SNIPS</td>
<td>Construction, Standards and Rules of the former USSR</td>
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<tr>
<td>TAG</td>
<td>Theme Advisory group</td>
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<tr>
<td>USACE</td>
<td>United States Army Corps of Engineers</td>
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<tr>
<td>USBPR</td>
<td>United States Bureau of Public Roads</td>
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<td>USGS</td>
<td>United States Geological Survey</td>
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<tr>
<td>WSA</td>
<td>WS Atkins Consulting Engineers</td>
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<tr>
<td>WSPRO</td>
<td>FHWA Water Surface Profile modelling programme</td>
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</table>
PRINCIPAL NOTATION

A numerical subscript attached to a symbol usually indicates the location of the cross-section, or part of a cross-section, or the reach of a river according to context.

\( a \) Cross-sectional area of flow in a (part full) bridge waterway opening (m\(^2\)).
\( a_W \) Total cross-sectional area of a waterway opening when flowing full (m\(^2\)).
\( A \) Total cross-sectional area of flow in a river channel (m\(^2\)).
\( A_N \) Cross-sectional area of flow between the channel bed and normal depth line (m\(^2\)).
\( A_P \) Cross-sectional area of the submerged part of the piers (m\(^2\)).
\( B \) Net width (i.e. excluding pier width) of bridge opening at bed level at 90° to flow (m).
\( b_S \) Width between abutments of a skewed bridge, measured along the highway centreline (m).
\( B \) Width of river channel (m).
\( B_R \) Regime (Lacey) width of an alluvial channel measured at 90° to the banks (m).
\( B_T \) Top width of water surface between the river banks (m).
\( C, C_d, C_D \) Coefficient of discharge (dimensionless).
\( D_b \) USBPR method differential ratio to calculate the fall in water level across embankments.
\( d \) Flow depth from bed (invert) (m).
\( e \) Eccentricity (numerical ratio of abutment lengths, or conveyances or discharges).
\( f \) Darcy-Weisbach friction factor
\( F \) Froude number, or drag force in Chapter 6.
\( F_M, F_A \) Mean/average Froude number calculated from mean/average depth on floodplain.
\( F_N \) Froude number with normal depth flow (= \( F_4 \), dimensionless).
\( g \) The acceleration due to gravity (9.81 m/s\(^2\)).
\( h \) Height of water surface above the centre of curvature of an arch (m).
\( h_F \) Head loss due to friction (m).
\( H \) Elevation of water surface above datum level (m).
\( H^*\) USBPR method bridge afflux (m) without adjustment for piers, skew, or eccentricity.
\( H^*_D \) USBPR method afflux at a dual bridge (m).
\( H_1 \) Elevation above datum of water surface (with bridge) at section 1.
\( H_{1A} \) Elevation above datum of water surface (no bridge) at section 1 with abnormal stage (m).
\( H^*_1 \) Afflux at cross-section 1 with normal depth, \( H^*_1 = Y_1 - Y_N \) (m).
\( H^*_{1A} \) Afflux at cross-section 1 with non-uniform flow conditions (m).
\( H^*_3 \) Distance of the water surface below the normal depth line at section 3 (m).
\( J \) Proportion of bridge waterway blocked by piers or piles, or blockage ratio (HR method).
\( k \) USGS method adjustment factors (various subscripts) to base coefficient of discharge.
\( k^* \) USBPR method total backwater coefficient (dimensionless).
\( k^*_C \) USBPR method total critical depth backwater coefficient (dimensionless).
\( K \) Total conveyance of river channel (m\(^3\)/s), or friction factor in chapter 6.
\( K_b \) Conveyance of the part of the approach channel equivalent to the bridge opening (m\(^3\)/s).
\( K, K_A, K_N \) Yarnell, d'Aubuisson and Nagler coefficients for flow past piers.
\( K_R, K_Y \) Friction factor coefficients in Rehbock (1921) and Yarnell (1934) equations.
A numerical subscript attached to a symbol usually indicates the location of the cross-section, or part of a cross-section, or the reach of a river according to context.

L  Length of bridge waterway in the direction of flow (m), or reach length with subscripts.
M  Bridge opening ratio = q/Q or a/A or b/B or K_u/K (dimensionless).
M_L Limiting opening ratio (dimensionless) at which the flow is at critical depth.
n  Manning’s roughness coefficient (s/m^{1/3}).
P  Wetted perimeter of a channel (m).
q  Quantity of flow that can pass through the bridge opening unimpeded (m^3/s).
q  Discharge per metre width in Chapter 7 (m^3/s per m or m^2/s).
Q  Total discharge (m^3/s).
r  Radius of curvature of an arch, or radius of entrance rounding to waterway (m).
R  Hydraulic radius of channel (= A/P m).
R_S  Regime scoured depth of flow (m) corresponding to channel width B_R.
S_F  Longitudinal slope of total energy line (dimensionless).
S, S_O  Longitudinal slope of river bed (dimensionless).
Sc*  USBPR method afflux scour correction factor (dimensionless).
t  Thickness or width of a bridge pier.
V  Mean flow velocity (m/s).
V_C  Critical velocity (m/s), velocity when F = 1.0
V_N  Mean velocity when flow in a river channel is at normal depth (m).
V_u  Mean upstream approach velocity at either section 1 or 2 (m/s).
V_{2A}  Average velocity at section 2, in the opening, at the abnormal stage that would exist without the bridge (m/s).
X  Length approach embankment/abutments (m) for calculation of eccentricity.
Y  Depth of flow measured from the bed (m).
Y_C  Critical depth (m), corresponding to critical flow (F = 1.0) at minimum specific energy.
Y_d  Downstream depth measured above mean bed level on the channel centreline (m).
Y_M, Y_A  Mean depth, average depth (m). Numerical subscript indicates location of cross-section.
Y_N  Normal depth (m), e.g. as with uniform flow and predicted by the Manning equation.
Y_u  Upstream mean depth, the larger of the depths at sections 1 and 2 (m).
y_1, Y_1  Depth at section 1 (including the afflux) upstream of the bridge (m).
y_1A  Depth at section 1 without the bridge when abnormal stage exists (m).
Z  Vertical height of bridge opening (to the top of an arch) from mean bed level (m).
ΔE  Energy loss (W)
Δh  Difference in elevation of water surface between sections 1 and 3 (m).
ΔH  Differential head (m) across the bridge.
Δy  Representation for afflux used in chapter 6 (m).
Φ  Angle of skew, angle of bridge embankments or piers to the approach flow.
α  Kinetic energy correction coefficient, or contraction ratio in Yarnell (1934) equation.
β  Momentum correction coefficient.
PRINCIPAL NOTATION

A numerical subscript attached to a symbol usually indicates the location of the cross-section, or part of a cross-section, or the reach of a river according to context.

δ  Pier shape coefficient in Yarnell (1934) equation.
θ, η  Energy loss coefficients in Nagler (1917) equation.
λ  Dimensionless afflux ratio.
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SUPPORTING PROJECT RECORDS (PR)

Copies of technical papers commissioned as part of this study:

Annex 1. PR1 - A Review of Current Knowledge on Bridge Afflux (Knight)
Annex 2. PR2 - A Review of Hydraulic Model Implementation of Bridge and Culvert Afflux and Blockage (Samuels)
Annex 3. PR3 - A Review of Current Practice for Afflux Estimation in the USA (Benn & Bonner)
Annex 4. PR4 - A Review of Current Practice for Afflux and Blockage Estimation in the UK, Europe and Asia (Kirby and Guganesharajah)
Annex 5. PR5 - Review of SW Region Study – Prediction and Modelling of Structure Blockage during Flood Flows (Faulkner)
Annex 6. PR6 - Bridge Afflux Experiments in Compound Channels (Atabay and Knight)
1 BACKGROUND

1.1 Defra/Environment Agency Joint R&D Programme

This scoping study is part of the joint Defra/Environment Agency Flood and Coastal Defence Research and Development Programme. This R&D programme is unique to Defra and the EA in that it is jointly funded and managed by the two organisations as a single programme.

The approach of the R&D Programme follows the recommendations of the joint Defra/Agency Research Advisory Committee. The R&D Programme provides:

- A thematic structure of key subject areas for R&D – following the policy making/scheme development/asset management process in flood or coastal defence.
- Enhanced links between (a) R&D and (b) the groups in Defra and the EA (and related organisations such as consultants and local drainage authorities). This includes the need for Research Contractors to maintain awareness of other related R&D projects and Concerted Actions.
- Improved system for management and uptake of R&D so as to increase its effectiveness to Defra and the EA. This includes the maximisation of collaboration with other relevant research-commissioning and professional bodies, such as DTLR, Research Councils, the Institution of Civil Engineers (ICE) and the Chartered Institution of Water & Environmental Management (CIWEM).

In developing the new programme, Defra and the EA are determined to achieve an integrated and user-led R&D Programme. Key guiding principles are:

- Justifying research from a user viewpoint, particularly through researcher/practitioner panels, to identify and address research issues within the context of current practice,
- Thinking ‘sustainability’ – with technical, environmental, economic and social elements, and,
- Focussing on delivering benefits through enhanced performance or cost reductions, and ensuring good dissemination and implementation of research outputs.
- This project falls within the Engineering Theme of the joint programme, which is led by Dr Mervyn Bramley and advised by a Theme Advisory Group (TAG). The programme in each Theme is set out in the Theme Work Plan.

1.2 Other related research initiatives

As part of the joint Defra/Environment Agency joint R&D Programme there are several related scoping studies or R&D projects in progress that have relevance or linkages to this project. The main projects are summarised below.
1.2.1 Engineering Theme

- **W5A-057** Reducing uncertainty in river flood conveyance. Research Contractor is HR Wallingford. There is an identified input into the ‘Improved performance of existing one-dimensional modelling codes’ as part of Phase 2.

- **W5A-059** Concerted action on operation and maintenance of flood and coastal defences. Research Contractor is Posford Haskoning.

- **W5A-014** Design and operation of trash screens – Phase 3. Research Contractor is Posford Haskoning.

- **W5A-027** Fluvial Design Manual – Phase 2. Research Contractor is Binnie Black & Veatch.

- **W5B-023** Weirs – Best practice guidance. Research Contractor is Mott MacDonald.

- **W5-105** Benchmarking of river models. This updates earlier work undertaken in 1996 by the National Rivers Authority. The research contractors for this project are the Universities of Bradford, Leeds and Nottingham.

1.2.2 Risk Evaluation and Understanding of Uncertainty Theme


- **W5B-02** Risk Assessment of flood and coastal defence systems for Strategic Planning (RASP). Research Contractor is HR Wallingford. Due for completion in March 2004.


- **W5B-06** Performance and reliability of flood and coastal defence structures. Research Contractor to be appointed.

1.2.3 Flood Forecasting and Warning Theme

- R&D Project WSC 01/5 Real Time Modelling. WS Atkins is the appointed research contractor for this project, due for completion in March 2002.

1.2.4 Broad Scale Modelling Theme

- **W5F-01** Demonstration system for broad scale modelling tools and decision support systems for flood defence planning. Theme Leader for the research theme is Edward Evans. HR Wallingford is leading the software development of the modelling framework.
• Linked to this theme is Defra-funded work on developing the modelling and decision support frameworks in support of Shoreline and Catchment Flood Management Planning.

1.2.5 Policy Development Theme

• Monitoring, recording and analysing events (post event appraisal): Coordination, benefits and use study. Phase 1 (review, issues and recommendations for any further work/guidance). Research Contractor is Bullen Consultants/JBA Consulting. Scoping study due to be completed in August 2002.

The Agency also has the research on-going in R&D Project W6-061 Extension of rating curves at gauging stations using hydraulic models. The research contractor for this project is HR Wallingford and a scoping report was published in late 2001.

Project W5A-057 Reducing uncertainty in river flood conveyance has particularly strong linkages with this project and has identified a parallel programme of future research with important linkages to afflux and blockage. The Inception Report for this project was completed in July 2002. Some aspects of the format of the Conveyance Scoping Report have been adopted here in order to ensure consistency, and the contribution it has made to this study is gratefully acknowledged.
2 STUDY OBJECTIVES

2.1 Terms of Reference

The Environment Agency’s specification for this scoping study set out the following objectives:

(a) Identify and review current knowledge and research on afflux at bridges and large culverts.

(b) Identify and review current knowledge and research on the extent and effects of temporary blockages at bridges and large culverts.

(c) Review the EA’s South West Region Project ‘Risk Assessment of Structure Blockage during Flood Flows’.

(d) Identify any further work required to deliver robust algorithms for simulating afflux at bridges and large culverts in hydraulic modelling packages (e.g. for flood warning and for hydraulic design). Assess whether existing data is sufficient to support algorithm evaluation and development, or whether further data would need to be gathered.

(e) To identify the general benefits, and to specify where agreed, the further phase(s) of R&D and/or information dissemination to be carried out to:

   (i) Improve the accuracy of modelling of bridges and/or large culverts (without blockages).

   (ii) Improve the modelling accuracy of partially blocked structures.

   (iii) Develop technique(s) for identifying the likelihood of blockage of structures.

   (iv) Develop techniques for the rapid assessment of extent of flood risk areas associated with blockages.

   (v) Assist in prioritising operational resources during a flood event to minimise flood damages during blockages.

(c) To review the need to extend the project to include structures other than bridges and culverts.

(f) To advise on the format and synergies of any future research. In particular whether further investigations should be as a single project, or split into two or more, e.g. one on bridge afflux, one on structure blockage, etc.

The requirement for a two-page summary of best interim practice as determined from the Scoping Study was added to the brief in May 2002.

2.2 Study approach

The project team has involved both consultants (JBA, Mott MacDonald and WS Atkins) and researchers (Paul Samuels, Professor Donald Knight). In addition other experienced
practitioners (Vern Bonner, Chris Scott, John Riddell and Brian Faulkner) have made contributions.

Other contacts were also made with ‘end-users’ in the Environment Agency, Local Authorities, Consultants, Internal Drainage Boards and the US Federal Highways Administration and asset maintainers such as Railtrack and the Highways Agency.

The project was divided into several tasks as follows:

- Consultation with leading users and practitioners by means of a suite of targeted questionnaires and a workshop.
- Review of current theory and representation of afflux and blockage in commonly used software by specially commissioned papers from leading experts and by dialogue with model users and software developers.
- Review of current practice both within and outside of the UK in terms of published guidance and manuals.
- Identification of research needs for the targeted and longer-term research programmes.
- Development of a procurement strategy for the Targeted Programme, covering contract options, research organisations, programme management and user involvement in training/piloting.

In addition to this report, wider dissemination of this Scoping Study is sought by means of:

- A website for the project at [www.project-information.com](http://www.project-information.com).
- Presentations to the Technical Groups of the EA’s NEECA and Flood Risk Mapping Framework Alliances.
- Submission of a paper to the Chartered Institution of Water & Environmental Management – Rivers & Coastal Group meeting in May 2002.

2.3 Steering group and consultees

The study was project managed for the Defra/EA Joint R&D Programme by Andrew Pepper as external adviser on rivers and catchments to the Engineering Theme. Close links were maintained with Peter Spencer and Tilak Peiris (Flood Risk Modelling), Mervyn Bramley as Theme Leader and Project W5A-057 on river flood conveyance through Paul Samuels. These persons formed an informal steering group.
3 DEFINITIONS

3.1 Large bridges and culverts

The terms of reference specified that this study would be concerned with ‘large’ bridges and culverts because these are the most common structures studied in river systems. We have assumed that this includes structures with a notional diameter, width, or height greater than 1.5m. Openings with the prime dimension(s) less than 1.5m can usually be considered as pipes or simple orifices. However there is no definite limit, based on physical dimensions, between what constitutes a pipe or a culvert.

There are many definitions of what constitutes a ‘bridge’ and what constitutes a ‘culvert’. Which one is most appropriate depends on the purpose of study (e.g. structural assessment, hydraulic assessment or asset management). The definition used here is that a culvert has integral walls, soffit and invert (‘floor’). A culvert is also usually relatively ‘long’ (i.e. the length along the watercourse axis is several times larger than the span or width of the crossing itself). During flood conditions a culvert may often flow full or be surcharged at its entry or exit, but under normal flow conditions it will have a free water surface throughout its length.

![Figure 3.1: Physical features of a culvert](image)

Photographs of a box culvert – approach conditions (top) and the same culvert within a physical model at point of surcharge (bottom)
Bridges do not in general have an integral invert (although they may have one added as scour protection). They are usually relatively short – i.e. the distance from the upstream to downstream face is usually less than the span of the crossing. Under their usual flow conditions, bridges and culverts are rarely designed to form a hydraulic control to the flow. However, in extreme flood events, they may form a hydraulic control due to their constriction of the waterway cross section. Bridges are unlikely to develop pressure flow (i.e. ‘flow full’) although they may become surcharged at very high flows (i.e. water levels on the upstream are higher than the soffit). The design condition for bridges is usually for them to allow free surface flow beneath the structure.

![Figure 3.2: Physical features of a bridge](image)

Figures 3.1 and 3.2 provide an overview of some of the terms used to define the geometry of a bridge and culvert. The CIRIA Culvert Design Manual (CIRIA, 1997) provides a good overview of culvert types and the different hydraulic flow regimes.

### 3.2 Afflux

Prior to the placement of the bridge or a culvert across a watercourse, the water surface for a given flood discharge may assume a normal profile parallel to the bed (i.e. uniform flow) or a transitional profile due to other controls upstream or downstream (i.e. non-uniform flow). Due to the constriction in flow (and consequent energy loss) imposed by the presence of the structure, the water level at a location upstream of it (and unaffected by high local velocities caused by the constriction itself) will increase. The increase in water level provides the additional head needed to overcome the energy loss caused by the constriction; it is this process that creates afflux.
Afflux was defined in the project terms of reference as “the difference in water levels upstream and downstream of the structure – measured at a location unaffected by high local flow velocities caused by the constriction of flow”. However in this report, afflux is more strictly defined as the maximum difference in water level, at a location upstream of the structure, if the structure were removed. The afflux is thus defined as the maximum difference in water surface elevation between the original (uniform or non-uniform) and the increased levels (see Figure 3.3).

![Figure 3.3: Definition of afflux, head loss and energy loss (after Hamill, 1999)](image)

The numbered sections shown in Figure 3.3 correspond to typical locations of physical significance for flow through a bridge. Section 0 is a point upstream of the bridge where it has no effect on water level. Section 1 is the upstream point on the river centreline where the effect of the bridge on water levels is at a maximum (that is, the location of the afflux as defined above). Section 2 is a location at or near the upstream face of the bridge where the water surface passes through normal depth as it is drawn down through the opening. Contraction typically continues through the bridge, and section 3 indicates the point where the
flow reaches a minimum width. Section 4 is a location downstream where flow conditions are no longer directly affected by the bridge.

The significance of locations 1 to 4 in Figure 3.3 is somewhat simplified in the description given above. The upper panel of Figure 3.3 is based on the assumption of a uniform flow condition as this provides a convenient starting point to illustrate the development of afflux. In this case, the afflux as defined above can be written

\[ H_{1}^{*} = \max(Y_1 - Y_N) \]  

(3.1)

where \( Y_1 \) is the depth of water and \( Y_N \) is the normal depth, measured at locations such that the difference between the two water profiles is at a maximum. Where there are bed instabilities or non-uniform flow conditions, then water levels, rather than depths, provide a more general definition. Definitions of afflux for non-uniform flow conditions will be discussed below and in Chapter 6. For a more detailed discussion of the terms illustrated in Figure 3.3, the reader is also referred to Hamill (1999, Chapter 2).

3.3 Head loss

The head loss is the difference in the water surface elevation between any two specified points. The head loss across the structure (for example between sections 1 and 4 in Figure 3.3) can be written in the case of uniform flow as:

\[ \text{Head loss (sections 1 - 4)} = H_1 - H_4 \]

\[ = H_{1}^{*} + S_0 L_{1-4} \]  

(3.2)

Equation 3.2 suggests that the head loss is related to the afflux, \( H_{1}^{*} \).

The Environment Agency’s current BIS-A ‘Best Interim System’ river modelling software programs (i.e. HEC-RAS, ISIS and MIKE-11) only calculate head loss across a structure. To estimate afflux using these packages it is necessary to undertake two simulations for identical boundary and flow/roughness coefficients. One simulation should include the structure and one should be without the structure. The afflux is the difference between the estimated water levels at a suitable location upstream.

A not infrequent error is to quote head loss rather than afflux as the principal hydraulic effect of a bridge or culvert.

3.4 Energy loss

The more general way of calculating afflux or head loss at a structure is to work from the ‘energy grade line’ (shown in the lower panel of Figure 3.3). This allows the effects of non-uniform flow, varying channel cross section and controls other than normal depth to be taken into account. The energy loss across a structure measured between locations 1 and 4 in Figure 3.3 can be written as
Energy loss (sections 1 – 4) = \left[ H_1 + \frac{\alpha_1 V_1^2}{2g} \right] - \left[ H_4 + \frac{\alpha_4 V_4^2}{2g} \right] \tag{3.3}

where

- \( g \) is the acceleration due to gravity (constant),
- \( V_i \) is the flow velocity at section \( i \),
- \( H \) is the water surface elevation,
- \( \alpha \) is the kinetic energy correction coefficient, which accounts for non-uniform velocity distribution across the channel,

The use of the energy equation (3.3) to compute afflux is discussed in Section 6.2 of this report. The position of the afflux at a structure is usually an arc around the inlet, as shown in the photograph and sketch plan in Figure 3.4. In most practical situations only the afflux along the centreline of the river is estimated.

The culvert shown here is approximately 4m wide by 3m high.

**Figure 3.4: Position of afflux at a structure**

### 3.5 Backwater

In the USA, the terms afflux and head loss are rarely used. The preferred US term is ‘backwater’ which is defined as the rise in the water surface caused by the obstruction compared to ‘normal’ depth (i.e. the water surface without the obstruction). As the backwater is normally the maximum difference, it is the same as afflux as defined above. Another term that will be seen, usually in the context of culverts and in some of the US Army Corps of Engineers literature, is ‘swell head’.

In the UK, the term ‘backwater’ is generally used to describe the water surface profile upstream of structure – not quite the same as the above definition. Backwater is also used, in the context of ‘backwater effect’, to refer to the distance upstream of the structure where water levels are raised above normal.
3.6 Which term to use?

Head loss has the advantage over afflux or backwater in that it is directly measurable. Afflux or backwater requires the estimation of the water surface that would be present if the structure were removed, and so cannot be directly observed or measured. However, head loss can be misleading as a measure of the effect of a structure on water levels as it can include other losses such as those due to friction and the difference in bed elevation.

In this report and for the future research, we recommend that afflux (the greatest difference in water levels) as defined in Figures 3.3 and 3.4, and Equation 3.1 for uniform flow, is adopted as the primary measurement, especially for the collection of field data on bridge and culvert losses. The reasons for this are to avoid the multiple interpretations of the term backwater and to clearly differentiate the effects of the structure on water levels from the other effects such as friction and change in channel elevation that would be present without the structure.

3.7 Structure surcharge and overtopping

At extreme flood flows, or when the design capacity of the structure is significantly exceeded, it is possible that the structure becomes surcharged (i.e. the water surface is higher than the soffit) or it is even overtopped or by-passed (Figure 3.5).

![Overtopping of a bridge resulting in spill out of the channel and flooding of housing.](image1)

![Water flowing over a highway bridge deck.](image2)

![Water levels approaching the top of the arches of a multi-span bridge.](image3)

**Figure 3.5: Overtopping and surcharging of bridges**
A surcharged condition is sometimes a design condition for culverts (which are structurally more capable of withstanding the resulting hydraulic forces acting on the soffit). Bridges are rarely intentionally designed to surcharge, and the preference for dealing with high flows is to allow overtopping of the roadway or embankment before water levels reach the underside of the bridge deck. However, this is often not possible in the conditions that exist where development has encroached over the years onto the floodplain at a river crossing.

3.8 Blockage

Additional blockage of a structure, other than the ‘intentional blockage’ to flow resulting from the presence of the physical structure, can arise from several causes:

- Collection of floating debris at the abutments/piers or soffit. This is often referred to as ‘temporary blockage’ because the debris can be removed. Removal more often than not requires human intervention, rather than the floe clearing the debris away.

- Collection of floating ice, referred to as ‘ice jams’. This is relatively uncommon in the UK at present.

- Accumulation of bed load at the inlet, outlet, or beneath the structure.

In addition to blockage, other possible effects worthy of note are:

- Bulking the decrease in the density of water as a result of entrained air in highly turbulent conditions or due to large sediment loads and consequent increase in water levels.

- Local scour the lowering of bed levels around a structure usually due to the locally increased flow velocities.

For most UK situations, blockage by floating debris and accumulation of sediment at the bed are the most important material considerations (see Figure 3.6). In this study, the effects of both these types of blockage are considered with floating debris as the primary consideration.

The consideration of scour, bulking and ice jams would have involved considerable widening of the scoping study and have not been considered directly, as these are not though to be so important in the UK. Blockage due to the provision of screens is already dealt with in the Manual on Design and Operation of Trash Screens (Environment Agency R&D Report W5A-01).

This study has adopted a consistent approach to blockage. However, the most important point to recognise with blockage is that its occurrence is triggered by a number of different factors and it must be addressed through a risk-based approach.
3.9 Other structures

Afflux can also occur at other structures that restrict the flow, such as siphons or sluice gates. Knowledge of afflux at these structures is generally good where their purpose is to control water levels, and hence considerable effort has gone into establishing sound hydraulic design criteria.

Weirs are generally built for the specific purpose of creating an afflux, and they can have a considerable backwater effect. However, as weirs are a well-documented form of control, estimation of their effect on upstream water levels is usually more straightforward. As mentioned in Section 1.2, weirs are considered under the Engineering Theme by a separate research initiative, although their hydraulic analysis is not dealt with in depth.

Finally, discrete and short narrowings in a river channel such as that resulting from embanking or construction of channel walls may generate afflux if the available flow area is sufficiently restricted. If the narrowing is discrete, afflux calculations should be used. If however the channel changes occur over several cross sections, then the hydraulic effects are best considered within general water level calculations for non-uniform flow along the river channel of varying cross section.
3.10 Chapter Summary

This chapter has introduced some of the basic definitions and concepts used in bridge and culvert hydraulics relevant to afflux estimation.

The amount of afflux for a given structure varies with flow rate and will also differ depending on the location within and along the channel. For most engineering purposes, it is the maximum value that is required.

It is important when undertaking hydraulic calculations that the afflux effect of a structure is identified separately to other effects and that the location of afflux is correctly identified. Particularly care is required in the location of the points of calculation either side and through the structure.

As the primary concern for flood management is the effects of bridges and culverts on water levels at high flows, it is necessary to also consider, under the umbrella of afflux estimation, the possibility of surcharge and overtopping.

In a similar manner, the proposed future development of an Afflux Estimation System would be planned in such a way as to readily interface with standard river models.
4 POTENTIAL ‘USERS’ AND THEIR NEEDS

4.1 Introduction

Flood management activities can be considered under five main headings:

1. Regulation
2. Improvements
3. Strategic Planning
4. Flood Warning
5. Operations and Maintenance (including Incident Management)

The Environment Agency is seeking to introduce nationally consistent tools and procedures within its areas of operation. The EA also concurs with the views expressed by the ICE Presidential Commission on Flooding in their report ‘Learning to Live With Rivers’ (ICE, 2001). This emphasises the critical importance of estimation of flood water level in all aspects of flood risk management.

4.2 Needs in regulation and Development Control

The influence that the EA and other operating authorities exert on infrastructure and residential/industrial developments through the planning process and the statutory consent procedures of the Water Resources Act and the Land Drainage Act is significant. The prevention of inappropriate and unsustainable developments ensures that floodplain capacity is maintained. The current ‘norm’ is to require that any new structure over or in a watercourse has zero afflux at a defined flood magnitude (usually the 100-year event), or can be demonstrated to have negligible (adverse) influence on flood risk. If the proposals are considered to have an impact and they cannot be modified easily, mitigation measures (such as raising of flood defences) may be considered.

It is common for designers of new structures or consultants undertaking flood risk assessments to estimate afflux by computer modelling – usually using a software package. In practice it is not possible (and arguably it is not the role) of development control officers to check or verify these calculations, although they should have a knowledge of the methods approved by the EA and be able confirm the correct magnitude. There is therefore an important issue of ‘confidence building’. Is it possible to quickly and unambiguously assess whether an appropriate method has been used and that the estimated afflux is realistic? There are ‘back stop’ measures that are often applied (for instance insisting on a minimal blockage or opening ratio for any new structure, in addition to minimal afflux) and in some cases these are appropriate. However, these can be difficult to defend if challenged.

While no consent/regulation regime can always be entirely effective, the benefits of improved design procedures and design knowledge should benefit all developments, whether consented or unconsented.

4.3 Needs in flood risk mapping

Defra and the EA and have several on-going initiatives in flood risk mapping including the Section 105 flood risk mapping programme, the Indicative Flood Risk Maps (IFM) for
England and Wales, Catchment Flood Management Plans (CFMPs), mapping of the extent of major flood events (post event surveys), and the Extreme Flood Outline project. To varying degrees, all these initiatives (except perhaps the post-event flood mapping) rely on the estimation of afflux and blockage risk, with the Section 105 programme being the most dependent.

### Table 4.1: Summary of Flood Risk Mapping Initiatives in England and Wales

<table>
<thead>
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<th>Coverage</th>
<th>Risk Level Mapped</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flood Risk Map of England and Wales (IH 130)</td>
<td>All of mainland England and Wales and Isle of Wight</td>
<td>1% fluvial.</td>
<td>First attempt at national flood risk mapping. Data was used in the production of the IFM maps.</td>
</tr>
<tr>
<td>Indicative Flood Risk Maps</td>
<td>All of England and Wales</td>
<td>1% fluvial, 0.5% tidal Undefended.</td>
<td>1: 10,000 mapping scale. Original (1999) version largely based on IH 130 dataset (see below). Updates issued in 2000, 2001, 2002, 2003. The IFM maps also include S105 map and post event flood data where available.</td>
</tr>
<tr>
<td>Section 105 Mapping</td>
<td>Coverage of defined ‘hot spots’. Intended to identify floodplains on main rivers and in major urban areas of known flood risk.</td>
<td>1% fluvial, 0.5% tidal plus other levels (usually lower) as appropriate. In some cases, the effects of culvert blockage on water levels are considered.</td>
<td>Based on detailed hydrological and hydraulic modelling. 1:2,500 scale urban; 1:10,000 scale rural.</td>
</tr>
<tr>
<td>Post Event Flood Maps</td>
<td>Recent events mapped include the Midlands/Wales floods of Easter 1998, the Yorkshire Derwent floods of 1999 and the North East and South East floods of 2000.</td>
<td>Varies according to severity of event. Includes afflux and blockage effects where recorded.</td>
<td>Tend to be plotted at 1:10,000 or 1:2,500 scale.</td>
</tr>
<tr>
<td>Extreme Flood Outline</td>
<td>All of mainland England. Decision yet to be taken on Wales.</td>
<td>0.1% fluvial and tidal. Structures and blockage not considered.</td>
<td>1: 10,000 mapping scale. Exact method of analysis yet to be determined. Project expected to be completed by March 2003.</td>
</tr>
<tr>
<td>Catchment Flood Management Plans</td>
<td>Intended to cover all major river catchments in England and Wales by March 2007.</td>
<td>Various but 1% fluvial will be included in all studies. Possible climatic change considered.</td>
<td>Largely to be undertaken using the MDSF methodology based on flow routing. Unlikely to explicitly include afflux except in very specific cases. Fluvial equivalent of Shoreline Management Plans.</td>
</tr>
</tbody>
</table>

Note:
Indicated mapping scales are the recommended scale for printing. Most floodplain/flood risk data is available digitally and can be plotted at any scale.

The exceedance probabilities (or return periods) used in Section 105 mapping is generally 1% (100 years) for fluvial flood risk and 0.5% (200 years) for tidal flood risk. The Extreme Flood Outline used 0.1% fluvial and tidal.
Flood Outline will consider the 0.1% (1000 year) fluvial and tidal flood risk. At such flows many structures are likely to be overtopped. Table 4.1 provides a summary of the flood risk maps produced under the auspices of Defra and the EA.

The Indicative Flood Maps (which contain information from the more detailed Section 105 maps and historical events) increasingly influence other areas of the EA’s business. The IFM maps are issued to local planning authorities and are an important tool for deciding on which planning applications are referred to the EA. The maps in a slightly lower resolution form are also publicly available through the EA’s website. The IFM data is also used in the delineation of the EA’s flood warning areas and also in identifying Critical Ordinary Watercourses under the Defra High Level Targets initiative.

At a local level, the accuracy with which afflux is estimated can have a significant impact on the flood outline, particularly at flows in or around bankfull. Blockage of structures can have an even greater impact on flood risk areas.

Currently flood risk mapping commissioned by the EA is being produced under a national framework agreement, with an Alliance Board to promote partnering between the EA and the four consortia of framework consultants. The Alliance co-ordinates the methodologies used in the mapping process through a detailed written specification (National Flood Risk Mapping Specification, Issue 4, November 2001), and the advisory role of a Technical Group consisting of representatives from the EA and the Consultants.

The National Flood Risk Mapping specification currently requires afflux to be assessed using ‘appropriate methods’ and the ISIS, MIKE-11 and HEC-RAS programmes are suggested as appropriate modelling tools. The Agency’s National Survey Specification (Environment Agency, 2002) also provides guidance on appropriate survey techniques and advises on the positioning of cross-sections around structures. The specification includes the option to consider the effects of blockage on structures, and one region of the EA in particular (South West) routinely incorporates this in to its flood plain mapping commissions. It would be logical if consistent approaches were adopted by all regions.

Flood risk maps are also produced as part of more strategic studies such as the Catchment Flood Management Plans (CFMPs), and in capital strategy planning. These sometimes need to map to a level of detail where the effects of afflux and blockage would be discernable. In CFMPs, there can be a need for some consideration of afflux where flooding is known to be related to constrictions at particular structures.

### 4.4 Needs in Operations and Maintenance

Operations and maintenance staff of the EA and other operating authorities carry out important tasks of removing debris, vegetation trimming and general channel maintenance. They have to balance flood defence needs with those of conservation, fisheries, aesthetics and the available resources. With the development of a risk-based approach to maintenance, the need to target work on critical areas is becoming increasingly important. Such targeting would take into account the impacts of the work (on flood levels and the environment) and acknowledges the inherent uncertainty in the assessment process. The approach to specification and provision of these improved measures is being developed under the PAMS initiative (see Section 1.2.1).
A key factor in any targeted programme of maintenance for bridges and culverts (which will mainly relate to the control and removal of blockages) is whether the blockage is significant in flood defence terms. In some cases the negative visual/public impression given by debris blockages can be greater than any actual impact on flood risk. However, the maintainer requires a rational and consistent assessment procedure in order to underpin the maintenance regime adopted (in other words, can the procedures be justified if challenged). Another important consideration is that many temporary blockages occur during flooding where the feasibility of removal is more limited. This is discussed further in the Trash Screens Design Manual (draft report published by Environment Agency, R&D Project W5A-01, June 2003).

4.5 Needs in new works design

The Environment Agency and other operating authorities have major interests in improvement works to rivers, both for environmental considerations and for flood defence. Afflux and blockage are important considerations in many of the pre-feasibility, feasibility, detailed design and post-project appraisal of such schemes, especially in urban areas. Even relatively small changes in the stage/discharge relationship of a structure can have significant impact on the effectiveness and economic appraisal of improvement schemes. There are also wider benefits to UK Consultants working overseas and for others.

Some feasibility and design work is undertaken for the EA by Consultants, and a proportion is also undertaken in-house. Since 1999 major flood defence schemes have come under the remit of the EA’s National Capital Programme Management Service (NCPMS) and since November 2000 much of the design work is being undertaken under the EA’s NEECA Framework Agreement with leading Engineering Consultancy firms.

The benefits of more accurate level estimation in scheme design are two-fold. If afflux/blockage risk is underestimated then the scheme may be under-designed and the risk of flooding will be higher than assumed. On the other hand if afflux is overestimated then inappropriate works may be undertaken. In the context of structures such as bridges and culverts, even minor works to existing structures can be costly and highly disruptive.

Designers of new works include the IDB and local authority staff and specialist and non-specialist consultants. It is important to note that much new urban infrastructure is undertaken with one designer undertaking the road, foul and surface water drainage as one package. The designer may or may not be a specialist in open channel hydraulics, and the common design tools used (MicroDrainage and HydroWorks) are not necessarily the most appropriate for assessing the impact culverts and bridges may have on channel water levels.

4.6 Needs in Hydrometry

Flood discharge is a key parameter in flood defence planning and forecasting. Bridges and culverts can often play an important role in flow gauging as they can act as controls and help to contain the flow. Historically bridges were often used as locations for spot gaugings although the use of cableways and ultrasonics is now generally favoured. Improvements in the estimation of afflux could help in the planning of new gauge sites (in particular in avoiding inappropriate locations) and in generating theoretical ratings for ungauged locations. The results of the afflux research project may have relevance and a cross-linkage with the EA Water Resource Function’s R&D project W6-061 on ‘Extension of Rating Curves using Hydraulic Models’. In this respect, the need in hydrometry is for consistent and appropriate
methods for estimating afflux at all ranges of flow. The Agency’s other needs are much more focused on the higher flows where flow is approaching or exceeding bankfull.

4.6 Needs in Flood Warning

The role of afflux and blockage estimation in flood warning has already been mentioned in the discussion of flood risk mapping, operations and hydrometry, all of which are important inputs into the flood warning systems being used by the EA. As increasing use is made of models for flow forecasting, so the benefit from improvements in afflux and blockage estimation will become more pronounced. The effect that afflux has on river levels needs to be assessed in order that the observed and predicted flows can be properly interpreted. In a flood warning context, this assessment needs to take place in real time or shortened timescales.

4.7 Users outside the Flood Defence operating authorities

On the ‘Client’ side, the users of information on afflux and blockage include:

- Local Authority Development Control/Planning Departments
- Highway Authorities and the Highways Agency
- Navigation Authorities (e.g. British Waterways, Harbour Authorities)
- Railtrack
- British Railways Property Board
- Water Companies
- Developers of new infrastructure.

Many structures of interest to Flood Defence operating authorities are also considered by one or more of the above groups. No estimate was available of the number of structures involved in these sectors. Initial consultation has revealed that Railtrack alone has responsibility for something in the region of 21,000 bridges over water and probably at least double that number of culverts (although it should be noted that the railway industry uses a definition of a culvert that includes any structure with a span between 6 feet (1.8m) and 18 inches (0.45m) which is different to the definition used here). These structures are predominately more than 100 years old and the hydraulic design criteria are unknown and highly variable. The owner of these assets is also not an expert on flood risk or hydraulics and relies heavily on existing guidance.

However the railway, navigation and highways sectors do have an existing regime of technical guidance and are subject to independent audit. There is therefore an existing mechanism for improved guidance on afflux to be disseminated and adopted.

4.8 Research and training

The academic community uses the concept of afflux for teaching and research, but would not be classed as an end user in any practical sense. However, it will be essential to engage with, and obtain the support of, the academic community for delivery of the benefits of this research programme as they influence the education and training of future practising engineers, managers and modellers.
The EA also has a structured professional development programme for its staff and is increasingly looking to identify and demonstrate ‘core competences’. Afflux and blockage awareness should be one aspect of this programme for its flood defence and development control staff. The Association of Drainage Authorities is a focus for identification and dissemination of best practice for IDBs and will have interests in the dissemination of the findings of the afflux research project to this sector.

4.9 Typical target users for the research

As a result of the consultation process (see Appendix 2 for details), an impression has been gained of the range of ‘users’ of information on afflux and blockage. In order to assist in the identification of research needs general ‘categories’ of users have been identified.

The categories have not been based on existing sectoral/institutional structures but rather on the likely resources and technical requirements. Four categories are suggested:

1. The Manager/Asset Maintainer
2. The Flood Defence Professional
3. The (non-specialist) Designer
4. The River Modeller

There is of course an obvious further category of the specialist hydraulic designer/researcher and software developer. However, it is assumed that such professionals will already be familiar with the key issues and will be able to assimilate the results of this research without having the material specifically targeted for them.

Each of these user categories is considered in further detail below.

4.10 The Manager/Asset Maintainer

Groups that would typically fall into this category would be engineers and managers in charge of asset maintenance outside the river engineering sector – e.g. those working for highway authorities, the railway industry and developers. These practitioners need to understand the aspects of the assets they are concerned with that effect afflux.

Likely Needs:
- A non-technical definition of afflux and its causes.
- Tools for easy identification of when to consider afflux/blockage, probably as a trigger to determine when ‘expert’ or ‘outside’ help is required.

This category of user is highly unlikely to be a hydraulics expert or to have access to specialised design software. Afflux will be one of many other considerations they need to take into account. There may be little existing awareness of afflux or its effects both as a design and a management consideration.
4.11 The Flood Management Professional

Groups that would typically fall into this category are flood defence and development control engineers in the Environment Agency, IDB staff and local authority drainage staff.

Likely Needs:
- A non-technical definition of afflux
- A summary of the causes/indicators of afflux and blockage
- Tables showing the range of afflux/blockage that can be expected at ‘typical’ structures.
- Guidance on suitable methods of estimating afflux/blockage and how to specify this to others.
- Quick ‘vetting’ procedures for calculations submitted by others.
- Design tables/nomographs to estimate afflux/blockage.

This category of user is unlikely to be a hydraulics expert or to have access to or the time/training to use specialised software.

4.12 The (non-specialist) Designer

Groups that would typically fall into this category would be engineers working for consultancies and client design teams.

Likely Needs:
- Clear guidance on the selection of appropriate design criteria.
- Clear (prescriptive) guidance on the appropriate methods of assessing afflux and blockage.
- Case studies.
- Advice on the limits of existing methodology and where more specialist analysis is required.

This category of user is unlikely to have a specialist background in hydraulics but is likely to be a graduate with some knowledge of open channel and pipe flow.

A heavy reliance on software in design work would be expected, although access to specialist river/drainage models will be limited (most likely to be HEC-RAS or MicroDrainage).
4.13 The River Modeller

Groups that would typically fall into this category are the EA staff and technical members of Consultant’s teams working on NEECA and Flood Risk Mapping projects for the EA, and undertaking flood risk assessments for developers.

**Likely Needs:**
- In depth appraisal of the methods of estimating afflux and blockage and their limitations.
- Critical advice on using existing modelling packages to estimate afflux and blockage.
- Easy access to other expert practitioners and the latest research.
- How to accommodate afflux and blockage on a catchment scale.

For this user it is important that any advice is integrated with existing guidance such as the EA’s National Flood Risk Mapping Specification and Modelling Guidance.

This category of user may not have a background in hydraulics but is likely to be a university graduate or postgraduate. A noticeable trend in the sector is for the use of personnel from disciplines other than civil engineering as a result of recent increases in demand and the broader base from which such staff are now drawn. Typically university degrees are geography, environmental science and GIS.

A heavy reliance on software is typical, usually a river modelling package (ISIS, HEC-RAS, MIKE-11 and HYDRO).

The benefits of improved afflux and blockage estimation are discussed in Section 11.
Defra/Environment Agency
Flood and Coastal Defence R&D Programme

AFFLUX AT BRIDGES AND CULVERTS

Review of current knowledge and practice

Chapter 5 (Factors that contribute to afflux)
Chapter 6 (Methods of estimating afflux)

R&D Technical Report W5A-061/TR1

J R Benn, P Mantz, R Lamb, J Riddell, C Nalluri

Research Contractor:
JBA Consulting – Engineers & Scientists
5 FACTORS THAT CONTRIBUTE TO AFFLUX

5.1 Introduction

In this section, the most significant features that can contribute to afflux are discussed. These are based on a comprehensive literature review, the questionnaire replies and also consultations with practitioners and experts. It should be noted that many of the factors are themselves inter-related and so the distinctions made here are for illustrative purposes.

The hydraulic performance of a bridge or culvert is a function of the channel geometry, the structure geometry and the flow conditions. Figure 5.1 illustrates types of flow, showing the relationship between upstream and downstream water levels.

![Figure 5.1: Possible flow types through a bridge (the vertical differences between arrowheads represent the afflux).](image-url)
5.2 Type of control

Many types of flow can occur through bridge or culvert openings. These depend primarily upon the water levels upstream and downstream of the structure, the flow discharge, the extent of constriction and its shape. The water levels and the discharge at the structure are controlled either by the channel or by the structure (constriction) itself.

Channel control

For channel control (Figure 5.2) the relationship between stage (water level) and discharge is normally estimated by Manning’s equation,

\[ Q = \frac{A}{n} R^{2/3} S_f^{1/2} \]  (5.1)

where \( Q \) is the discharge in \( m^3/s \), \( A \) is the cross-sectional area in \( m^2 \), \( R \) is the hydraulic radius (\( = A/P \), \( P \) being wetted perimeter in m) in metres, \( n \) is the Manning roughness coefficient and \( S_f \) is the energy slope (which equals the bed slope in the case of uniform flow).

![Figure 5.2: Channel Control](image)

Structure control

If the opening provided is too small, the constriction (structure) itself controls the flow as if it were a sluice gate or orifice (see Figure 5.3). The discharge can then be written as

\[ Q = C_d a_w \left[ 2g(Y_u - \frac{Z}{2} + \frac{a_w V_u^2}{2g}) \right]^{1/2} \]  (5.2)

where \( C_d \) (\( = 0.35 \) to \( 0.6 \)) is the discharge coefficient, \( a_w \) is the total area of the opening flowing full, \( Y_u \) is the upstream depth and \( Z \) is distance between the soffit and the bed level.
When the upstream and downstream water levels are above the top of the opening, the flow is of the drowned orifice type. The flow type could change to pipe flow if the length of the opening is long enough (culverts), in which case the friction plays a role in equation 5.3 (see Novak et al, 2001).

Drowned orifice flow can be defined by:

\[
Q = C_d a_u \left(2g \Delta H\right)^{1/2}
\]

(5.3)

where \(\Delta H = \left[\left(Y_u + \frac{\alpha_u V_u^2}{2g}\right) - \left(Y_d + \frac{\alpha_d V_d^2}{2g}\right)\right]\), the subscripts u and d denoting upstream and downstream respectively.

### 5.3 The opening ratio

This is a measure of the obstruction the bridge or structure presents to the flow. The smaller the opening ratio (i.e. the more the structure is an obstruction to flow) the larger is the afflux. Figure 5.4 shows possible hydraulic variables affecting the bridge performance.

The constriction ratio, M can be written (Hamill, 1999) as

\[
M = q/Q = \frac{a}{A} = \frac{b}{B} = \frac{K_b}{K}
\]

(5.4)

for rectangular openings with no intermediate piers. In this equation, q is a hypothetical portion of the discharge at normal depth through the opening width and Q is the discharge at normal depth across the whole channel.
The symbol \( a \) is the bridge opening area with width \( b \), and \( A \) is the channel area with width \( B \). \( K \) is the conveyance given by:

\[
K_i = \frac{Q}{S_{F}^{1/2}} = \frac{A \text{ } R_i^{3/2}}{n_i} \quad (5.5)
\]

In the case of multi-span bridges, the above equations may be used with gross areas (ignoring the piers’ presence) and then introducing a coefficient. In all cases, use of normal depth is recommended. However, \( M \) can be evaluated from the observed water levels upstream of the bridge.

The calculation of \( M \) in the case of arch bridge openings is more complicated, as \( b \) changes with stage. Biery and Delleur (1962) suggest that \( M \) can still be evaluated using equation 5.4 by assuming \( M = q/Q \) or \( K_b/K \).

For a semicircular opening (arch) of radius \( r \) with the arches springing from bed level, with \( Y_s \) being the water depth in the waterway above the springings (see Figure 3.2), \( M \) can be written as (expressed in radians):

\[
M = \frac{Y_s(r^2 - Y_s^2)^{1/2} + r^2 \sin^{-1}(Y_s/r)}{BY_N} \quad (5.6)
\]

If the arch springer is above the bed, equation 5.6 transforms to

\[
M = \frac{r^2 [\sin^{-1}(h/r) - \sin^{-1}(d/r)] + h(r^2 - h^2)^{1/2} - d(r^2 - d^2)^{1/2}}{BY_N} \quad (5.7)
\]

where the centre of curvature of the arch is at some depth, \( d \) (m) below spring level and \( h \) (m) is the height of the water surface above the centre of curvature. The value of \( M \) can change the discharge coefficient (\( C \) or \( C_d \)) considerably (up to 30%), and hence the discharge through the waterway. Also, the type of flow changes significantly with \( M \) values.
In the cases of single and multiple arch bridges, HR Wallingford (1988) suggests two ‘blockage ratios’ ($J_1$ and $J_2$) which in turn are used to determine the afflux. $J_1$ is defined as the upstream blockage ratio (area of blockage of bridge at depth $Y_1$/area of flow) whereas $J_2$ is the downstream blockage ratio (area of blockage of bridge at depth $Y_3$/area of flow).

5.4 Froude number

The Froude number, $F$, is a measure of the ratio of inertial forces to gravity forces, and is defined by

$$F = \left[ \frac{aQ^2B_T}{gA^2} \right]^{1/2},$$

(5.8)

where $B_T$ is the top width of the water surface (m) between the banks. The type of flow is largely determined by the Froude number. The Froude number is unity at the point of control in an open channel where critical depth is formed (see Figure 5.2). In most river channels, the flow is subcritical.

Some afflux estimation methods (e.g. the USGS method) use the Froude number at the point of minimum cross sectional flow area, which may be within the bridge waterway; this location corresponds to cross section No. 3 in Figure 3.3, and the Froude number here is denoted $F_3$. With arched openings $B_T$ reduces with increase in stage and the solution becomes dubious. Hamill (1993) suggested the use of the bottom width of the arch when calculating $F_3$ and critical depths.

As the flow opening contracts, the bridge flow progressively changes to critical. The limiting (critical) contraction is suggested by Yarnell (1934) as

$$M_L = \frac{27F_1^2}{(2 + F_1^2)^3},$$

(5.9)

where subscripts correspond to the locations of cross sections 1 to 4 shown in Figure 3.3. Henderson (1966) modified this with the assumption that the momentums at sections 3 and 4 are equal, thus

$$M_L = \frac{(2 + 1/M_4)^3F_4^4}{(1 + F_4^2)^3}.$$  

(5.10)

Flow conditions that can occur through a structure are illustrated further in Section 10 (Figure 10.3).
5.5 Choking of bridge opening

Flow through a constriction may be dominated by the ‘choking’ phenomenon. Choking is usually associated with critical or supercritical flow at structures with particularly large reductions in waterway width. It occurs where the constriction in flow is more severe than the limiting contraction (as discussed above). The water level increases upstream of the structure to enable the transition from subcritical to supercritical flow. This is manifest in the real world as a rapid rise in the upstream water level for little or no change in discharge.

The process of choking can be described in terms of the relationship between depth at a section and the specific energy (which is the energy above bed level, equal to the sum of the depth of flow and velocity head). Figure 5.5 shows three such possible depth/energy relationships for different values of specific discharge (i.e. discharge per unit width).

For a given total flow, the effect of making a constriction narrower is to increase the specific discharge through the constriction as shown. For subcritical flow, this can be achieved by an acceleration (driven by an increase in upstream water level). The acceleration is balanced by a decrease in depth which means that there is no change in specific energy required. This situation is illustrated in Figure 5.5 by the transition from point ‘A’ to point ‘B’. Point ‘B’ has been chosen to be at critical depth (i.e. the limiting contraction). For a further contraction, the consequent increase in specific discharge cannot be accomplished at the same specific energy, but requires a shift to the right on the graph, representing an increase in specific energy. This increase in energy leads to an increase in upstream water depth. If choking occurs then the effect is thus to increase upstream water levels, making the afflux larger.

Figure 5.5: Relationship between depth and specific energy showing the effect of choking (after Hamill, 1999)

Most methods of calculating afflux treat the flow as subcritical and do not allow for choking. Choking is not easy to predict and the problem of debris caught on the piers or abutments would always aggravate the situation.
5.6 Ratio of waterway length to width (span) of opening

This ratio provides an indirect indication of whether pipe flow may occur (in the case of culverts) and whether flow re-attachment is likely as flow expands after contracting through the inlet. For arch bridges, the bottom width is used in its calculations. Large waterways (where \( L/b \) is large, say > 1.0) behave like culverts, and the afflux increases.

5.7 Rounded entrances

The hydraulic efficiency of bridge openings (i.e. the amount of flow for a given upstream water level) is improved by the provision of rounded entrances. Rounded entrances reduce the contraction and pass larger flows (increased discharge coefficients) without increasing the water level (see Hamill, 1997).

5.8 Pier shape

Pier shape directly influences the level of turbulence and hence head loss and head loss around the structure. Hence two piers of the same width (giving the same opening ratio) may have slightly different afflux. Rounding of piers or provision of sharp cutwaters not only reduces afflux but also may reduce the tendency of piers to collect debris (Figure 5.6). The latter may have a more marked effect on afflux than the pier shape itself.

![On this bridge, although the pier cutwaters are shaped to divide the flow, the large pier width (4m) still serves to create a discernable ‘bow wave’.](image1)

![Use of extended cutwaters on a bridge pier to reduce the collection of floating debris](image2)

**Figure 5.6: Effect of pier shape on water profile**

5.9 Eccentricity

Figure 5.7 illustrates the eccentricity (\( e \)) of the flow through the waterway. It is quantified as \( e = X_a/X_c \), but is best defined as \( e = Q_a/Q_c \) in terms of the flow discharge, or \( e = K_a/K_c \) in terms of the conveyance.

The effect of \( e \) on \( C_d \) values is particularly insignificant if \( e > 0.12 \) to 0.20.
Figure 5.7: (a) Eccentric crossing (b) Skewed crossing

5.10 Roughness

Channel roughness is an important parameter in river hydraulics as it has a significant role in determining the normal depth, and the tailwater and upstream water levels to a structure. For bridges, the roughness of the actual structure rarely has a significant influence and is often ignored (often for good reasons – see the box below). However, information on this parameter is useful in determining the energy correction coefficient, $\alpha$, particularly in the case of an irregular channel with different values of roughness coefficients attributed to different parts of its boundary. For long culverts, the roughness of the barrel can be an important consideration.

It is important to bear in mind that roughness, although a contributor to afflux, is rarely the prime cause. However an increase in water levels upstream of a structure can be easily achieved by increasing roughness and it is important that the user of computer software in particular is not tempted to use the roughness coefficients as a convenient surrogate. For most situations the roughness of a bridge or culvert is most likely to be less than the channel either side.

5.11 Scour

Scour may actually reduce the afflux levels, though it is detrimental to the bridge foundations. The afflux ($H^*_1$) computed in the absence of scour is adjusted by a correction factor ($S^*_C < 1.0$; Bradley, 1978).

In general, the narrower the openings then the larger the velocities and the finer the bed material. The effects of scour will then increase. Bridges and culverts can be both the cause
and the location for scouring. This scouring can lower the river bed significantly and can even undermine the foundations of the structure (Figure 5.8). The estimation of scour is dealt with comprehensively elsewhere (e.g. Manual on Scour at Bridges and other Hydraulic Structures, CIRIA, 2002) and has not been explicitly considered further as part of this project. However, the effects of scouring in terms of changes to bed level and hence channel section properties needs to be considered in some circumstances of afflux estimation.

![Bridge pier is 1.6m wide and 4m long. Direction of flow is from left to right](image1.jpg) ![General or natural scour resulting in exposure of bridge foundations.](image2.jpg)

**Figure 5.8: Example of scour at a bridge pier**

### 5.12 Blockage

Blockage is the most unpredictable factor leading to afflux and could take the form of anything from leaves and branches to whole trees, garden sheds, caravans etc (Figure 5.9). In addition to the increase in flood levels that the debris might cause, the hydrostatic pressures on the structure might also increase and lead to structural problems. Debris can also cause changes in the hydraulic performance of the structure and can exacerbate scour and damage due to hydraulic loading.

Debris trapped upstream or downstream of the structure can cause changes in river course and hence a change to flow presentation at the structure. The causes and effects of siltation, like that of scouring, although important are however outside the scope of this study.
Partial blockage of the railings of a footbridge.

Collection of floating debris at a bridge soffit during flood.

Increasing the risk of blockage by poor culvert design.

Figure 5.9: Examples of blockage
### Table 5.1: Variables that affect afflux

<table>
<thead>
<tr>
<th>Variable</th>
<th>Likely effect on upstream water levels/afflux</th>
</tr>
</thead>
<tbody>
<tr>
<td>Opening Ratio (M)</td>
<td>The smaller the opening ratio, the larger the afflux. If very high, may cause choking of the inlet.</td>
</tr>
<tr>
<td>Froude Number (F)</td>
<td>Generally the higher the Froude Number, the larger the afflux. (This is the Froude Number for flow in the channel, in the absence of the structure).</td>
</tr>
<tr>
<td>Choking of bridge opening</td>
<td>Afflux increases when flow is choked.</td>
</tr>
<tr>
<td>Length/breadth Ratio (L/b)</td>
<td>If ratio is high (L/b &gt; 1.0) then afflux generally increases</td>
</tr>
<tr>
<td>Rounding of the entrance/ Rounding of Piers</td>
<td>The smoother the entry, the less turbulence and hence the lower the afflux. Smoothing/rounding can also help to reduce blockage risk.</td>
</tr>
<tr>
<td>Eccentricity, e</td>
<td>For e &lt; 0.2, afflux is generally reduced.</td>
</tr>
<tr>
<td>Skew</td>
<td>The larger the skew of a structure relative to the direction of flow, the larger the afflux. For skew angles of less than 20°, the effect is usually negligible.</td>
</tr>
<tr>
<td>Roughness</td>
<td>The larger the roughness, the larger the afflux.</td>
</tr>
<tr>
<td>Scour</td>
<td>Generally reduces bed levels and hence decreases afflux.</td>
</tr>
<tr>
<td>Blockage</td>
<td>Decreases the opening ratio and increases turbulence at the entrance. Nearly always results in a higher water level.</td>
</tr>
</tbody>
</table>
6 METHODS OF ESTIMATING AFFLUX

6.1 Introduction

This chapter begins by describing the principle theoretical approaches to afflux calculation. The chapter then gives an overview of the main methods for calculating afflux for various types of bridge and culvert structures under different hydraulic conditions. The principles of afflux calculation are discussed here, whilst details of the specific methods are given in Appendix A. The discussion draws on an expert paper by Knight (2001), commissioned for this study to review current knowledge on bridge afflux, and attached as Annex 1 to this report. A similar paper by Samuels (2001), attached as Annex 2, reviews the implementation of afflux estimation in hydraulic models.

6.2 Theoretical approaches to afflux calculation

There are two main representations used for estimating the afflux upstream of a bridge. The first representation assumes that the afflux (ΔY) is a proportion of the kinetic energy of the flow through a bridge, thus:

\[ ΔY = K \frac{V^2}{2g} \]  

(6.1)

where K is a friction factor, V is the mean velocity through the bridge, and g the acceleration of gravity.

The second representation uses the afflux as an independent variable in representing the flow discharge (Q), thus:

\[ Q = CA \cdot f(ΔY) \]  

(6.2)

where C is a dimensional discharge coefficient, A is the area of flow through the bridge, and f(ΔY) is a function of the afflux. These methods apply to the pier bridges and embankment bridges, as described below. The basis for the representations is estimation of the energy loss caused by the structure, which leads to the increased upstream level or afflux that is required for a steady flow.

The rationale for the two representations of afflux is best illustrated by considering their application to steady, uniform flow in a river channel. In river hydraulics, the dimensionless relationships for steady open channel flow can be adapted from pipe flow equations (Roberson et al, 1997). For example, the head loss (hf) for turbulent flow in pipes is given by the Darcy-Weisbach formula as:

\[ h_f = f \frac{L}{D} \frac{V^2}{2g} \]  

(6.3)

where f is a pipe friction factor (which may be interpolated from a Moody diagram), L is the pipe length, D the pipe diameter, V the mean velocity of flow in the pipe and g the gravitational acceleration. This formula is adapted to channels by noting that the head loss in an open channel is given by \( (S \cdot L) \) where S is the channel slope and L the channel length. The pipe diameter (D) is replaced by 4R for a channel, where R is called the “hydraulic”
radius denoted by the area/perimeter ratio. By equating the head losses, the channel velocity and flow are is solved as

\[
V = \left(\frac{8g}{f}\right)^{0.5} \ast (RS)^{0.5}
\]

and

\[
Q = \left(\frac{8g}{f}\right)^{0.5} A \ast (RS)^{0.5}
\]

where \(A\) is the area of the channel. Thus \(\left(\frac{8g}{f}\right)^{0.5}\) is the discharge coefficient (C) for uniform flow, and the two representations are equivalent.

For bridge hydraulics, however, the friction factor and discharge coefficient depend upon many more variables (as discussed in Section 5), owing to the heterogeneity of the bridge structural geometry and incident flows. A unique solution, as for the case of steady flow river hydraulics, has therefore not yet been achieved.

6.2.1 Theoretical approaches – An example of the friction factor representation for a pier bridge

A “pier bridge” is defined herein as crossing the entire flood plain, being supported by several piers, and usually located in a rural setting. The resistance to flow is caused mainly by the presence of the piers (Figure 6.1(a) illustrates a simplified type). The laws for the conservation of mass and momentum have been applied to a simplified pier bridge (Montes, 1998), and it was shown that the result approximates the friction factor method.

![Figure 6.1: (a) Pier bridge (b) Embankment bridge](image-url)
The nomenclature for the analysis is illustrated in Figure 6.2(a) and the control volume in Figure 6.2(b). The momentum balance between sections 4 and 1 is given by:

\[ \rho Q(V_4 - V_1) = 0.5 \rho g B (Y_1^2 - Y_4^2) + F \]  \hspace{1cm} (6.6)

where \( \rho \) is the mass density, \( V_4 \) and \( V_1 \) are the mean velocities for each section, \( B \) is the width between piers, and \( F \) is the drag force on the bridge piers. The river bed shear resistance is neglected as it is considered much less than \( F \), which is quantified using a drag coefficient \( (C_d) \) that depends on the pier shape (of thickness, \( t \)), such that

\[ F = C_d (Y_1 t) V_1^2 / 2 \]  \hspace{1cm} (6.7)

This formulation assumes that \( V_1 \) is of the same order as the velocity between the piers. The momentum equation can now be written as

\[ V_1^2 / g Y_1 (1 - Y_1 / Y_4) = 0.5 (Y_4^2 / Y_1^2 - 1) - 0.5 C_d* t / B* V_1^2 / g Y_1 \]  \hspace{1cm} (6.8)

A dimensionless afflux ratio may be defined as \( \lambda = (Y_4 - Y_1) / Y_1 \), and thus the momentum equation may be written with the downstream Froude number \( (F_1 = V_1 / (g Y_1)^{0.5}) \) as a dependent variable.
\[ F_1^2 = \lambda (1 + \lambda)(2 + \lambda)/(\lambda + 0.5C_d/B(1 + \lambda)) \]  \hspace{1cm} (6.9)

Curves of \( \lambda \) against \( F_1 \) with \( 0.5C_d/B \) as a parameter are illustrated in Figure 6.3, and it seen how rapidly \( \lambda \) increases with \( F_1 \). This is analogous to the bow waves produced by a ship’s hull. The curves also indicate the significance of increased water levels with unstreamlined piers (high \( C_d \)), and thus the increased flooding risk. It was also shown by Montes (1998) that as \( \lambda \) is decreased below about 0.1, the momentum equation further reduces to:

\[ \Delta y \cong C_d/B[1 + F_1^2(1 + 0.5C_d/B)] \frac{V_1^2}{2g} \]  \hspace{1cm} (6.10)

Thus the momentum equation for a pier bridge is reduced to the friction factor method.

![Graph of dimensionless afflux vs. downstream Froude number](image)

**Figure 6.3:** Dimensionless afflux as a function of downstream Froude number and contraction parameter

### 6.2.2 Theoretical approaches – An example of the discharge coefficient representation for an embankment bridge

An embankment bridge is one whose sides contract the river channel on the flood plain and whose deck is supported by the abutments; it is usually located in an urban setting. The resistance to flow is caused mainly by the flow contraction upstream and the flow expansion downstream (Figure 6.1(b) illustrates a simplified type). The nomenclature, plan and elevation are shown in Figure 6.4, and differ from the above pier bridge example.
Figure 6.4: (a) Side elevation for the water surface of an embankment bridge (b) Plan elevation for the bridge contraction

The analysis involves the application of the conservation of energy between sections 4 and 2, then the conservation of momentum between sections 2 and 1 to evaluate the expansion energy loss. As in the previous example, the river bed resistance is considered negligible, although it may be included in the analysis if desired.

The energy equation between sections 4 and 2 is:

\[ Y_4 + \frac{Q^2}{2gA_4} = Y_2 + \frac{Q^2}{2gA_2} + \Delta E_{4-2} \quad (6.11) \]

where uniform flow is assumed at the sections (that is, the kinetic energy coefficients are assumed as unity for simplicity), and \( \Delta E_{4-2} \) is the energy loss between the sections. As \( \Delta E_{4-2} \) increases, the specific energy at section 2 decreases to its minimum value of critical flow. At this stage, section 2 now controls the flow and any further increase in upstream flow incurs an upstream flood condition. The flow is said to be ‘choked’ when critical flow occurs (see Section 5.5), and the contraction condition has been estimated (Montes, 1998) in terms of the upstream Froude number, \( F_4 = \frac{Q}{BY_4(gY_4)^{0.5}} \), thus:

\[ \frac{b}{B} = 1.838 \frac{F_4}{(1 + F_4^2/2)^{1.5}} \quad (6.12) \]

The momentum equation between sections 2 and 1 is written:

\[ Q(V_2 - V_1) = 0.5gB(Y_{1}^2 - Y_{2}^2) \quad (6.13) \]

where \( V_2 \) and \( V_1 \) are the mean velocities in the sections. If \( Y_4, Y_2 \) and \( \Delta E_{4-2} \) are known, the energy equation is sufficient to solve for \( Q \), and the momentum equation can be used to solve
for $Y_1$ and thus the afflux ($Y_4 - Y_1$). Unfortunately, it is $Y_1$ that is usually known from a normal flow or backwater condition, and the solution for $Y_2$ then becomes an iterative problem. Furthermore, the energy loss $\Delta E_{4-2}$ is a complex function of the bridge geometry and flow. It can however be analysed using the discharge coefficient method as follows.

The energy loss between sections 4 and 2 is due to frictional and eddy losses in the contraction, and is written in terms of a loss coefficient,

$$\Delta E_{4-2} = K_{4-2} \cdot \frac{Q^2}{2g(bY_2)^2}$$  \hspace{1cm} (6.14)

Substituting this term into the energy equation, an implicit equation for $Q$ may be written using the discharge coefficient representation, thus:

$$Q = C_b Y_2 \cdot \sqrt{2g(Y_4-Y_2 + \frac{Q^2}{2gA_4} - K_{4-2} \cdot \frac{Q^2}{2g(bY_2)^2})^{0.5}}$$  \hspace{1cm} (6.15)

The discharge coefficient $C$ has been evaluated for many contraction geometries and flow types by Kindsvater and Carter (1953), and is described more fully below. Note however that unless the momentum equation from section 3 to 4 is solved, then the true afflux ($Y_4-Y_1$) cannot be computed.

The preceding paragraphs in Section 6.2 have discussed theoretical principles applied to calculate afflux. Specific methods for afflux calculation, for different structures and flow conditions, are considered below in Sections 6.3 to 6.8.

### 6.3 Afflux calculation methods for pier bridges

Some of the early researches into bridge hydraulics were mainly concerned with pier bridges. The work of d’Aubuisson (1840) and Nagler (1917) are examples of the discharge coefficient representation, and the work of Rehbock (1921) and Yarnell (1934) are examples of the friction factor representation. They are summarised below.

#### 6.3.1 Discharge coefficient representation (pier bridges)

Following from the previous discharge coefficient representation (Equation 6.15), d’Aubuisson assumed that $Y_2 = Y_1$, and the energy loss $\Delta E_{4-2}$ was negligible, thus:

$$Q = C_A b Y_1 \cdot \sqrt{2g(Y_4-Y_1 + \frac{V_4^2}{2g})^{0.5}}$$  \hspace{1cm} (6.16)

Nagler (1917) expanded this relation by introducing the energy loss as a coefficient ($\eta$) for the upstream velocity head. He also assumed that $C$ was directly influenced by the downstream Froude number, $F_1 = \frac{V_1}{(gY_1)^{0.5}}$, thus:

$$Q = C_N b Y_1 \cdot (1-\sqrt{0F_1^2})[2g(Y_4-Y_1 + \eta V_4^2/2g)]^{0.5}$$  \hspace{1cm} (6.17)

The coefficients $\theta$ and $\eta$ are coefficients to account for the contraction ratio ($b/B$) and energy losses between sections 4 and 2. An approximate value of $\theta = 0.15$ was given by Nagler, and $\eta$ was estimated as:

$$\eta = 1 + 1.05 \tanh[4.5(1 - b/B)]$$  \hspace{1cm} (6.18)
The discharge coefficients $C_A$ and $C_N$ for the above representations were determined empirically, and depended principally on the pier shape. The most reliable values are those determined by Yarnell (1933) from a series of 2600 laboratory measurements.

### 6.3.2 Friction factor representation (pier bridges)

Both Rehbock (1921) and Yarnell (1934) used the friction factor representation for pier bridges, and assumed a functional relation for $K$ as:

$$K = K(\alpha, F_1, \delta)$$  \hspace{1cm} (6.19)

where $\alpha = 1-b/B$ and is a measure of the contraction, $F_1$ is the downstream Froude number, and $\delta$ is a pier shape coefficient. The functional forms of these coefficients were determined empirically as:

- **Rehbock:**
  $$K_R = (\delta - \alpha(\delta - 1))(0.4\alpha + \alpha^2 + 9\alpha^4)(1 + F_1^2)$$  \hspace{1cm} (6.20)

- **Yarnell:**
  $$K_Y = 2\delta(\delta + 5F_1^2 - 0.6)(\alpha + 15\alpha^4)$$  \hspace{1cm} (6.21)

Although these equations are similar, the Rehbock friction factor evaluates to about 50% of the Yarnell factor. Since the Yarnell factor is based on more data however, it is preferred. As for the discharge coefficient representation, the pier shape factor was based on varied geometries as given in Table 6.1 (after Yarnell, 1934).

<table>
<thead>
<tr>
<th>Pier Shape</th>
<th>$K_Y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Semi-circular nose and tail</td>
<td>0.90</td>
</tr>
<tr>
<td>Lens-shaped nose and tail</td>
<td>0.90</td>
</tr>
<tr>
<td>Twin-cylinder piers with connecting diaphragm</td>
<td>0.95</td>
</tr>
<tr>
<td>Twin-cylinder piers without connecting diaphragm</td>
<td>1.05</td>
</tr>
<tr>
<td>90° triangular nose and tail</td>
<td>1.05</td>
</tr>
<tr>
<td>Square nose and tail</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Note that these early pier methods were applicable for subcritical flows only, and cannot be used for high bridge flow conditions when the soffit becomes submerged.

### 6.4 Afflux calculation methods for embankment bridges

As for pier bridges, early research on embankment bridges can be clearly divided into the discharge coefficient and friction factor representations. The work by Kindsvater, Carter and Tracy (1953) and extended by Matthai (1967) used the discharge coefficient representation. This work emanated from the US Geological Survey (USGS) and was based on the results of a laboratory research programme at the Georgia Institute of Technology.

In contrast, the work by Bradley (1978), and recently modified by Kaatz and James (1997), used the friction factor representation. This method is used by the US Bureau of Public Roads (USBPR) for the hydraulic design of bridges. The methods are summarised below.
6.4.1 Discharge coefficient representation (embankment bridges)

The Kindsvater and Carter method attempted to evaluate the discharge coefficient (C) for a universal range of bridge openings. The results were based on an extensive laboratory study and its verification using 30 field sites. The discharge coefficient was derived as summarised in 6.3, thus:

\[ Q = C b Y_2 \star [2g(Y_4 - Y_2 + Q^2/2gA_4^2 - K_{4-2} \star Q^2/2g(bY_2)^2]^{0.5} \quad (6.22) \]

The procedure has been documented fully in hydraulics textbooks (Chow, 1981; French, 1986; Hamill, 1999) and only the main principles are given herein.

In general, bridge openings were classified into four types, as follows:

1. Vertical embankments and vertical abutments, with or without wingwalls.
2. Sloping embankments and vertical abutments.
3. Sloping embankments and sloping abutments.
4. Sloping embankments and vertical abutments with wingwalls.

For each type, the discharge coefficient was determined as a function of the bridge geometry and flow. For example

\[ C = C(b/B, L/b, r/b, \varphi, e, F_2) \quad (6.23) \]

where \( L \) is the length of the bridge contraction, \( r \) is the radius of curvature at the contraction entry, \( \varphi \) is the skew or angle that the axis of contraction differs from the flow direction, and \( e \) is the eccentricity or distance off-centre of the bridge opening from the axis of symmetry of the channel flow. When these design coefficients were evaluated for a field measurement, Kindsvater and Carter found that the field results differed by less than 5%. The method was also extended with further coefficients to multiple openings, spur dykes (guide walls at the bridge approach), and submerged bridges.

Note again that the method does not give afflux directly from the discharge calculation. The momentum equation or energy loss from section 2 to 1 is required to compute the downstream elevation for estimating afflux. As an alternative, Kindsvater and Carter (1955) gave an empirical method for estimating the afflux that used two design charts.

6.4.2 Friction factor representation (embankment bridges)

The Bradley method for estimating afflux begins by considering the energy equation between sections 4 and 1 (Figure 6.4), which can be written

\[ Y_4 + Q^2/2gA_4^2 = Y_1 + Q^2/2gA_1^2 + \Delta E_{4-1} \quad (6.24) \]

The friction factor representation is then used to estimate the energy loss,

\[ \Delta E_{4-1} = K \star (\alpha_3 V_{N3}^2/2g), \quad (6.25) \]

where \( \alpha_3 \) is the velocity distribution coefficient for non-uniform flow, and \( V_{N3} \) is a reference velocity equal to \( Q/A_{N3} \). The area \( A_{N3} \) is the hypothetical area in the contraction subtended by
the normal depth of the channel in the absence of the contraction. Thus \( V_{N3} \) can be readily calculated for a bridge obstruction in a flow at normal depth.

Substituting the friction factor representation in the energy equation gives the afflux as:

\[
\Delta y = K \times \left( \frac{\alpha_3 V_{N3}^2}{2g} + \alpha_3 \frac{V_3^2}{2g} - \alpha_4 \frac{V_4^2}{2g} \right)
\]

If it is assumed that the velocity distributions at sections 4 and 1 are similar, then \( \alpha_4 = \alpha_1 \). By conservation of mass, \( A_1 V_1 = A_4 V_4 = A_{N3} V_{N3} \), thus the above equation may be simplified to:

\[
\Delta y = K \times \left( \frac{\alpha_3 V_{N3}^2}{2g} + \alpha_4 \left[ (A_{N3}/A_1)^2 - (A_{N3}/A_4)^2 \right] \frac{V_{N3}^2}{2g} \right)
\]  

The flow variability is now governed by the velocity distribution coefficients, as opposed to the downstream Froude number as used in the discharge coefficient method. Ideally, \( \alpha_1 \) should be estimated from a velocity traverse, and a design chart can be used to estimate \( \alpha_3 \) as a function of \( b/B \). Thus given \( K \) and \( \alpha_4 \), the afflux equation may be solved iteratively.

The friction factor \( K \) is calculated as a function of the bridge geometry using incremental coefficients, thus:

\[
K = K_b + K_e + K_\phi + K_p
\]

where \( K_b \) is the main coefficient depending on the contraction ratio \( (b/B) \) and the geometry of the abutments (similar to the bridge opening types in the Kindsvater and Carter method). The other coefficients represent the influence of eccentricity, skew and the type and number of bridge piers.

In addition to estimating afflux for subcritical flows, Bradley (1978) has extended the method to the following situations:

1. Difference in water level across approach embankments.
2. Dual bridges.
3. Abnormal stage-discharge conditions.
4. Effect of scour on backwater.
5. Superstructure partially inundated.
7. Flow passing through critical depth.

Note that the method is more direct than the Kindsvater and Carter method, but it requires a knowledge of the velocity distribution for estimating the \( \alpha_1 \) coefficient.

### 6.5 Afflux calculation for arched bridges

There are two major studies concerned with the hydraulics of arched bridges. These are the work by Biery and Delleur (1962) and the Hydraulics Research study (HR, 1988). In general, both methods conclude with a simple, empirical, functional representation for afflux in the form:

\[
\Delta Y/Y_N = f(b/B, F_N)
\]
where the subscript N refers to the downstream section at normal flow. Thus the afflux is simply considered a dimensionless function of the bridge opening ratio and the downstream flow condition (given by the Froude number). The methods are therefore limited to normal, non-eccentric openings. A brief summary of each work follows.

6.5.1 Biery and Delleur method (1962)

This appears to be the first laboratory study on single span arch bridges. In addition to presenting equations for the afflux determination (as shown graphically in Hamill, 1999), the following factors were evaluated:

1. Variation of the distance from the bridge face to the section of the afflux with $F_N$ and $b/B$.
2. Variation of the distance between the sections of maximum and minimum water levels with $b/B$.
3. Variation of the coefficient of discharge for a semi-circular arch with $F_N$ and $b/B$.

6.5.2 HR method (1988)

The Hydraulics Research (HR) investigations extended the study of arched bridges to both field and laboratory investigations and to single and multiple arched bridges. The types of bridges analysed were:

1. Model single semicircular arches
2. Model single elliptical arches
3. Model multiple semicircular arches
4. Model multiple semicircular arches with different soffit levels
5. Prototype single arches
6. Prototype multiple arches

Instead of an opening ratio, HR defined a blockage ratio defined as the ratio between the structural blockage to flow and the total flow area. This ratio varies through the bridge due to the differing water depths and flow areas. As a consequence, three design charts were produced:

1. Variation of afflux with downstream Froude number and downstream blockage ratio for all bridges.
2. Variation of afflux with downstream Froude number and upstream blockage ratio for single arches.
3. Variation of afflux with downstream Froude number and upstream blockage ratio for multiple arches.

6.6 Modern computational applications of afflux calculation methods

The major problem with the above methods is that they all attempt to represent the energy losses in all three bridge ‘reaches’ shown in Figure 6.4, (i.e. section 1 to section 2 (the contraction), section 2 to section 3 (bridge waterway) and section 3 to section 4 (expansion)) with a single empirical coefficient. They therefore provide an order of magnitude estimate for afflux using hand calculation. With computer modelling, it is possible to solve complex water
level problems iteratively by dividing a river into reaches and solving from the upstream or downstream control (or boundary condition).

This same method can be used for the three bridge reaches, and thus a more accurate solution may be attained by estimating energy loss coefficients for each reach. Three examples are chosen to summarise these methods, namely the Schneider et al method (USGS, 1977) used in the WSPRO program (FHWA, 1986), and the energy and momentum methods used in the HEC-RAS program (USACE, 1995).

6.6.1 Schneider et al. (1977) method in WSPRO

The energy losses for each reach are represented in terms of the conveyance ($K_i$), where the subscript $i$ refers to the reach identification. The conveyance is defined by $Q = K_i S^{0.5}$, and thus the energy head loss for a reach is given by

\[ \Delta E_i = L_i \cdot S_i = \frac{Q^2 L_i}{K_i}. \]  

(6.30)

For the approach (contraction) reach, then friction losses are

\[ \Delta E_{4-3} = \frac{Q^2 L_{4-3}}{K_4 K_c}, \]  

(6.31)

where $L_{4-3}$ is tabulated, and $K_c$ is the smaller of conveyances between $K_2$ and $K_q$ ($K_q$ is the portion of section 4 conveyance contained within the bridge).

For the constricted (bridge) section:

\[ \Delta E_{3-2} = \frac{Q^2 L_{3-2}}{K_2^2}. \]  

(6.32)

For the expansion section, friction losses are:

\[ \Delta E_{2-1} = \frac{Q^2 L_{2-1}}{K_1 K_c} \]  

(6.33)

And also for the expansion section, turbulent losses are given by:

\[ \Delta E_{2-1} = \frac{Q^2}{2g A_1} [2(\beta_1 - \alpha_1) - 2\beta_2 (A_1/A_2) + \alpha_2 (A_1/A_2)^2] \]  

(6.34)

where $\beta$ is the momentum correction coefficient for a section.

6.6.2 Energy method in HEC-RAS (1995)

The energy method uses both the conservation of mass and energy for each bridge reach, thus:

Mass:

\[ Q_u = Q_d = V_u A_u = V_d A_d \]  

(6.35)

Energy:

\[ Y_u + \alpha_u \frac{V_u^2}{2g} = Y_d + \alpha_d \frac{V_d^2}{2g} + \Delta E_{u-d} \]  

(6.36)

where:

\[ \Delta E_{u-d} = LS + C (\alpha_u \frac{V_u^2}{2g} - \alpha_d \frac{V_d^2}{2g}) \]  

(6.37)
The subscripts u and d refer to the upstream and downstream sections for each reach. Note that S is taken as the average friction slope for the reach. This equation can be solved iteratively for each reach, provided that values are entered for the $\alpha$ and C coefficients.

### 6.6.3 Momentum method in HEC-RAS (1995)

The conservation of mss equation is again used, but now combined with the conservation of momentum, such that:

$$\beta_u Q_u^2/g A_u + Y_u' = \beta_d Q_d^2/g A_d + Y_d A_d + F_{ext}/\rho g + F_w/\rho g + F_a/\rho g$$  \hspace{1cm} (6.38)

where $Y'$ is the depth from the water surface to the centre of gravity of the flow section, $F_{ext}$ is the sum of streamwise drag forces such as bed and pier friction, $F_w$ is the streamwise, fluid weight force component, and $F_a$ is the streamwise force component due to different flow sections.

Note that since most observations of bridge flow profiles only include water level, then the energy equation is the most frequently used. A recent study by Brunner and Hunt (1995) has documented the evaluation of C coefficients in detail.

### 6.7 Extreme (high) flow methods

Extreme high flows are defined as those that submerge the bridge soffit. These may be further defined in order of increasing height as sluice flows, orifice flows, weir plus orifice flows, and totally submerged flows. The latter condition is a total flood condition for which a new flood plain geometry may well apply. The intermediate high flows use a discharge coefficient equation, since the hydraulics of sluiices, orifices and weirs have been previously established. It is therefore inevitable that solutions from each of the computer packages (HEC-RAS, ISIS and MIKE 11) should give similar results. The methods used by HEC-RAS only are therefore summarised below (Brunner and Hunt, 1995).

#### 6.7.1 Sluice flow

When water reaches the soffit level, a sluice gate type flow is initiated, and the discharge equation is given by:

$$Q = C_d A_{BU} \sqrt{2g(y_3 - z/2 + \alpha_3 V_3^2/2g)}$$ \hspace{1cm} (6.39)

where $C_d$ is the discharge coefficient, $A_{BU}$ is the area of the bridge opening at section 3, and $z$ is the vertical distance from the soffit to the river bed inside the bridge reach.

#### 6.7.2 Orifice flow

When both the upstream and downstream side of the bridge are submerged, the standard orifice equation is used:

$$Q = CA(2gH)^{0.5}$$ \hspace{1cm} (6.40)

where $H$ is the difference between upstream and downstream head. The discharge coefficient has a typical value of about 0.8.
6.7.3 Weir flow

Flow over the bridge and the roadway approaching the bridge is calculated using the standard weir equation:

\[ Q = CLH^{1.5} \]  \hspace{1cm} (6.41)

where \( L \) is the effective length alongstream of the weir, and \( H \) is the afflux. Orifice flow may be optionally added to the weir flow, if the bridge is not blocked. As the water elevation is increased downstream, the weir discharge is empirically reduced. When the weir becomes highly submerged, the computer program automatically switches to water surface calculations using the energy equation.

6.8 Culvert methods

As for bridge hydraulics, the hydraulics of culverts may be classified into low and extreme flows. Culvert hydraulics are well documented in standard hydraulic texts and are usually described in terms of different flow types depending on which type of control prevails. The three low inlet flows (Figure 6.5) may be identified as totally subcritical flow, subcritical barrel flow with outlet control to critical flow (then an outlet hydraulic jump), and critical barrel flow (then an outlet hydraulic jump) with inlet control. The extreme flows are those for which the conduit inlet becomes totally submerged. There are also three flow types leading to a critical flow outlet, a free outlet and a submerged outlet.

6.8.1 Low flows

Although the three river modelling packages in the EA’s Best Interim Systems (BIS) ‘A’ list (HEC-RAS, ISIS and MIKE 11) treat a culvert in the same way as bridge flow by considering three reaches (as in Figure 6.4), there are differences in the methods of each. As an example, the energy method used in HEC-RAS (Equation 6.37) includes the bed surface friction term, and therefore accounts for physically long entries and exits. In contrast, the energy method used in MIKE 11 uses a constant discharge with three coefficients for the entry, conduit and exit reaches. It does not explicitly account for bed surface friction effects in the entry and exit reaches, and may therefore be assumed relevant to short conduits only.

It appears that HEC-RAS is the most versatile package at present, since it has the most options. The model includes the following variables:

- A selection of nine common culvert shapes
- Single and multiple barrels
- Free surface flow with inlet or outlet controls
- Horizontal and adverse slopes
- The addition of silt to the inlet to simulate blockage

6.8.2 Extreme (high) flows at culverts

In common with the extreme flows for bridges, each of the three simulation packages model submerged inlet flows using the sluice, orifice and weir flow equations in sequence. Where
Figure 6.5: Unsubmerged and submerged inlet flows through a culvert

necessary, the models can also include road overflow around a structure, and this is integrated in HEC-RAS.

6.9 Measurement of afflux in the field

Reliable field data on afflux is very difficult to obtain, partly because it cannot be measured directly and partly because of the logistics of recording flow and levels at bridges and culverts at extreme flows. Ideally, the whole water surface profile is required from the start to the end of backwater water surface. The water surface profile (in the absence of the structure) can be estimated by backwater analysis and subtracted from the measured profile to determine the afflux. If data is measured only at a point then the position depends on the method of analysis. For example, the Kindsvater et al (1953) method requires water levels at sections 4 and 2 (Figure 6.4) whereas the Bradley at al (USBPR, 1978) method requires measurements at sections 4 and 1 (Figure 6.4).
With reference again to Figure 6.4, the position of section 4 is sufficiently far upstream of the structure and is normally located at one span (b) upstream of the structure face. The downstream section 2 is parallel to the contraction defining the minimum water depth, and in general is difficult to measure in the field. Section 1 is rather dependent on the nature of the flow expansion downstream of the structure. HEC-RAS suggests a rate of expansion of 1:4 (i.e. it should be at least 2(B-b) from the downstream face of the bridge).

Later studies (HEC Report RD-42, 1995) suggest reduced expansion ratios, giving the location of section 1 at around 2b from the downstream face of the bridge. In all cases, the variation of water levels across a section is usually ignored (which is not truly correct). See the report by Kirby and Guganeshrajah (2001) for further information.

### 6.10 Some examples of field measurements for afflux

Hamill (1997) quotes measurements (taken around 1736) at London Bridge indicating a fall (difference in water levels across the bridge) amounting to 1.45 m with an opening ratio, \( M < 0.5 \) (Table 6.2).

Hamill (1993) also measured afflux at a single arched bridge at Canns Mill in Devon and recorded values as high as 17mm for open channel control, 115mm for structure control and 270mm with the bridge acting as an orifice.

Bradley (1978) lists the head loss measurements underpinning his research, the smallest being around 50mm, in a range of 50-900mm.

The following field data sets on bridge afflux have been located as part of this review:

- Work by Kaatz and James (1997)
- Work by Hamill and McInally (1990)
- Work to develop the USGS method (Matthai, 1967)
- Work to develop the USBPR method (Bradley, 1978)
- Work undertaken to develop HR method for arch bridges (selected bridge sites, Brown, 1989)
- Work related to PhD studies at the University of Birmingham (Atabay and Knight, 2001). The experiments included different floodplain roughness conditions and different types of bridge opening, namely single opening semi-circular arch bridge, multiple opening semi-circular arch bridge, single opening elliptical arch bridge, single opening straight-deck bridge with and without piers including span widths.

Appendix 3 provides further details of these datasets.

The quality and usefulness of this data is to be reviewed in Stage 2 of the project. Note that no datasets have been located dealing with culvert blockage.

Another potential source of data on afflux could be from past commercial physical model studies. This should in particular provide information on the more unusual structures such as those that are skewed. While much of this data is proprietary, it would be worth investigating further.
Table 6.2: Examples of measured bridge afflux/head loss

<table>
<thead>
<tr>
<th>Location/ Photograph</th>
<th>Opening Ratio</th>
<th>Afflux/Head loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old London Bridge (C12th)</td>
<td>0.31–0.49</td>
<td>Fall (head loss) measured as 1.45m in 1736 (Hamill 1999) The difference was used to drive a water wheel</td>
</tr>
<tr>
<td></td>
<td></td>
<td>The range is due to the reference stage and the existence of ‘starlings’ or skirt ing around the piers as a form of scour protection.</td>
</tr>
<tr>
<td>Westminster Bridge, London</td>
<td>0.82</td>
<td>Measured by Labelye as 0.13m (130mm)</td>
</tr>
<tr>
<td>Kildwick Bridge, Yorkshire</td>
<td>0.52 – 0.68</td>
<td>Head loss measured by Yorkshire Rivers Authority as 0.5 – 0.6m in flood</td>
</tr>
</tbody>
</table>

6.11 Summary

The organisation of afflux methods described in this chapter may be summarised by author or method in Table 6.3.

Table 6.3: Methods for estimating afflux

<table>
<thead>
<tr>
<th>Class</th>
<th>Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier bridges</td>
<td>D’Aubuisson (1840) Nagler (1917) Yarnell (1934)</td>
</tr>
<tr>
<td>High flow Methods</td>
<td>Sluice gate flow Orifice flow Weir flow</td>
</tr>
<tr>
<td>Culvert methods</td>
<td>HEC-RAS ISIS MIKE 11</td>
</tr>
</tbody>
</table>
These afflux methods have in turn been implemented in the one dimensional river modelling programs adopted by the Environment Agency, as shown in Table 6.4.

Table 6.4: Afflux methods that appear in the BIS ‘A’ list models

<table>
<thead>
<tr>
<th>HEC-RAS</th>
<th>ISIS</th>
<th>MIKE 11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy method</td>
<td>ISIS Energy method</td>
<td>Yarnell (1934)</td>
</tr>
<tr>
<td>Momentum method</td>
<td>Extreme flow methods</td>
<td>Bradley (1978) - USBPR</td>
</tr>
<tr>
<td>Extreme flow methods</td>
<td>Culvert methods</td>
<td>Schneider et al (1977) - WSPRO</td>
</tr>
<tr>
<td>Culvert methods</td>
<td></td>
<td>Biery and Delleur (1962)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HR, Wallingford (1988)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Extreme flow methods</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Culvert methods</td>
</tr>
</tbody>
</table>

6.12 Key issues for Future Research

- Afflux formulae were developed for manual application and it is necessary for an experienced team of hydraulic experts and model developers to ‘interpret’ the algorithms to accommodate them in the computational codes.

- Benchmarking of different afflux formulae and their interpretations within model codes is not a straightforward exercise and can easily become embroiled in commercial rivalries or hampered by intellectual property right restrictions. The key issues are (a) to assess the limitations and the fitness for purpose of the various formulae for typical UK applications, and (b) to develop a widely accepted open code estimation system (which is available either for further development or embedding into commercial model codes).

- The uncertainty in afflux estimates is not well understood and needs to be quantified in terms of the different component types of uncertainty.

- Of the commonly used 1-d models, it is likely that the results of this research will most influence the development of ISIS and MIKE-11 and perhaps to some degree, HEC-RAS. The developers of HEC-RAS being part of a US Federal body are unlikely to be as responsive to UK requirements although they would no doubt be interested to learn of the outcome of any UK research. Defra and the EA will need to be pro-active in ensuring that any new methods are adopted in HEC-RAS.
Defra/Environment Agency
Flood and Coastal Defence R&D Programme

AFFLUX AT BRIDGES AND CULVERTS
Review of current knowledge and practice

Chapters 7 to 10
References
List of Figures and Tables
Appendices

R&D Technical Report W5A-061/TR1

J R Benn, P Mantz, R Lamb, J Riddell, C Nalluri

Research Contractor:
JBA Consulting – Engineers & Scientists
7 METHODS OF ACCOUNTING FOR BLOCKAGE

7.1 Causes of blockage

There are many causes of blockage, but all are a combination of a source of debris that can cause a blockage and a means of trapping that debris. Table 7.1 provides a list of the most common factors (listed in no particular order of significance) and Figure 7.1 an indication of the inter-relationship between source, management and physical features of a culvert.

Table 7.1: Common factors causing blockage of bridges and culverts

<table>
<thead>
<tr>
<th>Factors contributing to Debris Source Potential</th>
<th>Factors contributing to Blockage Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Stream slope</td>
<td>• Width/height</td>
</tr>
<tr>
<td>• Urbanisation</td>
<td>• Opening ratio</td>
</tr>
<tr>
<td>• Vegetation</td>
<td>• Shape</td>
</tr>
<tr>
<td>• Sinuosity of stream</td>
<td>• Skew</td>
</tr>
<tr>
<td>• Upstream structures</td>
<td>• Length</td>
</tr>
<tr>
<td>• Channel width</td>
<td>• Existence of piers/multiple culverts</td>
</tr>
<tr>
<td>• Trash screens</td>
<td>• Trash screens</td>
</tr>
<tr>
<td>• Access to the channel</td>
<td>• Availability of skilled operators</td>
</tr>
<tr>
<td>• Time since last flood</td>
<td></td>
</tr>
<tr>
<td>• Maintenance regime</td>
<td></td>
</tr>
<tr>
<td>• Climate change</td>
<td></td>
</tr>
</tbody>
</table>

Figure 7.1: Processes involved in blockage
7.2 Empirical methods

Two previous studies associated with the blockage of structures - by the Federal Highways Administration (FHWA) and the Environment Agency - have been reviewed as part of this Scoping Study. There is other documented work from Australia and New Zealand on debris at bridge piers that is referred to in the FHWA documentation. Figure 7.2 illustrates design examples of floating debris accumulations from the latter.

![Figure 7.2: Typical methods of representing floating debris at a bridge pier, showing a cross section elevation of assumed maximum debris accumulation (diagram shows end elevation in the direction of flow)](image)

7.2.1 FHWA Drift Model

This emanated from a US FHWA research programme on Drift accumulation. Diehl (1997) published the results of a study of large scale drift (debris) accumulation at bridges carried out by the USGS between 1992 and 1995, in co-operation with the Federal Highway Administration. The study included a review of published literature on drift, analysis of data from 2,577 reported drift accumulations, and field investigations of 144 drift accumulations that dealt with large, wooded catchments.

The purposes of the study were (1) to determine characteristics of drift and drift accumulations at bridges and (2) to develop a method for rating potential for drift accumulation at bridges as high, medium, or low, based on bridge and site characteristics.
7.2.2 Environment Agency SW Region Blockage Model

The second study dealing with blockage is the analytical “Blockage Risk Assessment” method undertaken for the Environment Agency South West Region in 1997-8. This study and a more recent update are detailed in the paper by Faulkner (2001). In summary, the study’s main elements are

- A spreadsheet based “blockage risk model” requiring various basic structural, hydrological and debris type attributes to which individual probability risks can be assigned,
- A decision tree analysis which directs the user to various actions depending on the blockage probability,
- A manual guiding the user on the application of hydraulic models to model blockage.

The study identified several major problems with identifying blockage risk. One is the lack of data and the second is that the probability distributions of the variables describing the influence of blockage are not independent, and the associated probabilities are subjective. This problem is not unique to blockage theory, and requires a consistent risk management framework aligned with the treatment of other risk issues.

Note on application of hydraulic models in the SW Region Blockage Method

The advice given in the SW Region Method (Section 7.5) on using hydraulic models (namely ISIS and HEC-RAS) for simulating blockage is misleading. For HEC-RAS it suggests using the blockage utility to ‘fill’ some of the upstream flow area. This approach however, will not change the cross-sectional area of the bridge or culvert and so can give a misleading result. Figure 7.3 illustrates this with results from a HEC-RAS model representing a 90% and 75% blockage by both a change in cross-section area of a bridge and by applying a blocked area to the upstream cross-section.

The best interim method for incorporating blockage in a 1-d model such as HEC-RAS is to change the dimensions of the opening to include the blockage as part of the physical outline of the structure.

Since the SW study, was undertaken, the ability to add floating debris to bridge piers and also sediment to the invert of a culvert has been added to HEC-RAS (version 3.1 upwards).

7.3 Drawbacks with empirical methods

While both the FHWA and SW Region models are good references and relatively simple to use, they only address blockage risk for individual structures. It is therefore difficult to apply the methods at a catchment or reach scale and in particular to integrate blockage risk with the flood risk from a particular flow probability to produce uniform ‘risk maps’.
Note blockage is the inverse of the opening ratio, M. e.g. 90% blockage = 10% opening ratio

Figure 7.3 HEC-RAS modelling of varied blockages

7.4 Computer models

The paper by Samuels (2001) describes in some detail how afflux and blockage is or can be represented in the main river modelling software packages. Blockage can only be directly modelled in the HEC-RAS program, as detailed below. Note however that blockage can be introduced manually to any computer model by simply reducing the size of the structural opening area. This size reduction can be made in a similar way for all models.

Table 7.2: Comparison of facilities for incorporating blockage in river models

<table>
<thead>
<tr>
<th>Blockage Capability/Method/Procedure</th>
<th>HEC-RAS</th>
<th>ISIS</th>
<th>MIKE11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floating debris at bridge piers (Applies a ‘cylinder’ around the pier rather than the triangle suggested by the FHWA)</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sediment in culvert invert</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manual estimation</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Transient blockage as a sluice gate</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Model Blockage of bridge/culvert opening (NB – blockage is actually only applied to the bounding channel section – see box on previous page)</td>
<td>✓</td>
<td></td>
<td>✓</td>
</tr>
</tbody>
</table>
As a general improvement to all the existing 1-D models, it would be useful to include the facility to block off parts of a bridge or culvert opening without having to change the unblocked geometry. Not only would this make it easier for the user but it will provide a clear audit trail.

Such a feature should be relatively easier to add to the software and could include the facility to specify a different roughness for the blockage and to select whether to apply the blockage throughout length of the bridge/culvert or just to the inlet or outlet. The debris could also be fixed or allowed to move with the calculated water level.

Such software enhancements should not require any particular action by the EA as it requires no research as such.

7.5 Feedback from users

The survey of users of the SW Region method (Faulkner, 2001) produced the following feedback:

- Trash screens as a major cause of blockage – i.e. they do not reduce blockage, they just make it more manageable and move the location from the barrel of the culvert or inlet to the screen.

- Poor inlet conditions (which might include for example badly aligned wing-walls, zones of ineffective flow, and piers or other ‘obstructions’) all seem to be influential factors.

- Anecdotal evidence from the survey suggests that whilst absolute size of structure is important (obviously a very large opening is less prone to blockage, and the result of blockage is less noticeable with regard to afflux), there is also a relative aspect to blockage with regard to the ratio of the capacity of the culvert to accommodate extreme flood events higher than its design flow. For example, a large open structure may appear to have negligible blockage potential, but a large hydraulic load imposed on this structure by a comparably large flood (with associated large debris etc.) may create conditions for a comparably large blockage problem.

- This issue of the flexibility of systems and/or structures to accommodate extreme flows is central to the development of Performance-Based Asset Management Systems (PAMS) and to the adoption of risk-based approaches to management of flood defence systems. This involves assessments of residual risks beyond the design (service) flow associated with the standard of protection. Thus the management of residual risk associated with a structure (defined as likelihood x consequence) where likelihood may be low but undesirable consequence is high, may need careful consideration.

- The extent to which high flows at or near the full capacity of the structure influence blockage probability is not well understood or quantified at present, but it seems an important area for further research. Intuitively, this must be a critical scenario, since at these flows, any blockage is likely to have a rapid and marked influence on the headwater afflux, and hence flood potential as a consequence. Similarly, low flows,
even with a high degree of blockage, are very much less sensitive in terms of afflux, and hence (flood) risk.

These conclusions are supported by the results of the original work for the South West study whereby a systematic sensitivity analysis of low and high flows through a theoretical ‘standard’ culvert (2.5 m x 2.0 m) was carried out in HEC-RAS. The ratio of the percentage increase in water level to percentage increase in blockage was nearly linear with a gradient of 1 for low flows, whereas in high flows, the ratio became more of a power function, where a 50% blockage created a 100% increase in headwater.

7.6 Physical models

Physical model studies of blockages are very difficult because of the need to scale correctly the buoyancy and dimensions of the blockage. Reproduction of a floating blockage as opposed to a fixed blockage is particularly problematic if varied flow rates are to be used. One way forward would be to carry out different experiments with different degrees of reduction in entrance area – say 30%, 50%, 70% and 90% and draw some conclusions. Also, the blockage could be at the invert or at the soffit. These could be simulated with a given amount of blockage area. Perhaps additional trash screens could also represent the blockage. A careful and systematic study would be needed and would need to include a sensitivity analysis.

A physical model study would aid understanding in two aspects:

a) Confirmation of whether the characteristics of blockage due to floating debris are different to that due to opening ratio – in other words are the current approaches of representing blockage as merely a smaller bridge or culvert valid?

b) The statistical derivation of afflux effect due to blockage, following analysis with a fixed number of variables, i.e. structure type, channel shape, blockage ratio, media type and extent of blockage.

Physical modelling of the effects of blockage from top down obstruction (floating debris) compared to invert up (siltation) of structures would determine the relative sensitivity of $C_d$ to form of blockage. Radial flow approach occurs in arched structures at a progressive increase in water level and attendant reduction in the free surface. Flow contraction results in a higher $C_d$ and increased susceptibility to top down blockage.

An extensive testing programme would be required and there is therefore a need to prioritise research. It is suggested that afflux would be studied by targeting certain ‘most common’ or ‘most at risk’ structures. Testing would be carried out on a tilting bed flume, constructed to allow a combination of varying structures, channel shapes and bed gradients to be installed. Blockage would be investigated by physical model testing of say three low to high risk bridge/culvert structures (ranging from box culvert to single arch to double arch), with a range of blocking media (large to small, single items and combinations).

The findings of laboratory studies need relating to the blockage ratio, $M$, shape of culvert/bridge opening, piers etc and to the water surface levels downstream of the structure (tailwater level). The laboratory scale models need to be realistic and some prototype (field) data is still necessary to verify the results. It has in fact been found from existing tests at the
University of Strathclyde that the modelling must be done at near prototype scale, since the scale effects of both debris size and Reynolds Number are critical (Riddell, 2002, personal communication).

7.7 Direct measurement of blockage

As discussed in Section 6.10, the measurement of afflux in the field is difficult and for blockage it is even more problematic. Certainly ultrasonics to measure flow and the water surface profile would not be suitable and reliance would need to be made on measurement of water level alone. In addition to make the data useful, the amount of blockage would need to be recorded. This would probably best be achieved through CCTV or timed photography.

7.8 Conclusions on the management of blockage

Blockage is a material consideration when considering the effects of a bridge or a culvert at high flow. It should not be an ‘add on’ and it is necessary to introduce the discipline of always considering blockage as an essential part of flood management planning. As discussed in Section 9, this is consistent with the risk-based framework for flood management and planning that Defra and the operating authorities are now introducing and in particular the need to manage the residual risks beyond the service design standard.

The key questions relating to blockage are:

1. What causes the blockage?
2. What is the risk of blockage?
3. How much blockage should be considered?
4. How should the blockage be represented in the calculations?
5. How can blockage be managed?

The answer to the first question is already well known in general, but needs to be considered in relation to the type of catchment that is being considered. In the UK blockage is usually caused by floating debris from natural and anthropogenic sources, which collects on the piers and abutments and at the soffit of bridges and culverts. Blockages from sediment accumulations, ice and large obstructions (everything from caravans being washed downstream to whole trees) are much rarer.

The risk of blockage is much more difficult to answer and research into the subject is severely hampered by the difficulty of obtaining useful data. The risk of a particular structure blocking (which is a key question for new structure design) is a subtly different issue to the additional flooding risk blockage may present along a whole watercourse. This latter point focuses the question to ‘what is the additional risk that blockage adds to that purely resulting from a given flow occurring?’ We feel the latter should be the target of additional research as it would help to address how blockage should be included in flood risk maps. However, it is recognised that the issue of blockage at bridges and culverts must be considered as a subset of the performance of the catchment system as a whole.

The question of how much blockage to be considered can never be precisely defined as each blockage is by its nature a unique combination of circumstances. The two extremes are the easiest to deal with. The structure and catchment upstream may be such that the possibility of blockage is so small as to be considered negligible. This could be the case of many bridges
where the opening ratios are very large and the soffit levels are above the ‘design’ flood level. Or the structure may be at a risk of complete blockage (particularly the case of culverts with inadequate screens). Both these extremes can be considered using existing methods – the fully blocked situation by changing the analysis to one of conveyance estimation or consideration of weir flow and the unblocked by using standard bridge and culvert formulae. The most common scenario will be blockages of between 0 and 100%.

The exact allowance used is left to ‘engineering judgement’ or perhaps to empirical models such as the SW Region method. Such approaches are most suitable for studies of single structures. For studies at a reach or catchment scale, the amount of blockage needs to consider both the potential of blockage at a particular structure but also whether the resulting risk is appropriate to the return period being studied. The approach to management of blockage also needs to evolve within the overall framework for managing risk with uncertain or incomplete information.

The representation of blockage can be relatively straightforward once the amount of blockage to be considered is already known. If the blockage is assumed to primarily be a constriction effect, then it is a case of simply altering the opening area of the bridge or culvert in the hydraulic calculations. Whatever tool is used (e.g. hand calculation or river model) this can usually be achieved quickly – even if ‘blockage’ is not an option on the menu. The analyst is therefore left with only having to consider which part of the structure to apply the blockage and how to adjust the loss coefficients in the hydraulic model used. There is currently little guidance on the latter and most coefficients are based on extrapolations from data obtained from different ‘fixed’ opening widths rather than those generated from temporary floating blockages. A useful area of research therefore appears to be to collect some data on how large temporary blockages change water levels and whether existing approaches used to considering blockage represent this effect with sufficient accuracy.

The management and operational aspects of blockage is therefore a key issue for the operating authorities the future approach needs to be developed within the overall framework for performance-based asset management, and the approach that is being adopted for decision-making with uncertain or incomplete information (see Section 9).
8 CURRENT PRACTICE

8.1 Background

This chapter sets out current practice in calculating afflux and blockage, both in the UK and elsewhere. It aims to identify possible gaps between the current state of scientific knowledge and practical application of methods.

Current practice has been assessed by means of commissioning a series of expert papers (see Benn and Bonner, 2001; Kirby and Guganesharajah, 2001) and consultation with key practitioners. Consultation has been by issue of a set of questionnaires and also direct interview. Data from post-course reviews of over 50 training courses on hydraulic modelling given by JBA were reviewed. A workshop was also held to elicit the views of consultants and managers concerned with afflux and blockage on current issues and needs. This chapter presents the main points to emerge from the consultation.

In determining current knowledge and practice it is important to recognise the following:

- Current normal practice may not always be best practice.
- Methods used are strongly influenced by what is available in the most widely-used software packages. However, while a broader range of methods are available for hand calculation than are available in such software, time constraints usually preclude their use.
- Practice has been found to vary between the EA Regions and Areas and individual consultants, Local Authorities and Internal Drainage Boards. In some cases this is due to historical procedures but in some cases may reflect the dominant issues in a local area.
- New methods and drawbacks with existing methods may not be widely disseminated.

8.2 UK Practice

Two versions of the questionnaire on afflux were issued – a short version aimed at engineers and managers and a longer, more detailed version aimed at modellers. All the questions in the short version were included in the long version and supplemented by others asking more detailed questions of modelling practice. A more specific questionnaire on the EA’s South West Region Culvert Blockage study was also sent to known users and specifiers of this method.

A detailed analysis of the Afflux questionnaires returned is given in Appendix 2. It is suggested from the replies that:

- 50-65% of them need to estimate/measure afflux. The most common types of structures analysed are single and multiple culverts; single span and multi-span bridges; rectangular openings; arched and skewed bridges; bridges with embankments across floodplains and outfalls with culverts.
- The main reasons for analysis are to assess flood risk management, design of new structures, estimation of flood water levels and computation of discharges.
- Measurements of water level were normally taken at some distance upstream and downstream of the structure.
The estimations of afflux are usually carried out using the energy equation (around 16%), momentum equation (around 34%), USGS (around 16%), USBPR (around 18%) and HR method (around 16%).

The majority of the consultants seem to be using software (mainly HEC-RAS and ISIS) and sometimes hand calculations assisted by charts (around 30-40%).

The majority (around 80%) exhibit poor confidence in their estimates although around 10% of them are having good confidence.

Most practitioners are concerned with the blockage effects (mainly for the design of new structures, operations and maintenance and flood risk assessment).

Most blockages are by leaves, twigs, trees and bushes, small rubbish and occasionally large rubbish and silt.

Blockage of screens was considered to be the most important scenario.

Around 30% of practitioners incorporate blockage in their designs (about 50-90% of loss of area).

The confidence in estimations of water levels due to blockage is rather poor.

Major concerns were expressed over the lack of information on new screens and their blockage, interaction between scour and afflux, field data, discharge and other coefficients, and reliable site measurements.

The Environment Agency and consultants rely heavily on standard river modelling packages which in turn only include a few options for estimating afflux. There is a lack of understanding by many model users, and those who rely on model outputs, of the basis of calculation and assessment techniques for afflux.

8.2.1 Consultation workshop

The workshop centred around the needs of afflux and blockage representation for flood risk mapping and river modelling. It was held at Posford Haskoning’s offices in Peterborough on 13 December 2001. A presentation was made by Jeremy Benn and Kandiah Guganesharajah of the project team on the findings from the expert papers and the questionnaire survey. A structured discussion was then held between the delegates comprising representatives from all the EA’s Flood Risk Mapping Framework Consultants and three of the EA’s regions. The workshop identified the following points:

**Key users**

- Development control officers in the EA, IDBs and local authorities were ranked as the potential key users of improved afflux and blockage assessment methods. Development control was also ranked as the area which would benefit most from new research.
- Flood risk mapping and river modelling for flood defence schemes were ranked as the second and third most important users.

**Lack of clear guidance on the most appropriate methods to use for afflux estimation**

- There are few text books (the exception being Hamill’s 1999 book on Bridge Hydraulics) which succinctly cover all the main methods of afflux estimation and clearly identify their limitations.
- Text books (US editions in particular) do not cover arch bridges.
• The consideration of blockage is often an afterthought rather than being addressed as part of the basic design condition formulation.

**Aflux estimation methods**

• Concerns that there is no ‘benchmark data’ and it is difficult for inexperienced modellers to get a feel for how much afflux should be expected.
• Concerns that the HR afflux at arch bridges method is based on a limited dataset.

**Computational modelling**

• There is a need to reduce uncertainty in model calibration.
• The transition between free surface and orifice/overtopping flow introduces inconsistencies.
• There is a need to improve visualisation of the representation of bridges and culverts in 1-d models
• Advice on choice of coefficients to represent blockage and where the blockage should be placed is needed.
• Little use of 2-d or 3-d models except for scour modelling.

**Flood risk mapping**

• Users were not sure how blockage should be represented on flood risk maps.
• No consistency between Agency Regions on whether and how blockage should be considered.
• The Agency’s current topographic survey specification is unclear on the key features of a bridge and culvert, and the channel side that requires capture.

**Field data instrumentation and collection**

• Lack of calibration data.
• There are few arrangements in place to collect data on bridge afflux and little expertise on the appropriate methods of doing so.

The FRMA consultants were requested at the meeting to search through current and past flood studies for any datasets which may be of use in future afflux and blockage research.

The EA and its Framework Consultants will also be a useful source for locating example sites for the investigation of afflux and blockage. It was recognised that there would be little afflux data, but modellers would know reaches where afflux/blockage issues exist, and whether background information such as survey data and flow estimates are available.

**8.2.2 Feedback from model users (via training course reviews)**

Feedback received from delegates on ISIS, HEC-RAS and MIKE-11 training courses also provides some useful pointers on the representation of bridges and blockages in these models. A summary of this feedback is given below.

• The most difficult task in modelling structures is ensuring that an appropriate representation of the constriction is included in the model. This involves careful
consideration of cross-section location (which should ideally be done before the
survey is undertaken!) and checking that the interpolation between cross-sections that
the models use in their numerical calculations reproduce the flow paths that will occur
in the real world. Poor visualisation of the interpolated river sections in models makes
this task harder.

- The most readily available information on a culvert and bridge is its opening area and
  geometry. Some models will only allow standard shapes to be represented and some
  modellers feel that this leads to an unnecessary loss of key information.

- The preference for models is for bridge/culvert units to represent all possible flow
  conditions (free-surface, orifice/pipe flow and overtopping).

- There is a balance to be achieved between having a wide variety of bridge and culvert
  units in the model and ‘forcing’ the modeller to do some thinking or hand calculation
  beforehand. Making data entry easy has clear advantages in terms of ease of use and
  minimising input errors, but equally there is a drawback that the modeller does not
  consider each structure on its own merits. DHI (the developers of MIKE-11) argued
  for many years that it was unwise to offer bridge units because of the wide variety of
  structures – preferring instead for the modeller to input a rating curve developed
  outside of the programme. Users largely agree that afflux estimation must be
  integrated into modelling and done intelligently.

- All the models give poor information on the hydraulics at a bridge or culvert. For
  instance, the constriction ratio is not a standard output variable nor is afflux. This is
  largely because the reporting formats are geared towards giving information at a single
  section rather than in a reach or across several cross-sections.

- Many modellers view the bridge/culvert units as a ‘black box’ and a significant
  minority are unaware of what generates afflux (a common view is that it is the choice
  of pier/bridge and entry/exit loss coefficients that are most critical).

- HEC-RAS is the only model that allows the use of more than one bridge modelling
  method in a single run. In ISIS and MIKE-11, separate runs are required to compare
  say USBPR with the Energy Method.

- There is general disappointment with the EA R&D W88, undertaken in 1995, on
  benchmarking of hydraulic models. The report in general terms concluded that most
  of the models tested were ‘fit for purpose’ and it was not possible to truly
  independently test each model for accuracy due to the lack of agreed datasets. This
  study is also now out of date with new releases of ISIS, MIKE-11 and HEC-RAS
  having been released in the last few years. The Agency currently has an R&D project
  (W5-105) due for completion in September 2003 to update this benchmarking study. It
  is hoped that the results of this will be more measured.

8.3 Europe

Little new research was uncovered from literature searches. DHI make use of the common
methods described above in their MIKE11 program (the bridge representation in the software
has recently been revised):
• Biery and Delleur method (arch bridges)
• HR method (arch bridges)
• Nagler (piers)
• Yarnell (piers)
• D’Aubuisson (piers)
• FHWA WSPRO (a modified version of the FHWA’s WSPRO computer program)
• USBPR
• pressure flow
• road overflow
• submerged bridges (using the momentum equation)

Other European research includes Skogerboe and Hyatt (Analysis of submergence in flow measuring flumes, 1973). They proposed the use of a submerged flow equation of the form:

\[ Q = \frac{C_b g^{1/2} b (y_1 - y_3)^{1.5}}{-(\log S)^n} \]  

(8.1)

which was compared with the USBPR and USGS methods by Fiuzat and Skogerboe (Comparison of open channel constriction ratings, 1984).

A problem with the method is the limited amount of data available for the coefficients \( C_2 \), \( S \) and \( n \) for different conditions.

A formula developed by D’Aubuisson (1840), which has been widely applied and is still in use, can be written as follows

\[ Q = C_b b_2 Y_3 \sqrt{2gh_u + v_1^2} \]  

(8.2)

where
- \( C_b \) - empirical discharge coefficient
- \( b_2 \) - total width of unobstructed channel at constriction
- \( Y_3 \) - depth of channel downstream of constriction
- \( g \) - gravity
- \( h_u \) - difference in head upstream and downstream of constriction
- \( v_1 \) - velocity of flow upstream of constriction

Refer to Figure 3.1 for details of symbols.

### 8.4 Commonwealth of Independent States (CIS) – former USSR

An exception to the widespread use of the USGS or the USPBR methods or those outlined in references such as Chow and Henderson is in the former Soviet Republics. Here research has historically been less well publicised in the Western research community, partly owing to language barriers but also due to political barriers. In these CIS countries, designs use as their basis sets of documents called Standards and Norms, known as SNIPS (Construction Standards and Rules, of the former USSR). These are in principal similar to British Standards...
in that they set out standards to be adopted for design. However, particularly in the field of hydraulics, they go beyond the sort of detail to be found in British Standards to define the hydraulic methods to be used. In this respect they are more akin to the US Hydraulic Engineering Circulars.

The scientific basis of the Standards and Norms is research (and practical experience) undertaken in the former USSR. In many CIS countries the Standards and Norms have not been updated since independence and so they date back to the 1980s. The relevant SNIP for hydraulic design is SNIP 2.06.01-86, Hydraulic Structures – Basic Clauses/Provisions for Design.

Detailed information gained on afflux estimation was obtained from an extract from Hydraulics Manual (V. A. Balshakhov, 1984). No translation from Russian was available. This makes reference to work by Y. B. Homjak (full reference not known). The emphasis in design is the calculation of a safe bridge width to minimise afflux and limit velocities to less than the critical velocity for bed movement to prevent scour, rather than the estimation of afflux at existing structures. Cases are considered for free and drowned flow.

The basic equation used is:

\[ Q = mb_m \sqrt[2]{2gH_0^{1.5} \sigma_3} \]  

with \( m \) dependent on the type of bridge and other coefficients determined from work by Homjak.

One of the exceptions to the lack of uptake of Russian research is the work of Izbash. His work was concentrated in the fields of rockfill and river closures for dams. However, Izbash and Khaldre (Hydraulics of River Channel Closures, 1970) developed a method similar to the USGS method for the hydraulic analysis of channel contraction but which also analyses the energy recovery and additional friction loses downstream of a constriction.

Downstream energy recovery is assumed to be:

\[ \left( \frac{A}{a} \right)^2 V_3 \]  

or:

\[ \left( 1 - \frac{a}{A} \right)^2 V_3 \]

The method determines:

\[ \Delta H = (H_f)_u + (H_f)_c + H_f + H_{sd} \]

where

\( \Delta H \) - additional head loss due to bridge

\( (H_f)_u \) - increase in bed friction upstream

\( (H_f)_c \) - increase in bed friction in constriction
(Hf)d - increase in bed friction downstream
Hsd - kinetic energy lost due to sudden expansion.

Each of these is determined in terms of:

\[
\frac{V^2}{2g}
\]

so that an equation of the form:

\[
Q = C_d A \left(2gK\Delta H\right)^{1/2}
\]

(8.8)

can be used.

To calculate friction losses, the length of the expanding flow zone between Sections 3 and 4 (Figure 2.1) is determined and then the friction loss calculated using a geometric mean of the friction gradients at Sections 3 and 4. This method uses a relatively simple calculation procedure to determine C_d however it would be valid to use a C_d value derived from the USGS method.

The method has the advantage in mathematical modelling of a smooth transition from free surface to submerged conditions because C_d depends only upon the contraction ratio for the contraction.

8.5 North America

There is plenty of useful advice in the USA and Canada for the hydraulic design of culverts and bridges, and much of it is readily available via the Internet. As well as the USACE manuals, many State Departments of Transport produce design manuals for bridges and culverts. The guide to bridge hydraulics of Niell (1973) is well regarded in Canada. Much of the advice is aimed at new structures and producing designs that will result in little of no afflux or risk of blockage.

If the structure encroaches upon a designated 100-year ‘base floodplain’, there is a National Code of Federal Regulations (CFR) that governs the hydraulic design. In these cases the practice is to design bridges so that backwater (afflux) does not exceed 0.3m (1 ft) at a 100-year discharge. A ‘freeboard’ allowance between the design water level and the underside of the bridge deck is also required to allow the passage of floating debris. The Primary Design Reference is the FHWA Design Series No.1 – Hydraulics of Bridge Waterways, which uses the Bradley (1978) or USBPR method.

For other bridge scenarios the hydraulic design criteria is usually to pass the 2% probability (50-year flood), with adequate freeboard to the lowest structural member to pass debris and/or the 1% probability (100-year) flood. Sometimes the lowest structural member and design waterway area are controlled by the effects of bedload and debris rather than the 100-year water surface (e.g. rivers with the risk of floating trees or in desert where there is significant risk of ‘bulking’ of flows at alluvial fans).
Culverts are approached differently to bridges and are usually designed to utilise the available head or freeboard for the 100-year flow, providing headwater does not rise above an elevation that would cause objectionable backwater depths or outlet velocities. Debris must also be able to pass through the culvert and a headwater pool cannot be tolerated. Lately, fish passage has become a major consideration, and in these situations, culverts are designed more like bridges to emulate natural stream flow. A primary national reference is the FHWA Design Series No.5 – Hydraulic Design for Highway Culverts for selecting a culvert size for a given set of conditions.

For debris, field review is considered crucial. This includes consideration of the upstream watershed and all drainage structures along the watercourse, with a thorough maintenance record search and review of flood history. The resulting rise in water surface due to ‘bulking’ and debris is left to engineering judgement and input into the hydraulic model accordingly.

8.6 Australasia

Australia has, in the past, used the USBPR method as the main method for afflux estimation. Work has also been carried out by G R McKay in Australia (Bridges and culverts reduced in size and cost by use of critical flow transitions, Cottman & McKay, 1990) to develop an innovative concept of minimum energy loss bridges and culverts to reduce afflux and increase discharge for a given opening. The concept is that wide shallow sub-critical flows are converged by an inlet transition and accelerated through the structure into high velocity critical flow and then decelerated downstream. The theory is that at critical depth the flow is hydraulically smooth with virtually no head loss other than friction.

There are practical issues to be considered such as siltation, debris and scour and there are certain conditions where minimum energy loss bridges and culverts are unlikely to be suitable. They are particularly suited to bridging or culverting relatively wide shallow sub-critical flows from ephemeral streams and overland flow. They have been used successfully in Australia, but conditions in the UK often do not match the preferred conditions outlined above.

8.7 Indian Sub-Continent

There has been a long history of research and field experimentation in hydraulics in the Indian sub-continent, with an emphasis on irrigation and drainage canals and structures. Designs are based on a mixture of research, empirical results and practical knowledge.

Frequently a detailed estimation of afflux will be inappropriate and various rules of thumb have been developed instead. For example, in the Practical Civil Engineers’ Handbook (edited by Khanna, P.N., 1982) for initial calculations, the afflux at bridges is given as “60 cm in alluvial and deltaic regions, 90 to 120 cm in trough regions, and higher in steep reaches of rivers with boulders and rocky beds”.

Flood embankments at the upstream end of bridges are increased in height to take account of afflux. Approach embankments to bridges are also raised to take account of afflux. The rise in water level is approximated to:
where $V$ is the average velocity of the river.

The empirical Molesworth formula (1871) is sometimes used:

$$H_{1}^* = \frac{V^2}{2g}$$ \hspace{1cm} (8.9)

where $V_a$ is the velocity in unobstructed river (m/s), $A$ is the natural (unobstructed) waterway area (m$^2$), and $a$ is the contracted area (m$^2$).

Other formulae used are:

$$H_{1}^* = \left[ \frac{V_a^2}{17.9} + 0.015 \left( \frac{A}{a} \right)^2 - 1 \right]$$ \hspace{1cm} (8.10)

where $V_a$ is the velocity in unobstructed river (m/s), $A$ is the natural (unobstructed) waterway area (m$^2$), and $a$ is the contracted area (m$^2$).

In Irrigation Engineering and Hydraulic Structures (Santor Kumash Garg, 1998), it is proposed that flow through bridges be estimated either using a broad crested weir formula or an orifice equation. The former being used where the afflux is greater than 25% of the downstream water depth and the latter where the afflux is less than 25% of the downstream water depth.

From the above reference for broad crested weirs, the discharge $Q$ is given by:

$$Q = 1.71 C_d L \left( Y + \frac{V_a^2}{2g} \right)^{3/2}$$ \hspace{1cm} (8.12)

where $V_a$ is the approach velocity, $Y$ is the upstream water depth, and $C_d$ the coefficient of discharge.

For orifice flow, the discharge $Q$ is given by:

$$Q = C_o \sqrt{2g \cdot L \cdot Y_d \left( h + \left( 1 + e \right) \frac{V_a^2}{2g} \right)^{1/2}}$$ \hspace{1cm} (8.13)

Values of $e$ and $C_o$ are provided in graphical form for different constriction ratios.
8.8 Key issues for future research

The results from the questionnaire, workshop and other consultations have enabled the following potential elements of a research programme to be identified:

- Research is needed to provide clear guidance on the most appropriate choice of afflux estimation methods, and to ensure transparent application of appropriate methods.

- Much of the standard reference material refers to large bridges over wide floodplains. For the UK, reference material needs to include single span arches in an urban environment.

- Field/prototype data is difficult to collect and clear guidance is needed for pre-planning data collection.

- It is worth investigating further the cost, and value to future methodological research, of obtaining data held by organisations that have undertaken physical modelling work, as much of the work will relate to non-standard structures.

There is a question of why, despite a large number of research projects using physical models over the last 20-30 years, there has been little change to the basic methods of afflux representation. Why have these studies not been more useful, or if they have been why have the findings not been taken up? The answer appears to lie partly in the fact that the methods are not easy to use and also in the fact that they stop short of providing what the end-user wants – namely obvious application to common known problems.

- Future research and development must therefore be designed with the end user in mind.
9 AFFLUX ESTIMATION AND DECISION MAKING IN A RISK–BASED FRAMEWORK

9.1 Risk and performance concepts

Defra and the Environment Agency manage the risk of flooding, and now recognise the value of considering the performance of systems of defences within risk-based methods of planning and design. Key concepts are performance (achieving a desired outcome) and risk (the chances and consequences of failing to do so). Through the Defra/Agency R&D programme, a consistent framework for the planning, design and operation of fluvial systems using concepts of risk, performance and uncertainty has been established.

Improvements in tools and techniques for assessing the hydraulic performance of structures and reaches need to link into this general framework. This is outlined in the recent R&D Technical Report FD2302/TR1 ‘Risk, Performance and Uncertainty in Flood and Coastal Defence’ (HR Wallingford, 2002) which has provided a review of concepts and methods.

Current thinking is leading towards a tiered approach to risk analysis in flood and coastal defence. This tiered approach has three levels of analysis with linked use of data:

- High level methods, based on nationally available data sets, and providing a national assessment of flood risk.
- Intermediate methods, making use of modelled and measured data and pitched at the ‘strategy study’ level for catchment and shoreline management planning.
- Detailed methods, using simulation of particular defence systems at the reach level for a wide range of scenarios, and pitched at identifying the optimum management interventions (either maintenance or improvement works) to achieve target reductions in flood risk.

Although the precise realisation of the tiered approach is a matter of on-going development, it will be delivered through tools such as the Risk Assessment of Flood and Coastal Defence Systems for Strategic Planning (RASP) and Performance Based Asset Management Systems (PAMS) methodologies. The R&D programme outlined in following chapters of this report proposes a number of outputs that will be available for use detailed analysis or representation of afflux within the tiered approach. A possible mapping of afflux R&D outputs onto the tiered approach for risk analysis is suggested in Table 9.1.

<table>
<thead>
<tr>
<th>Level</th>
<th>Afflux methodology</th>
</tr>
</thead>
<tbody>
<tr>
<td>High (Tier 1)</td>
<td>Not considered</td>
</tr>
<tr>
<td>Intermediate (Tier 2)</td>
<td>‘Afflux Adviser’ for critical issues</td>
</tr>
<tr>
<td>Detailed (Tier 3)</td>
<td>‘Afflux Estimator’</td>
</tr>
</tbody>
</table>
Mapping R&D outputs for structure and reach performance into the tiered approach will help to ensure suitable integration of the different models and analysis methods to be used for risk analysis of flood defence systems.

An important feature of the risk-based approach to planning and design is that it considers chances and consequences of failure over a range of possible scenarios. One useful tool to encapsulate this type of analysis into decision-making tools is the probability distribution of failure occurring for a given load, where loading could be, for example, flow rate or water level. This ‘fragility curve’ can provide a description of defence responses over a range of conditions. The use of fragility curves for representing component response with a flood management system will enable an appropriate assessment to be made of system response and the effects of different management interventions, including the effects of afflux and blockage.

9.2 Uncertainty

Uncertainty, which can be thought of very generally as characterising the dispersion of values that could reasonably be attributed to a measured or predicted quantity, can be broken down into three main component types for analysing flood defence systems, namely natural variability, knowledge uncertainty and process uncertainty.

Natural variability underlies most flood risk analysis, and contributes to the uncertainty about afflux and blockage, not least through randomness in the frequency and magnitude of flood flows. Although flood frequency is plainly fundamental to flood risk analysis, it can however be separated out to some extent from the hydraulic performance of structures per se.

Knowledge uncertainty about afflux includes uncertainty about process models, resulting in the existence of many different methods for estimation, and uncertainty about data, especially the lack of ‘benchmark’ information on measured afflux. This contrasts for instance with conveyance estimation where at least there is more general agreement on formulae.

Process uncertainty associated with the different afflux estimation methods is not necessarily as significant as it first appears. There are clearly differences in afflux estimates depending on the choice of a particular set of equations and the parameters within them – but these generally do not lead to variations of more than 10-150mm for in-bank flow. Where process uncertainty is most marked is when flow reaches bank-full, or water levels reach or exceed the bridge/culvert soffit. In these situations current practice is almost universally to use an orifice or weir representation. Whether this is a valid approach to assessing the effects of structures on water levels at high flows is a key issue, and one that requires further investigation.

This study has shown that much of the uncertainty about afflux associated with data can be attributed to the location of the cross-sections used in calculations, particularly where river models are used. This can be addressed by providing clearer guidance and training, and ensuring that existing guidance such as the EA’s National Survey Specification and Flood Risk Mapping Guidelines are updated accordingly.

Bridges and culverts come in a multitude of sizes, shapes and interact with the river flow in numerous ways. It will never be possible to derive universal approaches that will fit all these situations exactly. A reasonable aim should be to develop procedures that will adequately
address the most common structures/scenarios but will also clearly identify the ‘special’ cases which require the use of specialist approaches such as physical modelling or 2-D or 3-D computer modelling.

The uncertainties identified above are reflected largely in the confusion generated by having several different methods for afflux estimation and the lack of ‘benchmark’ information on measured afflux. The research programme developed within this scoping study has been designed to advance on this position. Key recommendations to address this shortcoming are:

- Clearly setting out the available methods with the limit of their validity and known ‘pros’ and ‘cons.’
- Providing a tool that allows rapid comparison of the valid methods and provides a clear visualisation of the process.
- Providing reference examples of structures stating the afflux and the features of the structure that have most influence on the afflux

This study has illustrated that all the above factors are material considerations, but that in a significant number of cases it is uncertainty on what method to use, and how to represent a structure, and uncertainty on the amount of blockage to allow which is of most concern to users.

Process uncertainty is greatest in terms of how structures are modelled once water levels approach the opening soffit, overtop the banks or the bridge/culvert roadway/embankment.

The estimation of uncertainty is an active area of research in the flood defence sector, particularly so in flood forecasting and estimation, see for example, Aronica et al (1998), Romanowiez & Beven (1998) and Lamb et al. (2002). Specifically in the context of flood level estimation, there have been investigations of some of these sources of uncertainty either individually or in combination, see for example, Burnham & Davis (1990), Defalque et al. (1993), Samucis (1990, 1995) and Yang and Kung (1994).

A full framework for the combination of all sources of uncertainty, including flood discharge and blockage estimation, may emerge through the outputs of R&D in the long term. In the medium term, it is important that practical means, such as the fragility curve, are developed to represent uncertainty within decision-making tools. For example, the fragility curves can be accompanied by confidence intervals and these ranges of uncertainty may be combined, although there remains a choice of methods for doing this. A desirable outcome of uncertainty estimation in this context would be to give an expected range of uncertainty (e.g. as measured by the standard deviation of the process) together with an estimate of the upper bound.

9.3 Tolerance of uncertainty, its consequence, management and risk

The following paragraphs draw directly upon comments made in the R&D Scoping Study on River Channel Conveyance (Evans et al., 2001), which are applicable also to afflux and blockage. Any future afflux estimation system will be used in conjunction with the Conveyance Estimation System, which is now under development.
A separate issue is the tolerability of uncertainty in the end use of afflux or blockage estimations. It is possible that for some users, a greater degree of uncertainty is permissible enabling simpler methods to be used, less field data to be gathered or less intensive calibration of the parameters. Hence, in parallel with the framework for the estimation of uncertainty, there should be an assessment of the resilience of the usages of afflux estimation to uncertainties in the results. This will facilitate the identification of the return on investment of effort and resources in reducing uncertainty in conveyance estimation in different contexts, particularly when considering overall system response over a range of flow conditions.

The effects of uncertainty in the estimation of afflux differ with the various business sectors of the EA. The potential consequences of uncertainty and typical current methods of mitigation for the uncertainty are listed in Table 9.2 below.

Table 9.2: Potential consequences of uncertainty in afflux

<table>
<thead>
<tr>
<th>Flood management function</th>
<th>Consequence of uncertainty</th>
<th>Typical current mitigation strategy</th>
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<tbody>
<tr>
<td>Flood defence design</td>
<td>Under capacity of defences leading to potential failure below the design standard or over-capacity (e.g. over-design) potentially leading to morphological problems or lower economic return than planned. Over-design of capacity of defences leading to lack of implementation of schemes due to excessive cost.</td>
<td>Undertake sensitivity analyses and add a freeboard to allow for under capacity or blockage</td>
</tr>
<tr>
<td>Maintenance</td>
<td>Inadequate or excessive maintenance activities at bridges and culverts, possibly unnecessary disruption to aquatic and riparian habitats or insufficient capacity of the watercourse leading to increased flood risk.</td>
<td>Maintain to a defined programme</td>
</tr>
<tr>
<td>Strategic Planning</td>
<td>Flood mapping misrepresenting risk and providing an inadequate tool for planning and information on possible flood risk. Inadequate (or over necessary) development control, loss of professional and public confidence in the EA’s technical abilities. Broad-scale CFMP modelling results may not always include afflux, which may conflict with more detailed analysis.</td>
<td>Give suitable “health warnings” on use of IFM and CFMP modelling outputs.</td>
</tr>
<tr>
<td>New Works/Development Control</td>
<td>While the EA is only responsible for the works it constructs, and the issue of a consent is not a warranty of design, inadequate culvert and bridge design leads to the EA requiring to take legal action or mitigation in terms of providing flood warning.</td>
<td>Clearer design guidance, and ‘simple’ rubrics to allow the quick review of designs.</td>
</tr>
</tbody>
</table>

The potential economic benefits of the use of improved methods will come from the potential to alter the mitigation strategies as the degree of uncertainty is reduced. The sensitivity of project decisions to uncertainty in afflux estimation also needs to be established. Strategic
decisions made early in the project life cycle can have far reaching consequences and it is at this early stage that uncertainties in information and data are greatest.

There is a close relationship between uncertainty and flood risk management in that the greater the uncertainty, the greater may be the probability of the project or maintenance activity not achieving its objective. This is linked to the confidence of the performance of the scheme or process to meet its intended objectives. Thus, optimisation of performance and the confidence with which performance can be delivered is linked inexorably with understanding and controlling uncertainty.
10 DISCUSSION OF MAIN FINDINGS

10.1 Introduction

This chapter discusses the findings of the scoping study and the key conclusions and recommendations that have been set out in the Executive Summary. The main points can be summarised very briefly as follows.

- There are several relevant methods for estimating afflux, and this leads to a large uncertainty with regard to selecting appropriate methods,
- There is a paucity of data on blockage and some for bridges at bank-full or higher,
- Blockage of bridges and culverts lead to large afflux at opening ratios less than 50%,
- There is a need to consider the frequency of blockage at catchment scale,
- There is a user need for both a rapid and a detailed afflux estimation method.

These key issues are discussed in greater detail below.

10.2 Variability of afflux estimates for an existing structure

The variability of afflux estimates is illustrated in Figure 10.1, which details computational results using the USBPR, HR Arch Bridge, Yarnell, WSPRO, Energy and Momentum methods. The data are from a multiple arched bridge on the River Leam in Leamington Spa. The USBPR and HR Arch methods were applied using ISIS and the other methods using HEC-RAS.

What is interesting is that the differences in the afflux estimates are small (no more than ±300mm of the average). It is noted that the USBPR method is not strictly suitable for this bridge shape and channel geometry (and it ignores pressure flow beneath the bridge). If the USBPR method is excluded then the variation in afflux estimates is less than ±150mm. Water levels reach the soffit level of this bridge at around 130m$^3$/s

The differences in water levels/afflux once the bridge is surcharged or overtopped are largely due to the differences between the schematisation of an ISIS and HEC-RAS model. The chart in Figure 10.1b shows upstream water level for the same set of model results. Once the bridge overtops the methods revert to weir or orifice equations. Note that all water levels are within ±300mm of the average value.

Although this example is far from comprehensive it does confirm feedback from many modellers that afflux estimates tend to vary more as a result of model schematisation (i.e. spacing of cross-sections) than choice of method, the exception being if a totally inappropriate method is used (for instance using HR Arch Bridge on a rectangular opening).

Work done by Atabay and Knight (2001) using laboratory data also confirmed that for in-bank flow the variation between the afflux estimates from standard formulae compared to the flume data was small.
10.3 Existing guidance

The existing method recommended by the UK Highways Agency (HA, 1995) for calculating Bridge Afflux is that proposed by Bradley (1978). This method has become known as the USBPR method, and was shown in Chapter 8 as being the standard for both USA and Australasia. The principal advantage for the method is that it can be solved by hand with the aid of nomographs; it may also account for such variables as bridge opening ratio, the number of piers, eccentricity and skewness. The major disadvantages are that the method is laboratory derived and applies to US bridges that are not necessarily representative of the common UK bridge types. There is now an opportunity to provide the user with a suite of methods that are more fit for UK purposes.
10.4 Paucity of data

The currently available afflux data for arch bridges are summarised in Figure 10.2.

Figure 10.2: Plot of HR (1988) field data for (a) single and (b) multiple arch bridges

*Note: J1 = blockage ratio = inverse of the opening ratio. Hence J1 = 0.6 indicates 60% of the cross sectional area of the channel is blocked by the structure.*

The data were collected by HR Wallingford (1988) following a data request from over 50 Regional Water Authorities. As described in Chapter 6, the HR dimensionless afflux (afflux divided by the upstream water level) is simply related to a function of a bridge opening ratio (J1) and an upstream Froude number (Fr). A total of 66 sets of bridge data were collected by HR Wallingford, and these consisted of 15 single arched bridges and 51 multiple arched
bridges. Figure 10.2 illustrates the unreliability of the data used and the lack of information for the very highest flows/opening ratios. The poor fit of the prototype data to the ‘design curves’ also shows clearly the disadvantages of using data collected from post-event flood surveys for afflux studies – i.e. data that often measures headloss not afflux and which has a substantial measurement error.

10.5 Influence of blockage on afflux

The differing flows through a hypothetical bridge were illustrated in Figure 5.1, which is also relevant at this point. For ‘low’ flows below the soffit (flow types 1 to 4 in Figure 5.1), the critical depth will vary with stage and will be higher in the contracted flow reach, as shown.

Rating curves can be modelled for the entire range of flows, as shown in Figure 10.3 (which is derived from a 1D model for a hypothetical channel with infinite sides). The influence of the bridge opening ratio (M = area of bridge opening/area of channel) can also be modelled. It is then seen how the upstream levels (and thus the afflux) increase significantly when M < 50%. It follows that blockage giving more than 50% reduction in the bridge opening area (e.g. curves for M=25% and M=10%) will cause a significant increase in afflux.

10.6 Blockage frequency

Past studies on blockage (for example, Faulkner, 1998) have looked at the cause in terms of flood variables, structural geometry and debris type. For Section 105 Flood Mapping, an estimate is required for the number of culverts or bridges that are, on average, blocked (or partially blocked) in a catchment for a given flood frequency.

Figure 10.3: Bridge Rating curves for various opening ratios (M)
Statistical methods of extreme value analyses are used to estimate flood frequencies. It is probable that the number of blockages is dependent upon the flood frequency i.e. the more extreme the flood, the higher the number of structures blocked. It is therefore relevant to estimate the blockage frequency in terms of the number of bridges and culverts blocked against return period. Research in this area is also less demanding on data. What is required is the frequency of regular debris clearance, a flow or rainfall record and information on which culverts/bridges require the removal of debris after a flood so that some broad correlation can be obtained. This information should be readily available from existing Agency and Local Authority databases.

10.7 Afflux estimation

It follows from the previous chapters that there is a user need for clear guidance on afflux determination, but there are considerable existing problems. Users have been classified in Chapter 4 to groups who require a rapid assessment of afflux, and groups who require a detailed and best available method for the design assessment and modelling of afflux. The former groups are the Manager/Asset Maintainer and the Flood Defence Professional, and the latter groups are the Designer and the River Modeller.

This scoping study has indicated that there is confusion on the subject of afflux and the best methods to use, but that in practice, this may not be as significant as is thought. Representation uncertainty (on what method to use and how to represent the structure) and the allowance made for blockages are the more important issues, with process uncertainty regarding the behaviour of structures at very high levels of blockage or out-of-bank flow more important than the selection for example of USBPR over HR afflux at arch bridges.

10.8 Summary of key findings

Drawing on the preceding discussion, and conclusions reached throughout this report, the key findings of the scoping study can be summarised as follows.

- There is confusion as to the definition of afflux and how it differs from headloss. For any flow, afflux is defined as the maximum difference in water level, at a location upstream of the structure, if the structure were removed.

- There is a general lack of confidence of users when estimating afflux. Users are unsure as to which is the ‘best formula’ to use in particular situations and many have no ‘feel’ for how much afflux to expect at high flows.

- Existing guidance is poor, either because it involves using several information sources, or because it is overly complex or too design-orientated.

- The most typical structures analysed are existing arched bridges and arched culverts. The most critical locations are considered to be in urban areas or reaches with formal flood defences, particularly where the structures may be subject to blockage. The flow conditions of most interest are bank-full or when structures are overtopped.

- The most typical tools available for estimating afflux are hand calculation or a 1-D river model such as MIKE-11, ISIS or HEC-RAS.
• Some of the most important users of afflux information are development control officers within the Environment Agency, Internal Drainage Boards and Local Authorities. Few of these users have access to river models and are unlikely to be able to make use of them even if they were available. Developers (and their consultants) are also important users of afflux data and estimation tools.

• The implementation of existing afflux formulae in river modelling software is poor. It is generally not possible to readily compare different formula and it is not made clear to the user the relative importance of the various input variables as regards their affect on afflux. The significance of the opening ratio, angle of approach and tailwater depth in particular are not highlighted. The significance of cross-section spacing on the afflux calculations is not readily apparent to the user.

• The available datasets on afflux are largely from laboratory studies and are poorly documented. There is little awareness or agreement of how afflux should be measured in the field – probably because it is rarely done. The currently available data is predominantly for bridges and is extremely variable in quality. Field data is required for bridges and culverts where overtopping occurs and for structures blocked with floating debris in order to confirm the adequacy of existing estimation methods. To obtain better field data will require a specifically targeted effort. All datasets could be improved by being clearly linked to blockage ratio and tailwater depth.

• Blockage is considered an important issue, particularly with regard to trash screens and culverts, and this adds to the uncertainty of afflux calculation. There is currently no consistency on when or where blockage should be a consideration in an analysis. There is no guidance on how blockage should be best addressed in flood risk mapping.

• There is a reasonable degree of confidence among users and experts in the SW Region ‘Blockage Risk’ model, which was examined under this study.

• A ‘typical’ UK estimator of afflux/blockage has a minimal background in hydraulics and is unlikely to have used hand methods for afflux estimation.

• There are limitations to using physical/laboratory models to estimate afflux. These limitations are more pronounced for blockage. Physical models will still require prototype (field) data for validation.

There is an opportunity for Defra and the EA to establish best practice in the consideration of the effects of bridges and culverts at high flows relatively quickly and at low cost. A programme of Targeted Research over an 18-month time fame was identified during the scoping study. This research would be highly cost beneficial to flood defence operating authorities. A further programme of Strategic Research over a three to four year time scale is proposed to address inadequacies in understanding and hydraulic theory. This research is also cost beneficial and would be suitable for collaborative programmes with academia.

Until the research is carried out, Appendix 4 of this scoping study provides details of best interim guidance for the estimation of afflux and blockage.
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In addition, the Authors are grateful to the enthusiasm and assistance of Dr Mervyn Bramley, Andrew Pepper, Tim Palmer and Dr Tilak Peiris for their comments at all stages of this project.

ENDPIECE

‘Engineers are too prone to select empirical formulas and coefficients from handbooks and apply them to entirely irrelevant cases, never inquiring as to the natural limitations on the applicability which intelligent use would place on them. Intelligent extension of experimental formulas and coefficients to practical problems is the highest type of engineering, but the blind application of formulas smacks of student days.’

Quotation by Nagler reproduced in Hamill (1999).
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APPENDIX A: SUMMARY OF THE MAIN EXISTING METHODS AVAILABLE FOR ESTIMATING AFFLUX

There exist several theoretical and empirical methods (see Kirby and Guganesharajah, 2001) to estimate afflux. The theoretical approaches depend on the principles of energy and momentum conservation. In contrast, the empirical ones are formulated by laboratory scale physical models supplemented with field data. The methods are summarised below.

A.1 Method based on conservation of energy (as in HEC-RAS, 1995)

In general the methods proposed by this approach need appropriate energy loss (at entry and exit) coefficients (see Table A.1).

Table A.1: Entry and exit loss coefficients

<table>
<thead>
<tr>
<th>Source</th>
<th>Type of transition</th>
<th>Entry loss coefficients</th>
<th>Exit loss coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open-Channel Hydraulics</td>
<td>Warped type</td>
<td>0.10</td>
<td>0.20</td>
</tr>
<tr>
<td>(Chow, 1982)</td>
<td>Cylinder-quadrant type</td>
<td>0.15</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Simplified straight line type</td>
<td>0.20</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>Straight line type</td>
<td>0.30</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Square ended type</td>
<td>0.30+</td>
<td>0.75</td>
</tr>
<tr>
<td>HEC-RAS manual</td>
<td>Gradual transitions</td>
<td>0.10</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>Typical bridge sections</td>
<td>0.30</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Abrupt transitions</td>
<td>0.60</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Figure A.1: Elevation view of channel constriction (energy analysis)

Referring to Figure A.1.1 Bernoulli’s equation between sections 1, 2 and 3 with appropriate losses gives
\[ d_1 + \alpha_1 V_1^2 / 2g = d_2 + \alpha_2 V_2^2 / 2g + \Delta h_{1-2} \quad (A1) \]

between sections 1 and 2, and
\[ d_2 + \alpha_2 V_2^2 / 2g = d_3 + \alpha_3 V_3^2 / 2g + \Delta h_{2-3} \quad (A2) \]

between sections 2 and 3, where \( \Delta h_{1-2} \) and \( \Delta h_{2-3} \) are entry and exit losses respectively.

Manipulating equations 1 and 2 we can get the difference in water levels between 1 and 3.

Here friction losses are usually ignored although where they are significant they may be included using an equation such as Manning’s. In this case we may have to use two more sections within (or just upstream and just downstream) the constriction. The loss coefficients depend on the types of entry and exit, and Table A.1 suggests possible values. Also shown in the table are the values suggested by the HEC-RAS (1995) manual.

**A.2 Method based on conservation of momentum (as in HEC-RAS, 1995)**

The momentum equation accounts for momentum, pressure forces, gravity, drag and friction.

![Figure A.2: Elevation view of channel constriction (momentum analysis)](image)

Referring to Figure A.1.2 the momentum equations can be written as:
\[
A_i \dot{y}_i + \beta_i \frac{Q_i^2}{g A_i} = \sum A_{2i} \dot{y}_{2i} + \sum \beta_{2i} \frac{Q_{2i}^2}{g A_{2i}} + \sum A_{w} \dot{y}_{w} + \sum D_i + \sum F_{12i} - \sum W_{12i} \quad (A3)
\]

between sections 1 and 2,
and:

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\[
\sum [A_{2i} \dot{Y}_{2i} + \beta_{2i} \frac{Q_{2i}^2}{gA_{2i}}] = \sum [A_{3i} \dot{Y}_{3i} + \beta_{3i} \frac{Q_{3i}^2}{gA_{3i}} + F_{23i} - W_{23i}] \\
\] (A4)

between sections 2 and 3, and:

\[
\sum A_{3i} \dot{Y}_{3i} + \sum \beta_{3i} \frac{Q_{3i}^2}{gA_{3i}} + \sum A_{4i} \dot{Y}_{4i} - \sum F_{34i} + \sum W_{34i} = A_4 \dot{Y}_4 + \beta_4 \frac{Q_4^2}{gA_4} \\
\] (A5)

between sections 3 and 4.

In the equations, \( \dot{Y} \) is the centroid of area, \( F \) is the friction force, \( D \) is the drag and \( W \) is the gravity. The coefficient \( \beta \) is the momentum correction coefficient. The subscripts \( u \) and \( d \) denote upstream and downstream.

The continuity equation between sections 1, 2, 3 and 4 can also be written as

\[
Q_i = \sum Q_{2i} = \sum Q_{3i} = Q_4 \\
\] (A6)

Both the momentum and continuity equations can be solved provided the appropriate coefficients are known.

When the entry is submerged, an orifice type equation is used, and when the structure is submerged the weir flow equation is used.

**A.3 Yarnell’s method (1934)**

For subcritical flows (Class A flows) the afflux is calculated from

\[
H_1^* = KY_4 F_4^2 \{K + 5F_4^2 - 0.6\} \{\alpha + 15\alpha^4\} \\
\] (A7)

where \( K \) is the pier coefficient (function of its shape – see Table A.2), and \( \alpha \) is the channel contraction ratio = \((1 - b/B)\) or \((1 - M)\).

<table>
<thead>
<tr>
<th>Pier Shape</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Semi-circular nose and tail</td>
<td>0.90</td>
</tr>
<tr>
<td>Lens-shaped nose and tail</td>
<td>0.90</td>
</tr>
<tr>
<td>Twin-cylinder piers with connecting diaphragm</td>
<td>0.95</td>
</tr>
<tr>
<td>Twin-cylinder piers without connecting diaphragm</td>
<td>1.05</td>
</tr>
<tr>
<td>90° triangular nose and tail</td>
<td>1.05</td>
</tr>
<tr>
<td>Square nose and tail</td>
<td>1.25</td>
</tr>
</tbody>
</table>

**Table A.2: Values of Yarnell’s pier coefficients.**

With supercritical flows the afflux can be calculated by allowing for an energy loss between 1 and 3 (critical flow at section 3) as \( K_P V^2/2g \), \( K_P \) being the pier coefficient (see Henderson, 1966) of about 0.35 with square ends and 0.18 with rounded ends. Yarnell provided charts (Yarnell, 1934) for different shapes of piers in order to compute the afflux in subcritical
flows, knowing M, the contraction ratio of the opening, V and F at section 4 (the normal flow section where the bridge influence is absent). Further development of Yarnell in supercritical flows also produced design charts to compute the afflux as a function of Y and F at section 4, pier shape and the normal depth Froude number at the limiting contraction when the flow in the opening is at critical depth i.e., $F_3 = 1.0$. Other works such as by Nagler (1918) and d’Aubuisson (1840) are primarily used for discharge computations and a large amount of information is available in classical text books. All these works are restricted to rectangular or deck type bridge openings.

A.4 Biery and Delleur method (1962)

Biery and Delleur (1962) worked with the hydraulics of single span arch bridges ignoring effects of skewness, entrance rounding, and piers. Only uniform flows were considered. The result of their study is mostly provided as design charts from which the afflux can be estimated knowing the normal depth and Froude number and the opening ratio, M. They also formulated a discharge equation through a semicircular arch in a rectangular channel, where the upstream depth and the radius of the arch are known. The discharge coefficients were also established as a function of M and normal F.

A.5 Hydraulics Research method (HR, 1988)

This work to evaluate the afflux is mainly based upon laboratory tests of single and multiple arched openings (Brown, 1985 and 1989). The results were then compared with field measurements from selected bridges. Charts were produced showing the relationship between $H_1^*/Y_4$, $F_4$ or $F_N$ and the upstream ($J_1$) or downstream ($J_2$) blockage ratios.

A.6 Kindsvater et al (1953) or US Geological Survey (USGS) method

This method was developed for rectangular openings and deck type bridges and is based on an extensive laboratory study and evaluated with different field data. The field data is from the southern USA (Mississippi, Louisiana and Alabama) and has been written up in a series of exceptionally well-documented monographs (USGS, 1953). It should be noted however that the bridges studied were all major highway bridges, with rectangular openings and crossing wide and heavily-wooded floodplains.

The USGS method considers types of opening, eccentricity, skew, entrance rounding, piers, wingwalls and submergence of the opening, all under normal subcritical flows. The bridge is basically treated as a gauging device (with no outflanking). This method needs the water levels at sections 1 (normally located at a distance b upstream of the structure) and 3 (located at the minimum cross sectional area of flow) and $F_3$. Normally the information on Y and F at section 3 is difficult to obtain.

Through the energy and continuity equations the following discharge equation can be obtained:

$$Q = CA_3 \left[ 2g \left( \Delta h + \frac{\alpha_i V_i^2}{2g} - h_F \right) \right]^{1/2} \quad \text{(A8)}$$
where $C$ is the coefficient of discharge (a function of contraction, $a_3$ and $h_E$). $A_3$ is the gross cross sectional area of flow (including the part occupied by piers) at 3. $\Delta h$ is the water level difference between 1 and 3 and $h_F$ is the friction loss between them. Matthai (1967) extended these studies to different types of embankments and abutments with and without wingwalls and produced charts to determine the correction coefficients introduced to $C$. Also, he introduced correction factors to take into account the eccentricity, skew, multiple openings, and submergence.

### A.7 Schneider et al. (1977) or FHWA WSPRO Method

This method is not used much outside the USA despite being available as one of the bridge modelling options in HEC-RAS. The energy losses for each reach are represented in terms of the conveyance ($K_i$), where $i$ refers to the reach identification. The conveyance is defined by $Q = K_iS^{0.5}$, and thus the energy head loss for a reach is given by:

$$\Delta E_i = L_i \cdot S_i = Q^2L_i/K_i$$ \hspace{1cm} (A9)

For the approach (contraction) reach, then friction losses are:

$$\Delta E_{4-3} = Q^2L_{4-3}/K_4K_c$$ \hspace{1cm} (A10)

where $L_{4-3}$ is tabulated, and $K_c$ is the smaller of conveyances between $K_2$ and $K_q$ ($K_q$ is the portion of section 4 conveyance contained within the bridge).

For the constricted (bridge) section:

$$\Delta E_{3-2} = Q^2L_{3-2}/K_2$$ \hspace{1cm} (A11)

For the expansion section, friction losses are:

$$\Delta E_{2-1} = Q^2L_{2-1}/K_1K_c$$ \hspace{1cm} (A12)

And also for the expansion section, turbulent losses are given by:

$$\Delta E_{2-1} = Q^2/2gA_1^2 [2(\beta_1 - \alpha_1) - 2\beta_2(A_1/A_2) + \alpha_2(A_1/A_2)^2]$$ \hspace{1cm} (A13)

where $\beta$ is the momentum correction coefficient for a section.

### A.8 Bradley (1978) or US Bureau of Public Roads (USBPR) method

This method is also basically applicable to rectangular openings and deck type bridges, although more versatile than USGS method described earlier. The method assumes normal depth conditions to fix the afflux due to the introduction of a constriction (bridge or culvert) of opening ratio, $M$. The development is based on comprehensive laboratory data (Liu et al, 1957) and verified by field data. Bradley (1978) later elaborated the study considering skew, eccentricity, abnormal water levels, dual bridges, flows with critical depths, water level differences across the approach embankments, and submergence of the openings.
The method requires the bridge opening ratio, $M$, the normal depth velocity head at section 2, $\alpha_2 V_{N2}^2 / 2g$, and some empirical coefficients from charts. It is based on the energy and continuity equations applied between the upstream and downstream sections.

A.9 Summary

Table A.3 below summarises the methodologies available with their relative merits and demerits.
Table A.3: Summary of the methodologies available to estimate afflux

<table>
<thead>
<tr>
<th>Type</th>
<th>Theoretical or Laboratory or Field study</th>
<th>Adopted in the following Agency BIS river modelling packages</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy principles</td>
<td>Theoretical</td>
<td>HEC-RAS, ISIS, MIKE 11</td>
<td>Knowledge of loss coefficients essential (see Table A.1)</td>
</tr>
<tr>
<td>Momentum principles</td>
<td>Theoretical</td>
<td>HEC-RAS and MIKE 11</td>
<td>Momentum and drag coefficients involve uncertainty; however, some information available in literature</td>
</tr>
<tr>
<td>Yarnell</td>
<td>Empirical, laboratory study; supported by limited field data</td>
<td>HEC-RAS and MIKE 11</td>
<td>Information on pier coefficients essential; application limited to rectangular openings and deck type bridges</td>
</tr>
<tr>
<td>Biery and Delleur</td>
<td>Empirical, laboratory based; supported by limited field data</td>
<td>MIKE 11</td>
<td>Incorporates analysis of arch bridges; single arches only; limited to normal flows without eccentricity, skew etc.</td>
</tr>
<tr>
<td>Hydraulics Research (HR) method</td>
<td>Empirical, laboratory based; supported by data from selected bridge sites within UK</td>
<td>ISIS and MIKE 11</td>
<td>Arch bridges only – single and multiple arches</td>
</tr>
<tr>
<td>US Geological Survey (USGS) method</td>
<td>Laboratory work; supported by field data</td>
<td>HEC-RAS</td>
<td>Primarily aimed at discharge computations; rectangular openings with eccentricity, skew, piers, wingwalls and submergence; difficult to estimate afflux</td>
</tr>
<tr>
<td>US Bureau of Public Roads (USBPR) method</td>
<td>Theoretical basis using energy and continuity equations</td>
<td>ISIS and MIKE 11</td>
<td>Study extended to include several possibilities including dual bridges; difficult to incorporate more natural channels; more versatile method.</td>
</tr>
<tr>
<td>FHWA WSPRO Method</td>
<td>Theoretical basis using energy and continuity equations</td>
<td>HEC-RAS</td>
<td>Essentially a variant of USGS.</td>
</tr>
</tbody>
</table>
APPENDIX B: QUESTIONNAIRE SURVEYS

Fourteen long and eight short questionnaires were returned. The common answers for the 22 total are plotted as histograms below, and the overall results are summarised in words.

Question 1: On average, how often do you need to estimate or measure afflux?

- Most observers estimate afflux 3-10 times per year
- Most observers measure afflux 1-2 times per year

Question 2: What are the most common types of structures that you need to estimate/measure afflux?

- Most observers estimate/calculate afflux at 1-10 times per year for skewed bridges
- Most observers estimate/calculate afflux at 1-10 times per year for multiple culverts
**Question 3** What are the main reasons you need to estimate/measure afflux?

- Most observers estimate/calculate afflux at 1-10 times per year for the design of a new structure

**Question 4** If you have measured afflux where were the water levels recorded?

- Most observers record the water levels at some distance upstream and downstream of the structure
Question 5  Which of the following formulae/methods of estimating afflux do you use?

- Most observers use the momentum method 1-10 times per year or more for estimating afflux

Question 6  What aids do you use to estimate afflux?

- Most observers use a design chart or nomograph at 1-10 times per year for estimating afflux
Question 7  What methods of validation or calibration of afflux estimates do you use?

- Most observers use field measurements at 1-10 times per year to validate afflux estimates

Question 8  How much confidence do you have in estimates/measurements of afflux?

- Most observers think that estimates for afflux are within 25% - 50%
- Most observers think that measurements for afflux are within 10% - 25%

Question 9  What factors affect the confidence you have in your estimates of afflux?

- Errors in estimated discharges
- Inability of calibration of hydraulic models
- Type and shape of the structure differ considerably
- Methods of analysis differ considerably
- River plan form nearer to the structure
- Size of afflux
- Flood plain conditions
- Lack of high flow data for calibration
- Water levels non availability
- Approach conditions
- Scour and afflux interactions – not much information
- Visitation to the site during flood flows
- Estimation of contraction or blockage ratios
- Assumption of normal depths
- Treatment of supercritical flow is limited
- Afflux in complex channels
- Vagueness and ambiguity in the available methods and guidelines
- Choice of head loss coefficients
- Degree of blockage
- Comparison with standard formulations such as USBPR, HR to be cautious
- New MIKE 11 has very wide range of possible formulations – comparison to this with each formula should be interesting.
- Poor or insufficient site data
- Application of standard coefficients to non-standard applications
- Size of watercourse/structure
- Complexity of structure
Question 10  On average, how often do you need to estimate/measure blockage of culverts?

- Most observers need to estimate the blockage of culverts at 1-2 times per year
- Most observers never need to measure the blockage of culverts

Question 11  How important do you consider blockage of culverts to be in the following situations?

- Most observers think that blockage is very important in the design of new culverts

Question 12  What do you consider are the most common blockage material at culverts and bridges?

- Most observers think that silt or sediment at 1-10 times per year are the most common blockage material
Question 13  What do you consider are the major causes of blockage?

- Most observers think that Trash screens are the major cause of blockage for more than 10 times per year
- Most observers think that Large debris are the major cause of blockage for 1 - 10 times per year

Question 14  Do you make an allowance for blockage in hydraulic analysis/design?

- Most observers allow for blockage using professional judgement
Question 15  How would you make allowance for blockage in calculations?

- Most observers allow 0%-20% of cross-sectional area blockage in low risk situations
- Most observers allow 20%-50% of cross-sectional area blockage in high risk situations

Question 16  Where would/do you assume the blockage is?

- Most observers think that blockage is at the soffit

Question 17  How much confidence do you have in estimates or measurements of blockage?

- Most observers think that estimates for blockage are within 25% - 50%
- Most observers think that measurements for blockage are within 25%-50%

Question 18  How do you use the results of a blockage analysis?

- Most observers use the results as an input for estimating freeboard

Question 19  Any other comments

- Relatively crude head loss estimates are sufficient
- Other factors like water levels, channel roughness and flow estimates are necessary – a degree of uncertainty exists
- There are different degrees of blockages and their consequences
- River Chelt – none of the culverts have screens
- Proper maintenance is essential
- Blockage of new screens is inevitable in certain cases
- SW region guide for blockages is not well used
- No scour is accounted for – practically no data is available
• A system of blockage handling within flood risk mapping is useful
• Afflux could be estimated from a survey of the bridge
• The new MIKE 11 includes wide range of formulations – it could be used for sensitivity analysis
• More field data must be available
• Physical model testing of different arch shapes and configurations is needed to establish variation in Cd or K*
• Reliability of site measurements must be investigated
• Maintenance is essential for flat catchments
• Agricultural fertilizer bags can be a problem
• A National database of all culverts known to have blockage problems is needed

Organisations that completed the afflux questionnaire

EA National Flood Risk Mapping consultants and NEECA consultants
• JBA Consulting Engineers & Scientists
• Mott MacDonald
• WS Atkins
• HR Wallingford
• Binnie Black and Veatch
• Jacob Gibb

Other UK consultants
• Brian Faulkner (Integrated Water Management)
• ATPEC
• Hydrotec Consultants
• Tony Green

Other UK public organisations
• Department of Agriculture and Rural Development - Rivers Agency (Belfast)

Environment Agency offices
• Warrington
• Leeds
• Preston
• Newcastle
• Bodmin

Local Authorities and Internal Drainage Boards
• Bedfordshire IDB
• Lindsey Marsh Drainage Board
• Witham Fourth IDB

UK academic/research organisations
• University of Hertfordshire
• University of Plymouth

Other academic/research organisations
• Danish Hydraulic Institute
APPENDIX C: AVAILABLE DATA ON AFFLUX AND BLOCKAGE

C.1 US data

1. Nagler (1917) presented the results of 256 experiments on 34 different bridge models.

1. Yarnell (1934) conducted 2600 laboratory experiments with pier bridges.

2. Kindsvater, Carter and Tracy (1953) used Laboratory data from the Georgia Institute of Technology.


4. Bradley derived USBPR method (1978) from 1400 laboratory experiments at Colorado State University (Liu, Bradley and Plate, 1957) enhanced with USGS field data from 50 bridge measurements (from 36 bridges).

5. Schneider et al (USGS, 1977) used USGS field data from 20 sites in the southern US with wide, vegetated floodplains.

6. Hydrologic Investigations Atlases (USGS, 1978, 1979) gave observed water surfaces for 17 different flood events at 13 different field sites.

7. Biery and Delleur (1962) conducted a laboratory study on arched bridges.


C.2 UK data

1. HR Wallingford laboratory study on single and multi-span arched bridges (HR, 1988) with about 200 runs.

2. HR Wallingford field study following a UK survey that located 192 bridges giving afflux problems. 66 data sets were located.

3. Hamill (1993) reported continuous field observations from a single bridge in Devon.

APPENDIX D: BEST INTERIM GUIDANCE FOR ESTIMATING AFFLUX AND BLOCKAGE

D.1 Afflux definition

When a structure such as a bridge or culvert is built in a stream, there is a local loss of stream energy. This is due to the fluid friction in contact with the structure, and the stagnation zones that border the contracting and expanding flow sections upstream and downstream of the structure. This local loss of energy is compensated by an increase in stream potential energy immediately upstream of the structure. The maximum increase in water level above the undisturbed stream is called the afflux. Structures are normally designed so that the afflux is kept to a minimum, as it is a source of flooding.

The above situation is illustrated in Figure D.1 for subcritical flow through a bridge. The depth of steady flow in a straight, uniform channel without a structure is given by the normal depth, \( Y_N \). When a bridge structure is introduced, there is a contracting flow energy loss from section 1 (whose flow depth is \( Y_1 \)) to section 2, a bridge flow energy loss from section 2 to section 3, and an expanding flow energy loss from section 3 to section 4. The afflux is given by \( Y_1 - Y_N \), and for steady flow, this energy loss is compensated by the increased potential energy from section 0 to section 1.

![Figure D.1: Flow energy loss due to a bridge and afflux](image-url)

D.2 Blockage definition

Blockage is the restriction of a structural opening (either in area or volume) and is caused by transported debris. Design conditions rarely prevail when structures become blocked, since the afflux increases and creates a higher flood risk. Common causes of blockage are sediment,
natural debris such as trees and rubbish tipped into a watercourse. Trash screens on culverts can be a significant source of afflux, particularly if poorly designed or when they become blocked themselves.

The factors that can affect afflux are listed in Table D.1.

**Table D.1: Factors that affect afflux**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Likely effect on upstream water levels/afflux</th>
</tr>
</thead>
</table>
| Opening Ratio (M)                     | The smaller the opening ratio, the larger the afflux. If very high, may cause choking of the inlet.  
Froude Number (F)                     | Generally the higher the Froude Number the larger the afflux.                                                                                                                                 |
| Choking of bridge opening             | Afflux increases when flow is choked.                                                                                                                                               |
| Length/breadth Ratio (L/b)            | If ratio is high (L/b > 1.0) then afflux generally increases.                                                                                                                      |
| Rounding of the entrance/ Rounding of Piers | The smoother the entry the less turbulence and hence the lower the afflux. Smoothing/rounding can also help to reduce blockage risk.                                              |
| Eccentricity, e                       | For e < 0.2, afflux is generally reduced.                                                                                                                                           |
| Skew                                  | The larger the skew of a structure relative to the direction of flow, the larger the afflux. For skew angles of less than 20°, the effect is usually negligible.                   |
| Roughness                             | The larger the roughness, the larger the afflux.                                                                                                                                    |
| Scour                                 | Generally reduces bed levels and hence decreases afflux.                                                                                                                           |
| Blockage                              | Decreases the opening ratio and increases turbulence at the entrance. Nearly always results in a higher water level.                                                             |

**D.3 Data requirements for bridges**

The main data requirements for estimating the afflux at bridges are the design flow, stream and structural cross-sections, and an estimate of the amount of blockage that may occur.

The recommended cross-sectional survey positions are indicated in Figure D.1 as Section1 to Section 4. Sections 2 and 3 should be located a distance of a few metres from the bridge openings, and Sections 3 and Section 4 are located a distance equal to the width of the bridge obstruction to the flow.
The current method of estimation for blockage at bridges is to use “engineering judgement”. Although blockage may be calculated by reducing the bridge opening area or bridge flow cross-section, a magnitude for the restriction must be estimated. The local variables of potential debris size, quantity of debris being transported and the bridge opening size and existence of piers must be considered. For example, a narrow bridge located in a remote, forested area may suffer frequent blockage by floating trees, whereas a wide bridge in an urban area may suffer minimal blockage.

D.4 Data requirements for culverts

The data requirements for estimating the afflux at culverts are similar to those for bridges. For the estimation of blockage the ‘Environment Agency SW Region Blockage Model’ (The Environment Agency, 1998) is recommended.

D.5 Hand calculation methods for Afflux

Section 6 of this Scoping study detailed the methods used in afflux calculations. These have been reorganised into methods available for calculation by hand (Table D.1) and those used by computer models (Table D.2). Inevitably, the hand calculation methods are more approximate than computer methods, as they attempt to represent the energy losses in all three reaches (Figure D.1, Cross Section 1 to Cross Section 2, Section 2 to Section 3 and Section 3 to Section 4) with fewer empirical coefficients. Nevertheless, they provide an order of magnitude estimate for afflux.

The older methods for afflux estimation (D’Aubuisson, 1840; Nagler, 1917; Biery and Delleur, 1962) are not recommended. The Yarnell (1934) method is still recommended for Pier bridges, as it is derived from a very large database of measurements. The Pier, Embankment and Arched bridge methods are all applicable for low flows only (that is, flows below the bridge soffit). Detailed examples of these hand calculation methods are given in the textbook by Hamill (1999).

For high flows, the formulae given in Equations 6.39, 6.40 and 6.41 for sluice, orifice and weir flow are recommended. These flow types are well researched and require less empirical coefficients than for the low flow equations. However, the weir flow estimate must include some judgement for the simultaneous flow through the bridge. Inevitably, the latter judgement cannot achieve the same accuracy as the iterative techniques used in the computation models.

Many examples for the estimate of culvert afflux exist in the hydraulic engineering textbooks (for example, Chow, 1981; Novak et al, 2001). However, these have been standardised in the

<table>
<thead>
<tr>
<th>Structure class</th>
<th>Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier bridges</td>
<td>Yarnell (1934)</td>
</tr>
<tr>
<td>Embankment bridges</td>
<td>Kindsvater (1953) - USGS</td>
</tr>
<tr>
<td>Arched bridges</td>
<td>HR Wallingford (1988)</td>
</tr>
<tr>
<td>High bridge flow</td>
<td>Sluice gate and Orifice flow</td>
</tr>
<tr>
<td>Culverts</td>
<td>CIRIA Manual (1997)</td>
</tr>
<tr>
<td></td>
<td>Bradley (1978) - USBPR</td>
</tr>
<tr>
<td></td>
<td>Weir flow</td>
</tr>
</tbody>
</table>

For high flows, the formulae given in Equations 6.39, 6.40 and 6.41 for sluice, orifice and weir flow are recommended. These flow types are well researched and require less empirical coefficients than for the low flow equations. However, the weir flow estimate must include some judgement for the simultaneous flow through the bridge. Inevitably, the latter judgement cannot achieve the same accuracy as the iterative techniques used in the computation models.
“CIRIA Culvert Design Manual” (1997). The latter is therefore recommended for the estimation of culvert afflux.

D.6 Computer model calculation methods for Afflux

Since computer models can iterate for each reach to find the solution for implicit equations (as in the “standard step” steady flow method), their application enables greater complexity than is possible with hand calculation. Nevertheless, the models require the input of more empirical coefficients, and the uncertainty may accumulate to that of the hand calculations.

Table D.3 Recommended methods for computer model calculation of afflux

<table>
<thead>
<tr>
<th>Structure class</th>
<th>HEC-RAS</th>
<th>ISIS</th>
<th>MIKE 11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier bridges</td>
<td>Yarnell (1934)</td>
<td>Yarnell (1934)</td>
<td></td>
</tr>
<tr>
<td>Embankment bridges</td>
<td>Schneider (1977) - WSPRO</td>
<td>Bradley (1978) - USBPR</td>
<td>Schneider (1977) - WSPRO Bradley (1978) - USBPR</td>
</tr>
<tr>
<td>All bridges</td>
<td>Energy method</td>
<td>Energy method</td>
<td></td>
</tr>
<tr>
<td>All bridges</td>
<td>Momentum method</td>
<td></td>
<td></td>
</tr>
<tr>
<td>High flow methods</td>
<td>High flow methods</td>
<td>Weir &amp; Orifice/ Conduit</td>
<td>Weir &amp; Orifice/ Conduit</td>
</tr>
</tbody>
</table>

The same classification of low and high flows applies for the computational methods. Since the high bridge flow methods used for each of the 3 models are similar, the estimate of afflux is similar. Therefore only one computer model need be used. For low flows however, both different bridge types and methods are computed for each model (Table D.3). The most convenient model is that of HEC-RAS, since it computes all 4 low bridge flow methods for each bridge class in a single computation. The HEC-RAS output may also include the method with the highest afflux, and thereby accelerate this solution. In contrast, the ISIS and MIKE11 models require repeated computations for the different methods of the same bridge class.

D.7 Particular Points to Note

When using hand methods or computer models, the following aspects of the calculation should be reviewed carefully:

- Are the cross-sections positioned correctly? Review the change in the cross-sectional area of flow on the approaches/exit of the structure.
- How has any blockage been accounted for (it should be modelled by a change in the cross-sectional area at the entry)
- What is the variation in afflux between the estimation methods chosen? Are the methods used valid for the type of structure?
• Does the structure overtop? If so, is the effective overtopping length representative of what will occur?

References

