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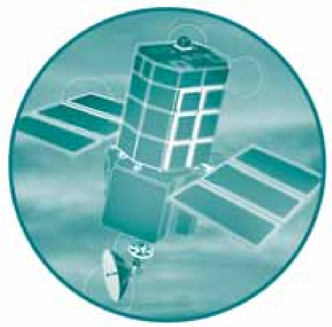

Afflux at bridges and culverts

Review of current knowledge and practice

Annex 4:
A Review of Current Practice for Afflux and Blockage Estimation in the UK, Europe and Asia

R&D Project Record W5A-061/PR4

Department for Environment Food and Rural Affairs

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A Kirby and K Guganesharajah, Mott MacDonald Ltd.

Research Contractor:
JBA Consulting – Engineers & Scientists

Publishing Organisation
Environment Agency, Rio House, Waterside Drive, Aztec West, Almondsbury,
Bristol, BS32 4UD.

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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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A. Kirby and K. Guganesharajah, Mott MacDonald Ltd.
The research contractor was JBA Consulting – Engineers & Scientists, South Barn, Broughton Hall, Skipton, North Yorkshire, BD23 3AE.
Tel: 01756 799919 Fax: 01756 799449 Website: www.jbaconsulting.co.uk

Environment Agency's Project Manager
The Environment Agency's Project Manager for Project W5A-061 was: Andrew Pepper, External Advisor, Engineering Theme.

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EXECUTIVE SUMMARY

The review paper forms part of the joint Defra/Environment Agency R&D Scoping Study into the Hydraulic Performance of Bridges and Other Structures, including the Effects of Blockages at High Flows. The paper considers general approaches to afflux estimation. Current practices for afflux estimation in use worldwide (with the exception of the United States) are reviewed. Methods of afflux measurement are discussed and key areas for future research are identified.

Methods for estimating afflux can be divided into empirical and theoretical methods. Empirical methods based on laboratory experiments and field data are in use in many parts of the world and often provide a first order estimate of afflux. The theoretical methods divide into methods based on conservation of momentum and methods based on conservation of energy and usually form the basis of algorithms for computer modelling.

Methods based on conservation of momentum have not been as widely used as those based on conservation of energy though they have certain advantages over conservation of energy methods. Their use should be investigated further. Computational Fluid Dynamic (CFD) models will have the capabilities in the near future of providing experimental data instead of, as well as complementing, physical models. The process of blockage by debris has not received significant attention in the past and there is a key link to be made by future research with the Defra/ Environment Agency Risk and Uncertainty R&D Theme. Good experience and training of hydraulic modellers in bridge hydraulics remains a key requirement if current methods of afflux estimation are to be used reliably.

CONTENTS

1 INTRODUCTION

This review paper has been prepared as a mechanism for feeding information into the Environment Agency R&D Scoping Study into the Hydraulic Performance of Bridges and Other Structures. The primary objective of the review paper is to discuss current practices in afflux estimation and blockage at a worldwide level. The review paper excludes a detailed discussion of UK and US practice and bridge representation in hydraulic modelling programs since these aspects are being covered by others in the study team. The specific objectives of the review paper are:

- to review current practice for estimating afflux and assessing blockage risk in Europe and Asia;
- to provide data on afflux measurements made in the field;
- to advise on the methods of measuring afflux and blockage in the field;
- to provide comment on the requirements for future research work priorities and assist in deriving indicative costs for blockage assessment.

On the latter point some comments on future research are provided within the review paper though it is envisaged that further comment will be provided following discussion of all the review papers.

2 RESEARCH APPROACH

2.1 Work Tasks

Contacts were made with representatives from academic and industrial organisations in a number of countries throughout the world. The aim when choosing a country to contact was to obtain a reasonable geographic spread. Individuals were queried informally on their experience of afflux and blockage and requests were made to source references that would typically be used in practice. Individuals in the following countries were contacted:

Australia
Denmark
Hong Kong
India
Khazakstan
New Zealand
Pakistan
Tajikistan
Thailand

In addition independent consultants working internationally were contacted and a request for information was sent to the Rivers Group – an international circle of people involved in water resources, corresponding by email.

A literature review was carried out to identify current texts and research that are used for afflux and blockage estimation, though this concentrated on identifying the extent of non-UK and non-US literature (covered by others). Literature searches included: ICE Library, British Library, International Commission of Irrigation and Drainage (ICID), the world wide web, Cambridge University Library, Mott MacDonald's technical library and Aqualine. In addition photographs and other information were sourced from previous and ongoing Mott MacDonald projects. A selection of photographs are presented as Plates at the end of the Paper.

The response to requests for information has been patchy and it is hoped that further information will be received after preparation of this First Draft.

2.2 Definitions and Notation

Afflux was defined in the contract documentation 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 afflux is more strictly defined as the maximum difference in elevation of the water surface, at a location upstream of the structure, with and without the structure. This is different from the head loss across the structure as can be seen in Figure 2.1. For the purposes of this review paper the definitions and notation used in Figure 2.1 have been used. In some literature reviewed it is unclear whether a distinction between head loss and afflux has been made. Where possible this has been mentioned in the text.

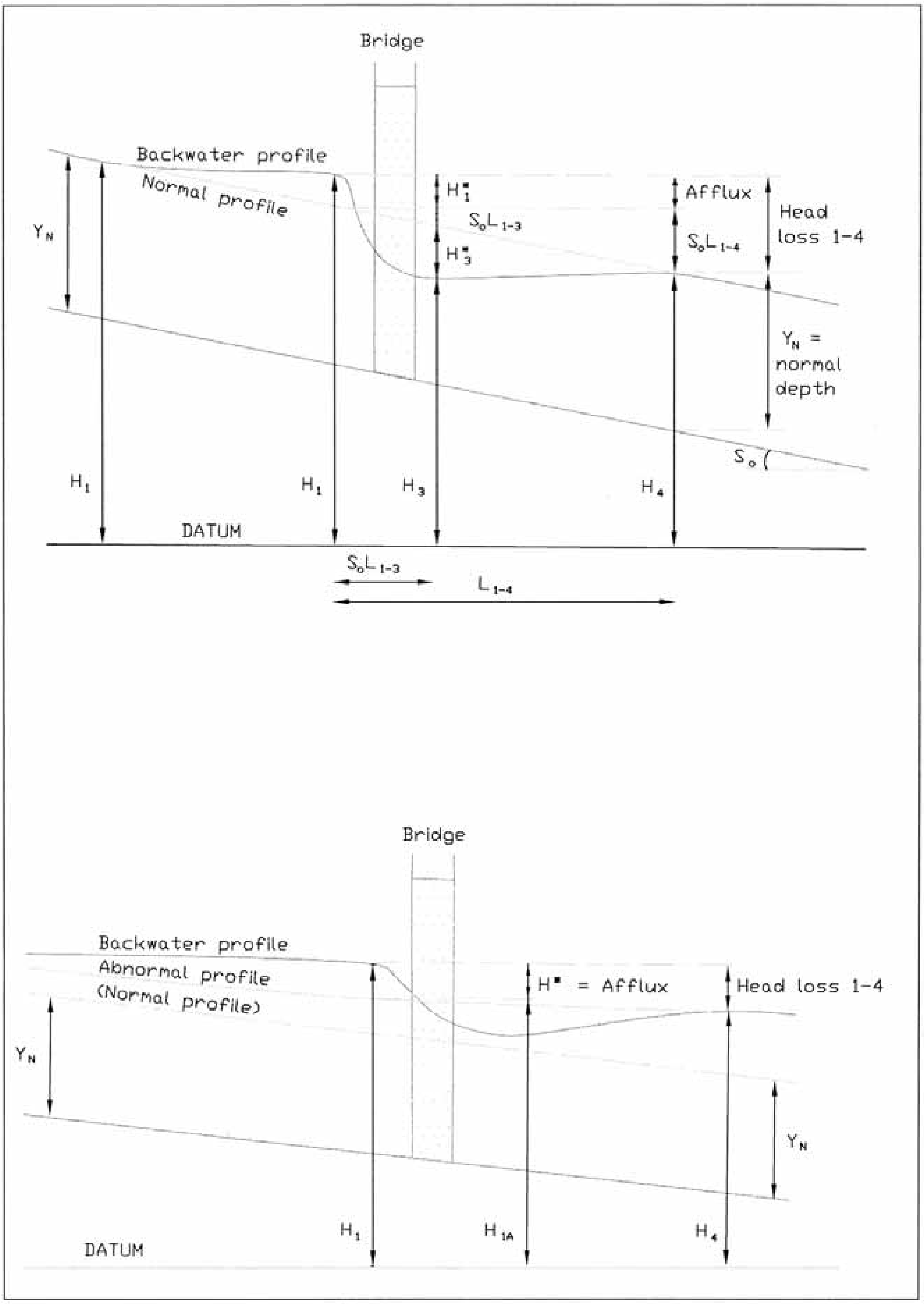

Figure 2.1 Definition of afflux used in Paper (from Hamill, 1999)

3 GENERAL APPROACHES TO AFFLUX ESTIMATION

3.1 Theoretical and Empirical Methods

There are theoretical and empirical methods available in various forms to estimate the afflux when flow passes through bridge constrictions. The theoretical methods can be categorised under:

- Methods based on conservation of energy
- Methods based on conservation of momentum

The main advantage of the momentum balance approach is that losses due to expansion or contraction are allowed for in the equation, whereas the energy equation approach requires appropriate coefficients to estimate the entry and exit losses. There are instances under high flow conditions when the upstream water level rises above the bridge soffit and the flow regimes is governed by pressure or orifice flow. The orifice flow can be categorised under free flow or submerged flow conditions.

The empirical relationships are generally based on laboratory experiments or field data or a combination of both. The relationships that have been developed generally include terms for discharge coefficients, the velocity in the unobstructed waterway and factors related to constricted and unobstructed areas of flow.

The drag on bridge piers also contributes to the afflux. The force of drag on piers can be included in the momentum equation by using appropriate drag coefficients based on the shape of the pier.

Experiments on drags and losses to date have been based on physical models with field verification. Recent advances in computational fluid dynamics (CFD) models mean that within the near future they will have a major role in further experimental work. They can become the standard approach in the near future to analyse the afflux associated with various bridge configurations.

3.2 Method Based on Conservation of Energy

In this method the energy balance is considered for the upstream section, bridge constriction and the downstream section of the river (Figure 3.1). Applying Bernoulli's equation to these sections we obtain:

Section 1 and 2

$$d_1 + \alpha_1 \frac{v_1^2}{2g} = d_2 + \alpha_2 \frac{v_2^2}{2g} + \Delta h_{12} \qquad [1]$$

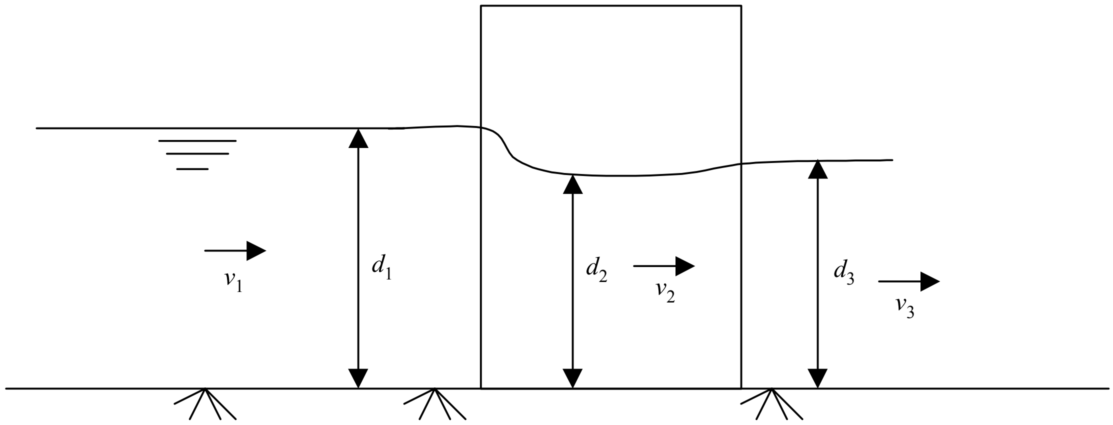

Figure 3.1 Elevation view of channel constriction (energy analysis)

Section 2 and 3

$$d_2 + \alpha_2 \frac{v_2^2}{2g} = d_3 + \alpha_3 \frac{v_3^2}{2g} + \Delta h_{23} \qquad [2]$$

where:

d_1	-	depth of flow upstream of constriction
α_1	-	energy coefficient upstream of constriction
v_1	-	velocity of flow upstream of constriction
d_2	-	depth of flow within the constriction
α_2	-	energy coefficient within the constriction
v_2	-	velocity of flow within the constriction
d_3	-	depth of flow downstream of constriction
α_3	-	energy coefficient downstream of constriction
v_3	-	velocity of flow downstream of constriction
Δh_{12}	-	entry loss
Δh_{23}	-	exit loss

From equation [1] and [2] the difference in water level between section 1 and 3 is given by the following expression:

$$\Delta h_a = \alpha_3 \frac{v_3^2}{2g} - \alpha_1 \frac{v_1^2}{2g} + \Delta h_{12} + \Delta h_{23} \qquad [3]$$

The entry loss can be estimated using a factor for the velocity head difference between the upstream section and constriction. Similarly the exit loss can be estimated using a factor for the velocity head difference between the constriction and the downstream section. The factors associated with entry and exit loss are available in several publications (e.g. Chow, 1982) and are summarised in Table 3.1.

If the energy losses due to fiction are significant then the energy equations should be applied by making allowance for frictional losses. In this case two sections within the constriction are required at the upstream and downstream end of constriction.

Table 3.1 Entry and exit loss coefficients

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

3.3 Method Based on Conservation of Momentum

The momentum equation accounts for the momentum, pressure force, frictional resistance and gravitational force. The application of the momentum equation is given for the four sections shown in Figure 3.2. For subcritical flow the calculations should commence from downstream and proceed upstream. For supercritical flow the calculations should be carried out from upstream to downstream. The momentum equation is derived below for a steady condition.

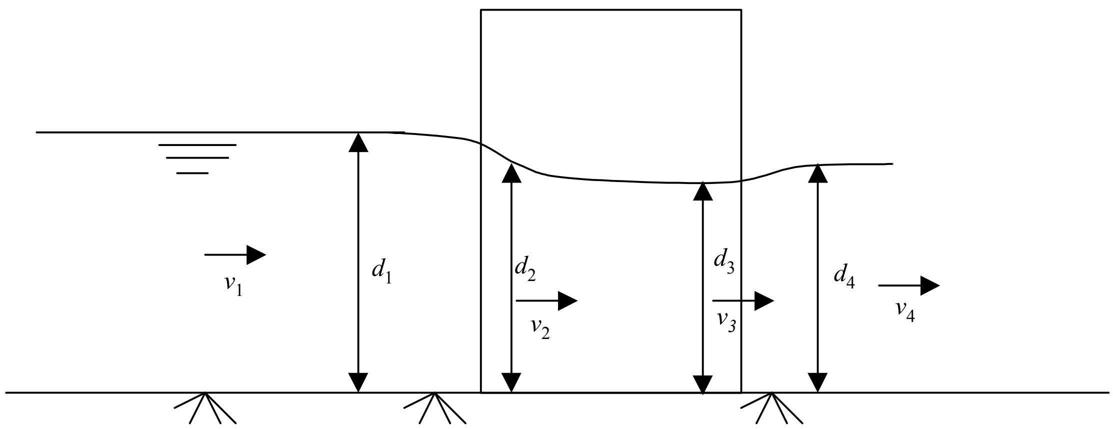

Figure 3.2 Elevation view of channel constriction (momentum analysis)

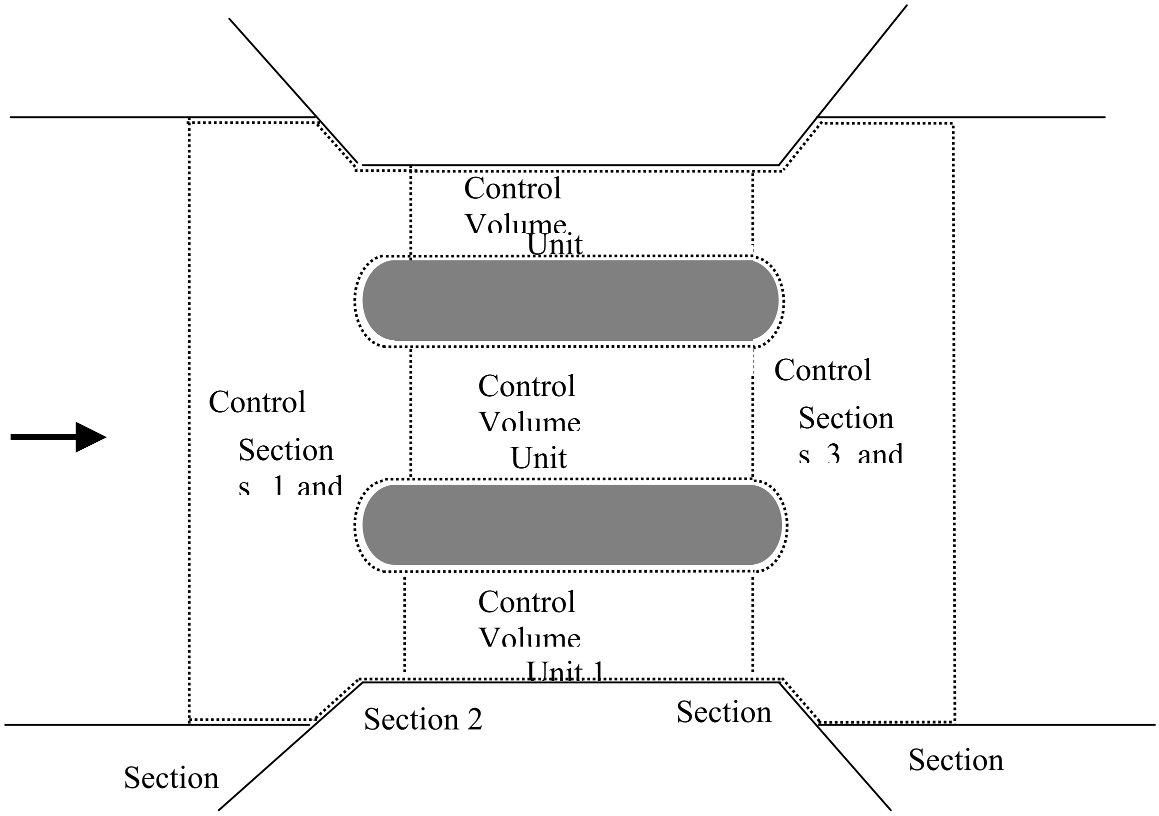

Figure 3.3 Plan view of channel constriction (momentum analysis)

Section 1 and 2

The momentum equation is applied by considering the control volume shown in Figure 3.3 between sections 1 and 2.

$$A_1\bar{Y}_1 + \beta_1 \frac{Q_1^2}{gA_1} = \sum A_{2i}\bar{Y}_{2i} + \sum \beta_{2i}\frac{Q_{2i}^2}{gA_{2i}} + \sum A_{ui}Y_{ui} + \sum D_i + F_{12i} - W_{12i} \qquad [4]$$

Where:

A_1	-	Flow area upstream of the constriction
$\bar{Y}_1$	-	Centroid of the flow area upstream of the constriction
Q_1	-	Upstream flow rate
g	-	Gravity
β	-	Momentum coefficient
$_i$	-	Unit
$_{2i}$	-	Element at the constriction
A_{ui}	-	Obstructed area of upstream side of pier or abutment
$\bar{Y}_{ui}$	-	Centroid of obstructed area of upstream side of pier
D_i	-	Force due to drag
F_{12i}	-	Force due to friction
$W_{12\mathrm{i}}$	-	Force due to the weight of water in the direction of flow

The continuity equation is given by the following expression:

$$Q_1 = \sum Q_{2i} \qquad [5]$$

Alternatively the control volume between sections 1 and 2 can be subdivided in the same way as the control volume has been divided between sections 2 and 3 in Figure 3.3. The flow distribution between each unit would need to be known. In this case certain assumptions are required to close the system of equations.

Section 2 and 3

In this case the momentum equation is applied to the individual units (Units 1, 2 and 3 in Figure 3.3) as described below:

$$A_{2i}\overline{Y}_{2i} + \beta_{2i}\frac{Q_{2i}^2}{gA_{2i}} = A_{3i}\overline{Y}_{2i} + \beta_{3i}\frac{Q_{3i}^2}{gA_{3i}} + F_{23i} - W_{23i} \qquad [6]$$

Continuity equation:

$$\sum Q_{2i} = \sum Q_{3i} \qquad [7]$$

Section 3 and 4

As for the control volume between sections 1 and 2 the momentum equation can be applied to the control volume between sections 3 and 4.

$$\sum A_{3i}\overline{Y}_{3i} + \sum \beta_{3i}\frac{Q_{3i}^2}{gA_{3i}} + \sum A_{di}Y_{di} - F_{34i} + W_{34i} = A_4\overline{Y}_4 + \beta_4\frac{Q_4^2}{gA_4} \qquad [8]$$

Continuity equation:

$$\sum Q_{3i} = Q_4 \qquad [9]$$

As described for sections 1 and 2 the control volume could also be sub-divided and the momentum equation applied to individual units.

3.4 Comments on Energy and Momentum Methods

In order to close the system of equations and to obtain the solutions it has to be assumed that the water level in sections 1 and 2 are same across the sections. An iterative technique may be employed to obtain the appropriate flow distribution between units. The iterative procedure should aim to obtain the same water level upstream for a given water level downstream.

Further work is required to determine energy and momentum coefficients. The future of such work is in the use of CFD models for experiments with limited use of physical models to validate the CFD models.

3.5 Orifice Flow

Orifice flow occurs when the water is in contact with the upstream edge of the deck of the bridge and the water level in the constriction is below the soffit of the bridge. In this case supercritical flow can prevail in the constriction. The orifice flow in the system can be expressed by the following relationship:

$$Q = C_d A \sqrt{2g\left(d_1 + \alpha_1 \frac{v_1^2}{2g} - d_2\right)} \qquad [9]$$

where:

Q	-	discharge through the bridge
C_d	-	coefficient of discharge
A	-	net area of bridge opening
d_1	-	depth of flow upstream
α_1	-	energy coefficient
v_1	-	velocity in the channel upstream (section 1)

The coefficient of discharge in equation [9] can vary from 0.35 to 0.6. In certain literature the value of d_2 in equation [9] is modified as $0.5d_2$ (HEC-RAS manual).

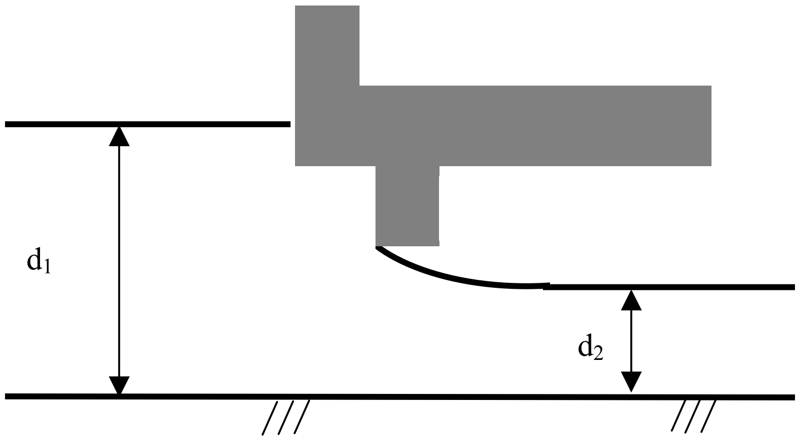

Figure 3.4 Orifice flow

3.6 Submerged Flow

Submerged flow occurs when the bridge soffit on the upstream and downstream edges of the bridge is submerged. This is similar to pressure flow in pipe. For this case the flow equation can be given by the following equation:

$$Q = C_d \sqrt{2g(\Delta h)} \qquad [10]$$

The head loss Δh in the above expression is given by:

$$\Delta h = \left(\left(\alpha_1 \frac{v_1^2}{2g} + h_1 \right) - \left(\alpha_2 \frac{v_2^2}{2g} + h_2 \right) \right) \qquad [11]$$

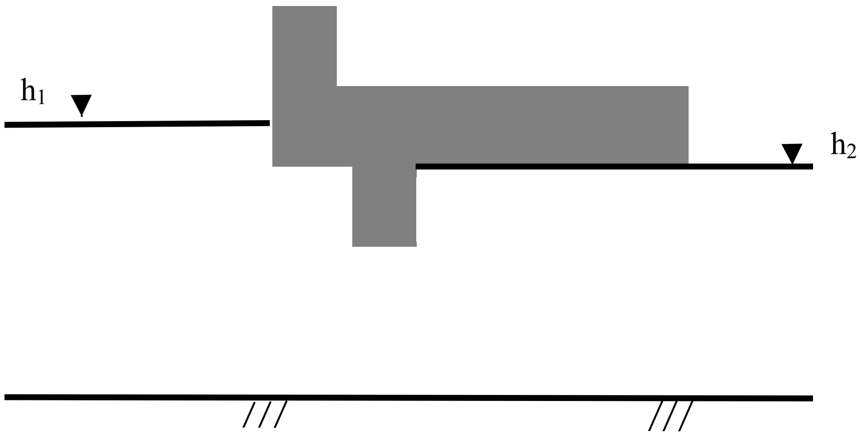

Figure 3.5 Submerged flow

3.7 Methods Based on Empirical Relationships

There are a number of methods in use based on empirical relationships. These include:

- Biery and Delleur method (arch bridges)
- HR method (arch bridges)
- Nagler (piers)
- Yarnell (piers)
- D'Aubuisson (piers)
- Molesworth (constrictions)
- Homak (constrictions)

These are discussed further in Chapter 4.

4 CURRENT WORLDWIDE PRACTICE IN AFFLUX ESTIMATION

4.1 General Practices

It was clear from discussions with practitioners that afflux estimation by hand or spreadsheet is not a particularly common design requirement and is more commonly carried out as part of developing a computational hydraulic model. Where hydraulic models are used the choice of afflux method used is set by the range of program options available. The choice of which method to use within a program appears to be dictated by the following criteria

- applicability of method to the type of bridge and flow being analysed;
- user preference or confidence in a particular method;
- method which gives stable and realistic results.

A lack of model robustness when dealing with flow through bridges was cited as a reason for using a basic energy loss method over other methods in order to give a stable model, believable results and allowing judgement to be used during model calibration by adjusting loss coefficients within realistic bounds.

Where specific hand or spreadsheet calculations are required reference would be made to internationally accepted technical hydraulic literature such as Open-Channel Hydraulics (Chow, 1981) and Open Channel Flow (Henderson, 1966). This leads to the use of three main afflux methods:

- basic energy equation;
- USBPR method;
- USGS method.

It is not the intention of this review paper to discuss the latter two methods as they will be covered by other parts of the study. Suffice to say that the energy equation can be derived from elementary fluid mechanics and is explained in a number of "standard" hydraulics texts such as those above and Open-Channel Hydraulics (French, 1986) and is summarised in Chapter 3. The USBPR method is derived from energy and continuity principles and is contained in Hydraulics of Bridge Waterways (Bradley, 1978). The USGS method is also derived from energy and continuity principles and is contained in Measurement of peak discharge at width contractions by indirect methods (Matthai, 1967).

Other methods mentioned by practitioners include:

- rules of thumb
- Biery and Delleur method (arch bridges)
- HR method (arch bridges)
- Nagler (piers)
- Yarnell (piers)
- D'Aubuisson (piers)
- Use of weir and orifice flow equations

Flow over bridge decks is commonly considered as broad crested weir flow.

In the above respects general practices appear to match the types of practices encountered in the UK. Other practices are described in the following sections.

4.2 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 Skogerboae and Hyatt (Analysis of submergence in flow measuring flumes, 1973). They proposed the use of a submerged flow equation of the form:

$$Q = \frac{C_2 g^{1/2} b (y_1 - y_3)^{1.5}}{-(\log S)^n} \qquad [12]$$

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 was developed by D'Aubuisson (1840) which has been widely applied and is still in use. Refer to Figure 3.1 for details of symbols.

$$Q = C_b b_2 d_3 \sqrt{2gh_a + v_1^{\,2}} \qquad [13]$$

Where:

C_b	-	empirical discharge coefficient
b_2	-	total width of unobstructed channel at constriction
d_3	-	depth of channel downstream of constriction
g	-	gravity
h_a	-	difference in head upstream and downstream of constriction
v_1	-	velocity of flow upstream of constriction

4.3 Indian Sub-continent

There has been a long history of research and field experimentation in hydraulics in the Indian sub-continent, though 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 (ed P N Khanna, 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:

$$H_1^* = \frac{V^2}{2g} \qquad [14]$$

Where:

V	-	average velocity of the river.

The empirical Molesworth formula (1871) is sometimes used:

$$H_1^* = \left[\frac{V_a^2}{17.9} + 0.015\right]\left[\left(\frac{A}{a}\right)^2 - 1\right] \qquad [15]$$

Where:

V_a	-	velocity in unobstructed river (m/s)
A	-	natural (unobstructed) waterway area (m^2)
a	-	contracted area (m^2)

Other formulae used are:

$$H_1^* = \frac{V^2}{2g}\left(\frac{W^2}{c^2 L^2} - 1\right) \qquad [16]$$

Where:

V	-	velocity of normal flow in channel
W	-	width of stream at High Flood Level (regime depth)
L	-	linear waterway under the bridge
c	-	coefficient of discharge through the bridge, taken as 0.7 for sharp entry and 0.9 for bell mouthed entry

from Bridge Engineering, (S Ponnuswamy 1986, source unattributed).

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:

$$Q = 1.71 C_d L \left(y \quad + \frac{V_a{}^2}{2g} \right)^{3/2}$$

[17]

Where:

V_a	-	approach velocity	
y_u	-	upstream water depth	
C_d	-	coefficient of discharge, which is given as:	
		Narrow bridge opening without floors	0.94
		Wide bridge opening with floors	0.96
		Wide bridge opening with no bed floors	0.98

For orifice flow the discharge Q is given by:

$$Q = C_o \sqrt{2g}.L.y_d \left(h + (1+e)\frac{V_a{}^2}{2g} \right)^{1/2}$$

[18]

Values of e and C_o are provided in graphical form for different constriction ratios.

4.4 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 . Homyak (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:

$$Q = mb_m \sqrt{2g} H_0^{1.5} \sigma_3$$

[19]

with *m* dependent on the type of bridge and other coefficients determined from work by Homyak.

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:

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

[20]

or:

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

[21]

The method determines:

$$\Delta H = (H\ )_u + (H_f)_c + (H_f)_d + H_{sd}$$

[22]

Where:

ΔH	-	additional head loss due to bridge
$(H_f)_u$	-	increase in bed friction upstream
$(H_f)_c$	-	increase in bed friction in constriction
$(H_f)_d$	-	increase in bed friction downstream
H_{sd}	-	kinetic energy lost due to sudden expansion

each of these is determined in terms of:

$$\frac{V_3^2}{2g}$$

so that an equation of the form:

$$Q = C_d A_3 (2gK\Delta H)^{1/2}$$

[23]

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.

4.5 Australasia

Australia primarily uses the USBPR method as the main method for afflux estimation. Austroads (the national association of road and transport authorities in Australia) bases its method for calculating backwater, contained in Waterway Design (1994), on the USBPR method (Bradley, 1978).

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 being that at critical depth the flow is hdyraulically 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 they 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. Conditions in the UK tend not to match the preferred conditions outlined above, though their use would be appropriate elsewhere in the world.

5 MEASUREMENT OF AFFLUX IN THE FIELD

5.1 What to measure

It is often difficult to obtain reliable field data to validate laboratory data in river hydraulics and afflux is no different. The measurement of afflux in the field is complicated by the fact that it cannot be measured directly. What can be measured is the head loss across the structure. A backwater analysis then has to be undertaken to estimate what the water surface profile would be without the structure. This can then be subtracted from the measured profile to determine the afflux.

Afflux measurement is also complicated by the uncertainty concerning at which location upstream and downstream of the structure to measure the water surface profile.

Water level measurement is required upstream of the bridge at Section 1 the point of maximum afflux and then downstream of the bridge at either Section 3 or 4. If water level measurements are taken at Section 4 they must be sufficiently far downstream to avoid the area between Sections 3 and 4 where the flow is expanding against an adverse hydraulic gradient.

The USGS method, which was developed from extensive laboratory study backed up with at least 30 field sites, was developed as a method to use constrictions as flow measurement devices rather than specifically to estimate afflux. Field measurements were made at Sections 1 and 3 to determine discharge. Afflux can then be deduced using a backwater analysis.

The USBPR method was also developed from extensive laboratory study backed up with field measurements (some carried out by the USGS). The method considers water levels at Sections 1 and 4.

It is therefore clearly important to understand where in particular along the longitudinal profile of a river a set of field measurements have been taken and if measurements are to be undertaken what they intend to measure.

Upstream of the bridge the water level measurements at Section 1 must be sufficiently far upstream to be in the natural un-constricted channel and therefore not affected by the contraction of flow through the constriction between Sections 1 and 2 which draws down the water surface profile. The USGS method suggests that Section1 should be located one span upstream of the face of the structure. The USBPR method provides a graphical method for determining the location of Section 1. However a modification of the USBPR method, sometimes called the modified Bradley method defines Secion1 as one span upstream of the structure face (Kaatz and James, 1997) as per the USGS method.

The downstream section, Section 3, should be parallel to the contraction and defines the minimum area, it is therefore normally between the bridge abutments. This can make measurement difficult in practice as it may be beneath the bridge deck and difficult to access during high flows. Fixed instrumentation can overcome this problem.

There is no absolute rule to define the location of the downstream section, Section 4 and depends on the nature of the flow expansion. HEC-2 suggested a rate of expansion of 1:4

(width:length), thus Section 4 should be at least 2(B-b) from the downstream face of the bridge. However Kaatz and James (1997) cast doubt on this assumption by finding that it leads to an overestimation of afflux compared with values they measured at nine sites under thirteen floods. It should also be noted, for example, that the WSPRO program assumes Section 4 is one span downstream.

Water level measurements are normally taken at the edge of a river and the variation in water level across a section is ignored. The implications of this may need to be considered to improve the reliability of measurements.

5.2 Practical Issues

There are number of practical issues to be considered when measuring afflux in the field. The first issue is to identify suitable bridges that will yield useful measurements. Where field measurements are required for a specific study at a particular bridge this is not a problem, though the value of the results particularly, for example, where the afflux is small may be questionable. Where more general field measurements are required then identifying suitable bridges requires investigation of potential sites.

To obtain meaningful measurements requires a reasonable head loss across the bridge, otherwise the uncertainty over the accuracy in water level measurement will affect or dominate the results. For example the smallest head loss measured in the field measurements listed in Hydraulics of Bridge Waterways (1978) is 50 mm with the majority of measurements of head loss being between 100 mm and 900 mm. Matthai (1967)gives a warning that the USGS method should not be used where the head loss (Sections 1 to 3) is less than 150 mm. Whilst there are some bridges and in particular small culverts in the UK where losses will be over 150 mm there are also many others where head losses are considerably lower. Even at traditional small multiple span arch bridges, head losses are frequently relatively small.

A problem encountered by all hydraulic modellers when schemetising a bridge in a model is that the bridge does not fit neatly into a type detailed in the standards methods. It may have unusual approach or exit conditions or it may have an unusual configuration of piers or abutments. Similarly when choosing a field site the researcher has to consider whether these non-standard configurations will invalidate the data by making it inapplicable to other bridges.

Under normal flow conditions few bridges in the UK will have a large head loss across them, so the next problem to obtain useful field data is to experience a range of flow conditions to generate a good dataset. This depends on the number of installations (greater numbers giving a greater chance of experiencing a flood); duration of field trial (longer duration giving a greater chance of experiencing a flood) and choosing a site that is more likely to experience a range of flows owing to the type of river or catchment where the bridge is located.

Fixed installations will normally involve the location of a transducer (pressure or ultrasonic) or float and shaft encoder in a stilling well connected to a data logger or telemetry outstation. Measurements can be taken at regular intervals (often 15 minute intervals). A power supply is required, though unless the station is required long term, battery power can be used. Where a long term station is required remote telemetry can be set up using a modem and telephone line or mobile phone - GSM (Global System of Mobile Communications) modem. Adequate damping of the water level fluctuations using a stilling well is important to collect reliable data.

A less costly alternative to the above type of installation is a maximum water level gauge. This involves a gauge containing water sensitive tape housed within a stilling well. After a flood the maximum water level as indicted by the tape can be recorded by hand. The limitations of this are that only a single water level can be recorded and the site has to be visited and a manual reading taken after each event. It does though avoid the need to go out and gauge during floods.

Gauging during floods is routinely carried out by Hydrometry Units in the Environment Agency. Many have or are preparing service level agreements with the Flood Forecasting and Warning function to identify the needs and priorities for gauging during floods. The priorities will concentrate on flood defences, properties at risk and gauging to aid forecasting so resources are unlikely to be readily available to measure afflux.

Gauging during floods is difficult owing to problems of access and safety and having the resources available at short notice (potentially 24 hours a day, 365 days a year) to carry out the work. Such gauging is normally carried out by manually reading staff gauges. Probably the main problem with this is obtaining an accurate reading due to water level fluctuations, the gauge board may also be inaccessible during floods so that gauge is read from a distance further reducing the accuracy. Water level measurement post flood is also possible from trash marks but is generally less reliable than the other methods.

5.3 Field datasets

Field datasets encountered in technical literature include:

- work by Kaatz and James (1997) comprising measurements at 13 flood events at 9 sites
- work by Hamill and McInally (1990) at Canns Mill Bridge, detailed assessment and comparison of methods
- work to develop the USGS method (Matthai, 1967) verification at 30 sites
- work to develop the USBPR method (Bradley 1978) the first edition of the work was based on limited field measurements carried out by the USGS to verify model tests the second edition was then prepared using additional field data at nearly 40 sites.

6 BLOCKAGE

Relatively little research was found that looked at bridge or culvert blockage by debris. The main focus of concern in the US with debris has been the increase in scour that results rather than the effect on water levels. The USGS conducted a study, in co-operation with the Federal Highway Administration (FHWA) into the problem of drift (debris) accumulation. The study was conducted between 1992 and 1995, and included a literature review, an analysis of data from 2,577 reported debris accumulations, and field investigations of 144 debris accumulations. The finding are contained in the Potential Drift Accumulation at Bridges (www.usgs.gov). Though there are some interesting findings of the study, such as Table 6.1, their applicability to UK rivers is doubtful since many of the US rivers are wide with dense woody vegetation in the floodplain and along the river edge.

Table 6.1 Percentage of channel blocked by debris at USGS study sites

Percentage of channel blocked	Number of bridges (and as %) by State				
	Indiana	Maryland	Massachusetts	South Carolina	Tennessee
75 – 100	3 (0.1)	1 (0.1)	0 (0.0)	35 (1.0)	6 (0.2)
50 – 75	2 (0.1)	1 (0.1)	0 (0.0)	28 (0.8)	18 (0.5)
25 – 50	28 (1.2)	7 (0.8)	1 (0.1)	100 (2.9)	74 (2.1)
5 – 25	104 (4.3)	51 (5.8)	16 (2.1)	409 (11.7)	282 (7.9)
0 – 5	133 (5.6)	62 (7.1)	37 (4.9)	481 (13.8)	422 (11.8)
0	2124 (88.7)	757 (86.1)	702 (92.9)	2445 (69.8)	2779 (77.5)
Total	2394 (100)	879 (100)	756 (100)	3498 (100)	3581 (100)

Source: Potential Drift Accumulation at Bridges (www.usgs.gov).

The Blockage Risk Model developed during the study by South West Region, Risk Assessment of Structure Blockage During Flood Flows as outlined by Faulkner and Waller (1998) appears a valuable development in UK practice. This effectively provides a screening process to identify structures at high risk of blockage. This is combined with an assessment of the consequences of higher upstream water levels to give a decision making tool to decide whether blockage should be taken into account in hydraulic analysis. What it does not do appear to do is seek to advise on the degree of blockage that should be considered. It is this latter area where fairly arbitrary judgements are being made and further research would be valuable.

Blockage is normally analysed hydraulically as a percentage reduction in flow area, often over the full length of the structure. A more accurate representation is likely to be a reduction in flow area at the inlet/upstream face of the structure only.

The location of the blockage in the river cross section has an effect on the magnitude of afflux. Six types of blockage were assessed in the work for the above study. Common assumptions in practice are that it either reduces the flow area over the full water depth (corresponding to a reduction in structure width) or it reduces the flow area from the water surface downwards. The former is generally easier to model. The latter is sometimes simplified as a reduction in soffit level. This can be considered to correspond to a blockage only occurring at high water levels where floating debris is prevented from passing through the barrel by the structure soffit.

An analysis of blockage at Welsh Bridge in Shrewsbury on the River Severn was recently carried out by Mott MacDonald. It considered 20 % and 40 % blockage of the open bridge area using the two common assumptions above. The analysis showed only small differences between the different types of blockage but a threefold difference in head loss between a 20 % and 40 % blockage.

Blockage should be considered as an element making up the uncertainty in the prediction of water level. As such it should be analysed within a risk framework along with other uncertainties in roughness, flow etc. At the moment when it is taken account of it is normally only analysed in a sensitivity test. Where the consequences of blockage are relatively low this is an appropriate level of analysis. Where the consequences of blockage are high a more detailed analysis may be appropriate. Risk Performance and Uncertainty in Flood and Coastal Defence – A Review (HR Wallingford, 2001) usefully sets out the range of types of risk analysis that could be used. A Monte Carlo analysis is one type of analysis that appears to lend itself to analysing blockage since it is possible to consider blockage in terms of different probabilities of percentage reductions in flow area.

7 CONCLUSIONS AND FUTURE RESEARCH

The method based on conservation of momentum has not been used as widely used as methods based on conservation of energy (such as USGS and USBPR). HEC-RAS does allow the option of the use of momentum method, but this is the only known hydraulic modelling package that does. The use of the momentum method has advantages over the energy method and its use should be investigated further.

In the near future CFD models will have the capabilities to reliably model flow through bridges and can be used instead of physical models for experimental work. Such development of CFD models is already ongoing, for example, refer to 3-Dimensional modelling of overtopping at bridge crossings (Richardson et al, 1999) and this would be a useful area for further research.

The process of blockage of structures by debris appears to not have received significant investigation. The development of the South West Region blockage risk analysis is a valuable step in the right direction. Structure blockage should be considered in a risk framework as an element of uncertainty in the hydraulic performance of a river. As such there is a key link to be made by future research with the Defra/EA Risk and Uncertainty R&D Theme. The key element of further research should be to develop guidance on the degrees of blockage possible under different catchment and structure conditions.

It is clear that application of different methods can give widely different results. Partly this is because the methods tend to have been verified only for certain hydraulic conditions (e.g. flows, size of river, magnitude of afflux). Understanding the limitations of the methods is an important part of the hydraulic analysis. The improved use of appropriate methods will depend on good experience and training of hydraulic modellers. This confirms that hydraulic analysis is not at a stage (and maybe never will be) where models can be used in a "black box" approach by untrained staff.

The problem of estimating afflux and the effects of blockage in practice must be placed in the context of the problem being solved and the magnitude of other uncertainties in hydraulic performance such as flow and roughness uncertainty. In many cases the consequences of the uncertainty in afflux estimation or blockage are considerably less than the uncertainties in other parameters. This means the focus of our attention should be on reducing the largest uncertainties.

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10 PLATES

Plate 1 – Flow around bridge piers – River Severn

Plate 2 – Supercritical flow through bridge – Cambodia

Plate 3 Bridge blockage by boat – River Soar

Plate 4 – Debris can take many forms – Kilkenny, Ireland

Plate 5 – Flood flows at arch bridge – River Cam

Plate 6 – Flood flow at bridge – Cambodia