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Review of current knowledge and practice

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A Review of Current Knowledge on Bridge Afflux

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Donald W Knight, School of Civil Engineering, The University of Birmingham

Research Contractor:
JBA Consulting – Engineers & Scientists

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

Tel: 01454 624400 Fax: 01454 624409 Website: www.environment-agency.gov.uk

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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.

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

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Donald W Knight, School of Civil Engineering, The University of Birmingham.
The research contractor was:
JBA Consulting, 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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1 INTRODUCTION

One of the common tasks of a river engineer is to make estimates of water level based on a recorded or simulated flood discharge. A review of the methods for doing this has recently been undertaken by Knight (2001) for the Environment Agency, and is contained in an allied scoping study report on 'Reducing uncertainty in conveyance estimation' (HRWallingford, 2001). In that study various methods are reviewed that enable the conveyance capacity of river channels, or the stage-discharge relationship at a given cross-section of a river, to be determined for both in-bank and out-of-bank flows. The stage-discharge relationship (H v Q) is particularly important in river engineering, as it not only links discharge with water level in flood routing models, but also it is frequently used to obtain estimates of water level for extreme flood events via extrapolation of the rating curve. In the earlier scoping study (HRWallingford, 2001), the H v Q relationships were assumed to be primarily governed by the roughness properties of the river and the cross-section shape of the river channel, based on prismatic river cross-sections. Consequently any features that locally influence water level, such as hydraulic structures, or where there are significant changes in cross-section, were ignored.

In flood events, it is these local features that may cause significant increases in water level. Since our current knowledge of the hydraulic losses occurring at certain structures is still imperfect, the estimation of water levels in many river engineering problems remains problematic. Our knowledge is limited not only by the enormous variety in the types of hydraulic structure encountered in river engineering, but also by the difficulty of obtaining high quality field data during extreme flood events and by the complexity of the 3-D fluid processes involved. Good estimates of water level are of particular importance in many areas of river engineering, as for example in the production of accurate flood risk maps, in the exercise of local planning procedures, and in the design of appropriate flood alleviation works. Since computational models are used in many of these processes, the calibration and verification of 1-D, 2-D and 3-D mathematical river models is clearly important. The companion expert report by Samuels (2001) should be consulted for a review of the implementation of afflux in computational hydraulic river models. Current practice for estimating afflux is described in the companion expert reports by Kirby & Guganesharajah (2001) and Benn & Bonner (2001). This report focuses on a review of our current knowledge concerning the increase in water level, or afflux, arising from bridges and large culverts, and their consequent influence on backwater profiles. Some conclusions are drawn concerning gaps in our knowledge and suggestions made as to a future research programme that addresses some key issues.

This report only considers the generic type of hydraulic structure loosely defined as 'bridges and large culverts', due to their similarity and common occurrence in the UK. The bridge afflux problem is a well-known one and has been studied by many researchers over many decades. The extent of such work over the last 160 years may surprise modern engineers, and is therefore illustrated in Table 1 by listing a brief selection of key researchers and relevant texts. A more substantive list is given in the references.

Table 1. A brief history of work on afflux at bridges and related topics

1840	*d'Aubuisson & Weisbach*: early experiments on different types of bridge pier
1918	*Nagler*: 256 experiments on different pier shapes, 34 models
1934	*Yarnell*: 2600 experiments, pier nose shape, tail shape, length and angle of skew
1953/5	*Kindsvater & Carter*: channel blockage, cross section shape & roughness - USGS method
1957	*Liu, Bradley & Plate*: normal depth, opening discharge ratio - Colorado State University
1960	*Bradley*: hydraulics of bridge openings - FHWA method
1962	*Biery & Delleur*: single span arch bridge
1973/8	*Bradley*: - USBPR method
1973	*Neill*: book - 'Guide to bridge hydraulics', University of Toronto Press, Canada
1977	*Knepp*: spur dikes, abutment shape, Froude No, FHWA, USGS & HEC-2 methods
1985/7	*HR Wallingford*: arch bridges - HR method
1991	*Breusers & Raudkivi*: book - Scouring, Hydraulic Structures Design Manual, No 2, IAHR
1995	*US Corps*: 1D & 2D modelling, revised HEC method
1997	*Hoffmans & Verheij*: book - 'Scour Manual', Balkema
1999	*Hamill*: book - 'Bridge Hydraulics', E & FN Spon
2000	*Melville & Coleman*: book - 'Bridge Scour', Water Resources Publications, LLC
2002	*Wu & Guo*: Choking flow phenomenon in lateral contractions, Beijing Flood Conference.

The extent and variety of the work listed in Table 1 indicates the abiding interest and importance of this topic among river engineers, as well as indicating the lack of any definitive answers to the many problems associated with bridge hydraulics. Indeed, it is possibly because the problem of afflux is such an old one, and therefore regarded as being an 'old chestnut', that many engineers imagine that all must be known about this topic by now and that consequently it is only necessary to consult an existing corpus of knowledge and literature for design information. However, as will be shown herein, there is plenty of work still to be done on this topic and new saplings have arisen in recent years that allow us to re-interpret previous work and to relate it to our current understanding of open channel flow hydraulics.

2 PARAMETERS THAT INFLUENCE AFFLUX

Standard textbooks such as Chow (1959), Hamill (1999) and Henderson (1966) contain much background information, as does the Stage 1 (draft) report to the Agency by JBA Consulting Engineers & Scientists (Benn & Bonner, 2001). It is assumed that the reader is familiar with these textbooks and report, and consequently only the conceptual approaches and the implications of selecting certain parameters are discussed herein. Additional reports, such as those by Sheikh (1997), and Atabay & Knight (2002), provide additional information specifically on bridge afflux, as does the paper by Knight & Samuels (1999).

There are certain key parameters that have been adopted by many researchers involved in studying the hydraulic behaviour of water flowing through constrictions. Some of the principal hydraulic variables are shown in Fig. 1, taken from Hamill (1999). In order to facilitate understanding, the same notation is adopted in this report as in the Stage 1 report, the notation being taken originally from Hamill (1999), even though it is not altogether standard or suited for 3-D analysis. These key parameters may be conveniently grouped under two major headings, geometric and flow, although as will be shown later, this distinction is somewhat blurred. They are however presented and discussed briefly here under these two traditional headings.

2.1 Geometrical parameters

2.1.1 Bridge opening ratio, M

The bridge opening ratio, M, is defined as a ratio of discharges, $M = q/Q$, where q = unfettered flow through the gap as if the bridge or constriction was not in place and Q = total flow in the river. Thus $M = 1.0$ when there is no effect arising from a constriction, and $M < 1.0$ when there is an effect. See Fig 1(a). The bridge opening ratio should be distinguished from the blockage ratio, m, commonly defined as $(1 - q/Q)$, giving $m = 0$ when there is no blockage, $m = 1.0$ for full blockage. It should be noted here that in evaluating q, the depth is usually taken to be the normal depth in the river, Y_n, and the width, b, is associated with that particular portion of the river occupied by the bridge opening. The value of M is therefore strictly dependent on the flow distribution within the river channel, which in turn is generally influenced by the geometric shape of the river channel, the roughness distribution within it and the approaching flow conditions upstream. Thus the proportion of total flow occurring in a given width, located at an arbitrary lateral position within a river cross-section, is complicated by the 3-D nature of the flow distribution, as illustrated either by standard river flow hydrometric gauging procedures or by methods for determining the lateral variation of depth-averaged velocity and boundary shear stress across rivers (Knight, 2001). Technically M should therefore be regarded as a ‘combined’ flow/geometric parameter, and not as a purely geometric one.

Most ‘standard’ textbooks dealing with the issue of afflux do not consider the 3-D nature of the flow, and assume that in a simple rectangular channel, M is simply equivalent to the width ratio, b/B, (i.e. $M = b/B$), where b = waterway opening and B = total channel width. It should be noted that even for a rectangular channel, this is never strictly true because of the boundary layers that develop both from the sides and from the bed of the channel. The value of M would vary from a minimum when b is located near the walls, to a maximum when b is located at the channel centreline. The numerical values would also vary with depth (i.e. discharge), as the aspect ratio of the channel changes from a large value at low depths to a lower value at high depths (discharges). In the case of a compound channel M would correspond to the percentage of the total flow occurring in the main river channel, $\%Q_{mc}$, i.e. excluding the flow on the floodplains. For those cases where there are significant depth variations within the main channel or constricted zone, the theoretical determination of q becomes particularly problematic when based on flow proportioning rather than simple area proportioning. It thus appears that M, one of the most important geometric parameters or geometric/flow parameters adopted by previous researchers, needs some revision in the light of our knowledge and computational ability to compute 3-D flow distributions.

Since the alignment of the flow with the bridge/channel boundary is important, some consideration must be given to the macro flow field, as well the local flow field immediately adjacent to the bridge or culvert. The effect of the macro flow field is traditionally simulated somewhat crudely by two parameters, the eccentricity, e, and the skewness, ϕ, whereas the very local or micro flow field is traditionally simulated by a rounding parameter, r, or chamfering parameter, w.

2.1.2 Eccentricity, e

To some extent the eccentricity parameter, e, which equals the ratio of the bridge abutment lengths intruding from either side of the channel, X_a/X_c, is one traditional attempt at accounting for some of the 3-D effects of flow distribution on the coefficient of discharge. See Fig. 1(e). The eccentricity, e, was originally regarded as a purely geometric parameter (e.g. Kindsvater et al, 1953), but later e has been assumed by some (e.g. Matthai, 1967) to be equal to the ratio of flow discharge occurring either side of the bridge opening, Q_a/Q_c, in direct proportion to the specified lengths, in a somewhat similar way to the parameter M. However, from the comments made in 2.1.1, it should be clear that the link between discharge and length is not a simple linear one, and will also vary with depth in both simple and compound channels.

Experimental work in rectangular shaped channels by Kindsvater & Carter (1955), Matthai (1967) and Bradley (1978) indicate that when $e > 0.12$-0.2, or when one embankment length is less than 5 to 8 times the other, there is little reduction in discharge performance. For $e = 0$, the USGS method states that the discharge reduction is only around 5%. However, it should be noted that the eccentricity parameter, e, does not strictly deal with all possible geometries and flow conditions. For example, it is possible to have $X_a = 0$ in Fig. 1(e), but different values of X_c, all giving $e = 0$. In the case of $X_a = 0$, i.e. when there is no embankment on one side of the channel, the vena contracta effect would be suppressed on that side, leading to a smaller contraction laterally and therefore an increase in discharge, provided X_c is not too large. However, when $X_a = 0$ and X_c is large, or even for some very small values of e, the flow from one side towards the gap would cause a very large contraction, reduce the discharge capacity, and might even be sufficient to produce supercritical flow locally. It would therefore seem sensible to redefine the eccentricity in such a way that introduces the ratio of gap width to embankment length, b/X_c or b/X_a in addition to the ratio X_a/X_c. It should be noted that e is usually defined in such a way that it never exceeds unity, the longer embankment length always being placed in the denominator.

In the case of flow in prismatic channels with two floodplains, the link between proportionate discharge and floodplain width will be especially complex, due to the lateral exchange of momentum between the main river channel and its associated floodplains. At flow constrictions, where typically embankments extend across the full width of both floodplains, the momentum exchange is further complicated by the flow spilling off the floodplain upstream of the bridge and onto the floodplain downstream of the bridge. These flow exchanges, which also occur in compound channels with converging or diverging floodplains, will directly affect the momentum-force balance and introduce additional energy losses. The theoretical determination of *e*, based on flow proportioning, is therefore especially difficult for overbank flow in natural channels with floodplains.

2.1.3 Skewness, ϕ

For cases where the bridge crossing is not normal to the predominant flow direction, an angle of skew, ϕ, is traditionally introduced, as shown in Fig. 1(f), where $\phi = 0^o$ when the embankments are perpendicular to the flow. As illustrated in Fig. 2, the gap width normal to the mean direction of river flow is then given by $b = b_s \cos\phi$, where b_s is the span width of the opening between the abutments of a skewed bridge, as measured along the highway centreline. It should be noted that the width b will have an impact on the determination of q (and hence M), and again highlight the confusion between geometric and flow parameters in traditional approaches. The alignments of the ends of the abutments also become important, as illustrated in Fig. 2, for skewed opening types 1, 2 & 3.

In the case of a skewed opening of type 1A, the water level will be higher on the left-hand side of the channel (looking downstream), since a dead zone will develop there. A second dead zone, larger than the upstream one, will occur on the right-hand side of the channel immediately downstream of the right-hand embankment. Since the water level in the upstream dead zone on the left-hand side will generally be higher than the average water level in the main river channel upstream, due to the recovery of the velocity head, it may therefore have consequences for the onset of submergence, as this generally occurs on the side that has the higher upstream water level.

The hydraulic efficiency of the waterway crossing is affected by skewness in a number of ways. Although the span width, b_s, increases with ϕ, the flow now has to turn through an angle to enter the gap. Furthermore, the orientation of the ends of the abutments may cause additional eddying via separation, as well as contraction effects downstream, if the alignments of the end faces are unsuitable. See Fig. 2, in which b = 10m for the normal opening ($\phi = 0^o$), as well as in 1A and 2A, giving b_s = 14.1m for $\phi = 45^o$ in both 1A and 2A. The flow through skewed openings is therefore complex. In 1A and 2A the discharge will increase relative to the discharge through the normal opening, but not in direct proportion to b_s/b, arising from the effects of changing the direction of the flow normal to the waterway opening, b_s, as well as the effects of separation due to possible differences in the alignment of the ends of the abutments. In 1B and 2B there will be a corresponding decrease in discharge relative to the discharge through the normal opening, but again not in proportion to b/ b_s. It should be remembered that it is not uncommon for a river course to change its direction during a flood, and therefore the alignment of the bridge abutments and mean skew angle of the river flow to the embankments will vary and will need to treated with care.

Experimental results by Matthai (1967) indicate that the effect of skewness is most significant when ϕ is around 45°, producing a possible 30% reduction in discharge. Once a crossing is fully submerged, the influence of skewness is however reduced, as the opening then behaves more like an orifice, with the flow separating strongly from the underside of the bridge (soffit or deck) causing the flow to contract strongly in a vertical plane rather than a horizontal plane. In these circumstances the discharge capacity is then related to b_s, the width of the opening between abutments, and less dependent on ϕ. It thus appears that the 'effective discharge width' of a skewed crossing is not always as obvious as it seems, and care should be taken in evaluating the 3-D flow field under free or submerged conditions. As always in hydraulics, a good understanding of fluid flow behaviour and a good visual memory bank of different mechanisms, are pre-requisites for sound analysis or sensible application of software.

2.1.4 Rounding of piers, entrances to bridges or culverts

The introduction of a corner radius, *r*, or a chamfer length, *w*, into a waterway crossing is illustrated in Fig. 1(d). It is known that the upstream rounding of piers and entrances to both bridges and culverts significantly influences the flow characteristics of such structures, by preventing or delaying fluid separation at otherwise sharp changes in geometry. Small local changes, such as rounding or chamfering, will therefore generally reduce the contraction effect, increase the discharge capacity, and this increase is usually related to the parameter M. The increase is particularly large when the structure is operating under submerged conditions, i.e. orifice flow. The bridge opening ratio, M, is usually calculated for simple geometrical shapes, such as arch bridges, by assuming that the flow is wholly normal to the structure and that the flow area is identical to the geometric area. On this basis equations for the variation of M with depth for various types of arch bridge may be produced (e.g. Hamill, 1999).

The influence of r/b and w/b on the discharge coefficient, and hence the discharge capacity, is given by Kindsvater et al. (1953), Kindsvater & Carter (1955) and Hamill (1997). The USGS results indicate that the greatest increase in discharge capacity through entrance rounding will occur in narrow waterways (e.g. discharge increase of 20% for $M = 0.2$ & $r/b = 0.14$). As in all hydraulic flows, the streamlining of the macro and micro flow fields is important for enhancing discharge capacity and reducing losses. In the context of bridge crossings, streamlining is especially difficult to ensure throughout the full depth range, as the direction of the approaching flow may change significantly as the floodplains become inundated upstream of the bridge.

2.1.5 Length to breadth ratio, L/b

The length to breadth ratio, L/b, of a waterway crossing is shown in Fig. 1(c), where L is the length between the upstream and downstream faces of the constriction, or waterway length, and b is the width of the bridge opening or a characteristic height dimension of the culvert. The length to breadth ratio, L/b, is important for distinguishing between bridges and culverts, and common practice dictates that a culvert is defined by having an integral invert and $L/b > 1$, enabling it to behave hydraulically as a pipe with 6 main categories of flow (Chow, 1959; IWEM, 1989; CIRIA, 1997). The length is important in determining where re-attachment of the flow to the sides or soffit of a bridge takes place, which generally enhances the discharge capacity. The USGS results indicate that the increase in discharge is strongly related to L/b and M values and may be as high as 30%. A large culvert is taken here to mean $b > 1.5$m (or an equivalent height or diameter), and provided $L/b < 1$ it will behave in a similar manner to a bridge.

2.1.6 Channel shape

For a single waterway opening, the cross-section shape of the river channel is important as it will affect the length of piers subjected to drag resistance, the choking behaviour, the kinetic energy correction coefficient values, as well as the determination of Q, the total conveyance capacity, q, the zonal conveyance capacity corresponding to the width b without the bridge present, the normal and critical depths of flow, etc. Although these considerations are important, it should be remembered that in many cases, even without an integral invert, the shape of the waterway opening is often configured into a relatively simple shape by the construction process. The customary convergence and acceleration of the flow towards a constriction generally helps to re-distribute the velocity and homogenise the turbulence. In many cases, such as in critical depth flumes and weirs, new boundary layers are assumed to begin once the throat section is reached. The turbulence characteristics of accelerating and decelerating flows in non-prismatic channels are unknown. Diverging flows, such as those that occur immediately downstream of bridges, are particularly difficult to deal with from a theoretical perspective. Arch bridges will generally affect the flow more than those with rectangular openings, due to the separation and contraction effects produced by the upper surface as the depth of flow reaches the crown.

The traditional method for dealing with the channel shape and 3-D influences in standard 1-D open channel theory is to account for it by including kinetic energy and momentum correction coefficients. However, these are unlikely to be appropriate when dealing with complex flows through bridges. The vertical and lateral distributions of velocity within a river channel are clearly important when attempting to determine drag forces and resistance parameters. The boundary shape and the 'no slip' condition are in fact the boundary conditions required for the solution of any flow field. This again emphasises the link between geometric and flow parameters, and why in essence they should all be treated under a unified approach. Since most of the experimental work on bridges and culverts has been conducted in channels of simple prismatic shape, generally rectangular or trapezoidal, there is a need to break away from this traditional approach and to investigate more realistic cross-sectional shapes. The most obvious one is to consider bridges in natural channels with floodplains. A considerable number of bridges in the UK fall into this category (e.g. see Pool bridge, River Wharfe, given as Fig. 1 by Knight & Samuels, 1999), in which the general river shape is a compound channel and the embankments are perched higher, giving multiple openings at different invert levels. Apart from the experiments at the University of Birmingham, summarised in a separate report (Atabay & Knight, 2002), there is a dearth of investigative work on bridge crossings in compound channels. A large corpus of knowledge now exists on flows in compound channels, arising from EPSRC supported work in many UK universities and through the Flood Channel Facility at HR Wallingford (Knight, 1992, Knight & Shiono, 1996). Simply relying on all the previous empirical evidence gained mainly from studies in rectangular channels, and listed in Table 1, is perhaps one of the greatest weaknesses in current design practice.

2.1.7 Multiple openings

For a waterway crossing with multiple openings, the division of flow has to be determined or assumed prior to any analysis. Where there are multiple openings with different invert and drowning characteristics, this will be problematic. As already commented upon in 2.1.6, the answer lies in estimating the flow field, by 'extended 1-D' analytical methods that give the lateral distribution of the approach flow in the streamwise direction. Alternatively, the flow distribution may be estimated using 2-D depth averaged models that give both the magnitude and the direction of the approaching flow, or by using full 3-D CFD modelling.

2.1.8 Blockage

One remaining geometric issue governs the shape of the opening, namely the blockage. How this is conceptualised will clearly influence the discharge characteristics and any additional head loss arising from debris. Blockage may occur through floating debris trapped against the deck or soffit, trash against piers, ice accumulation, siltation or other extraneous sources, such as cars and caravans. Modelling these by simply deceasing the waterway area is clearly not always appropriate, and other methods should now be attempted such as those that proportion the area to the floating debris (i.e. varying the area of blockage from the water surface downwards), those that include vertical or horizontal strips of blockage, those that consider random blocked area patches and those that mimic siltation by reducing the area from the bottom upwards. Care needs to be taken in recognising the impact that these different geometric inserts might have on the hydraulic behaviour and functioning of the structure. Some attention should also be paid to collecting evidence of where bridges have failed due to debris accumulation, impact forces (overturning moments) and scour. There is very little experimental data on measured forces on bridges with which to check any momentum-force balance approach, and what is available often relates to specific case studies, such as required in low cost bridge crossings in India or Australia (e.g. Roberts, Freer-Hewish & Knight, 1983). Although various software purport to simulate blockage, there is no validation evidence known to the author to support such numerical results.

2.2 Flow parameters

2.2.1 Types of flow

A fundamental distinction needs to be drawn between free flow, submerged flow and drowned flow through bridges, as illustrated in Fig. 3. At low discharges, with the channel controlling the flow, the flow is referred to as an open channel type flow, as shown for a subcritical state in Fig. 3(a). When the discharge and upstream water level reach a value at which submergence of the upstream face occurs, then the hydraulic behaviour switches from a free flow state to a sluice gate-like flow state, as shown in Fig. 3(b). If re-attachment occurs on the underside of the bridge soffit, then the contraction effect within the bridge opening will be affected, typically causing the whole area at the outlet to become a flow area and thereby changing the longitudinal pressure gradient, as shown in Fig. 3(c), type 2. The flow is now referred to as a drowned orifice type flow, with both the structure and the channel controlling the flow. Depending on the tailwater level, the downstream face of the structure might also become submerged, as shown in Fig. 3(d), type 1. At very large discharges, some flow may also occur over the bridge parapet or roadway, leading to overtopping or bypassing of the structure. In this case the flow is part orifice-like, through the bridge opening, and part weir-like, over the bridge parapet and any adjoining embankments. Thus during a flood event, the simulation of these different types of flow must be reproduced by the numerical model, with different algorithms and clear distinctions made between them at key water levels relative to the structure. One of the weaknesses in a previous benchmarking study on bridge afflux undertaken by the Agency (1995) was that this was not done. In that study, purportedly undertaken to validate software, a single discharge was used for a hypothetical bridge for which there was no verification data.

The case of supercritical flow through a waterway is less common and not dealt with in detail herein. However, should supercritical flow occur, then bridges have to be treated very carefully as the depth of flow will increase in the constriction (i.e. the opposite to that shown in Fig. 3(a). Under these circumstances the water level may reach the soffit, and then a hydraulic jump is likely to form on the upstream side of the bridge, thereby creating a new control point. Where bridges are sited in supercritical flow on bends (e.g. Blaenau Ffestiniog), then splitter walls may be required to reduce the super-elevation so that the water surface does not reach the level of the bridge soffit. The 6 types of flow that can occur through culverts (Chow, 1959, IWEM, 1989, CIRIA, 1997) are likewise not discussed in detail herein, as these are peripheral to the main issue being addressed, that of afflux at bridges with a L/b value of less than 1.

Other ways of categorising flows through bridges are annotated in the USBPR method, based on the work of Bradley (1978). Since this is one of the most widely used for determining the afflux through a constricted waterway, this is summarised as follows:

Type I	subcritical flow throughout a constriction
Type II	Type II A (critical at throat only, i.e. choking at throat)
	Type II B (critical for a short distance downstream, then a jump and subcritical flow further downstream)
Type III	choking at throat, then supercritical flow downstream

The different flow mechanisms that occur in different flow regimes means that knowledge from sources other than bridge experiments have something to offer. For example, vertical sluice gates, Tainter gates and other similar structures also offer clues about drowning characteristics, since these structures are not dissimilar to the upstream conditions of certain bridge types. See Escaramia et al. (1993) and Knight & Samuels (1999) for selected references on this particular topic. Furthermore, contraction and expansion phenomena also occur in many other types of fluid flow, and it is suggested that any subsequent literature search in Stage II of the Scoping study be widely based. Turbulent shear flows and boundary layers form two key categories of fluid flow, and a substantial number of textbooks are devoted wholly to these particular topics, e.g. Cebeci & Bradshaw (1977), Reynolds (1974) and Schlichting (1979).

2.2.2 Froude number

Sub and supercritical flow are defined in relation to the Froude number, *Fr*, which is traditionally defined in terms of the gross parameters related to the cross section geometry ($Fr^2 = Q^2T/(gA^3)$, where T = top width at water surface). The inclusion of the kinetic energy coefficient, α, is not recommended as it again confuses geometric and flow concepts. Although common in the literature, being a traditional attempt at trying to represent what is essentially 3-D flow as 1-D flow, it is technically incorrect and wholly inappropriate when dealing with critical flow in channels with a complex geometry. See Yuen & Knight (1990) and Blalock & Sturm (1980) for further details. It is not uncommon in rivers with floodplains to have sub and supercritical flow existing within the same cross section.

The Froude number is important, not only in distinguishing between different types of flow, but also because it occurs in certain types of afflux equation based on the momentum approach (e.g. in Eq. (9) shown later).

2.2.3 Reynolds and Weber numbers

The Reynolds number, *Re*, is generally of less importance than the Froude number, *Fr*, except when interpreting small-scale experimental results. The drag forces on small experimental piers are known to be subject to scale effects, and some of the early experiments listed in Table 1 will be influenced by this consideration. Small-scale experiments are also influenced by surface tension effects, which are governed by the Weber number, *We*. There is some evidence (Escaramia et al., 1993; Knight & Samuels, 1999) that *We* influences the contraction coefficient for sluice gates.

2.2.4 Choking

Although not strictly a flow parameter, the type of flow through a constriction may be dominated by the choking phenomenon. Since the specific energy, depth and discharge per unit width relationship is theoretically fixed for a particular shape of cross section, the flow within a constriction may become critical if the gap width is too small. Further reduction in the gap width will only maintain the hydraulic control at that point, but cause the water level upstream to rise significantly. This choking phenomenon is therefore one aspect of afflux behaviour that needs to be considered as a related topic.

Standard hydraulic theory applied to flow in a rectangular channel of breadth B, in which a pier of width b is placed, will lead to the equation for choking

$$\sigma^2 = \frac{27Fr_1^2}{\gamma^3(2+Fr_1^2)^3} = M_L^2 \qquad (1)$$

where σ = (B-b)/B, Fr_1 is the upstream Froude number and γ is a factor to account for any energy loss between the upstream section (1) and the throat (2), such that $\beta E_1 = E_2$, where γ < 1 and E refers to the specific energy. It should be noted that despite the use of b as defined here for the pier width, and not the gap width, σ becomes the same as the bridge opening ratio, M. Thus for known B, Fr, and γ, the value of σ, and hence the width of the bridge pier, b, or limiting opening ratio, M_L, that will cause choking may be determined. Eq (1) is similar to that presented by Hamill (1999), but with the energy loss term included. Many more complex equations may be produced based on similar principles (e.g. Wu & Molinas, 2001; Wu & Guo, 2002).

Eq (1) also highlights the conceptual difficulty of defining appropriate cross sections between which any analysis should take place. Too often assumptions are made about the hydraulic behaviour, and quite arbitrary sections selected for no other reason except for that is always been done that way in the past. Current practice is not always a guide to best practice. It is partly this mindset, as well as a lack of going back to first principles, that has probably led to confusion over distinguishing between afflux and head loss at a bridge.

2.3 Definition of afflux

Fig. 4 shows one traditional cross section numbering scheme adopted for constricted waterways, with cross-sections 1-4 positioned at strategic places for analysis (as in HEC-RAS R&D, 1995, but with reversed notation to make it compatible with standard practice and the USBPR method). Section 1 is traditionally taken upstream of the bridge, prior to the commencement of any contraction of the streamlines due to the bridge. Section 2 is generally taken on the upstream face of the bridge, with Section 3 either on the downstream face or at the position of the vena contracta. Section 4 is taken some distance further downstream, typically at the end of the expansion of the streamlines where the flow returns to normal depth conditions in the river channel. Fig. 5 shows a similar numbering scheme for flow past bridge piers, taken from the companion scoping study report on current practice for afflux estimation by Mott MacDonald (Kirby & Guganesharajah, 2001). Figs 6 & 7 show the corresponding conceptual longitudinal water surface profiles through such constrictions, as envisaged by Chow (1959) and Hamill (1999) respectively. Fig. 8 shows an actual water surface profile through a constricted bridge crossing in a compound channel, taken from Atabay & Knight (2002). Figs 9-13 show some actual water surface profiles through different model bridge types, taken from Atabay & Knight (2002), and Figs 14-16 show some actual bridges during flood events. Figs 9-11 show open channel type flow, corresponding to Fig 3(a), whereas Fig.

14 shows sluice gate type flow, corresponding to Fig 3(b), and Fig. 13 shows orifice type flow, corresponding to Fig 3(c).

It will be noted that in the vicinity of a bridge the flow is rapidly varying, as shown by Figs 6-8. The water surface is raised above its normal level upstream, but dips below the normal level downstream before recovering to the normal depth value as dictated by the channel conveyance capacity. In the immediate vicinity of a constriction, the flow area contracts, increasing the kinetic head, thus lowering the water surface. Downward momentum and continuing lateral contraction downstream of the obstruction sometimes accounts for the 'overshoot' of the water surface. The length of the constriction, and the L/b ratio, are important here. Separation is illustrated in Figs 10 & 11 for an elliptical and a twin arch semi-circular model bridges with no rounding on the upstream faces. This tends to accentuate the contraction effect, both lateral and vertical. Because of the rapidly varying water surface elevation, it is obvious that precise water levels on the upstream and downstream faces (sections 2 & 3) vary considerably according to the particular structure. Although these water levels are easy to measure in practice, they are a generally poor guide to either energy loss or afflux.

The afflux is defined strictly as 'the maximum difference in elevation of the water surface, at a location upstream of the structure, with and without the structure', i.e. $Y_1 - Y_n$ in Fig. 7. It should be noted that this is different from the energy loss across the structure, $H_1 - H_4$, a popular misconception among river modellers. A prerequisite for determining afflux is therefore a method for calculating the normal depth, $Y_{n,}$ in a channel of arbitrary shape, which is not as straightforward as it seems once 3-D effects are included. See for example a previous scoping study report for the Environment Agency on 'Conveyance capacity in 1-D river models' by Knight (2001). The precise location of Section 1 for calculating afflux, or position 4 for calculating head loss, or indeed intermediate sections like 2 & 3 for related analysis, all require thought before applying either the momentum or the energy equation.

3 CONCEPTUAL APPROACHES TO DETERMINING AFFLUX

3.1 Theoretical background

The analysis of flow through a bridge or under a sluice gate has been restricted here to that in a rectangular channel, in order to illustrate the application of the two basic theoretical approaches. In this simplified and illustrative analysis, only 3 cross-sections are specified, upstream (1), somewhere within the constriction (3), i.e. at the throat or place of maximum contraction, and downstream (3), as given elsewhere by Knight & Samuels (1999).

The momentum conservation principle gives

$$\frac{1}{2}\rho g Y_1 (Y_1 B) - \frac{1}{2}\rho g Y_3 (Y_3 B) - F_D = \rho Q (V_3 - V_1) \tag{2}$$

where F_D is a drag force, which has to be defined independently. Typically F_D is given by

$$F_D = C_D \frac{1}{2}\rho V_1^2 (J_1 B Y_1) \tag{3}$$

where J_1 = blockage ratio = (blockage area at depth Y_1)/(BY_1) (4)

Alternatively, the energy conservation principle gives

$$Y_1 + \alpha_1 \frac{V_1^2}{2g} = Y_3 + \alpha_2 \frac{V_3^2}{2g} + \Delta E \tag{5}$$

where α = kinetic energy correction coefficient and ΔE = energy loss across bridge (not afflux, which is ΔY {= Y_1 - Y_3}), which again has to be defined independently by ancillary equations. Typically inlet and outlet losses are given by

$$\Delta E = k_i \frac{V_2^2}{2g} \quad (6) \qquad\qquad \Delta E = k_o \frac{(V_2 - V_3)^2}{2g} \quad (7)$$

The equation for conservation of mass (continuity) is

$$Q = Y_1 B V_1 = Y_3 B V_3 \tag{8}$$

Equation (8) may be combined with either Equations (2) & (5), along with the ancillary equations for drag force or energy loss, to give the afflux across the structure. For example, the result of combining the momentum and continuity equations gives an equation for the bridge afflux, ΔY, which shows that it depends on the downstream Froude number, Fr_3, the upstream blockage ratio, J_1, and a drag coefficient, C_D. Neglecting the $(\Delta Y/Y_3)^3$ term, which generally is small, gives an equation for the afflux, ΔY, in the form

$$\frac{\Delta Y}{Y_3}=\frac{\left\{Fr_3^2-1\right\}+\left[\left\{Fr_3^2-1\right\}^2+3C_D J_1 Fr_3^2\right]^{1/2}}{3} \tag{9}$$

An iterative procedure is required to solve Eq. (9), since J_1 is used instead of J_3, thus requiring knowledge of the upstream water level. In other words the afflux is implicit in Eq (3), since the upstream water depth is used instead of the normal depth downstream, thus accounting for the iterative procedure. It will be appreciated that the empirical coefficients, C_D, k_i & k_o, in Equations (3) (6) & (7) are not simple parameters, but depend on many other coefficients, dealing with pier shape, bridge opening ratio, eccentricity, skewness, soffit clearance, embankment alignment, scour and local details.

In order to illustrate some of the commonly adopted assumptions used when applying the energy method to flow through constrictions, the approach given in Chow (1959) is reproduced here, as it forms the basis of the USGS method. This method was primarily concerned with using backwater effects to gauge flows, a use which is still relevant today (Archer, 1990). The discharge at the vena contracta, regarded as Section 3 in Fig. 6(a), is given by

$$Q=C_c A_3 V_3 \tag{10}$$

where C_c is a coefficient of contraction and A_3 is the flow area at that position. By applying the energy principle, the discharge may be related to the maximum water level difference, Δh, across the constriction, as given below and defined in Figs 6(b) & 6(c).

$$Q=CA_3\sqrt{2g(\Delta h-h_f+\alpha_1 V_1^2/(2g))} \tag{11}$$

Note here that the head loss due to channel friction, h_f, is also included in the analysis. The afflux, shown as H_1^* in Fig 6(b) may thus be determined provided H_3^* is known. The general coefficient C, may be related to C_c by the ancillary equation

$$C=\frac{C_c}{\sqrt{(\alpha_3+k_e+k_p}} \tag{12}$$

where k_e relates to energy losses in the upstream separation zones ($h_e=k_e\left(V_3^2/2g\right)$), and k_p relates to deviations form hydrostatic pressure conditions. The general coefficient, C, is given by

$$C=C^{'}K_F K_r K_W K_\phi K_e K_x K_y K_j K_t \tag{13}$$

where the $C^{'}$ is the standard value of the coefficient of discharge and the various K factors adjust it for skew, eccentricity, etc. The effect of the various geometric parameters is thus deemed to be multiplicative. Further details are given in Chow (1959) and Hamill (1999).

The afflux, H_1^*, is then determined by an iterative procedure based on the difference in water levels between sections 1 & 3 (See Fig. 7), using the relationship

$$\Delta h = H_1^* + H_3^* + h_f \qquad (14)$$

where the three remaining terms, Δh, H_3^* and h_f have to be determined by empirically derived ancillary equations or graphical relationships.

3.2 Methods for determining afflux

Various methods for determining afflux have been devised, the main ones being as follows:

- Energy equation method (various authors)
- Momentum equation method (various authors)
- USGS method
- USBPR method
- FHWA WSPRO method
- HR method (arch bridges)
- Biery & Delleur method (arch bridges)
- Pier loss methods - Yarnell, Nagler and d'Aubuisson

Although it is not appropriate to reproduce details of these methods here, since they are well documented elsewhere, it should be appreciated that there are several conceptual differences between them, so that the affluxes computed by the various methods will differ according to the method used, the coefficients adopted, and the expertise of the user in applying the method to both standard and non-standard situations. It is suggested that a thorough review of these methods be undertaken, including a critique of their theoretical bases and their underlying assumptions, both implicit and explicit. The review should also include a section on how our current knowledge of 3-D flow behaviour might enhance our understanding of what is essentially a group of 1-D methods. Some of the conceptual difficulties have already been highlighted in Sections 2.1 & 2.2 when discussing various geometric and flow parameters.

3.3 Critique of 1-D methods for determining afflux

The USGS method is based on extensive laboratory and field measurements for flow through waterway openings with a rectangular cross-section. It was primarily developed as a method for gauging flows at structures, and therefore relates conditions upstream (1), normally taken at a distance upstream of one span opening, b, to those at the vena contracta (3), normally taken to occur between the bridge piers or some way downstream. When used as a method for determining afflux, it relies wholly on two empirically determined relationships, one for the backwater ratio, H_1^* /Δh, which depends on the opening ratio, M, and the channel roughness, and another for the adjustment factor, k_c, which also depends on M. These 'geometric' parameters, are again really 'flow' parameters which are required in order to eliminate the term H_3^* in Eq. (14). The method also relies on knowing the precise conditions at the vena contracta (3), as the geometric parameters there are used in the determination of the Froude number, Fr_3, which features elsewhere in the methodology. For openings that are non-rectangular, or those with unusual velocity fields, this is problematic. The extension of the method to arch bridge types and flows in compound channels is likewise going beyond the bounds of sensible extrapolation. A more radical approach is therefore needed to account for the more complex flow conditions pertaining to these situations.

The USBPR method, which is non-iterative, and therefore simpler to apply than the USGS method, is based on the energy relationship applied between sections 1 & 4. The afflux, H_1^*, is then based on the normal depth at section 2, with the loss due to the bridge, h_b, expressed in terms of a standard kinetic head term (i.e. $h_b = k^*(\alpha_2 V_{N2}{}^2/(2g))$, with qualifying coefficients to account for all the geometric parameters that might influence k^*. The method also relies heavily on the kinetic energy correction coefficients, α_1 and α_2, at sections 1 & 2, as well as the channel areas at sections 1 & 4, as all of these appear in the basic afflux equation, often given as:

$$H_1^* = k^*\alpha_2\left(\frac{V_{N2}{}^2}{2g}\right) + \alpha_1\left[\left(\frac{A_{N2}}{A_4}\right)^2 - \left(\frac{A_{N2}}{A_1}\right)^2\right]\left(\frac{V_{N2}{}^2}{2g}\right) \tag{15}$$

In Eq. (15), the subscript N2 refers to the normal depth condition at section 2, taken to be the upstream face of the bridge opening, perpendicular to the mean direction of flow. Bearing in mind the comments made earlier about flow distribution in channels, the simple USBPR monogram presented by Bradley (1978) is unlikely to cover as wide a range of cases as might be experienced in practice. Kinetic energy correction coefficients are known to vary considerably.

The HEC-RAS method (1995), is based on conditions between sections 1 & 4, and therefore relies heavily on estimated contraction and expansion loss coefficients for the flow between sections 1 & 2, and between 3 & 4 respectively. It also includes an attempt at accounting for the flow distribution and some 3-D effects through the use of a 2-D depth-averaged model. However, much the same criticisms that are made of the USGS and USBPR methods may be made here. In particular, these loss coefficients were estimated from a 2-D model, applied to a non-typical compound channel with a very narrow main channel, and without any reference to the wealth of literature on the flow characteristics and physics of flow in such types of channel (Anderson et al., 1996). Moreover, the computed results for the coefficients, based on a USA Masters thesis, show considerable scatter. Consequently, the coefficients must be treated with caution. For flow in simple shaped channels, the HEC-RAS method does include orifice and overtopping flow (Samuels, 2001), but, like other methods, without the combined flow being verified on the basis of laboratory studies.

The methods aimed at determining afflux specifically at arch bridges (Biery & Delleur, 1962; Brown, 1988), are useful for the many types of arch bridge that exist in the UK. However, masonry arch bridges are common in many countries and therefore these methods will find use in many different contexts. The HR method, based on the momentum approach, suffers from the fact that the coefficient of drag in Eq. (3) is pier based and other drag forces are needed in Eq. (2) to account for frictional losses as well as lateral momentum exchanges between any floodplains in the contraction and expansion phases. These might be included in Eq. (2), based on the extended SKM or EDM methods. A further criticism is that all the experimental work undertaken at HR, upon which this method and algorithms within ISIS are based, was restricted to flow in simple rectangular shaped channels, with the arch bridges always aligned perpendicular to the flow, and in the case of multiple opening experiments, with the inverts of all the openings at the same level. This is not typical of most river channels, where the bridge either occupies the main channel and smaller openings are on the floodplain, or the bridge spans a river with varying depth so that the openings are of different size and elevation. There is much that could obviously be done, both experimentally and computationally to rectify this situation, but such work would have to be justified on a cost/benefit basis. The Biery & Delleur method is simple to apply, but less well founded experimentally than the HR method. It suffers from much the same criticism as made of the HR method, i.e. the lack of experimental data for compound channels or unusual opening geometries and is also heavily reliant on the Froude number.

Those methods based on the early experimental work of Yarnell, Nagler and d'Aubuisson have found use predominately in later developments and in those methods specifically based on the energy approach. Without such early work, the influence of the many geometric coefficients described in 2.1 would still be unknown.

4 STRATEGY FOR IMPROVING KNOWLEDGE

4.1 Knowledge gaps

It is likely that 1-D methods of analysis will continue to be employed in river engineering due to their simplicity in use and undoubted relevance in certain situations. However, the limitations of 1-D methods for analysing some open channel flow problems are becoming increasingly apparent, especially when flow variations are required within the channel. It is suggested that some of the more useful 2-D approaches, that are based on depth-averaged velocity be applied to flow in rivers to determine q and Y_n in particular. Of these methods, the coherence method, COH, (Ackers 1993), the lateral division method, LDM, the Shiono & Knight method, SKM (Knight & Shiono, 1996), the exchange discharge method, EDM (Bousmar, 2002; Bousmar & Zech, 1997) are the most promising. The EDM and SKM methods offer a lot in terms of basic conveyance calculations and distribution of flow parameters across the cross-section of a river. Full 3-D methods, such as based on CFD codes might also be useful, but caution is advised on their applicability to cases where the turbulence is non-standard.

Many comments have been made about the confusion between geometric and flow parameters. This confusion must be resolved by a more thorough critique of 1-D methods than that given in 3.2 and by making practitioners more aware of the limitations of the traditional 1-D methods. It is suggested that many 1-D methods may be extended to include some 3-D effects for flows occurring in channels with a complex geometry, in much the same way as advances are being made in conveyance estimates, following decades of work in compound channels. A significant step change in our understanding may now be made by going back to first principles and by using our current knowledge concerning 3-D flow effects in channels of varying shape. In particular attention should focus on the following:

- lateral distribution of velocity or flow in channels of varying shape
- more realistic channel geometries (e.g. rivers with floodplains)
- exchange mechanisms for converging and diverging floodplains (i.e. influence on contraction and expansion losses for embankments on floodplains)
- arch bridges, unusual shapes of opening and lower pier shape
- transcritical conditions (choking), especially in strongly eccentric conditions
- influence of different types of blockage (e.g. surface debris, siltation, partial area, etc.), and how it should be modelled
- full submergence conditions downstream, i.e. drowned flow

Some particular issues, given in a short review paper by Knight & Samuels (1999), are also listed here for completeness:

Knowledge gaps concerning bridges:
- arch bridges, unusual shapes of opening,
- surcharging, orifice type flow with soffit submerged
- overtopping, with combined underflow and overflow
- bypassing, frequent problem in many flood simulations
- abutment slopes and embankment alignment on floodplains
- bridge alignment, debris, scour
- bridge length & soffit type/clearance levels

What is not in standard texts for bridges and related structures such as sluices:

- stage-discharge relationships for overbank flow. See Knight & Shiono (1996).
- definition of critical depth in a compound channel. See Yuen & Knight (1990).
- multiple openings at different invert levels & drowning characteristics
- non standard geometry, both upstream and downstream

4.2 Research & development programme

The practitioner within the Agency and the academic will inevitably view any R&D programme differently. Perhaps the most obvious beginning point is to consider whether afflux in UK rivers with typical UK bridge openings is a significant problem. This could be done by the Agency undertaking a desk study for bridges at a few specific sites where there is a perceived problem, and estimating the afflux by traditional methods. This should cover a suitable flow range that includes all the three types of flow illustrated in Fig. 3. Due allowance should be taken for possible future variability in climate or flow conditions as envisaged 50 years hence. If the estimates are greater than 0.5 m then this suggests that improving afflux equations is worthwhile. To date, the most serious problem related to bridges in flood conditions in the UK has been failure due to scour (e.g. Glanrhyd & Inverness bridges).

Notwithstanding the outcome of the desk study into the seriousness of afflux at bridges in the UK, some attempt should be made at improving the application of existing theories. In order to appreciate the different approaches, and the influence of the various geometric and flow parameters on afflux, it is suggested that a simple spreadsheet aid be developed, possibly with suitable graphics, so that the engineer can obtain a ‘feel’ for the importance and relevance of any terms in any afflux equation. A series of training manuals on river hydraulics should be developed, so that a ‘cultural’ change, as well as a ‘technical’ change, may be induced in engineers engaged in modelling rivers or designing flood alleviation works. This would have particular benefits in improving the calibration of river models, making them more useful in real time forecasting, flood simulations of extreme events and in the production of more accurate flood inundation maps.

With regard to research topics, and any related research programme, these might be dealt with loosely under the headings of theoretical, field and laboratory studies. It is taken for granted, however, that there will be strong interaction between these categories, so that any field and laboratory results will feed into any theoretical and numerical work, and vice-versa. It may well be appropriate to also consider dividing the research programme into ‘targeted’ and ‘strategic’ components, in much the same way as the Agency has recently done for the work on ‘Reducing uncertainty in river flood conveyance’ (HR Wallingford, 2001).

With regard to theoretical work, specific tasks requiring investigation are:

- thorough review of existing methods for determining afflux and head losses at bridges.
- improvement of existing 1-D methods through the inclusion of current knowledge of 3-D effects in river channels.
- Relate q/Q with bridge opening ratios and eccentricity for various standard channel shapes by obtaining lateral distributions of velocity and flow in channels, possibly using the Shiono & Knight method (SKM).
- Relate q/Q with bridge opening ratios and eccentricity for highly irregular channel and

opening shapes by 2-D/3-D modelling.

- investigate use of the exchange discharge method (EDM) for analysing exchange mechanisms for converging and diverging flows on floodplains, and relate this to the contraction and expansion losses for embankments on floodplains.
- repeat Agency benchmarking study (1995) for 1-D models in order to test algorithms within existingsoftware (e.g. (ISIS, HEC-RAS & MIKE11) against a wider range of benchmark and hypothetical overbank flow cases, covering all flow regimes. This could interact with the 'spreadsheet' approach outlined earlier.
- investigate application of 2-D models to skewed bridges and flow through openings of variable geometry in channels with and without floodplains.
- investigate use of 3-D models for bridge afflux studies, concentrating on free surface profiles, separation phenomena, overall head loss and afflux. This will require significant knowledge on turbulence characteristics and turbulent coefficients.
- assess the impact of scour by suitable 'sensitivity' tests and by relating 'actual' scour results to the above topics. Although scour was considered to be outside the remit of the current scoping study, it could have a significant impact on afflux via certain geometric parameters, particularly those related to the shape of the opening.

With regard to fieldwork, specific tasks requiring investigation are:

- review previously collected 1985 data and its analysis (see Sheikh, 1997 and Knight & Samuels, 1999). Much of these data, and that on the ensuing laboratory studies on arch bridges, are held at the University of Birmingham in part paper and part electronic form.
- review data collected since 1985, particularly in the light of the floods in 1998 and 2000.
- identify certain sites (e.g. 6 for bridges, covering illustrative types, and 6 for culverts) for new detailed field studies on a limited number of bridge types. The aim should be to collect that kind of high quality data that is actually needed to resolve afflux problems, and to compare the measurements with numerical results obtained from a parallel modelling study of the same sites.

With regard to laboratory work, specific tasks requiring investigation are:

- review experimental afflux results contained in Atabay & Knight (2001). These data could also form part of any benchmarking exercise.
- obtain experimental afflux results in compound channel geometries (i.e. rivers with floodplains). Large scale (FCF) and small scale (university) flumes to be used.
- multiple openings at different invert levels & drowning characteristics
- arch bridges, unusual shapes of opening and lower pier shape
- transcritical conditions (choking), especially in strongly eccentric conditions
- influence of different types of blockage (e.g. surface debris, siltation, partial area, etc.), and how it should be modelled
- full submergence conditions downstream. i.e. drowned flow
- overtopping, with combined underflow and overflow
- selected topics from those already listed in 4.1
- encorage DEFRA/EPSRC to collaborate with the EA to obtain funding for the re-establishment of the Flood Channel facility (FCF) at HR Wallingford. This would have benefits in terms of strategic links with comparable work that will be undertaken on it for the EPSRC conveyance work. It will also reduce costs for the Agency.

5 CONCLUSIONS

This report has indicated that although the afflux problem is a ‘mature’ one, there are still a number of issues that require further study in order to reduce uncertainty in prediction. It is now possible to contemplate that a significant step might be taken in improving afflux prediction by applying knowledge of 2-D & 3-D flow structures to what are essentially 1-D methods. Such 1-D methods, with these improvements, are likely to go on being used in river engineering software for the majority of river flood risk assessments. Well designed software should take account of all the various coefficients for 'standard' structures, indicating their range of applicability, and how they might vary outside this range, and also how they might possibly apply to more complex situations where the approach flow and channel geometry is non-standard.

The report has also emphasised that, along with any technical advances, a ‘cultural’ change is required among river engineers in their understanding of rivers and the use of commercial software. Modelling open channel flows in rivers and urban drainage systems is still a skill that takes some years to obtain. It is hoped that by a deeper understanding and structured training, the level of expertise within the UK will be improved.

Finally it should be noted that our understanding of afflux in waterway crossings has been largely gained by exhaustive (and expensive) laboratory work by many dedicated researchers in a number of different countries over a considerable number of years. Although small-scale experiments must never be a substitute for seeking to acquire data at large-scale through fieldwork, it must be recognised that measuring afflux in flood flows presents considerable problems. Furthermore measuring sufficient parameters (e.g. velocity distributions, turbulence, coherent structures, boundary shear stress distributions, scour, etc.) in natural channels is simply not possible in sufficient spatial and temporal detail under flood flow conditions. This partly explains the rationale behind the well-focussed laboratory experiments described earlier, aimed at understanding flow structure and quantifying particular phenomena. If these are combined with field data, and linked to theoretical and numerical studies, then further advances in our understanding of afflux at bridges may be made.

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7 FIGURES

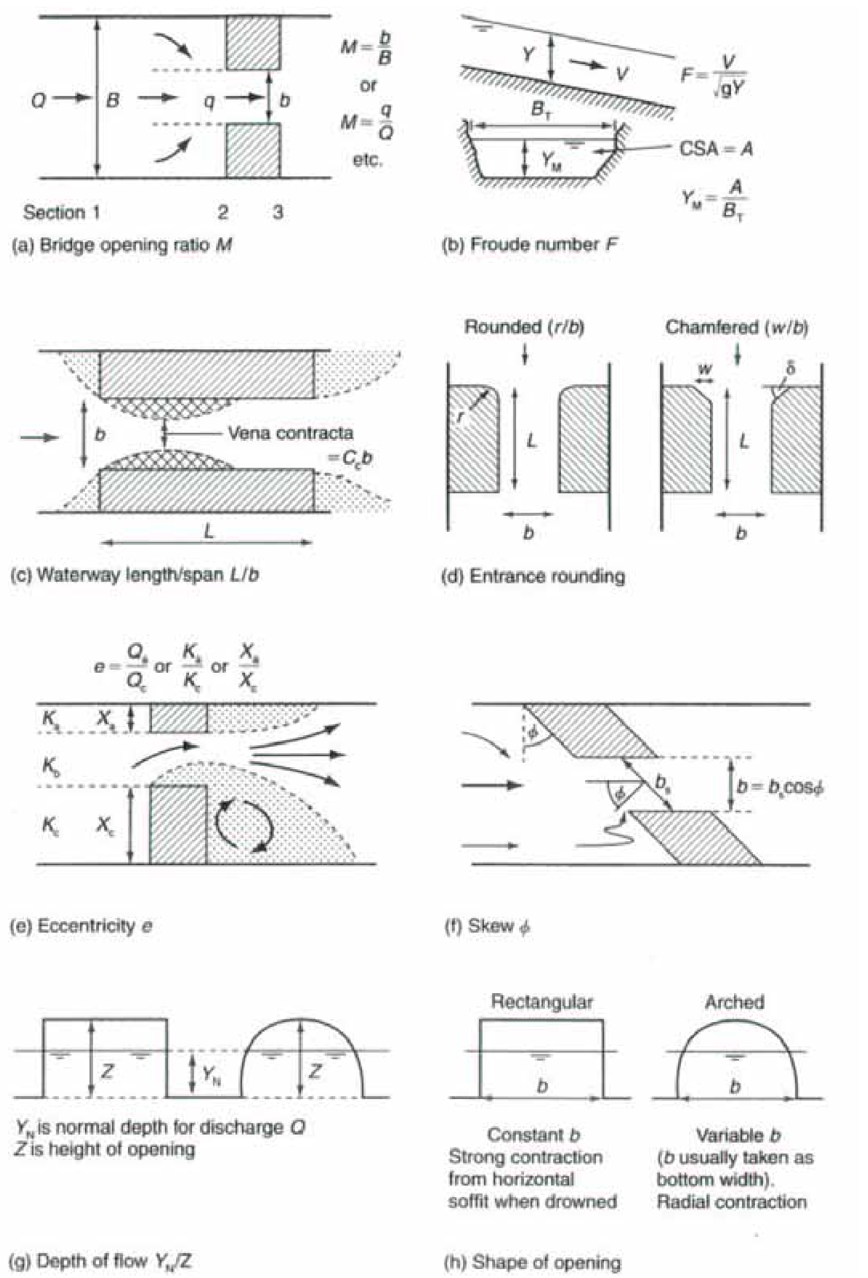

Figure 1: Summary of principal hydraulic variables affecting the hydraulic performance of a bridge *(after Hamill, 1999).* (a) Bridge opening ratio, M; (b) Froude number, *F*; (c) waterway length/span, *L/b*; (d) entrance rounding; (e) eccentricity, *e*; (f) skew, *φ*; (g) depth of flow, Y_N/Z; (h) shape of opening.

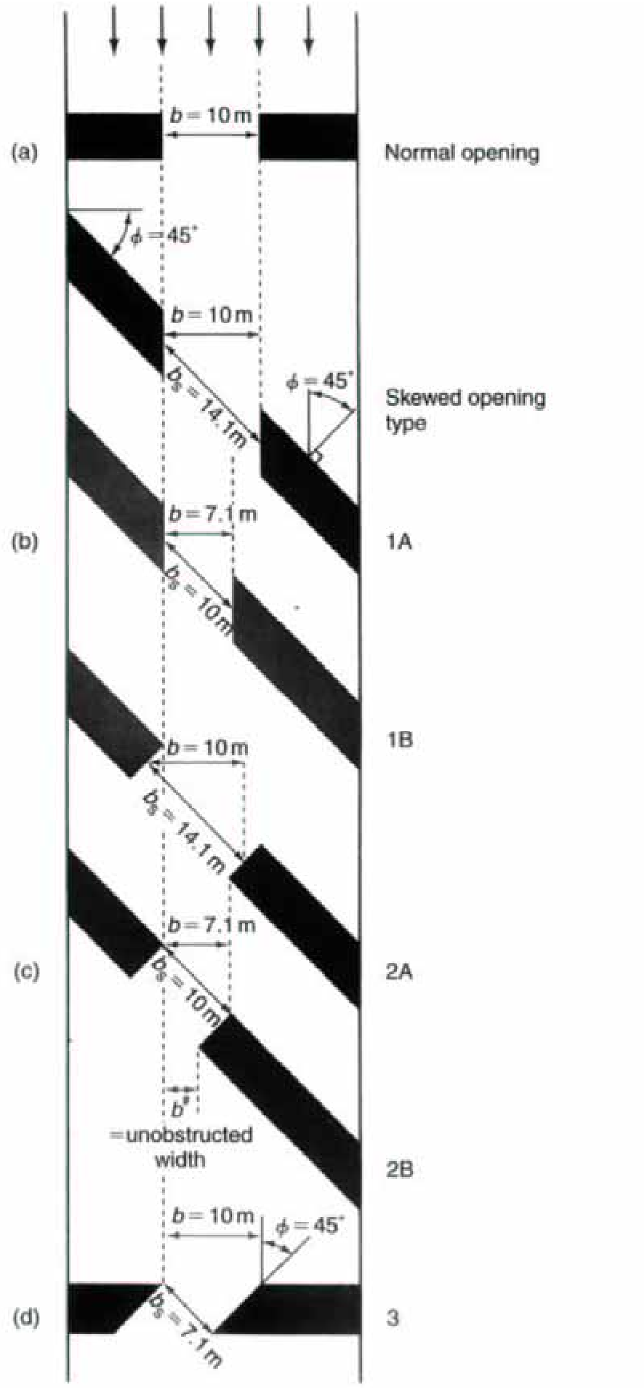

Figure 2: Comparison of normal and skewed crossings *(after Hamill, 1999)*. (a) Normal crossing; (b) type 1 skew with abutments parallel to the flow; (c) type 2 skew with abutments at 90° to approach embankments; (d) type 3 skew with abutments at an angle *φ* to both the flow and approach embankments.

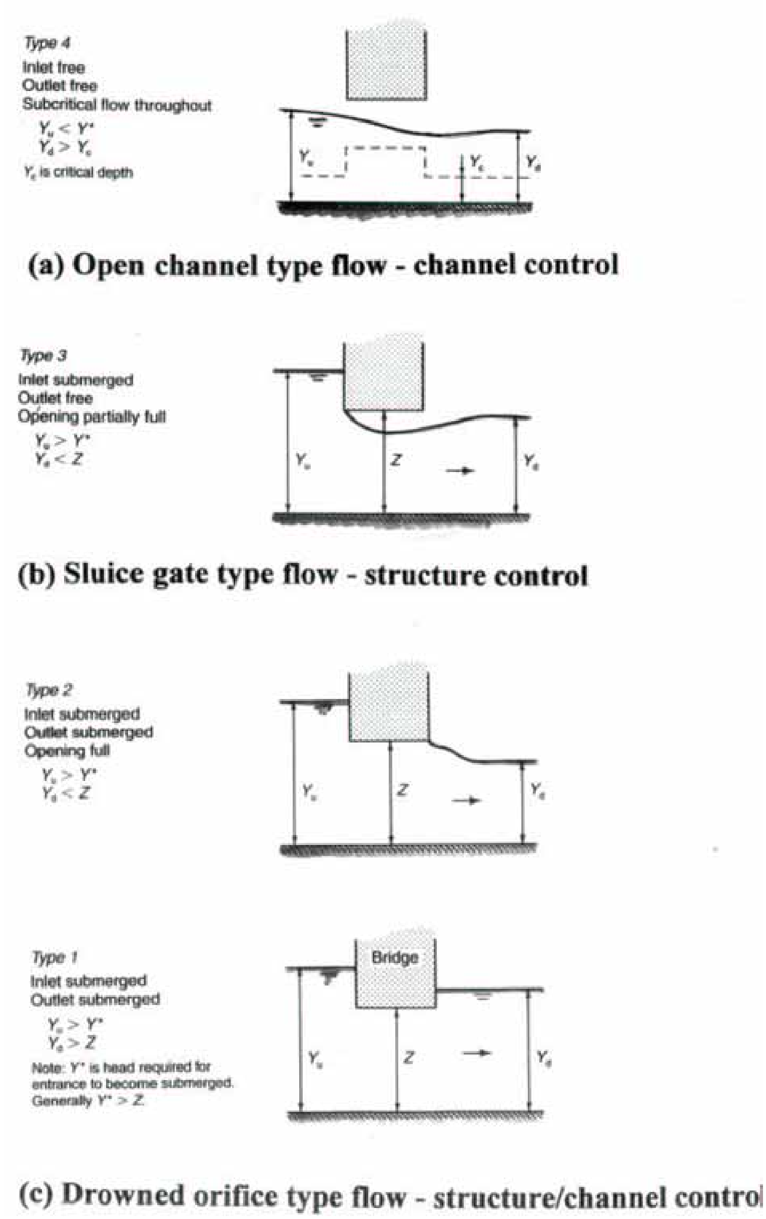

Figure 3: Types of flow through a bridge crossing ***(adapted from Hamill, 1999).***

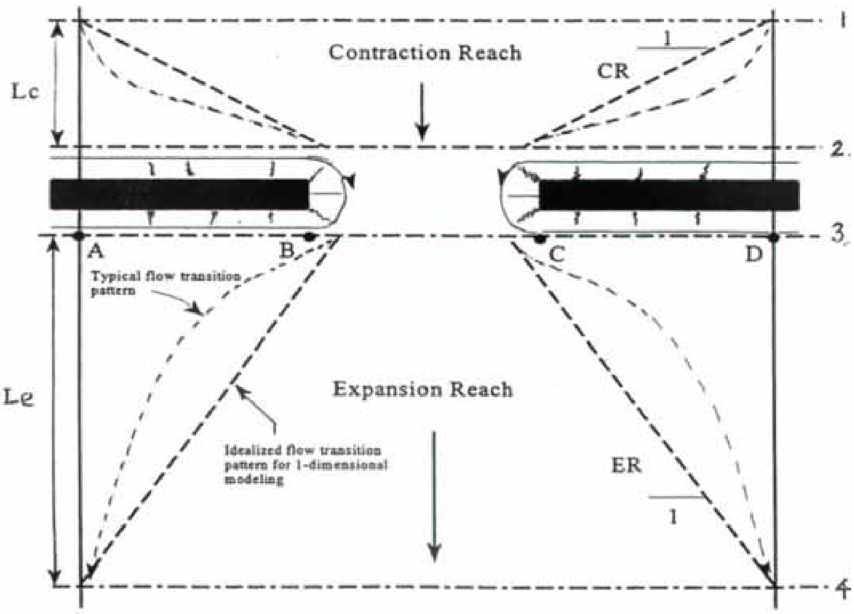

Figure 4: Numbering of cross sections (after HEC-RAS).

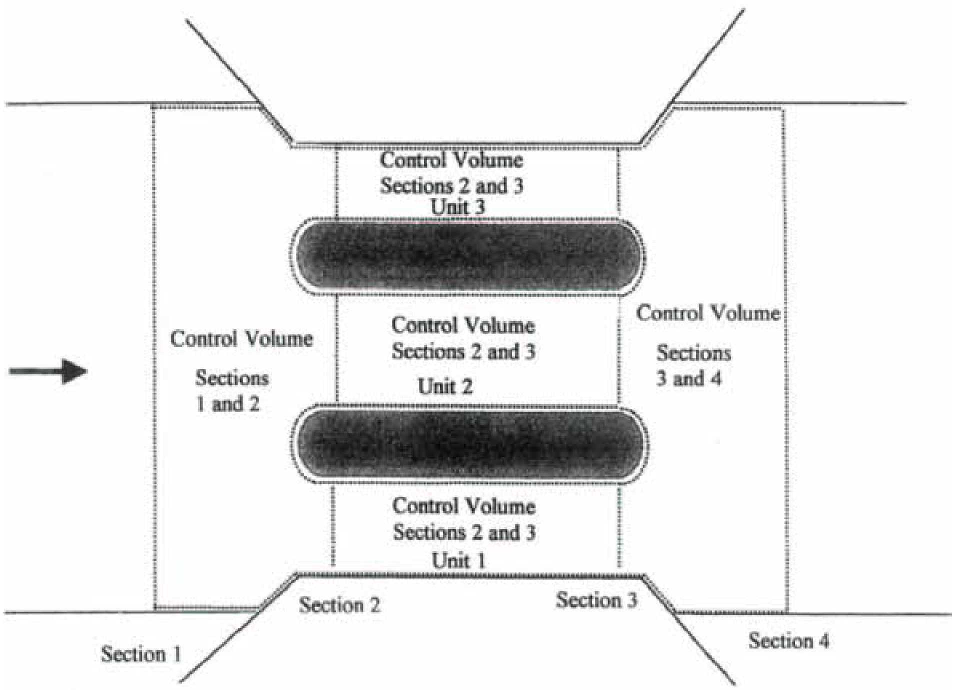

Figure 5: Numbering of cross sections (after Mott-Macdonald).

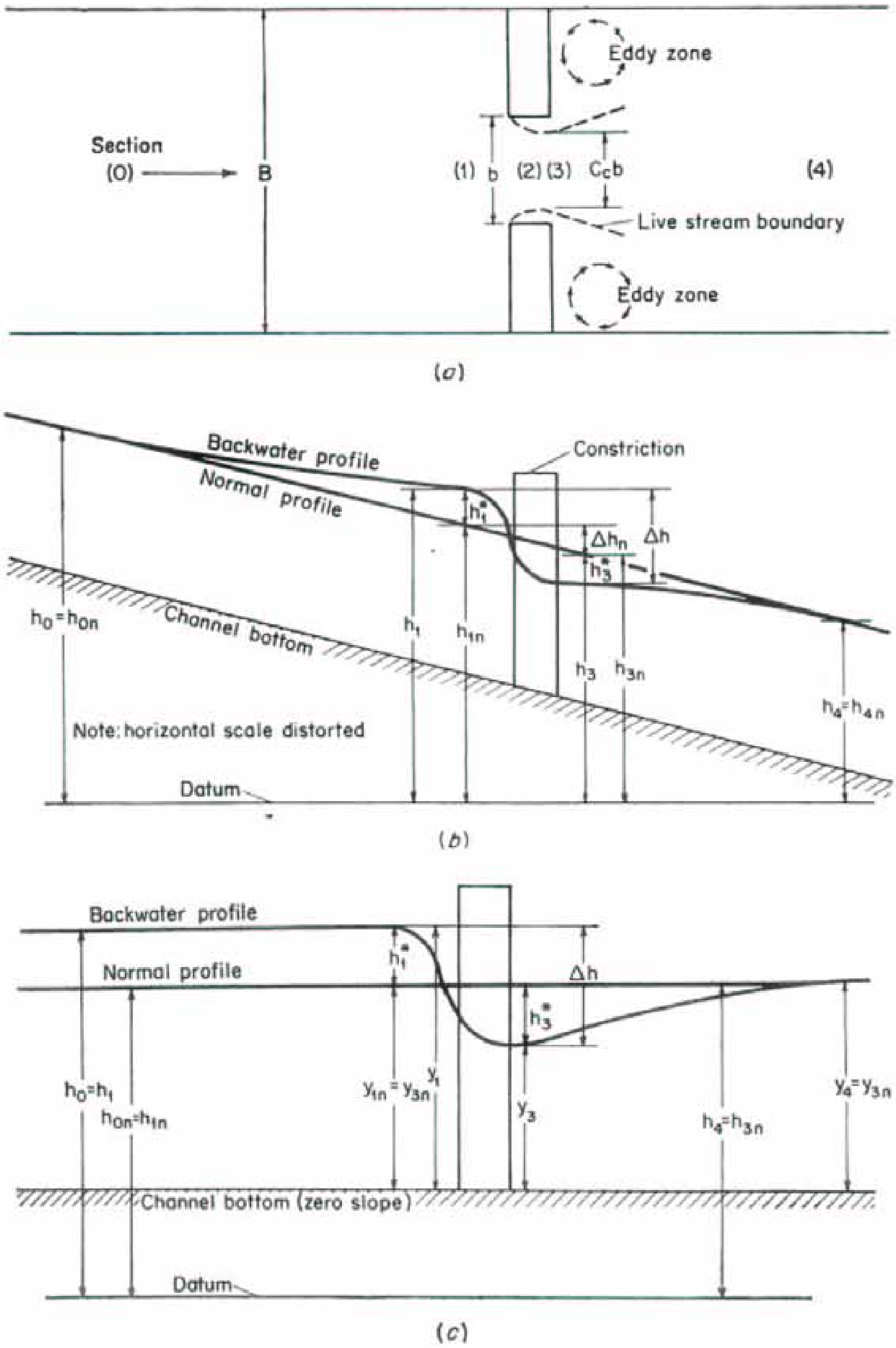

Figure 6: Longitudinal water surface profile through a bridge *(after Chow, 1959)*.

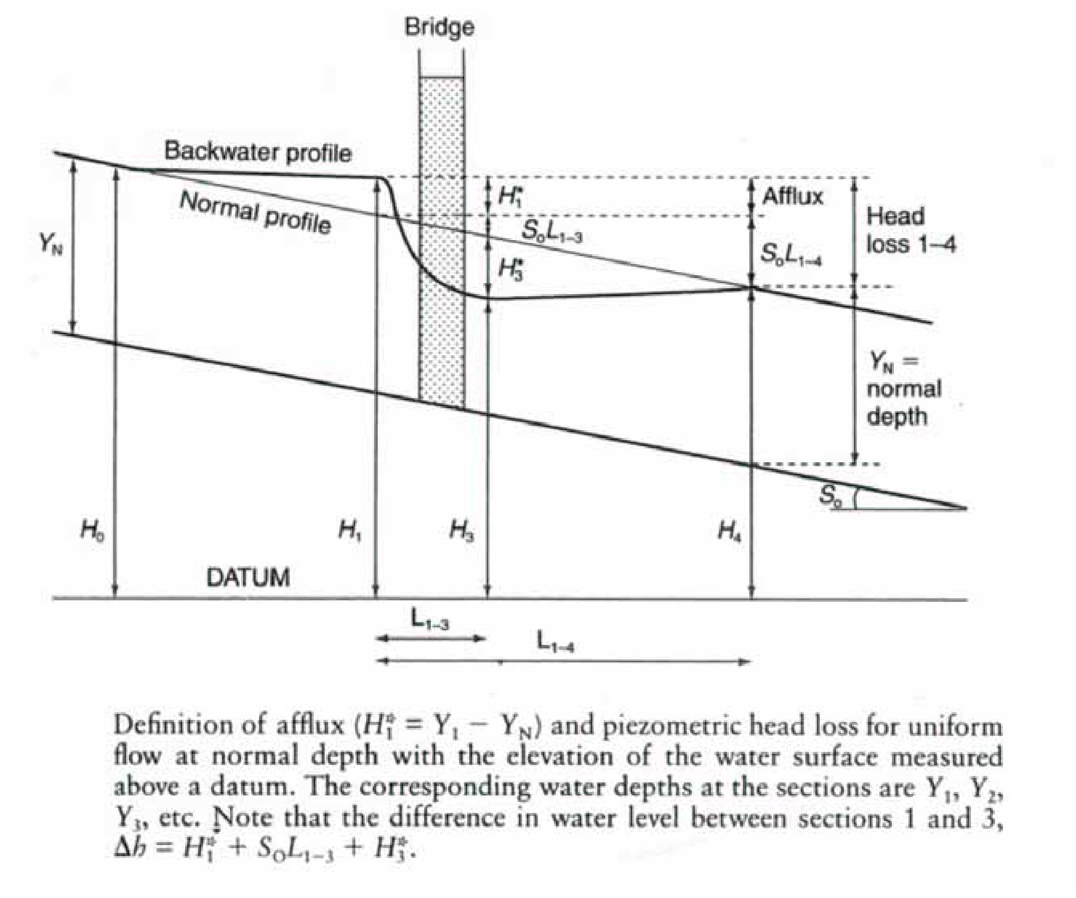

Figure 7: Longitudinal water surface profile through a bridge *(after Hamill, 1999).*

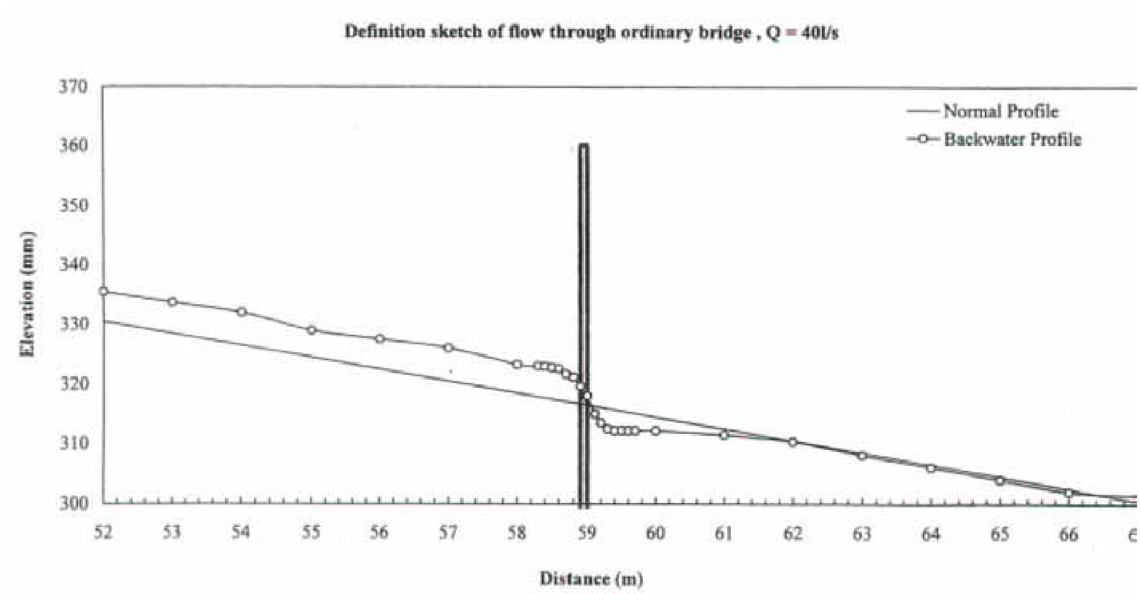

Figure 8: Measured longitudinal water surface profile through a bridge (Atabay & Knight, 2002).

Figure 9: Observed flow through a model bridge with circular piers in a compound channel (Atabay & Knight, 2002).

Figure 10: Observed flow through a model elliptical bridge in a compound channel (Atabay & Knight, 2002).

Figure 11: Observed flow through a model twin arch bridge in a compound channel, viewed from upstream (Atabay & Knight, 2002).

Figure 12: Observed flow through a model twin arch bridge in a compound channel, viewed from downstream (Atabay & Knight, 2002)

Figure 13: Observed high flow through a model twin arch bridge in a compound channel, viewed from downstream (Atabay & Knight, 2002).

Figure 14: Observed high flow through a single arch bridge in Italy.

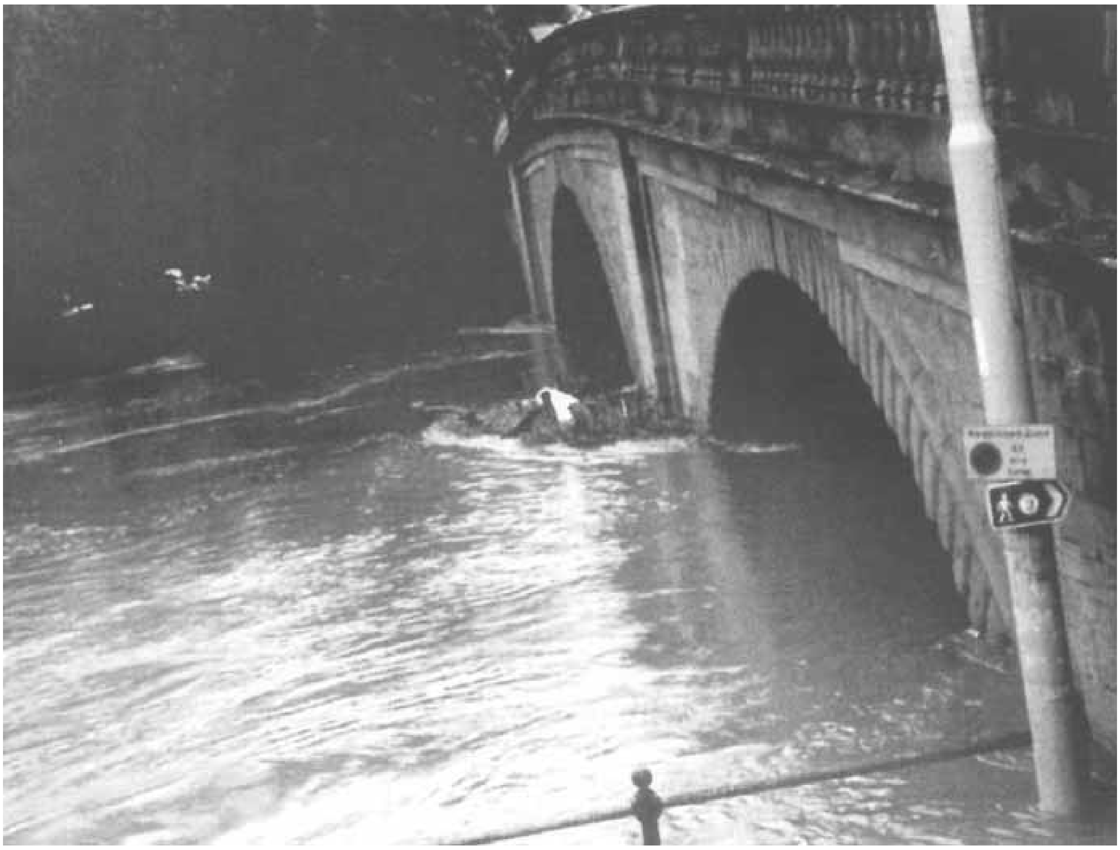

Figure 15: Observed flow through a multiple arch bridge on the River Severn at Bewdley.

Figure 16: Results of a catastrophic flood through a single arch bridge in Yorkshire.