



**Galip Seckin**  
 Associate Professor, Cukurova University, Balcali-Adana, Turkey



**Tefaruk Haktanir**  
 Professor, Erciyes University, Kayseri, Turkey



**Donald W. Knight**  
 Professor, University of Birmingham, UK

## A simple method for estimating flood flow around bridges

G. Seckin MSc, PhD, T. Haktanir MSc, PhD and D. W. Knight MSc, PhD, CEng, MICE, MASCE, MCIWEM

**The software program HEC-RAS from the Hydrologic Engineering Center (HEC) of the US Army Corps of Engineers is probably one of the most commonly used methods in the world for computing water surface profiles in rivers. The energy method (one of the four bridge subroutines within HEC-RAS) computes the bridge backwater (the upstream surface increase) by applying standard step calculations five times from the end of the expansion reach up to the beginning of the contraction reach, using two different transition loss coefficients and different reach lengths. The aim of this study was to estimate the backwater in a less cumbersome and practical way, without sacrificing accuracy. A one-step energy method is suggested, based on a comprehensive set of laboratory bridge backwater data from compound channels. The proposed method gives an absolute mean error of 10% when applied to these laboratory data and an absolute mean error of 25% when applied to field data collected by the United States Geological Survey including actual flood profiles through many bridges.**

### NOTATION

$A$	flow area (general)
$C$	coefficient of transition loss (general)
$C_c$	coefficient of contraction
$C_e$	coefficient of expansion
$C_{ti}$	proposed transition loss coefficient
$g$	gravitational acceleration
$h_1$	flow depth at section one
$h_4$	flow depth at section four
$K$	conveyance (general)
$L$	reach length
$M$	contraction ratio
$Q$	discharge
$S_f$	friction slope
$V$	average velocity
$WS$	water surface elevation
$\alpha$	velocity head correction coefficient

### 1. INTRODUCTION

Recent laboratory and field studies have shown that the energy equation used by the bridge subroutine in HEC-RAS<sup>1</sup> is capable of producing accurate estimates of water surface levels in river reaches constricted by bridges.<sup>2–4</sup> It is however, a multi-step procedure with the inclusion of parameters chosen by the user,

and this can lead to considerable inaccuracies in the application of the method.

The energy equation, based on application of the standard step method<sup>5</sup> to the transition reaches through a bridge waterway, is

$$WS_u + \frac{\alpha_u V_u^2}{2g} = WS_d + \frac{\alpha_d V_d^2}{2g} + f r_{u,d} + C \left| \frac{\alpha_u V_u^2}{2g} - \frac{\alpha_d V_d^2}{2g} \right|$$

where  $u$  denotes upstream section,  $d$  denotes downstream section,  $WS$  is water surface elevation,  $\alpha$  is velocity head correction coefficient,  $V$  is average velocity,  $g$  is gravitational acceleration,  $L$  is reach length between upstream and downstream sections,  $f r_{u,d}$  is the total frictional loss between the upstream and downstream sections ( $f r_{u,d} = L \times S_f$ , in which  $S_f$  is the average friction slope between two sequent sections) and  $C$  is the coefficient of contraction for the contracting reach and coefficient of expansion for the expanding reach.

For application of the energy method, the flow is considered in three parts embodied within four sequential sections through a bridge constriction as illustrated in Fig. 1. Cross-sections 1 and 4 determine the beginning of the contraction and the end of the expansion, respectively; cross-sections 2 and 3 are located a short distance upstream and downstream from the bridge. During the hydraulic computations, HEC-RAS automatically formulates two additional cross-sections, which are referred to as sections BD (bridge downstream) and BU (bridge upstream) inside the bridge structure (Fig. 1(c)).

The energy equation is applied to all reaches between any two sequential cross-sections, beginning at cross-section 4 and moving upstream to cross-section 1, to compute the water surface elevation of each cross-section as illustrated in Figs 1(a) and 1(c). The sequence of computations starts with a standard step application from cross-section 4 up to cross-section 3, which involves the all-important expansion coefficient that is crucial in determining the bridge losses occurring within the expansion reach. Four more standard step computations are then undertaken sequentially from section to section working upstream from section 4 to section 1. As shown in Fig. 1(c), section 1 is the section whose flow energy has risen and exceeded that for normal flow conditions in the absence of the bridge. This implies that the extra energy accumulated at section 1 will be used up by all the extra losses due to the contraction

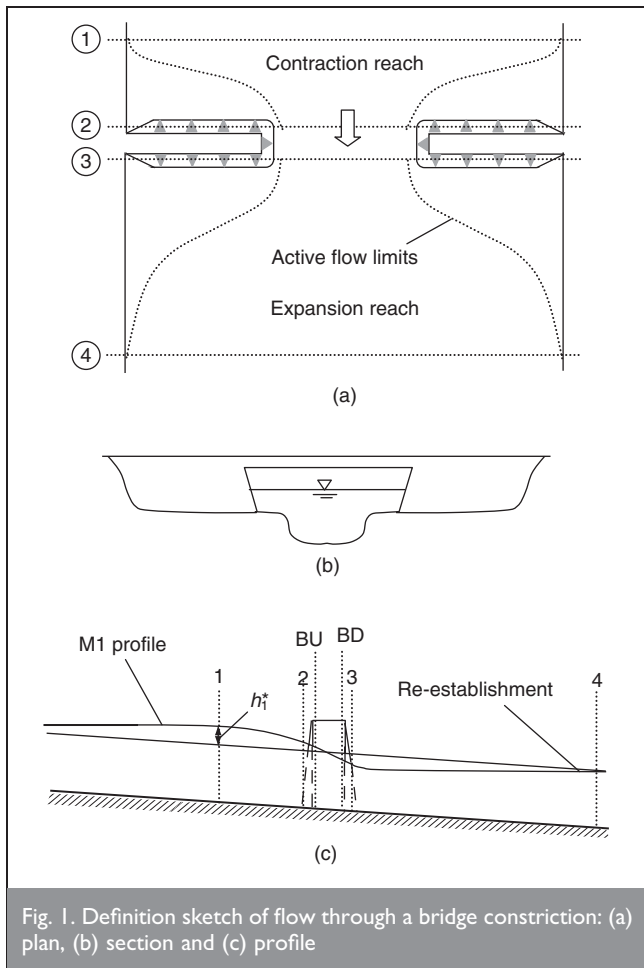


Fig. 1. Definition sketch of flow through a bridge constriction: (a) plan, (b) section and (c) profile

and expansion effects. Because flow is subcritical in most natural streams, it is assumed here that the increase of flow energy can be accomplished by increasing the flow depth, which occurs gradually through an M1 flow profile beginning at a section much further back upstream from section 1.

In summary, the energy method requires the application of standard step computations in five steps from section 4 to section 1, with two different transition loss coefficients and reach lengths for the expansion and contraction sections. Furthermore, if the special bridge subroutine is chosen, then the elevation difference between the upstream and downstream faces of the bridge structure, cross-sections BD and BU, is computed by the Yarnell equation. Normally, the special bridge routine is used for bridges having piers and the normal bridge routine is used for those without piers. The comparison and sensitivity studies applied by Kaatz and James<sup>3</sup> revealed that the special bridge routine will always yield smaller upstream surface increase (backwater) values for similar geometric configurations. Unfortunately, this produces yet more confusion in the computations through using either the normal or the special bridge routines. Furthermore, implementation of the Yarnell equation into the special bridge subroutine is not in accordance with the original model studies performed by Yarnell's research group in the 1930s.<sup>6</sup> The Yarnell experiments were performed in smooth rectangular channels containing piers of various numbers and configurations but without any transverse constrictions from embankments. Although natural channel reaches with wide floodplains and bridge waterways constricted by approach embankments do not resemble those of Yarnell's

experiments, the Yarnell equation is still used in HEC-RAS. In short, whether the normal bridge routine or the special bridge routine is used, considerable uncertainty arises in the model results due to the many required steps in the computations as well as the use of various empirical coefficients. The user therefore needs to make a number of difficult technical judgements. The bridge subroutines of HEC-RAS in their present form appear to be realistic but, in reality, they may yield disputable results because of the inclusion of (a) so many steps necessitating the usage of empirical coefficients and (b) so many cross-sections.

Two-dimensional (2D) computational models also may not yield the expected accuracy in the computed backwater commensurate with the amount of input data required and effort in preparing finite-element grids of the affected reaches. For example, after such an analysis, Hunt says 'It is reasonable to expect that in the near future one-dimensional models will remain the tool of choice for most practical bridge hydraulics studies. It should also be remembered that depth-integrated two-dimensional models are themselves approximations of the actual dynamics in any flow field'.<sup>7</sup> Similarly, Kaatz and James report 'The two-dimensional models require more data and effort on the part of the engineer as compared to one-dimensional model requirements. Thus, one-dimensional models are still the primary tool used by engineers to evaluate stream hydraulics'.<sup>3</sup>

The current study aims to evaluate transition loss coefficients as a whole, while reducing the number of computational steps and still increase maintaining a reasonable accuracy in the computed backwater. The backwater occurring at cross-section 1 is thus calculated with a single step and a single loss coefficient. The sum of possible errors inherent in so many intermediate steps of any one of HEC-RAS bridge routines, most of which require empirical coefficients and personal judgement, may be of the same order as the error in a one-step model. In developing this simplified model, comprehensive experimental data were used.<sup>4,8</sup> These data covered a wide range of subcritical flow conditions in a composite-section flume with roughness values from close to smooth to as high as  $n = 0.14$  ( $n$  being the Manning roughness coefficient).

## 2. SUMMARY OF EXPERIMENTAL WORK

Energy losses through a bridge constriction include friction losses, form losses, eddy losses and losses due to inertial forces resulting from changes in magnitude and direction of flow velocities during expansion and contraction. Correct quantification of these losses individually is virtually impossible, despite attempts to empirically determine them since the early 1920s.<sup>9</sup> Detailed laboratory studies have been undertaken with the financial support of government organisations such as the Federal Highway Administration (then the Bureau of Public Roads) and the United States Geological Survey (USGS).<sup>10,11</sup> Despite these efforts, comprehensive analyses and financial expense, it was later realised that laboratory-based models generally underpredicted the actual backwaters that occur in the field.<sup>3,12-14</sup> The main reason for this underprediction was because losses measured in ideal laboratory flumes were smaller than those in prototype structures under geometrically similar configurations. The major component of the total bridge loss is the expansion loss, and the expansion loss in real-life bridges is often greater because of



Fig. 2. Compound laboratory flume at the University of Birmingham used in testing bridge models (deck bridge model installed—from Seckin and Atabay 2005<sup>21</sup>)

vegetation and buildings in the overbank areas that are only inundated during severe floods. In short, the expansion reaches of the laboratory flumes did not have dynamic similitude because they were much smoother than those of natural reaches. Having realised this fact, some of these models were later refined using field measurements.<sup>12</sup> Field measurements are, however, prohibitively expensive and the data available are not always of sufficient spatial and temporal coverage. The field data mentioned in the literature are the more comprehensive sets taken by USGS on many bridge sites in the State of Mississippi in 1960s<sup>14</sup> and 13 bridges, again in southern US states.<sup>7,15</sup>

With the goal of refining the calculation of bridge backwater, a comprehensive series of laboratory flume experiments were designed and executed in the Hydraulics Laboratory of Birmingham University, UK (see Fig. 2). Firstly, all the reaches were of a compound geometry instead of either simple rectangular or trapezoidal single sections used in previous studies. Secondly, the roughnesses of the overbank areas were changed from almost smooth up to a value of  $n = 0.14$ , which would more realistically simulate the usually rough floodplain areas of natural reaches. Thirdly, many bridge models with various rectangular, elliptic and arch geometries were tested. Fourthly, flow depth and velocity measurements were taken very carefully. These recent data therefore covered a wider range, with better dynamic similitude, than previous studies.

Further details of the experimental set-up and procedures have been reported.<sup>4</sup> In summary, a series of model tests was conducted using a compound flume that consisted of a main channel and two symmetric floodplains. Water surface measurements were taken in two phases. In the first phase, uniform flow experiments, in the absence of bridges, were conducted to determine the channel bed roughness in the main channel and floodplains separately for five series of tests. Five different discharges were used in each series of tests. The channel bed surfaces in the main channel were smooth for two series of experiments. For the other three series, the main channel was roughened using different arrangements of metal meshes. The floodplains were also roughened with the same material for all series of experiments except the first one, which was referred to as the smooth bed case. Manning roughness coefficients  $n$  were computed for the compound cross-sections, after having measured all the relevant parameters for a wide range of uniform flows. The roughness of the channel section changed within the range  $n = 0.01-0.04$  and the roughness of the overbank areas changed within the much wider range of  $n = 0.02-0.14$ . The roughnesses of the floodplain sections were deliberately increased to quite high magnitudes, for example 0.14, in order to simulate as realistically as possible the heavily vegetated floodplains of natural rivers.

In the second phase, water surface measurements were taken for the same conditions of channel surfaces and discharge values but with the installation of the bridges. Four models were used in the testing programme, including single-opening semi-circular arch (ASOSC), single-opening elliptic arch (ASOE), multiple-opening semi-circular arch (AMOSC) and straight deck types of bridge models with and without piers (DECK and DECKP). Figs 2 and 3 show views of the model bridges used and Fig. 4 shows a typical



Fig. 3. (a) Elliptical model bridge. (b) Twin-arch model bridge

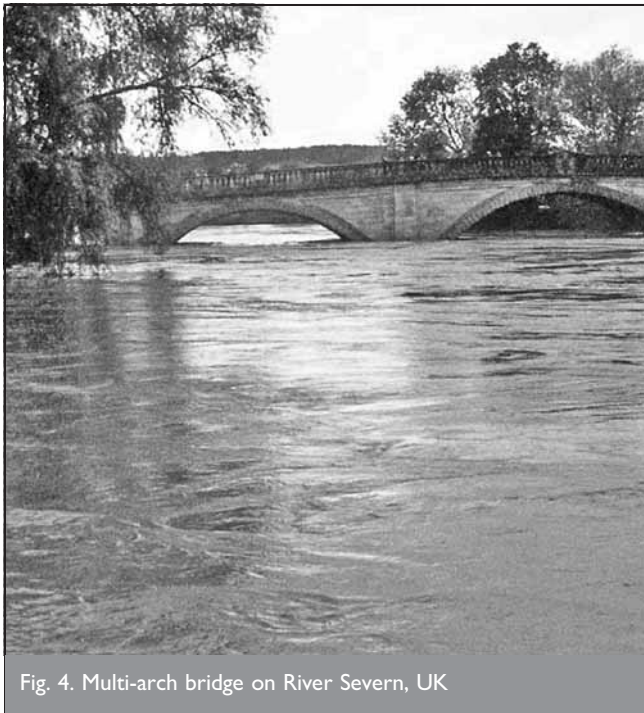


Fig. 4. Multi-arch bridge on River Severn, UK

multi-arch bridge during a moderate flood. In all, 50 tests were undertaken with the arch opening bridge models and 95 tests with the rectangular opening bridge models. All the bridge models were set initially normal to the flow direction.

Later, the range of experiments was extended using more roughness scenarios for the same flume and bridge models, as described elsewhere.<sup>4,8</sup> With these additional 43 experiments, also undertaken with the bridges set normal to the flow, a total of 188 experiments with subcritical flows resulted. The data from these different flow and geometric configurations are those used in the current study. It should be pointed out that 25 data points (which refer to the smooth floodplain cases) were excluded from the current analysis.

### 3. PROPOSED MODIFICATION TO THE ENERGY METHOD

HEC-RAS splits losses into contraction losses, friction losses and expansion losses. The expansion coefficient  $C_e$  and contraction coefficient  $C_c$  in equation (1) were used to calculate the magnitude of transition losses for the expansion and contraction reaches. A traditional rule for the lengths of the contraction and expansion reaches has been to use ratios of 1:1 and 1:4, respectively. Some recent studies argue that, as the 1:1 ratio for the contraction reach is reasonable, the expansion ratio could be about 1:2 instead of 1:4.<sup>7,15</sup>

For the contraction and expansion coefficients, values of 0.3 and 0.5, respectively, are suggested in most of the relevant HEC publications.<sup>1,16-19</sup> Hunt<sup>7</sup>, however, proposed a value of 0.1 and Seckin *et al.*<sup>13</sup> suggested 0.05 for the contraction coefficient; they presented regression relationships for the expansion coefficient, which suggested magnitudes much greater than 0.5 of HEC. This is significant, since the expansion loss is the dominant energy loss in bridge hydraulics.<sup>3,13</sup>

Rational quantification of additional losses is not easy, as they depend on various factors such as the amount of bridge

constriction, types of piers, types and density of vegetation and urbanisation in the expansion reach and hence its average roughness, presence of spur-dykes, skewness and so on. Some laborious attempts to compute the longitudinal and transverse water surface profiles around some natural river reaches by some 2D finite-element models accounting for friction and eddy losses in detail were not successful in accurately computing the actual water profiles and the backwater of some recorded flood events.<sup>7</sup> Despite the extensive 2D input data used and seemingly more powerful simulation, even the 2D models may not compute the actual backwater correctly. 1D models such as HEC-RAS therefore still preserve their practicality and usability.

Because of all the aforementioned ambiguities and complexities, it was decided to reflect the transition losses as a whole by multiplying the difference between the velocity head of the constricted section under the bridge and that of cross-section 4 by a single coefficient (symbolised as  $C_{tl}$ ) and to compute the backwater in a single step. This can be achieved by applying the energy equation between cross-section 1 and cross-section 4. In this case, assuming the channel bottom alignment of the natural reach is almost straight, the energy conservation equation between cross-section 1, the section of maximum backwater, and cross-section 4 can be written as

$$2 \quad h_1 + \frac{\alpha_1 V_1^2}{2g} + S_o L_{1,4} = h_4 + \frac{\alpha_4 V_4^2}{2g} + f r_{1,4} + C_{tl} \left| \frac{\alpha_{BD} V n_{BD}^2}{2g} - \frac{\alpha_4 V_4^2}{2g} \right|$$

There is a potential energy gain between sections 4 and 1 due to differences in the elevations of these sections, which equals  $S_o \times L_{1,4}$ , where  $S_o$  is the average channel bottom slope between sections 1 and 4 and  $L_{1,4}$  is the reach length between these two sections. Here, it is assumed that the energy loss due to friction  $f r_{1,4}$  (which equals  $S_f \times L_{1,4}$  where  $S_f$  is the average friction slope between sections 1 and 4) is approximately equal to the energy gain due to the potential difference between channel bottom elevations of sections 1 and 4. Because of variations in wetted perimeters and roughnesses of the representative cross-sections of the detailed models, the average friction slope between sections 1 and 4,  $S_f$ , may not be equal to the average channel bottom slope. Even so, the potential gain due to the channel bottom slope should be close to the overall frictional loss computed in the conventional way. This assumption will therefore bring about a negligibly small error, and the difference in the average channel bottom slope and the average friction slope will be accounted for within the overall loss coefficient,  $C_{tl}$ . The velocity  $V n_{BD}$  is the average velocity in the constricted section with an area under the original flow profile, which forms without the existence of the bridge. In natural reaches the unaffected flow is quasi-uniform, and the flow depths or water surface elevations at cross-sections 4, BD, and 1 without the existence of the bridge, will be routinely computed by stage computations. Equation (2) then becomes

$$3 \quad h_1 + \frac{\alpha_1 V_1^2}{2g} = h_4 + \frac{\alpha_4 V_4^2}{2g} + C_{tl} \left| \frac{\alpha_{BD} V n_{BD}^2}{2g} - \frac{\alpha_4 V_4^2}{2g} \right|$$

In equation (3),  $\alpha_1$  is close in magnitude to  $\alpha_4$ ,  $\alpha_{BD}$  is found using an empirical chart (see Fig. 5 of Bradley's work<sup>12</sup>) that estimates

$\alpha_{BD}$  as a function of  $\alpha_1$  and the contraction ratio  $M$ , defined as

$$M = \frac{K_q}{K_1}$$

in which  $K_q$  is the conveyance of the area equalling the bridge opening of section 1 and  $K_1$  is the total conveyance of section 1 for the water surface profile computed in the absence of the bridge. Because the friction slope at any section is an averaged value, valid for the entire cross-section by definition of 1D flow modelling, the ratio of conveyances of the constricted part and the natural part of the cross-section at section 1 is equal to the ratio of discharges through these two cross-sections. The smaller the contraction ratio, the greater the percentage of total discharge that will be forced to alter its natural stream lines and forced to experience contraction followed by expansion. Hence, the contraction ratio is a quantitative measure for the magnitude of the bridge constriction. Dynamically, the ratio of discharges is a more meaningful independent variable affecting the total extra energy loss due to contraction and expansion than simply the ratio of areas.  $M$  is thus the major factor in determining the empirical coefficient to compute the backwater in the method of Bradley.<sup>12</sup>

The velocity head difference in equation (3) reflects the degree of constriction. As the area under the bridge is narrower, the velocity head of the BD cross-section increases and the difference in velocity heads increases; this will cause a great transition loss to be computed. Conversely, for a mildly constricted bridge opening, which means the bridge deck covers most of the floodplain with rather short approach embankments, the velocity head of the BD cross-section is small, and the difference of these two velocity heads will also be small. Hence a small overall transition loss will be computed.

Having measured the actual velocities at many points in the cross-section BD by a current meter suspended from the downstream end of the bridge deck during many flood events at many sites, an empirical relationship between  $\alpha_{BD}$  and  $M$  and  $\alpha_1$  was developed (see Fig. 5 and B2 in Bradley<sup>12</sup>). The coefficient  $\alpha_{BD}$  was found to be slightly smaller than  $\alpha_1$ , but is still greater than 1.0. Because the average velocity at cross-section BD is inevitably high, the effect of  $\alpha_{BD}$  is considerable and its correct estimation is important.

Although  $h_1$  is slightly greater than  $h_4$ ,  $\alpha_1$  should be very close to  $\alpha_4$  because cross-sections 1 and 4 should be close in geometrical and roughness characteristics.

As already mentioned, Seckin<sup>4</sup> and Seckin *et al.*<sup>8</sup> performed 163 carefully controlled bridge model experiments in compound open-channel flumes with different configurations. The coefficient  $C_{ti}$  in equation (3) was solved using the measured values inserted for the other terms. The values of  $h_1$ ,  $h_4$ ,  $V_1$  and  $V_4$ , were taken directly from the experimental measurements. The coefficients  $\alpha_1$  and  $\alpha_4$  were computed for each experimental case using

$$\alpha = \frac{(A_t)^2 \left[ \frac{(K_{lob})^3}{(A_{lob})^2} + \frac{(K_{ch})^3}{(A_{ch})^2} + \frac{(K_{rob})^3}{(A_{rob})^2} \right]}{(K_t)^3}$$

where:  $A_t$  is the total flow area of the cross-section;  $A_{lob}$ ,  $A_{ch}$ ,  $A_{rob}$  are flow areas of left overbank floodplain, main channel and right overbank floodplain, respectively;  $K_t$  is total conveyance of cross-section; and  $K_{lob}$ ,  $K_{ch}$  and  $K_{rob}$  are conveyances of left overbank floodplain, main channel and right overbank floodplain, respectively.

The coefficient  $\alpha_{BD}$  was computed using the same procedure and curves developed by the US Bureau of Public Roads (USBPR) method (see Fig. 5 in Bradley<sup>12</sup>). Each  $C_{ti}$  value for each of the 163 different configurations was computed using equation (3).

#### 4. ANALYSIS OF EXPERIMENTAL RESULTS

The values of  $C_{ti}$  obtained from equation (3) ranged from 0.2 to 1.0 for the cases tested. This shows that the use of a 'standard' mean value for  $C_{ti}$  may cause large errors in estimating backwater. Many independent variables and combinations of variables (including discharges, roughnesses, Froude numbers, velocities, cross-section areas, flow depths and conveyances) were therefore then investigated, seeking significant correlations between  $C_{ti}$  as the dependent variable and these aforementioned quantities as independent variables.

As might be expected, regression analysis showed that there was greatest correlation between the transition loss coefficient  $C_{ti}$  and the contraction ratio  $M$ . Fig. 5 shows a plot of transition loss coefficient  $C_{ti}$  against  $1 - M$ . The best-fit equation for  $C_{ti}$  is

$$C_{ti} = 4.7795 \times (1 - M)^{1.9017}$$

for which the determination coefficient is  $R^2 = 0.83$ .

Equation (6) was then used in equation (3) and applied to 163 experimental data sets. The absolute mean error was 10%. The predicted values using equation (3) plotted against the measured values are shown in Fig. 6.

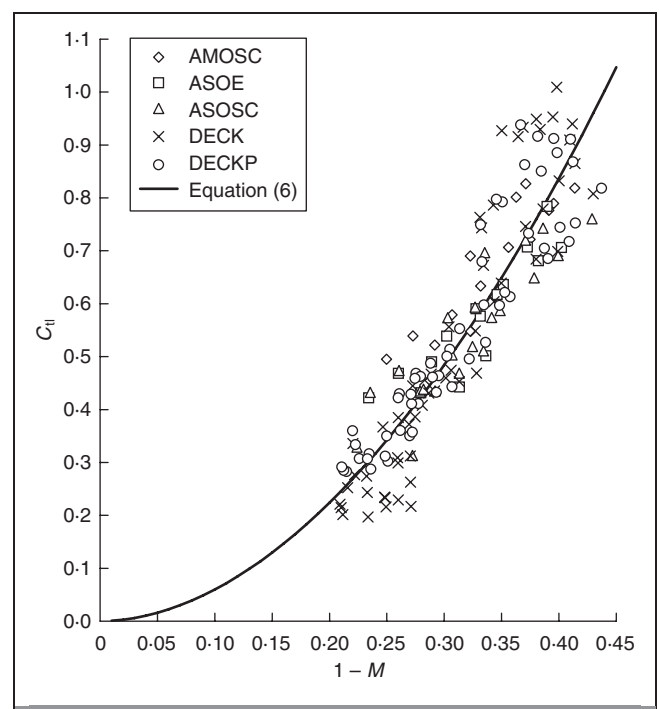


Fig. 5. Plot of transition loss coefficient ( $C_{ti}$ ) against contraction ratio ( $M$ )

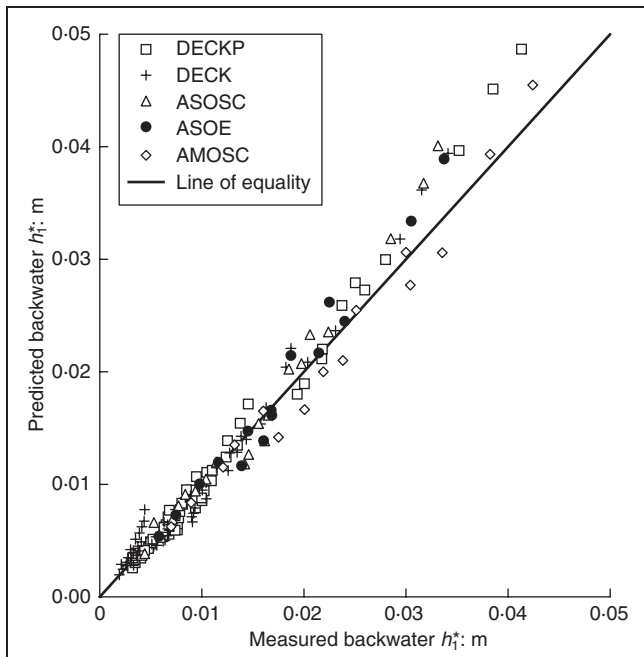


Fig. 6. Comparison of measured backwater values against the values

## 5. ANALYSIS OF FIELD DATA

### 5.1. General information about the data

During the period 1961–1965, the Mississippi District of Water Resources Division of USGS collected field data at 35 bridge sites during flood stages.<sup>14</sup> At some of the bridges, data were taken for more than one flooding event. The bridges were located on various

small and large tributaries of the Mississippi River. Water surface profiles, discharges, velocities and geometrical properties were measured. Unchoked subcritical (type IA) flow was observed at all of the bridges. The purpose of the study was to check the validity of two methods being used for bridge hydraulic computations, the USGS method and the USBPR method. Both methods had been developed after extensive laboratory model studies and the laboratory-developed coefficients and charts were to be checked using the field data. At the end of the study, it was concluded that both methods only gave around half of the observed backwater. Later, the USBPR method was revised and the design charts were modified in accordance with the data supplied by the USGS. Some of the design charts and figures were thus altered to fit the field values observed on these Mississippi River tributaries. The USGS method has not experienced such a remodelling.

Haktanir<sup>20</sup> repeated the computations for 26 observations of the USGS data using the modified USBPR method. The excluded portion of the data had some insufficient values; the part used by Haktanir is listed in Table 1. Haktanir's computations showed, as expected, that on average the backwater computed by the modified USBPR method gave very close values to the observed data.

### 5.2. Application of proposed equations to field data

In this work equation (3) was applied to the 26 field data used by Haktanir.<sup>20</sup> All of the terms in equation (3) except  $C_{t1}$  values were obtained from these data, and equation (6) was used to estimate the  $C_{t1}$  values. For the 26 cases, the backwater  $h_1^*$  was computed and compared with observed values. A plot of the measured backwater values against those predicted by equation (3) is shown in Fig. 7. A summary of the computed and observed backwater

Bridge site (1)	No. (2)	Date of flood (3)	Peak discharge: $m^3/s$ (4)	Bridge opening length: m (5)	Channel contraction ratio ( $M$ ): % (6)
Buckatunna Creek at State Highway 18 near Quitman	1	04/06/64	413	149	0.27
	2	04/07/64	264	149	0.31
Long Creek at State Highway 18 near Quitman	3	04/06/64	253	106	0.22
	4	04/07/64	118	105	0.25
Bowie Creek at Interstate 59 near Hattiesburg	5	02/22/61	1031	318	0.65
East Tallahala Creek at State Highway 528 near Bay	6	04/06/64	532	148	0.32
Tallahoma Creek at Interstate 59 at Ellisville	7	02/23/61	382	158	0.49
	8	04/08/64	326	158	0.51
Tallahala Creek at State Highway 42 Runnel town	9	02/25/61	881	363	0.71
	10	04/09/64	646	360	0.82
Wolf River at State Highway 26 near Poplarville	11	02/18/61	242	73	0.34
	12	03/02/64	89	72	0.48
Bogue Chitto at U.S. Highway 84 near Brookhaven	13	03/28/61	217	52	0.17
	14	03/02/64	82	52	0.26
West Hobolochitto Creek at State Highway 26 near Poplarville	15	11/14/61	204	127	0.78
	16	03/02/64	155	124	0.80
West Fork Tombigbee River at U.S. Highway near Nettletown	17	04/12/62	1769	379	0.45
	18	03/15/65	905	378	0.56
Yockanookany River at State Highway 35 near Kosciusko	19	12/18/61	433	165	0.22
Black Creek at State Highway 589 near Purvis	20	04/28/62	154	117	0.16
Upper Little Creek at U.S. Highway 98 near Columbia	21	04/28/62	110	95	0.65
Luke Flupper Creek at State Highway 528 near Bay Springs	22	04/06/64	137	49	0.22
Tallahala Creek at Interstate 59 near Laurel	23	02/23/61	510	178	0.32
	24	04/07/64	603	178	0.32
Leaf River at U.S. Highway 98 near McLain	25	02/26/61	3568	650	0.38
White Sand Creek at County Highway near Oakvale	26	03/29/61	450	59	0.67

Table 1. Description of bridge sites for field data analysis

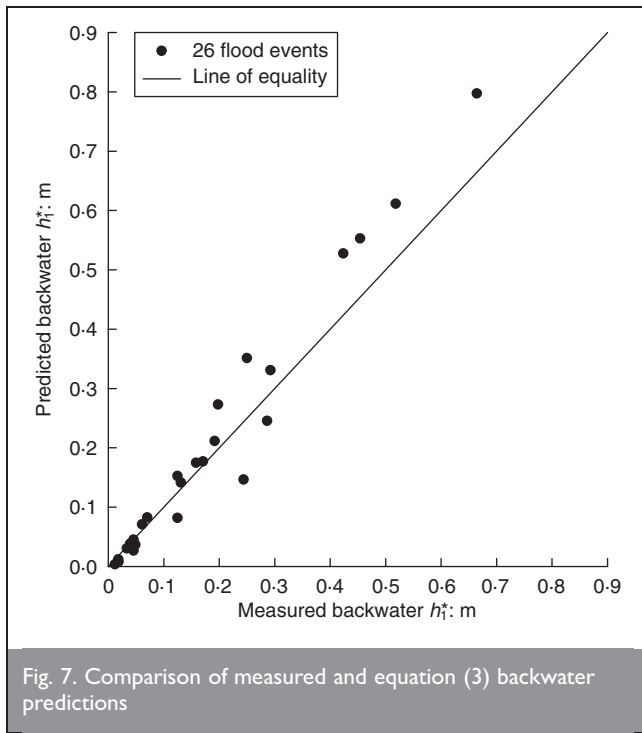


Fig. 7. Comparison of measured and equation (3) backwater predictions

results is also provided in Table 2. The absolute mean error of equation (3) was 25%.

The total transition loss coefficient proposed here is related to the magnitude of the contraction ratio  $M$  only. The effects of

eccentricity, skewness, abutment type, pier type and scour were not included in developing equation (6). As mentioned before, the amount of constricted area, defined by  $M$ , reflects the degree of contraction and expansion, and hence  $M$  is the major factor affecting the transition losses. The other parameters such as eccentricity, shape and area of piers have secondary effects on the transition losses. Their exclusion from the practical model proposed here will, however, naturally result in some loss of accuracy. Equation (6), developed on the basis of extensive laboratory work at Birmingham University, should yield plausible results for the backwater from either existing or planned bridges set normal to the flow.

Kaatz and James<sup>3</sup> investigated the performance and reliability of four popular methods: HEC-2 normal bridge routine, HEC-2 special bridge routine, WSPRO and modified USBPR methods. The study was based on 13 flood events at nine different bridge sites in southeastern USA. The average errors for these methods were 36, 26, 31 and 51%, respectively, when applied to these field data. The average error for the practical method outlined in this study (equations (3) and (6)) is 25%, which is acceptable when compared with the above reported values.

## 6. CONCLUSIONS

By proposing a lump-sum one-step energy equation for the reach of a natural stream affected by the constriction of a bridge, along with a simple formula relating an overall transition loss coefficient to the bridge contraction ratio, a practical method for estimating the bridge backwater has been developed. The proposed method yields an acceptable accuracy, with less

Bridge site (1)	Backwater results: m			
	No. (2)	Date of flood (3)	Measured (4)	Proposed equations (3) and (6) (5)
Buckatunna Creek at State Highway 18 near Quitman	1	04/06/64	0.52	0.61
	2	04/07/64	0.29	0.33
Long Creek at State Highway 18 near Quitman	3	04/06/64	0.45	0.55
	4	04/07/64	0.12	0.15
Bowie Creek at Interstate 59 near Hattiesburg	5	02/22/61	0.05	0.04
East Tallahala Creek at State Highway 528 near Bay	6	04/06/64	0.17	0.18
Tallahoma Creek at Interstate 59 at Ellisville	7	02/23/61	0.05	0.05
	8	04/08/64	0.03	0.03
Tallahala Creek at State Highway 42 Runnel town	9	02/25/61	0.05	0.03
	10	04/09/64	0.02	0.01
Wolf River at State Highway 26 near Poplarville	11	02/18/61	0.19	0.21
	12	03/02/64	0.04	0.04
Bogue Chitto at U.S. Highway 84 near Brookhaven	13	03/28/61	0.42	0.53
	14	03/02/64	0.07	0.08
West Hobolochitto Creek at State Highway 26 near Poplarville	15	11/14/61	0.02	0.01
	16	03/02/64	0.01	0.004
West Fork Tombigbee River at U.S. Highway near Nettletown	17	04/12/62	0.29	0.25
	18	03/15/65	0.12	0.08
Yockanookany River at State Highway 35 near Kosciusko	19	12/18/61	0.16	0.17
Black Creek at State Highway 589 near Purvis	20	04/28/62	0.06	0.07
Upper Little Creek at U.S. Highway 98 near Columbia	21	04/28/62	0.02	0.01
Luke Flupper Creek at State Highway 528 near Bay Springs	22	04/06/64	0.66	0.80
Tallahala Creek at Interstate 59 near Laurel	23	02/23/61	0.20	0.27
	24	04/07/64	0.25	0.35
Leaf River at U.S. Highway 98 near McLain	25	02/26/61	0.13	0.14
White Sand Creek at County Highway near Oakvale	26	03/29/61	0.24	0.15

Table 2. Summary of predicted and measured backwater results

computational effort and a smaller requirement of input data, in comparison with more detailed models such as HEC-RAS and USBPR.

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