

Backwater effect of multiple bridges along Huaihe River, China

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This study investigated the backwater effect of the eight bridges along the Huaihe River in China using physical model experiments. The experimental data obtained were used to derive equations to parameterise the backwater effect of a single bridge. The cumulative effect of two bridges was then analysed, using the backwater effect of a single bridge as a reference. It was found that, in order to eliminate the cumulative effect, the minimum distance between two bridges should be no less than 215 times the bridge pier width.

Notation

B	width of flow
B_1	width of main channel
B_2	width of compound channel
D	bridge pier width
H	water depth without bridge effect
h	afflux
K_θ, K_S	coefficients expressing pier alignment and shape
L	distance between two bridges
L_1	length of river reach with increased water depth
L_2	length of river reach with decreased water depth
m_1	coefficient depending on the distance between two bridges at a certain U/U_f
Q	flow discharge
S_e	slope of energy line
S_0	slope of channel bed
U	average flow velocity
U_f	average flow velocity for bankfull discharge
λ	blockage ratio (ratio of the area of the piers to the cross-sectional area of flow)
λ_L	horizontal model scale
λ_Q	discharge model scale

1. Introduction

Bridges act as an obstruction against river flow, resulting in alteration of the flow characteristics and a change in the original geometry of the nearby river bed. In subcritical flow, one typical kind of flow alteration caused by bridges is the backwater phenomenon, which results in an increase in the water surface level upstream and a reduction downstream of the

bridge; a wide longitudinal extent of the river reach is also affected. The interaction between bridge obstruction and river flow increases flooding risks and thus the probability of bridge damage, traffic disruption, loss of human life and economic losses. Researching the influence of bridges on flood control is therefore of critical importance.

The variation in the water surface level and the extent of the backwater-affected river reach are highly dependent on the river section, bridge geometry and characteristics of the flow and the floodplain (Luigia and Kebede, 2013). Many methods for investigating the backwater at bridge crossing sections have been developed. Yarnell (1934) developed the most widely used empirical equation for calculating the increase in water level caused by bridge piers. In that study, 2600 experiments were carried out considering the influence of several parameters on the afflux, including the shape of the piers, the width, the length, the angle and the flow rate. However, Yarnell (1934) did not compare the equation developed with the large amount of data collected through the experiments. Moreover, Yarnell's experiments involved a relatively small percentage of circular columns, which are now a commonly used pier shape. Through experimental studies using a large physical model, Charbeneau and Holley (2001) developed a new equation including a further two parameters as a modification of the equation proposed by Yarnell (1934). Suribabu *et al.* (2011) also suggested a modified equation, including one more parameter. Regrettably, neither of these two modifications gave a clear physical explanation of the newly added parameters in the equations. Charbeneau and Holley (2001) also reported that the length of the two-dimensional mound of water

immediately upstream of a bridge pier is no greater than the channel width.

Biery and Delleur (1962) developed a method for the prediction of the afflux at bridges based on laboratory studies using rectangular channels. However, it was found this method could lead to errors when applied to compound channels (Atabay *et al.*, 2008a, 2008b). Many other laboratory and field studies (Kaatz and James, 1997; Seckin, 2004; Seckin *et al.*, 1998) have shown that the energy equation used by the bridge subroutine in HEC-RAS (Hydrologic Engineering Center River Analysis System) (HEC, 2002) is capable of producing accurate estimates of water surface levels in river reaches constricted by bridges. However, considerable inaccuracies may arise in its application depending on the parameters chosen by the user (Seckin *et al.*, 2007). Raju *et al.* (1983) used experiments to investigate the effect of blockages on the drag coefficient of circular cylinders and obtained a relationship between the energy loss, afflux and drag force. Seckin *et al.* (2009a, 2009b) applied artificial neural network techniques to derive a regression-based formula for estimating bridge backwater based on laboratory and field data.

In many countries, rapid economic development has led to a sharp increase in traffic volume, thus requiring the construction of additional bridges across rivers and canals. Such construction could cause interactions with existing bridges and affect the characteristics of river flow and sediment motion (Wang *et al.*, 2016a, 2016b). Most of the studies cited above considered single bridges or piers, and little research considering the backwater of a group of bridges has been reported.

In the study reported here, a large physical model of the Huaihe River in China was constructed in order to investigate the effect of a group of bridges on backwater by measuring the water surface level along the plane of symmetry of piers located in the main channel. The data obtained from this study were applied to investigate two specific phenomena: (a) the effect of a single bridge and (b) the cumulative effect of a group of bridges on the backwater of the Huaihe River.

2. Experimental procedure

2.1 Experimental setup

The physical model, concerning a 22 km long reach of the Huaihe River from Bengbu sluice to Xinjiawan (Figure 1), was constructed in the State Key Laboratory of Hydrology-Water Resources and Hydraulic Engineering, Hohai University, China. The physical model is 50 m long and 4 m wide. The main model scales are shown in Table 1. The eight bridges along the river reach, in order from upstream, are Daqing Bridge (DQB), Chaoyang Bridge (CYB), K831 Old Bridge (K831OB), K831 New Bridge (K831NB), Jiefang Bridge (JFB), Tielu Bridge (TLB), Jinghu Bridge (JHB) and Changhuaiwei Bridge (CWB) (Figure 1(a)). K831OB and

K831NB are also named the K831 Twin Bridges (K831TB) because of the similar characteristics and the extremely small distance (25 m) between them. This reach of the Huaihe River was selected because of the dense distribution of bridges and the large possibility of their interaction.

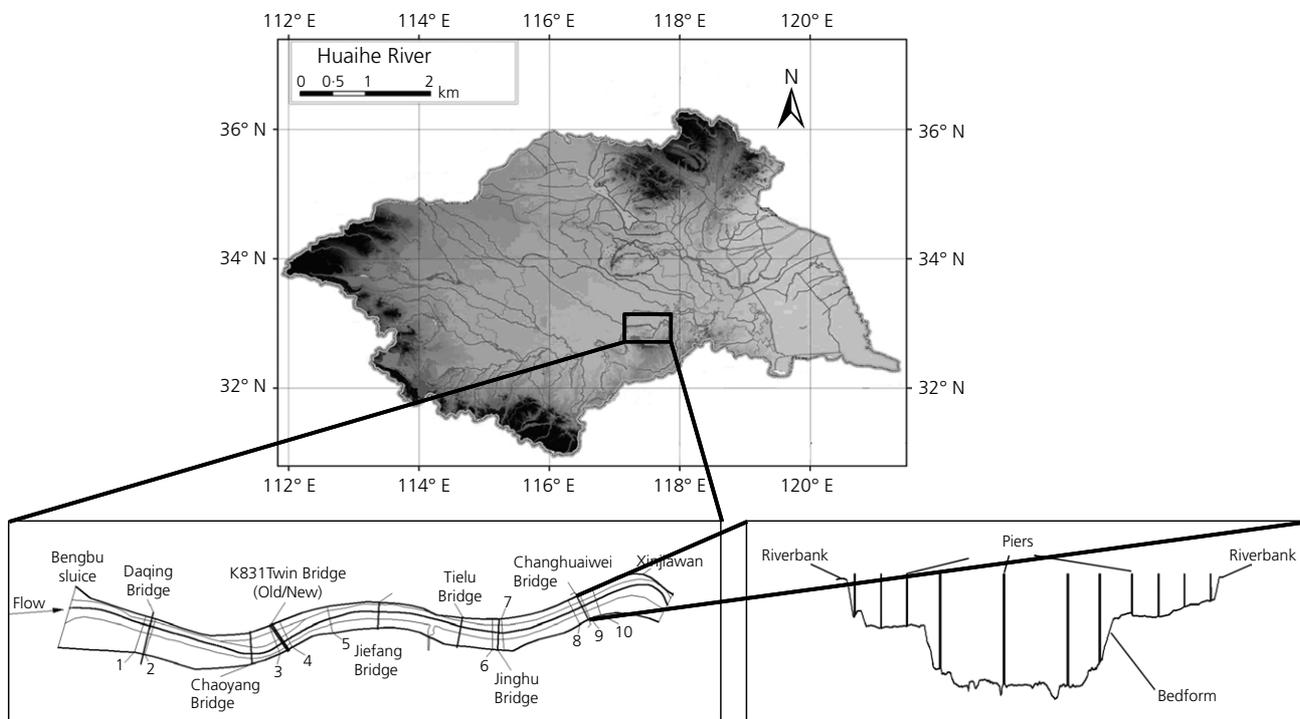
Prototypical landform data of the reach surveyed in 2013 were used to build the physical model. Since there is only one hydro-metric station established along this reach, the Huaihe River Commission of the Ministry of Water Resource (HRC-MoWR), in cooperation with Hohai University, measured the elevation of the water surface at ten temporary stations along this reach in 2014, as shown in Figure 1(a). The measured data were used to verify the reliability of the physical model. As the maximum discharge in 2014 was less than the bankfull discharge (BFD), differences between the landform of this reach in 2014 and 2013 could be ignored and the reliability of the physical model was verified using the water depths of this reach of the Huaihe River in 2014.

To represent various typical flows of the river, eight different model discharges (7.34, 8.70, 10.06, 11.46, 17.47, 26.20, 34.94 and 45.42 l/s) were used in the experiment. Using the discharge scale λ_Q shown in Table 1, the corresponding prototype discharges were 2100, 2490, 2880, 3280, 5000, 7520, 10 000 and 13 000 m³/s respectively. In this river reach, 3280 m³/s is the BFD rate and 13 000 m³/s is the discharge rate of a 100-year flood, which is the maximum discharge rate the bridges are designed against. The flow discharges were defined by a triangular weir and the experiments were performed under steady flow conditions. The model flow velocity range was 5.46–27.29 cm/s.

A comparison of the experimental and surveyed water surface elevations along the Huaihe River is shown in Figure 2. It can be seen that the experimental elevations of the water surface were slightly less than the surveyed values; this is attributable to the concrete floor of the physical model having a smaller roughness than the real river. Fortunately, the difference between the experimental water surface elevation and that of the real river was small and their variation tendencies were similar. Therefore, it was considered reasonable not to increase the roughness of the main channel in the physical model. Although effort was committed to minimising the differences between the model and the natural river, the Reynolds numbers were orders of magnitude larger in the real river because of the model scales.

2.2 Measurement of the backwater profile of the bridges

The water surface elevations along the plane of symmetry of piers located in the main channel were measured when the bridges were present in the physical model. The water level was measured by hook gauges of 0.1 mm precision. Measurements were made from 1 m upstream to 1 m downstream of the



(a)



(b)

Figure 1. The Huaihe River reach from Bengbu sluice to Xinjiawan: (a) location of the study area and test sites (1–10); (b) the physical model

Scale name	Model scale	Remarks
Horizontal scale, λ_L	400	Determined according to site conditions and test requirement
Vertical scale, λ_H	80	Determined based on geometry deformation limit condition
Geometry deformation, D_t	5	$D_t = \lambda_L / \lambda_H$
Discharge scale, λ_Q	286216.70	$\lambda_Q = \lambda_L \lambda_H^{3/2}$
Velocity scale, λ_V	8.94	$\lambda_V = \lambda_H^{1/2}$

Table 1. Main model scales

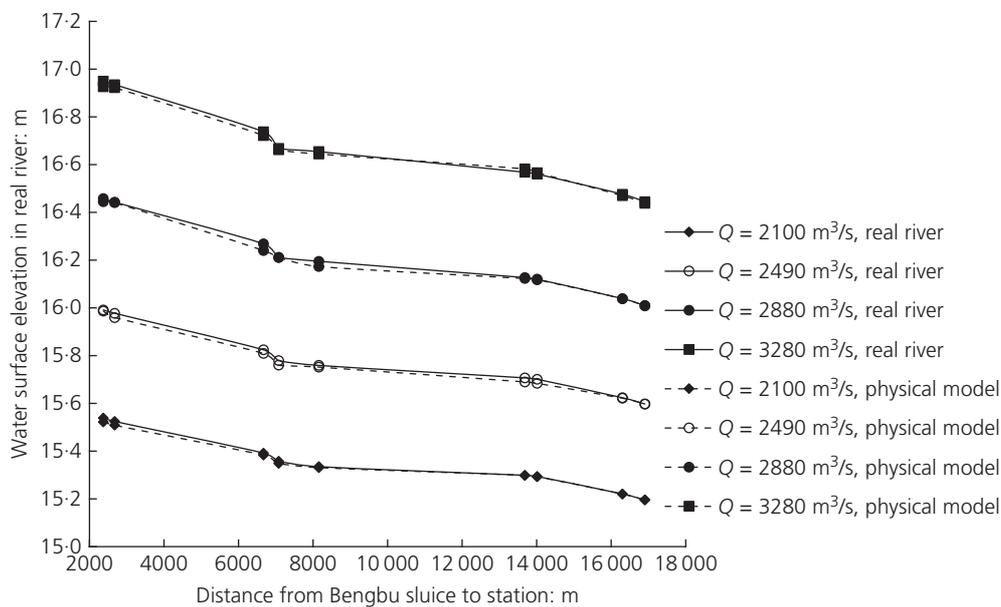


Figure 2. Comparison of the water surface elevation between the real river and the physical model

target pier at 1 cm intervals, as shown in Figure 3. During the experiments, the model bridges could be removed from the physical model so that the water surface elevations could be measured without them, using the same measuring points as when the bridges were present. Because of the limitation imposed by the decks of the model bridges, the beginning of both the downstream and upstream measuring points was set 3 cm from the piers. The pier widths of the model bridges were in the range 0.9–1.5 cm, according to the horizontal scale λ_L .

3. Dimensional analysis

In engineering applications, focus is mainly on the parameters of the backwater profiles of bridges – that is, the afflux (h), the length of the river reach with increased water depth (L_1) and the length of the reach with decreased water depth (L_2), as shown in Figure 3. Dimensional analysis for the afflux is explored in this section, which is not only used to derive empirical formulas but also to discuss the influence of

dimensionless groups on the parameters of the backwater of bridges. The afflux can be described by the following set of independent variables

$$1. \quad h = f[\text{flow}(Q, B, H, S_e), \text{pier}(D, K_\theta, K_S, \lambda), \text{channel}(B_1, B_2, S_0)]$$

where Q is the discharge, B is the width of flow, H is the water depth without the bridge where increased depth occurs, S_e is the slope of the energy line, D is the pier width, K_θ and K_S are coefficients expressing the pier alignment and shape, respectively, λ is the blockage ratio (ratio of the area of the piers to the cross-sectional area of the flow), B_1 is the width of the main channel, B_2 is the width of the compound channel and S_0 is the slope of the channel bed.

Generally, S_e and S_0 are determinants of the water surface elevation along a real river. For the Huaihe River reach

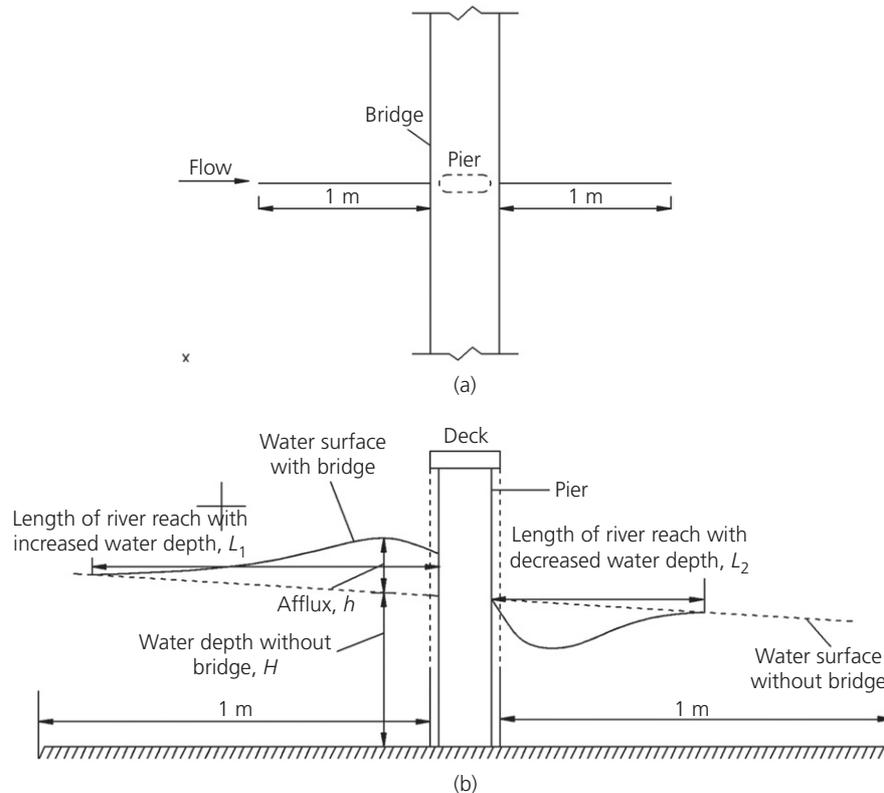


Figure 3. Measurement of the backwater of the bridges: (a) plane view; (b) side view

involved in this study – an alluvial river reach of 22 km length – the change of water surface elevation caused by the slopes is slight in subcritical conditions. Consequently, it could be assumed that S_e and S_0 have little influence on the afflux and thus they are ignored during the following analysis. Therefore, here, the afflux considers the difference resulting from the obstruction of the piers rather than the slope of the channel bed.

Considering the horizontal scale effects (λ_L), the shapes of the piers were unified as round-nosed, as their model sizes were so small that their exiguous differences could hardly influence the physical process; it was also difficult to express pier shape in the model. The HRC-MoWR rules that the skew angle of the flow direction resulting from pier alignment should be less than 10° ; therefore, it was considered that pier alignment had little influence on the afflux and thus K_θ and K_S were also not investigated in this study.

In a compound channel river, the variables H , B and B_1 are applied to express the complexity of the cross-sectional shape of the channel and the average flow velocity U is used in place of the discharge to express the flow characteristics. The blockage ratio λ is used to express obstruction to flow caused by the piers of a single bridge, especially for high velocities. Generally, a larger value of λ represents a more obvious

obstruction. The cumulative effect of the piers of a single bridge can be expressed by the blockage ratio λ . With the assumptions stated above, Equation 1 can be transformed into

$$2. \quad \frac{h}{D} = f\left(\frac{U}{U_f}, \frac{B}{H}, \frac{B}{B_1}, \lambda\right)$$

where U_f is the average flow velocity for BFD.

The experiments reported here were designed to contribute to the definition of Equation 2 and, ultimately, to evaluate the cumulative effects of a group of bridges, based on Equation 2.

4. Results and discussion

4.1 Change in water surface due to a single bridge

Firstly, only a single bridge was placed in the physical model each time. Then, the differential curves of the water surface with a single bridge and with no bridge were obtained, as shown in Figure 4 (the positive/negative value of the x -axis denotes upstream/downstream of the bridge). Figure 4 illustrates the differential curve of bridge DQB as an example, but the curves of the other bridges were similar. The values of h , L_1 and L_2 can be obtained from Figure 4. It can be seen that, as the discharge increases, these parameters increase as

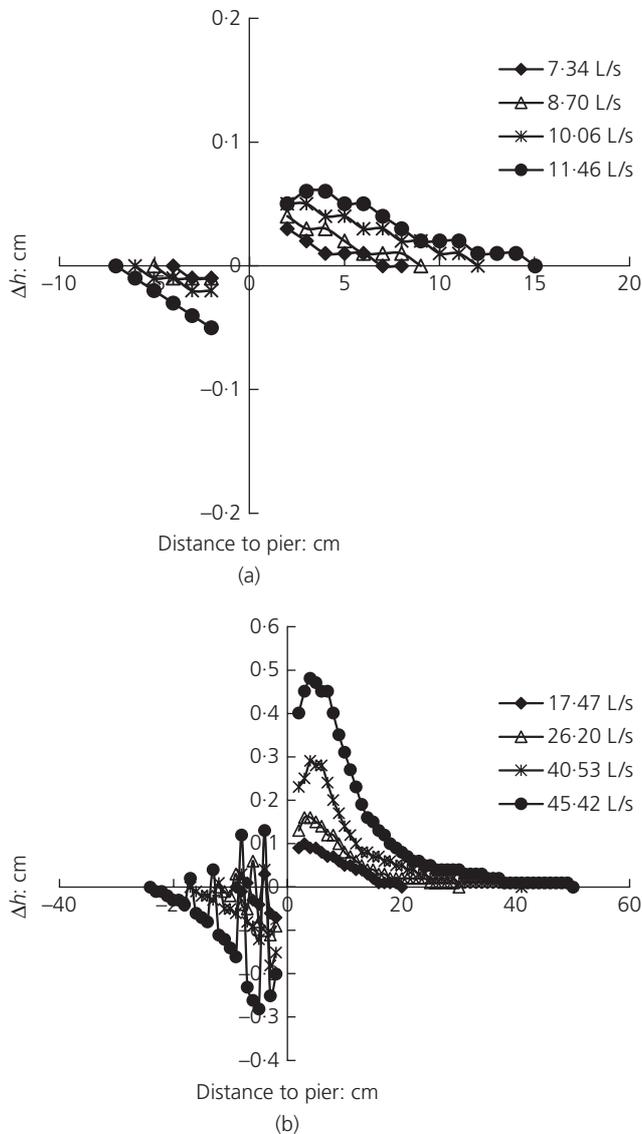


Figure 4. Differential curves of water surface between a single bridge (DQB) and no bridge: (a) discharges less than the BFD; (b) discharges exceeding the BFD

well. For each discharge less than the BFD (Figure 4(a)), the curve of the reach with decreased water depth decreases monotonically, but when the discharge exceeds the BFD, the curve of the reach with decreased water depth fluctuates. Because this phenomenon does not exist for discharges less than BFD and the gradient change of the differential curve for discharges less than BFD is much smaller than for discharges that exceed BFD, the following analysis is divided into two parts – discharges less than BFD and discharges that exceed BFD.

4.2 Afflux of a single bridge

The relationship between h/D and U/U_f is shown in Figure 5 for discharges less than the BFD. It was found that this

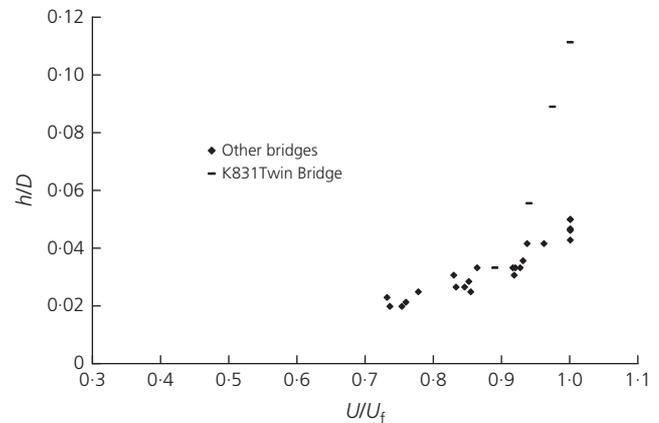


Figure 5. Afflux for discharges less than the BFD

relationship was similar for all the studied bridges, except for K831TB. Analysis of the landform of the Huaihe River revealed that the cross-sectional area of the main channel at K831TB was smaller than for the other bridges and thus the smallest discharge adopted in the experiment constituted an overbank flow in this section. Therefore, all the data for K831TB were treated as discharges exceeding the BFD.

To describe the relationship between h/D and U/U_f empirically, the following best-fit function was developed

$$3. \quad \frac{h}{D} = 0.00105e^{3.95U/U_f} \left(\frac{B}{H}\right)^{0.05}$$

as shown in Figure 6.

Equation 3 does not contain λ because it is too small (i.e. <6%) to have a significant influence on the afflux h when U/U_f is small.

Figure 7 shows the relationship between h/D and U/U_f for discharges that exceed the BFD, which indicates that h/D increases exponentially with U/U_f for each bridge. Because the shape of the cross-section of the flow is different for each bridge and the interaction between piers is obvious as the discharge increases, the curves of h/D and U/U_f for each bridge are dissimilar. Considering the above analysis, the experimental data were substituted into Equation 2 and the following relationship was obtained

$$4. \quad \frac{h}{D} = 0.0851e^{1.464U/U_f} \left(\frac{B}{(B_1H)^{0.5}}\right)^{2.2} (100\lambda)^{0.027}$$

Equation 4 considers the impact of the blockage coefficient λ on the afflux, based on the following: (a) when the discharge exceeds the BFD, the blockage coefficient will increase (up to

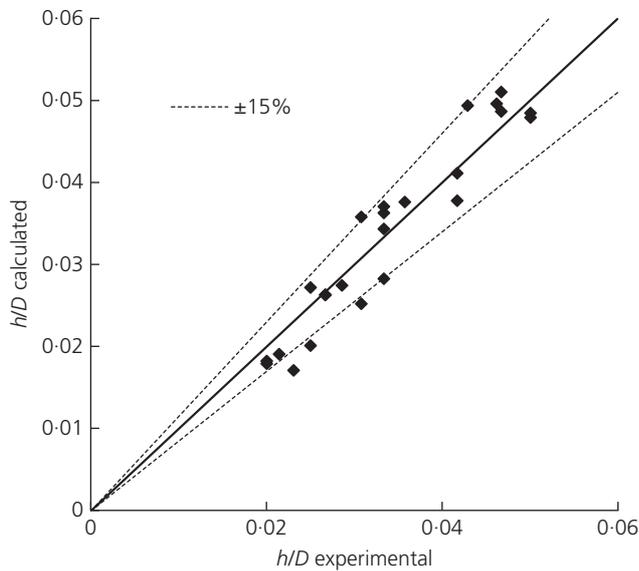


Figure 6. Comparison of experimental and calculated afflux for discharges less than the BFD

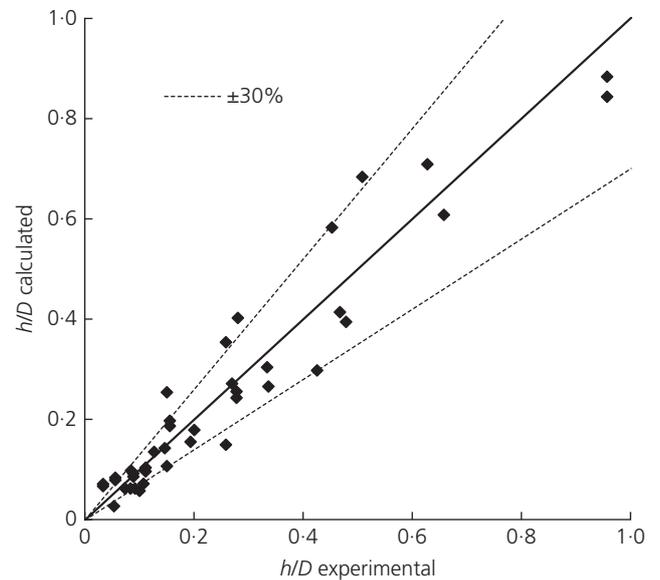


Figure 8. Comparison of experimental and calculated values for discharges exceeding the BFD

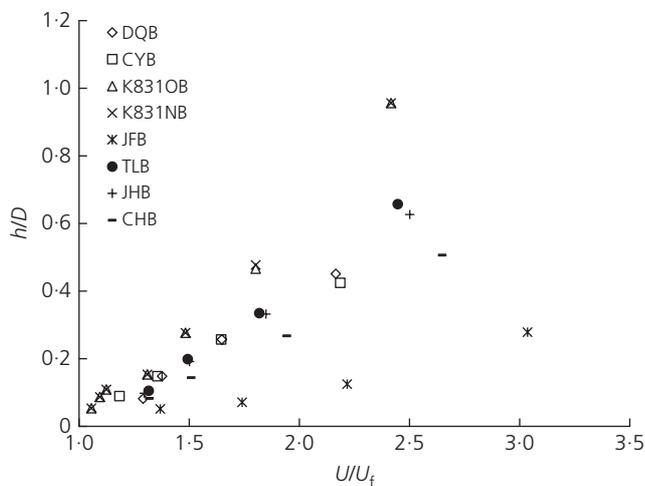


Figure 7. Afflux for discharges exceeding the BFD

9.11%) as piers located in the floodplain will also obstruct the flow; (b) as the velocity increases, the interaction among piers in a transverse arrangement becomes more obvious. Figure 8 shows the relationship between the observed h/D values and those computed using Equation 4.

4.3 Length of the reach with increased water depth and with decreased water depth

Based on the experimental data, it was found that the length of the river reach with increased water depth (L_1) and the length of the river reach with decreased water depth (L_2)

increase linearly with h/D ; the equations can thus be expressed as

$$5. \quad \frac{L_1}{D} = 281.76 \frac{h}{D}$$

$$6. \quad \frac{L_2}{D} = 167.12 \frac{h}{D}$$

as shown in Figures 9 and 10, respectively.

Here, L_1/D , L_2/D and h/D conform to an exponential relationship and the equations can be expressed as

$$7. \quad \frac{h}{D} = 0.0239e^{0.0624L_1/d}$$

$$8. \quad \frac{h}{D} = 0.0385e^{0.01096L_2/d}$$

as shown in Figures 11 and 12, respectively.

The correlation between L_2/D and h/D is not strong. However, in engineering applications, the focus tends to be mainly on the maximum possible value of L_2 for the decreased water depth, therefore Equation 8 represents an envelope that can be considered reasonable.

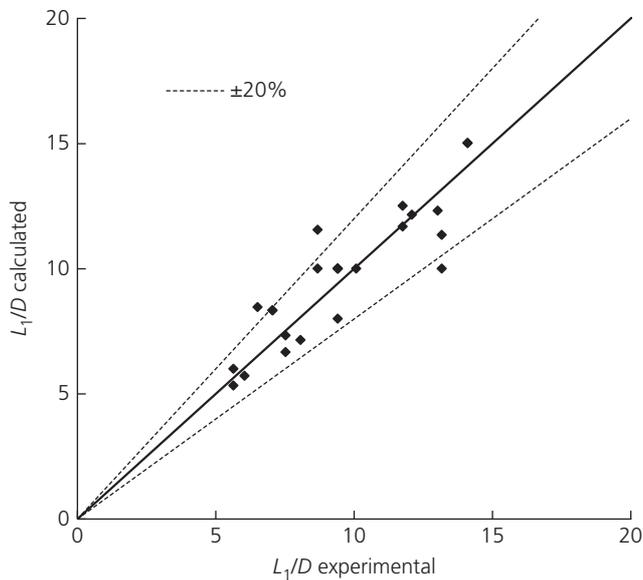


Figure 9. Comparison of experimental and calculated L_1/D for discharges less than the BFD

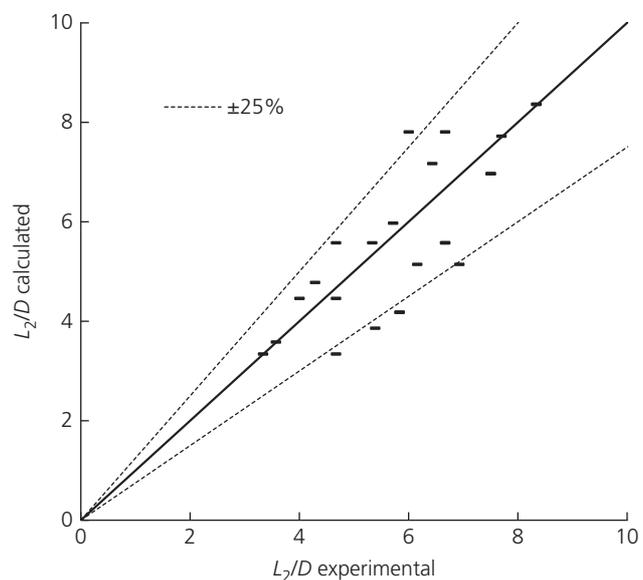


Figure 10. Comparison of experimental and calculated L_2/D for discharges less than the BFD

Obviously, the backwater characteristics of a single bridge with discharges less than the BFD are different from those with discharges exceeding the BFD. This may explain why some research conducted in a rectangular channel can lead to inaccuracy when applied to compound channels.

4.4 Cumulative effect of two bridges on backwater
Based on the analysis of the backwater of a single bridge, the cumulative effect of two bridges on backwater was investigated

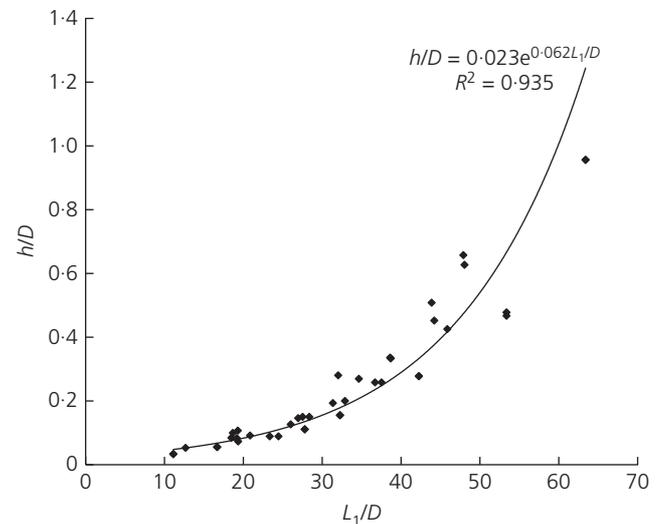


Figure 11. Relationship between h/D and L_1/D for discharges exceeding the BFD

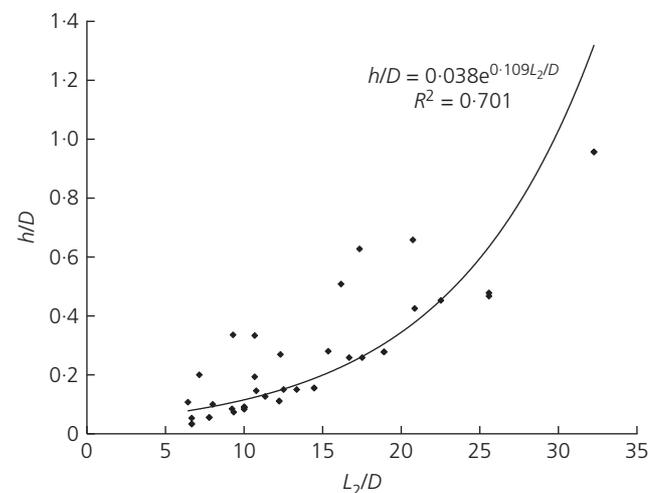


Figure 12. Relationship between h/D and L_2/D for discharges exceeding the BFD

for different combinations of two bridges. Only four cases showed an observable cumulative effect on the backwater of the upstream bridge under certain discharges, as shown in Table 2. For cases 2, 3 and 4, the values of h , L_1 and L_2 were the same as those for a single bridge when the discharge was less than 34.94 l/s (corresponding to 10 000 m³/s in full scale). When the discharge was increased to 34.94 l/s, the value of h was larger than that of a single bridge, while L_1 and L_2 remained the same. Finally, when the discharge was increased to 45.42 l/s (13 000 m³/s in full scale), the values of h and L_1 were larger than those of the single bridge, with the value of L_2 remaining the same. Therefore, it was established that the cumulative effect of two bridges becomes more obvious as

Case	Bridges	L/D	Model discharge: l/s	h	L_1	L_2
1	K831OB and K831NB	6.94	7.34–45.42	Y	Y	Y
2	CYB and K831OB	208.33	34.94	Y	N	N
			45.42	Y	Y	N
3	CYB and K831NB	213.54	34.94	Y	N	N
			45.42	Y	Y	N
4	TLB and JHB	214.28	34.94	Y	N	N
			45.42	Y	Y	N

Table 2. Cases for which a cumulative effect of two bridges existed ('Y' means the cumulative effect of two bridges existed; 'N' means the cumulative effect of two bridges did not exist)

discharge increases and that the cumulative effect is more significant on h and has least effect on L_2 .

A cumulative effect of two bridges on the backwater of the downstream bridge was not found in cases 2, 3 and 4. It is thus considered that the cumulative effect of two bridges is mainly on the upstream bridge. In case 1, K831OB is so close to K831NB that it is difficult to distinguish between the L_1 of K831NB and the L_2 of K831OB. Therefore, only L_1 of K831OB (the upstream bridge) and L_2 of K831NB (the downstream bridge) were obtained. Figure 13 shows the discrepancy in backwater parameters between K831TB and K831OB (or K831NB, as the values of K831NB were the same as those of K831OB under the single-bridge condition). The discrepancy becomes more obvious with an increase in discharge, meaning that the cumulative effect is more obvious as discharge increases. These findings agree with those derived from cases 2, 3 and 4.

The backwater parameters for case 1 can be expressed as

$$9. \quad \frac{h_t}{D} = 1.45 \frac{h}{D}$$

$$10. \quad \frac{L_{1t}}{D} = 1.29 \frac{L_1}{D}$$

$$11. \quad \frac{L_{2t}}{D} = 1.26 \frac{L_2}{D}$$

where the subscript 't' denotes two bridges, as shown in Figure 14. According to the coefficients in Equations 9–11, the cumulative effect on h is the most significant, which agrees with the results for cases 2, 3 and 4.

4.5 Maximum extent of the cumulative effect

According to the above findings, the cumulative effect of two bridges increases the backwater parameters compared with a

single bridge, which undoubtedly increases the difficulty of flood control. Therefore, determination of the maximum extent of the cumulative effect is obviously important for field engineering practice.

The conclusions in Section 4.4 reveal that the cumulative effect on the afflux h is the most significant. Therefore, to establish the maximum extent of the cumulative effect, all that is needed is to determine the threshold distance between the two bridges for which the value of h no longer changes (i.e. the same as single-bridge conditions). Since the cumulative effect occurs only when the discharge exceeds the BFD, Equation 4 for a single bridge is taken as a standard to obtain the threshold distance in the following analysis.

Firstly, Equation 4 is transformed into the following form, which describes the relationship between the afflux and flow condition for a single bridge in another way

$$12. \quad \frac{h}{D} / \left[\left(\frac{B}{(B_1 H)^{0.5}} \right)^{2.2} (100\lambda)^{0.027} \right] = 0.0851 e^{1.464 U/U_f}$$

Then, a comprehensive parameter to describe the afflux characteristics of either two bridges or a single bridge is defined as

$$13. \quad F(h/D) = \frac{h}{D} / \left[\left(\frac{B}{(B_1 H)^{0.5}} \right)^{2.2} (100\lambda)^{0.027} \right]$$

To derive the threshold distance between two bridges, the experimental results are illustrated in Figure 15 in terms of the relationship between $F(h/D)$ and U/U_f . In the figure, parallel lines are drawn at the maximum experimental values of U/U_f (corresponding to the maximum discharge of 45.42 l/s (13 000 m³/s in full scale)) for the four cases listed in Table 2. All the parallels have the same slope with the derivative of

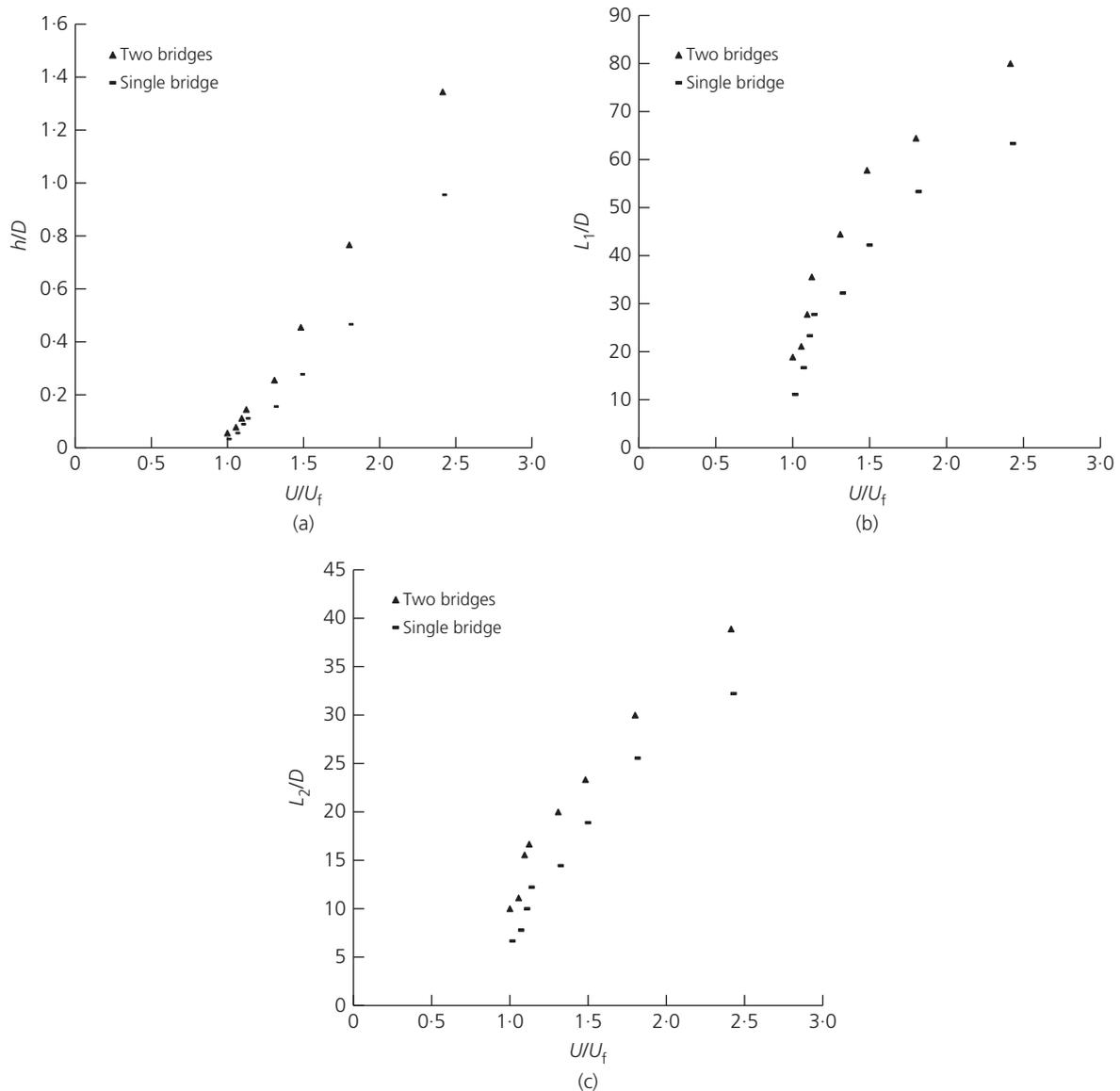


Figure 13. Discrepancy of backwater parameters between K831TB and K831OB: (a) h/D ; (b) L_1/D ; (c) L_2/D

Equation 12 at the maximum U/U_f (i.e. a single-bridge condition), so that we can obtain the maximum values of $F(h/D)$ for each case corresponding to the maximum U/U_f of the whole reach. Furthermore, the impact of the distance between two bridges L on the afflux was assessed. It was found that the maximum $F(h/D)$ of two bridges became closer to that of a single bridge as L/D increases, as shown in Figure 15.

Because of the different section shapes, the maximum experimental values of U/U_f for cases 2 and 3 are smaller than those of the whole reach. Thus, the maximum $F(h/D)$ for these two cases would be larger than the actual values under the exponential law, as they were also obtained using the ‘parallel method’

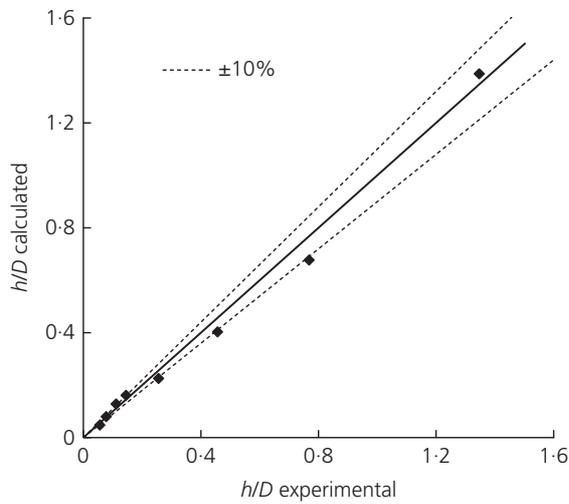
by extending to the maximum U/U_f of the whole reach. This kind of treatment is safer for engineering applications.

Based on the above analysis, let us assume that

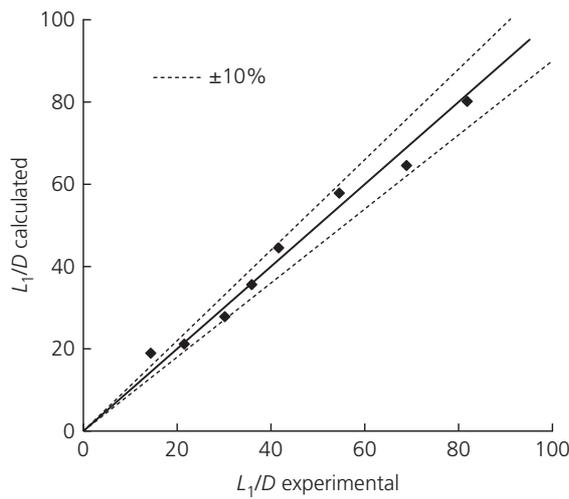
$$14. \quad F_t(h/D) = m_1 F_a(h/D)$$

where subscripts ‘t’ and ‘a’ denote two bridges and a single bridge, respectively, and the coefficient m_1 depends on the distance between two bridges (L) at a certain U/U_f .

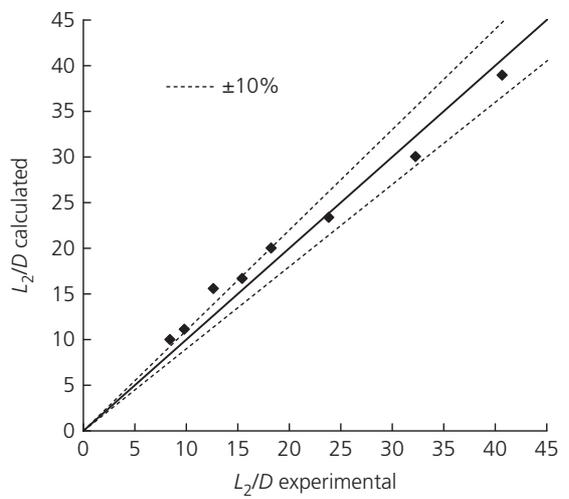
When $m_1 = 1$, it means that there is no difference between the two-bridge condition and the single-bridge condition, and



(a)



(b)



(c)

Figure 14. Comparison of experimental and calculated backwater parameters for K831TB: (a) h/D ; (b) L_1/D ; (c) L_2/D

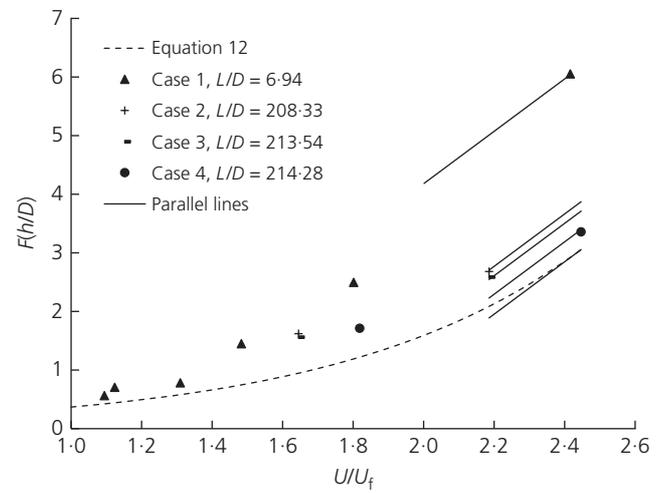


Figure 15. Variation of $F(h/D)$ with U/U_f

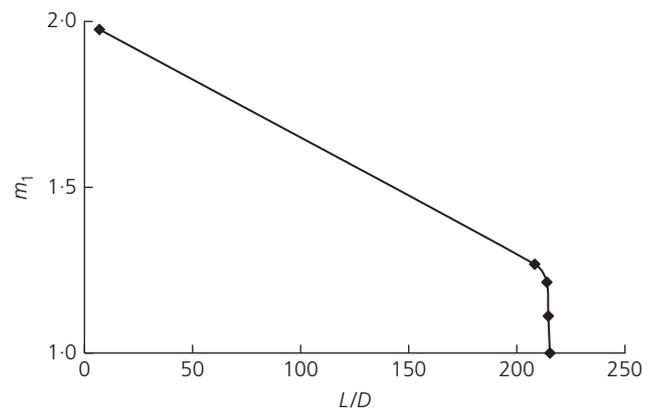


Figure 16. Variation of m_1 with L/D

the corresponding L could be considered as the threshold distance between two bridges at a certain U/U_f . Figure 16 shows the relationship between m_1 and L/D at the maximum U/U_f of the whole reach. Extending the curve to the x -axis ($m_1 = 1$) yields $L/D = 215$. This means that the cumulative effect disappears when $L/D = 215$ and thus this is defined as the maximum extent of the cumulative effect on bridge afflux under two-bridge conditions.

5. Conclusions

The following conclusions were drawn from this study of the backwater of a group of bridges on the Huaihe River in China.

- (a) When the discharge is less than bankfull discharge (BFD), Equations 3, 5 and 6 can be used to describe the parameters of the backwater of a single bridge. When the

discharge exceeds the BFD, Equations 4, 7 and 8 describe those parameters.

- (b) There is a cumulative effect of two bridges on the backwater for certain discharges and distances. The cumulative effect on the afflux is most obvious and least evident on the length of the reach with decreased water depth.
- (c) By analysing discrepancies between the backwater shapes of a single bridge and two bridges, it was established that the value of $L/D = 215$ represents the minimum separation between nearby bridges for the cumulative effect to disappear along the studied reach of the Huaihe River.

It is hoped that these experimental results will provide useful guidelines for future bridge construction and embankment maintenance on the Huaihe River. The cumulative effect of hydraulic structures increases the difficulty of flood control in the river, and the present results provide technical support as to how such effects can be avoided.

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