Backwater effect of multiple bridges along Huaihe River, China

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Abstract: This study investigates the backwater effect of the eight bridges along the Huaihe River in China using laboratory experiments. The experiments revealed obvious differences between backwater regulation when the discharge was less than the bankfull discharge compared with the condition of bankfull discharge. The experimental data obtained were first used to derive equations to parameterize the backwater effect of a single bridge. Then, the cumulative effect of two bridges was analyzed, with the backwater effect of a single bridge

used as a reference. It was found that to eliminate the cumulative effects, the minimum separation between the two bridges should be no less than 215 times the bridge pier width.

Keywords: backwater effect; bridges; cumulative effect; Huaihe River; hydraulic experiment

1. Introduction

A bridge can alter the natural geometry of a river section, becoming an obstacle to the river flow, which then has to change its own natural pattern. In subcritical conditions, this flow alteration can result in the backwater effect, which is an increase in the elevation of the water surface upstream of the bridge and a reduction downstream. The extent of the backwater-affected area and the magnitude of the increase of the elevation of the water surface are highly dependent on the river section, bridge geometry, and the flow and floodplain characteristics(Luigia and Kebede, 2013). Flood events at bridge crossings can cause traffic disruption, damage to property, and loss of human life and therefore, research on the effects of bridges in relation to flooding is of critical importance.

Many methods have been developed for the investigation of backwater at bridge crossings. For example, Biery and Delleur(1962)developed a method for the prediction of afflux at bridges based on laboratory studies using rectangular channels; however, this could lead to errors when applied to compound channels(Atabay et al., 2008a; Atabay et al., 2008b). Manyother 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 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 they obtained a relationship between the energy loss, afflux, and drag force. Seckin et al. (2009)applied artificial neural network techniques to derive a regression-based formula for estimating bridge backwater based on laboratory and field data.

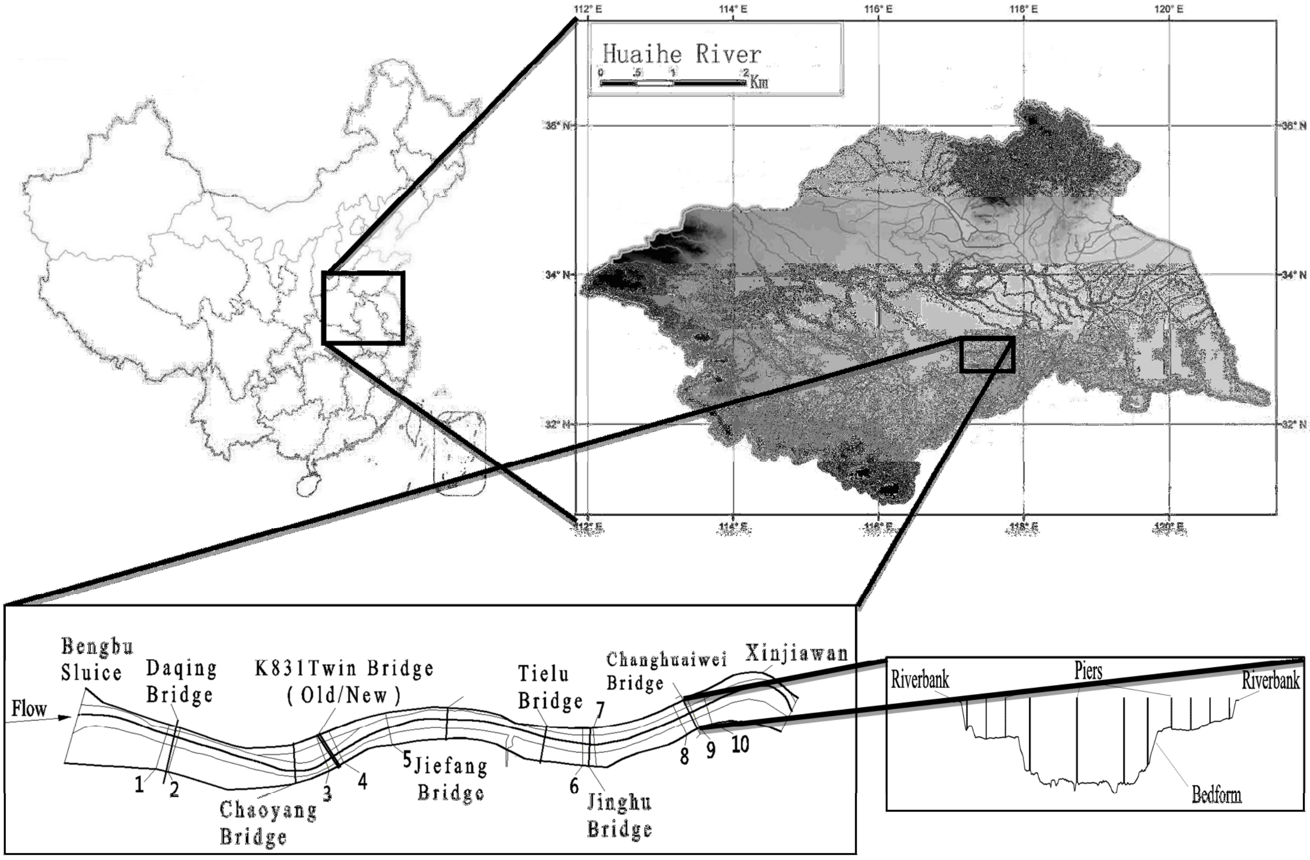
In many countries, rapid economic development has led to an increase in traffic volume that has required the construction of additional bridges across rivers and canals, which can cause interaction with existing bridges and affect the characteristics of river flow and sediment motion(Wang et al., 2015).Most of the above studies have considered single bridges and little research has been undertaken regarding the backwater of a group bridges. Therefore, in this study,a physical model of the Huaihe River in China was conducted to investigate the effect of a group of bridges on backwater by measuring the elevation of the water surface along the plane of symmetry of the piers located in the main channel. The data of this study were obtained to investigate two specific phenomena: (1)the effect of a single bridge and (2) 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 includes a section of the Huaihe River from the Bengbu Sluice to Xinjiawan (Fig. 1), which is constructed at the State Key Laboratory of Hydrology-Water Resources and Hydraulic Engineering, Hohai University, China and has dimensions of 50 m in length and 4 m in width. The main model scales are displayed in Table 1. Along the 22-km length of this reach of the natural river are eight bridges, which in order from upstream are the 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) (Fig. 1a).K831OB and K831NB are also named the K831 Twin Bridges(K831TB) because they have similar characteristics and they are separated by a distance of only 25m. This reach of the Huaihe River was selected because of the dense distribution of bridges and the large possibility of their interaction.

The landform of this reach of the Huaihe River surveyed in 2013 was used to construct the physical model. Because there is only one hydrometric station established on this section of the river, the Huaihe River Commission of the Ministry of Water Resource, in cooperation with Hohai University, measured the elevation of the water surface at ten stations along this reach in 2014, as shown in Fig.1(a). The measured data were used to verify the reliability of the physical model. Because the discharges in 2014 were less than the bankfull discharge, differences between the landform of this reach in 2014 and 2013 could be ignored and we verify the reliability of the physical model using the water depths of this reach of the Huaihe River in 2014.



(a) Location of the study area and test sites (1–10)

(b)The physical model

Fig.1. The Huaihe River reach from the Bengbu Sluice to Xinjiawan

Table 1. Main model scales

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

To represent different typical flows of the Huaihe River, eight different model discharges were used in the experiment (i.e., 7.34, 8.70, 10.06, 11.46, 17.47, 26.20, 34.94, and 45.42 l/s). Using the discharge scale λ_Q in Table 1, the corresponding prototype discharges were 2100, 2490, 2880, 3280, 5000, 7520, 10000, and 13000 m^3/s, respectively. In this reach of the Huaihe River, 3280 m^3/s is the bankfull discharge rate and 13,000 m^3/s is the discharge rate of a 100-year flood, which is the maximum discharge rates that the bridges are designed against. The discharges were determined using an orifice plate in the supply pipe, and the experiments were performed under steady flow conditions.

A comparison of the experimental and surveyed water surface elevations along the Huaihe River is shown in Fig.2. It can be seen that the experimental elevations of the water surface is slightly less than that of the surveyed values, which is attributable to the concrete floor of the physical model having a smaller roughness length than the real river. The traditional method for increasing the roughness is to glue gravel particles to the floor and these gravel particles force the local elevation of the water surface to increase. However, the diameters of the gravel particles and the pier widths(the pier widths of the bridges were 0.9–1.5cm, according to the horizontal scale λ_L) have the same order of magnitude. Fortunately, the difference between the experimental elevation of the water surface and that of the real river was small and their variation tendency was similar. Therefore, we considered it reasonable not to increase the roughness of the physical model. Although effort was committed to minimizing 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.

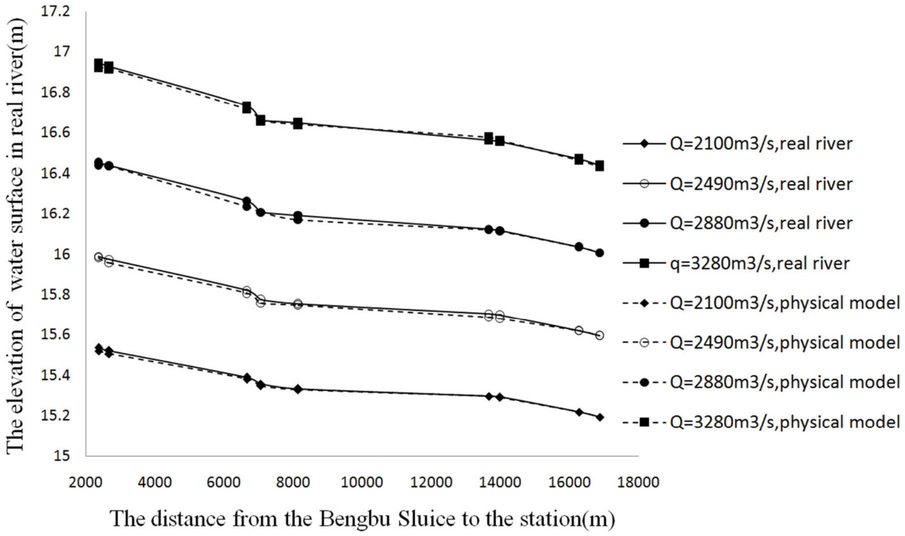


Fig.2. Comparison of the water surface elevation between the real river and the physical model

2.2 Measurement of the backwater profile of the bridges

Herein, the water surface elevations along the plane of symmetry of the piers located in the main channel were measured when the bridges were present in the physical model. A water level gaugewas used to measure the water surface elevations to the accuracy of $\pm$0.1 mm. Measurements were made from 1m upstream to 1m downstream of the pier at 1-cm intervals, as shown in Fig.3. However, during the experiments, the model bridges could be removed from the physical model and the water surface elevations 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 the upstream measuring points was set 2cm from the piers.

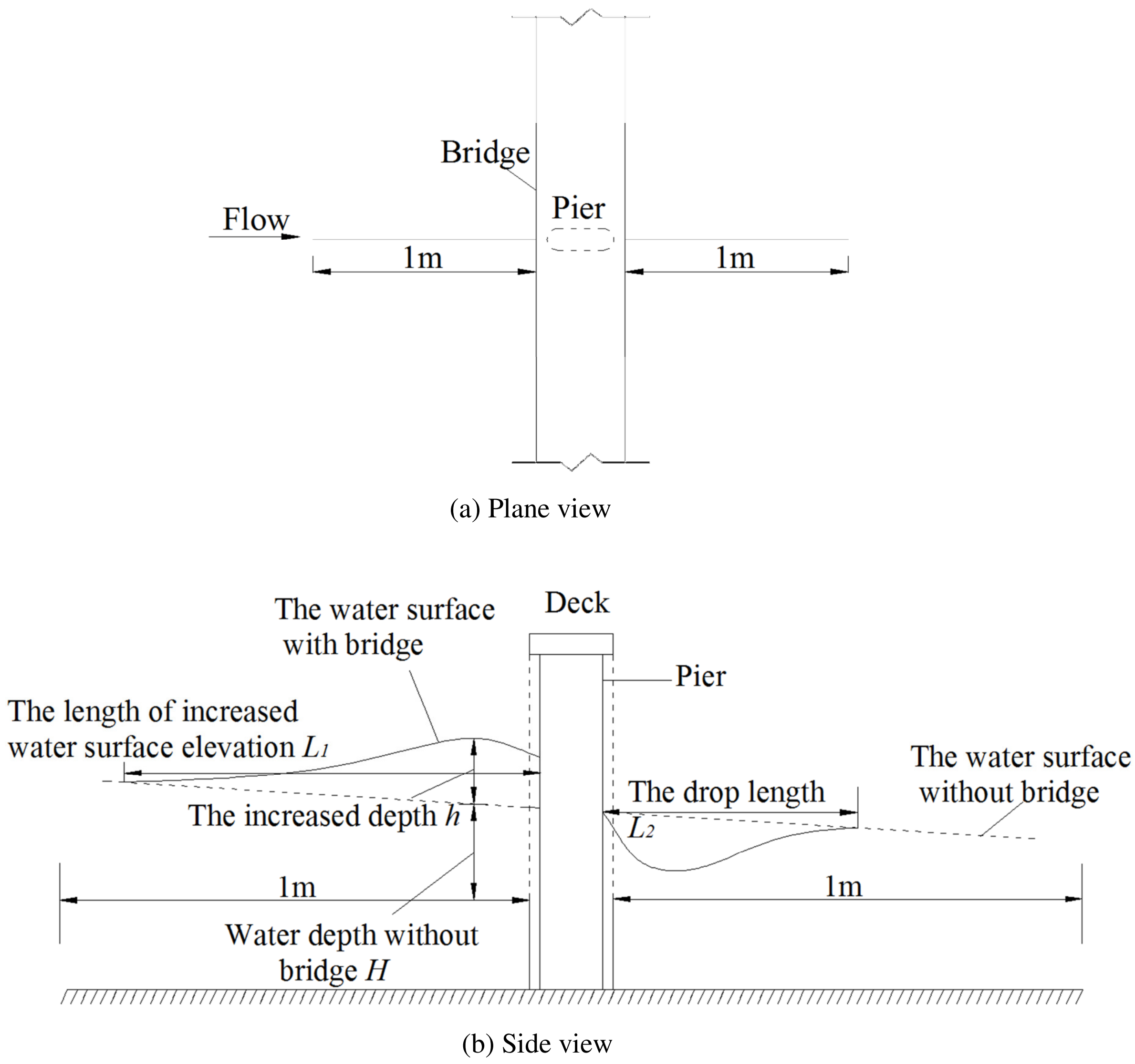


Fig.3. Measurement of the backwater of the bridges

3. Dimensional analysis

In engineering applications, the focus is mainly on the parameters of the backwater profiles of the bridges, i.e., the maximum increased water depth 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 Fig.3. The dimensional analysis for the maximum increased depth h 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 for the backwater of bridges.

The increased depth h can be described by the following set of independent variables:

$$h = f\left[flow(Q, B, H, S_e), Pier(D, K_\theta, K_S, \lambda), channel(B_1, B_2, S_0)\right], \tag{1}$$

where Q is the discharge; B is the width of the flow; H is the water depth without the bridge where the 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_e is the slope of the channel bed.

For the real river, S_e and S_e are determinants of the water surface elevation along the Huaihe River. For an alluvial river, the change of water surface elevation is slight in subcritical conditions. Herein, the increased depth is considered the difference resulting from the location of the piers rather than the afflux phenomenon resulting from constriction because the bridges across the Huaihe River are not arched and have only a few piers positioned within the river. Consequently, it is thought that S_e and S_e have little influence on the increased depth and thus they are ignored. This assumption is another reason why we did not increase the roughness of the physical model.

For the limitation of the horizontal scale λ_L, we unified the shape of the piers (round-nosed pier) because their sizes were small and their exiguous differences difficult to express in the models. The Huaihe River Commission of the Ministry of Water Resource requires that the skew angle of the flow direction resulting from pier alignment should be <10° and therefore, it was considered that pier alignment had little influence on the increased depth and thus, K_θ and K_S were not investigated in this study.

As 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 discharge to express the flow characteristics. The blockage ratio λ is used to express the obstruction to the flow caused by the piers of a single bridge, especially for high velocities. Generally, the larger the value of λ, the more obvious the obstruction. Under these assumptions, Eq.(1) can be transformed into:

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

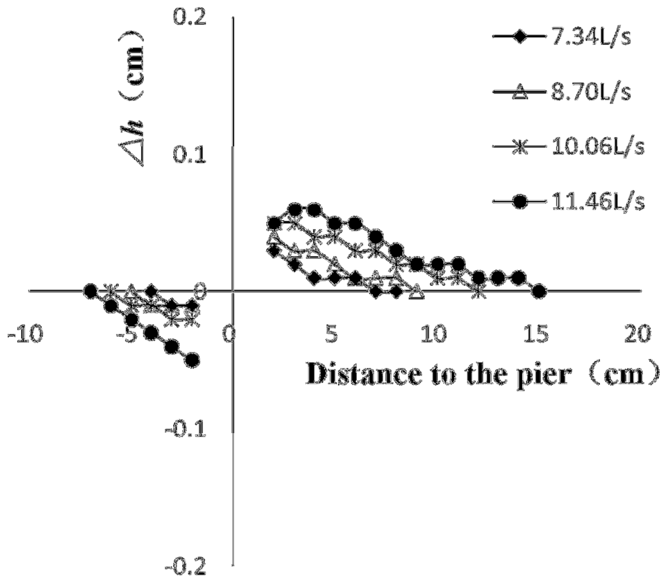
where U_f is the average flow velocity for bankfull discharge.

The experiments reported herein were designed to find the exact formulation of Eq.(2) and ultimately, to the cumulative effects of a group of bridges, based on Eq.(2).

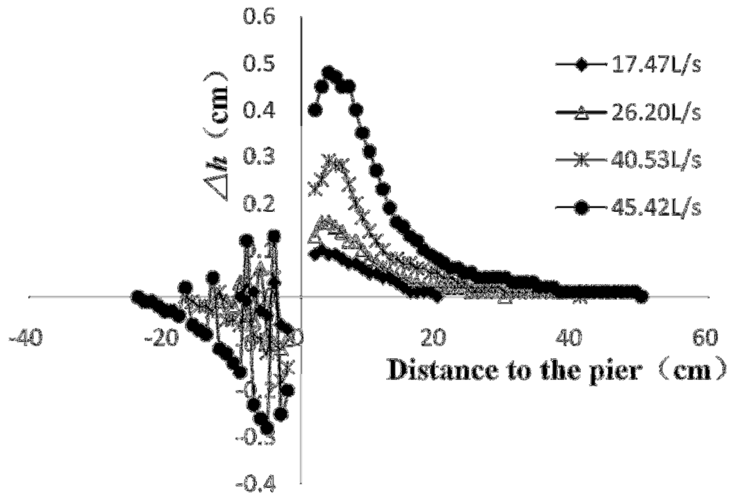
4. Results and discussion

4.1 The change in the water surface due to a single bridge

First, only a single bridge was placed in the physical model. Then, the differential curve of the water surface between a single bridge and no bridge was obtained, as shown in Fig.4(the positive (negative) value of the x-axis denotes the upstream (downstream) of the bridge).Figure 4 illustrates the differential curve of the bridge DQB as an example because the others curves were similar. The values of the increased depth h , length of increased water surface elevation L_1 , and drop length L_2 can be obtained from Fig.4. It can be seen that as discharge increases, all the parameters increase as well. For discharges of less than bankfull discharge, the curve of the drop decreases monotonically, but when discharge exceeds bankfull discharge, the curve of the drop fluctuates. This might be caused by the cumulative effect of strong vortices shed alternately from the rear of the piers(Breusers et al., 1977). The cumulative effect of the piers of single bridges is expressed hereafter bythe blockage ratio λ . Because this phenomenon does not exist for discharges of less than bankfull discharge, and because the gradient change of the differential curve for discharges of less than bankfull discharge is smaller than for discharges that exceed bankfull discharge, the analysis below is divided into two parts: discharges of less than bankfull discharge and discharges that exceed bankfull discharge.



(a) Discharges of less than bankfull discharge



(b) Discharges that exceed bankfull discharge

Fig.4 Differential curves of water surface between a single bridge and no bridge(DQB)

4.2 The increased depth h

The relationship between $\frac{h}{D}$ and $\frac{U}{U_f}$ is shown in Fig.5 for discharges less than bankfull discharge. It was found that this relationship for all the studied bridges was similar, 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 are treated as discharges that exceed bankfull discharge.

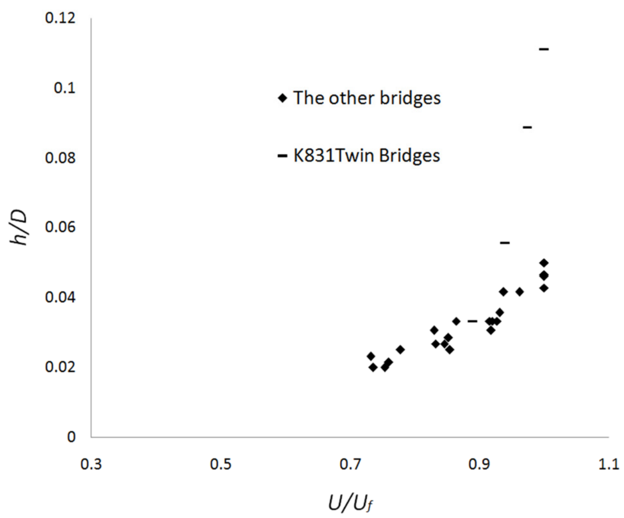


Fig.5 Increased depth for discharges of less than bankfull discharge

To describe the relationship between $\frac{h}{D}$ and $\frac{U}{U_f}$ empirically, a best-fit function was developed:

$$\frac{h}{D} = 0.00105e^{3..95U/U_f}\left(\frac{B}{H}\right)^{0.05}, \tag{3}$$

as shown in Fig. 6.

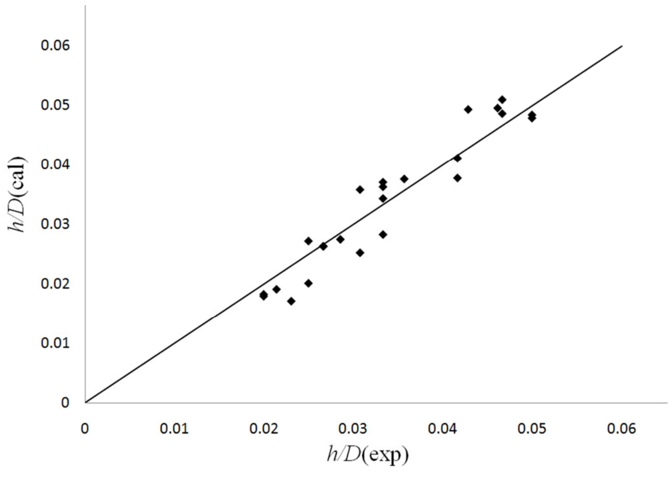


Fig.6 Comparison of the experimental and calculated increased depth for discharges of less than bankfull discharge

Equation (3) does not contain λ because λ is too small (i.e., <6%) to have significant influence on the increased depth h when $\frac{U}{U_f}$ is small.

Figure 7 shows the relationship between $\frac{h}{D}$ and $\frac{U}{U_f}$ for discharges that exceed bankfull discharge, which indicates that $\frac{h}{D}$ increases exponentially with $\frac{U}{U_f}$ for each bridge. Because the shape of the cross section of the flow is different for each bridge, and because the interaction between piers is obvious as the discharge increases, the curves of $\frac{h}{D}$ and $\frac{U}{U_f}$ for each bridge are dissimilar. Considering the above, the experimental data were substituted into Eq. (2) and the following relationship obtained:

$$\frac{h}{D}=0.0408e^{2.578U/U_f}\left(\frac{B}{(B_1H)^{0.5}}\right)^{2.7}(100\lambda)^{0.027}. \tag{4}$$

Figure 8 shows the relationship between the observed $\frac{h}{D}$ values and those computed using Eq. (4).

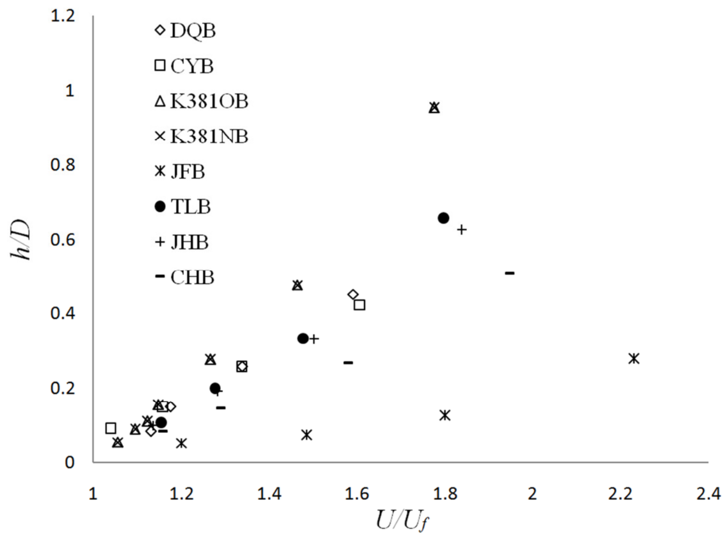


Fig.7. Increased depth for discharges that exceed bankfull discharge

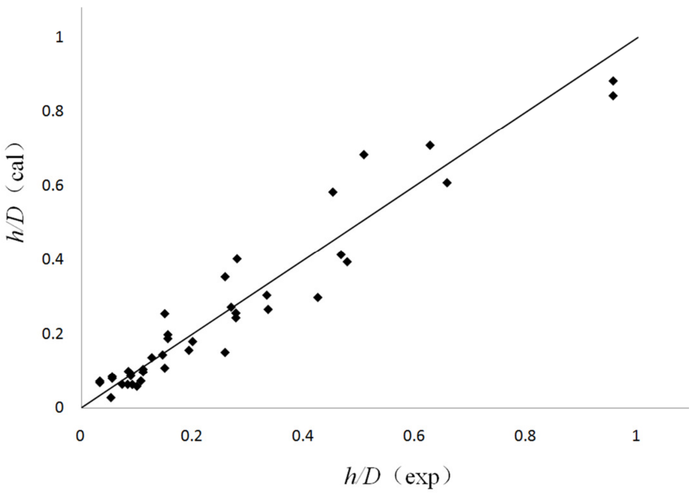


Fig.8 Comparison of the experimental and calculated increased depth for discharges that exceed bankfull discharge

4.3 The length of increased water surface elevation L_1 and drop length L_2

Based on the experimental data, it was found that L_1 and L_2 increase linearly with $\frac{h}{D}$; thus, the equations can be expressed as follows:

$$\frac{L_1}{D} = 281.76\frac{h}{D}, \tag{5}$$

$$\frac{L_2}{D} = 167.12\frac{h}{D}, \tag{6}$$

as shown in Figs. 9 and 10, respectively.

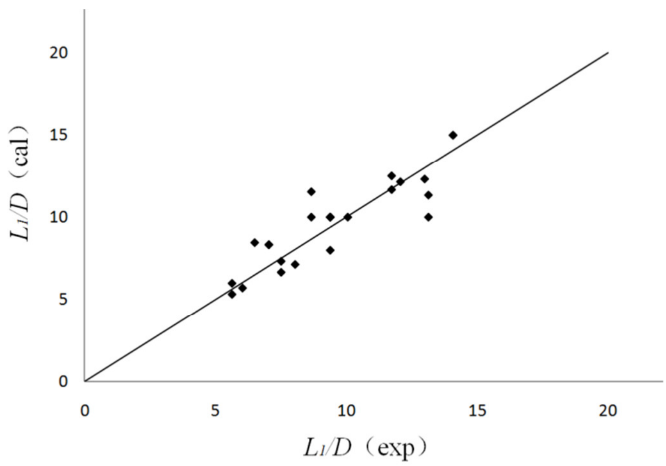


Fig.9 Comparison of the experimental and calculated L_1 / D for discharges of less than bankfull discharge

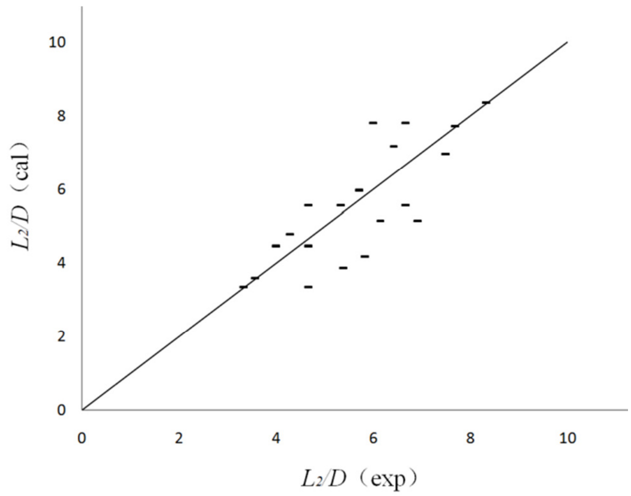


Fig.10 Comparison of the experimental and calculated L_2 / D for discharges of less than bankfull discharge

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

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

$$\frac{h}{D} = 0.0385e^{0.01096 L_1 / d}, \tag{8}$$

as shown in Figs. 11and 12, respectively.

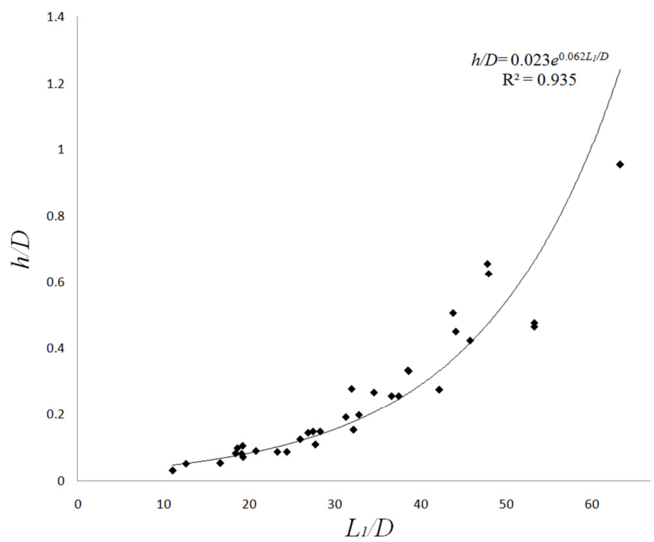


Fig.11 Relationship between h / D and L_1 / D for discharges that exceed bankfull

discharge

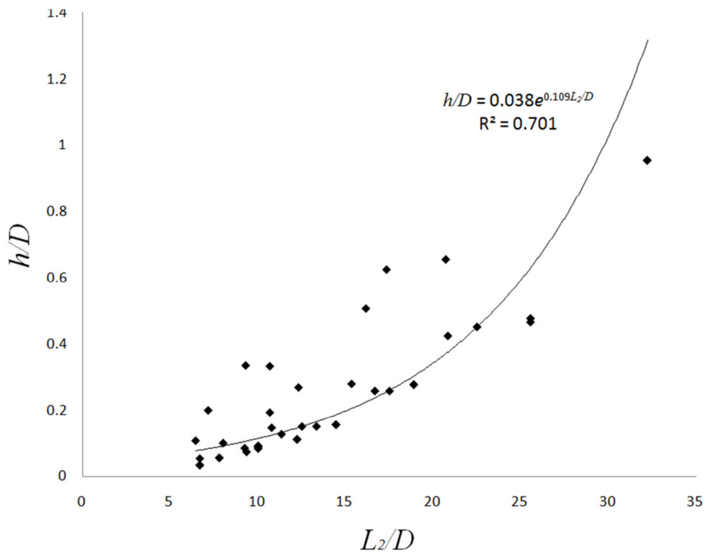


Fig.12 Relationship between h/D and L_2/D for discharges that exceed bankfull discharge

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 increased depth h and therefore, Eq.(8) represents an envelope that can be considered reasonable.

4.4 Cumulative effect of two bridges on backwater

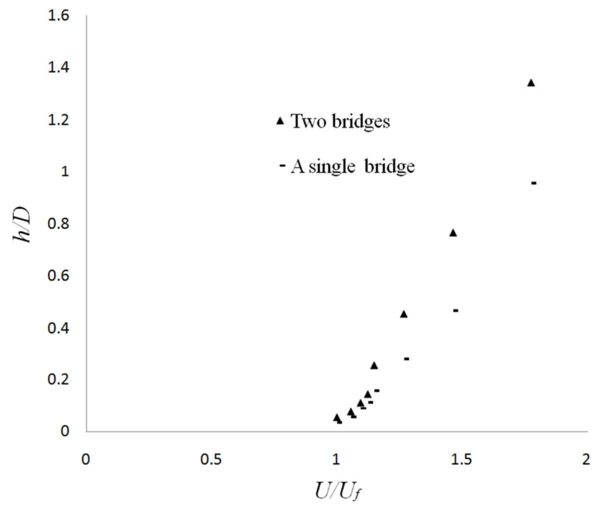
Based on the analysis of the backwater of a single bridge, the backwater of two bridges was investigated using combinations of two bridges. From all the tests, we found only four cases for which there was a cumulative effect of the two bridges on the backwater of the upstream bridge under certain discharges, as shown in Table 2. For cases 2, 3, and4, when the discharge was <34.94L/s(the natural discharge is 10,000m^3/s), the values of the increased depth h, length of increased water surface elevation L_1, and drop length L_2 were the same as for a single bridge. The value of h for two bridges was larger than for a single bridge when the discharge was 34.94L/s; the other values did not change. The values of h and L_1 for two bridges were larger than for a single bridge when the discharge was 45.42L/s(the natural discharge is 13,000m^3/s); the value of L_2 did not change. Therefore, it was established that the cumulative effect of two bridges is more obvious as the discharge increases, and that the cumulative effect is more obvious on h and least evident on L_2.

Table 2.Cases for which cumulative effect of two bridges existed

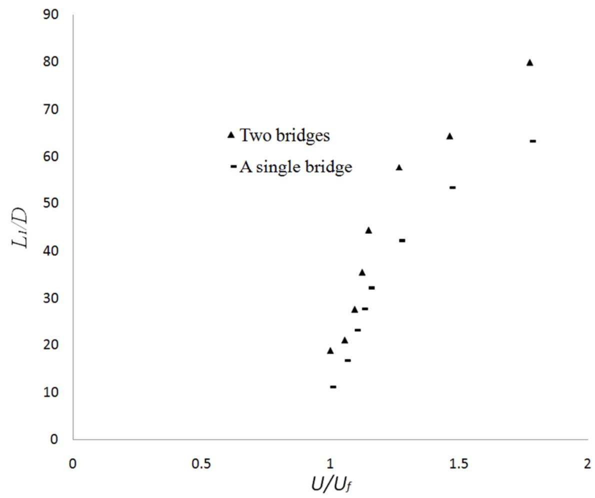
Case	Combined bridge	L/D	Model discharges (L/s)	h	L_1	L_2
1	K831OB, K831NB	6.94	7.34-45.42	Y	Y	Y
2	CYB, K831OB	208.33	34.94	Y	N	N
			45.42	Y	Y	N
3	CYB, K831NB	213.54	34.94	Y	N	N
			45.42	Y	Y	N
4	TLB, JHB	214.28	34.94	Y	N	N
			45.42	Y	Y	N

Note: “Y” means the cumulative effect of the two bridges existed and “N” means the cumulative effect of the two bridges did not exist.

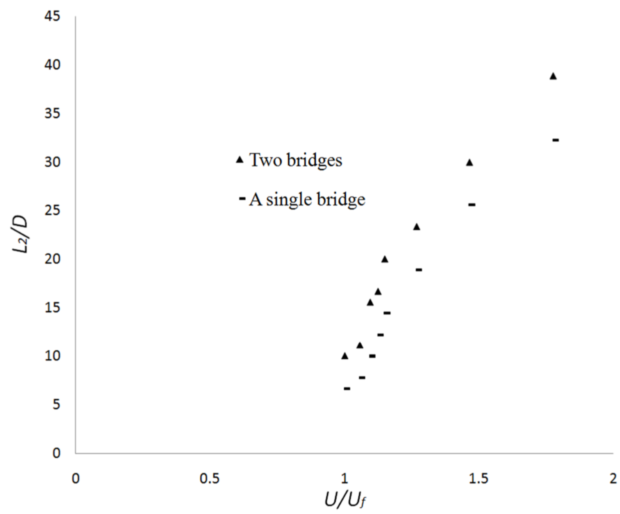
The cumulative effect of two bridges on the backwater of the downstream bridge was not observed in cases 2, 3, and4. Thus, it is considered that the cumulative effect of the two bridges is most evident on the upstream bridge. In case 1, because K831OBis very close to K831NB, the length of the increased elevation of the water surface L_1 and drop length L_2 overlap. We only obtained the values of L_1 for K831OB(the upstream bridge) and L_2 for K831NB(the downstream bridge). Figure13 shows the discrepancy of the parameters of the backwater between K831TB and K831OB(or K831NB because the values of K831NB were the same asK831OB). The discrepancy of the backwater becomes more obvious as the discharge increases, which means that the cumulative effect is more obvious as the discharges increase. These findings agree with those derived from cases 2, 3, and 4.



(a) h/D



(b) L_1/D



(c) L_2/D

Fig.13 Discrepancy of backwater parameters between K831TB and K831OB

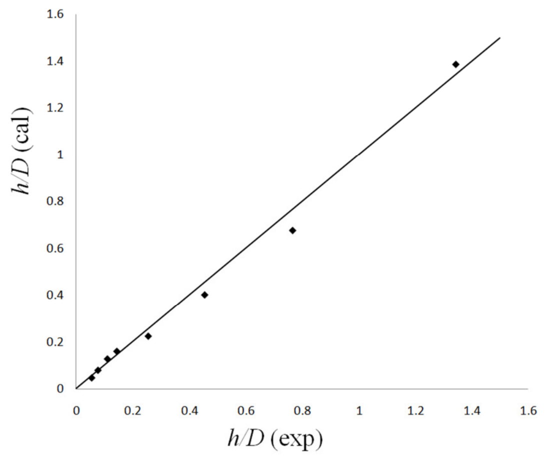
We found that the backwater parameters for case 1 could be expressed as follows:

$$\frac{h_s}{D} = 1.45 \frac{h}{D}, \tag{9}$$

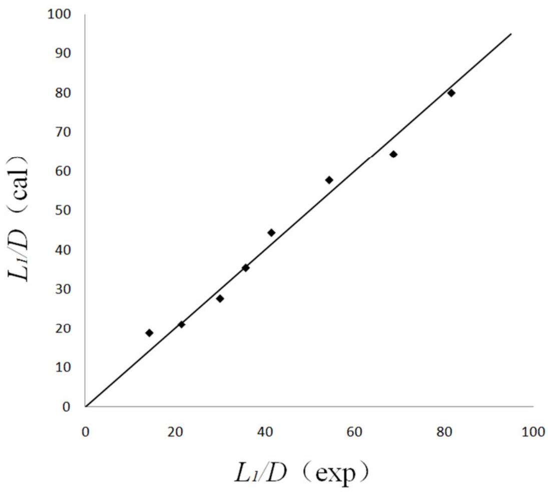
$$\frac{L_{1t}}{D} = 1.29 \frac{L_1}{D}, \tag{10}$$

$$\frac{L_{2t}}{D} = 1.26 \frac{L_2}{D}, \tag{11}$$

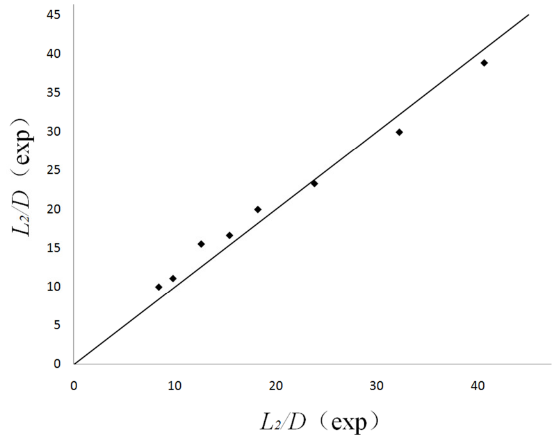
where the subscript t denotes the two bridges, as shown in Fig. 14.According to the coefficients in Eqs. (9)–(11), the cumulative effect on h is most obvious, which agrees with cases 2, 3, and 4.



(a) h / D



(b) L_1 / D



(c) L_2 / D

Fig.14 Comparison of the experimental and calculated backwater parameters for K831TB

4.5 Maximum extent of the cumulative effect

According to the findings above, the cumulative effect of two bridges increases the backwater parameters compared with a single bridge, which undoubtedly increases the difficulty of flood control. Therefore, the determination of the maximum extent of the cumulative effect is obviously important regarding field engineering practice.

The conclusions in section 4.4 reveal that the cumulative effect on the increased depth h is most obvious. Therefore, to establish the maximum extent of the cumulative effect, we just need to determine the distance between the two bridges for which the value of h no longer changes.

Because the cumulative effect is most evident for discharges that exceed bankfull discharge, Eq.(4) was transformed to the following form:

$$F\left(\frac{h}{D}\right)=\frac{h}{D}/\left(\left(\frac{B}{(B_1 H)^{0.5}}\right)^{2.7}(100\lambda)^{0.027}\right)=0.0408e^{2.57U/U_f} \quad .(12)$$

The experimental data in Table 2 were transformed according to Eq.(12), as illustrated in Fig.15. As L/D (the distance between the two bridges)increases, the value of $F\left(\frac{h}{D}\right)$ of the two bridges approaches that of a single bridge. Parallel lines are drawn in Fig. 15 based on the points for the cases given in Table 2 for the discharge of45.42L/s(the natural discharge is 13,000m^3/s),and these lines are parallel to the tangential line of the curve of Eq.(12). The distance of parallelism A between the lines for the cases in Table 2 and the line of Eq.(12) is computed, the physical meaning of which is the magnitude of the cumulative effect.

Figure 16 shows the relationship between the magnitude of the cumulative effect A and L/D. The limitation of the distances between the bridges in the real river means there are insufficient available points with which to develop an empirical formula that quantifies the relationship. However, we extend the curve in Fig.16 to the x-axis, which gives a value of $L/D = 215$ for a value of $A = 0$. This means the cumulative effect disappears when $L/D = 215$ and thus, this is defined as the maximum extent of the cumulative effect.

Furthermore, the increased depth associated with more than two bridges was measured when the discharge was 45.42L/s(the natural discharges is 13,000m^3/s), as shown in Fig.17(this figure simply presents some examples of changes related to the increase in the number of bridges). CYB in Fig.17 means we initially placed CYB in the model and then added other bridges according to their distance to CYB. Bridges closer to CYB were placed in the model preferentially. Figure 17 shows that for more than two bridges, the value of the increased depth remains unchanged. Therefore, it is considered that the cumulative effect of groups of bridges does not exist in this river reach.

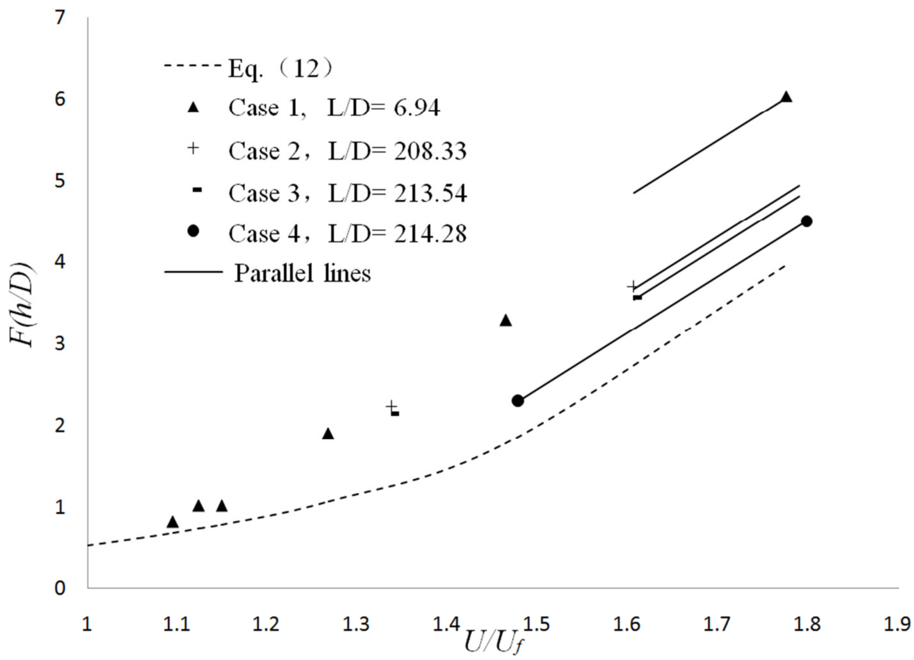


Fig.15 Variation of $F\left(\frac{h}{D}\right)$ with $\frac{U}{U_f}$

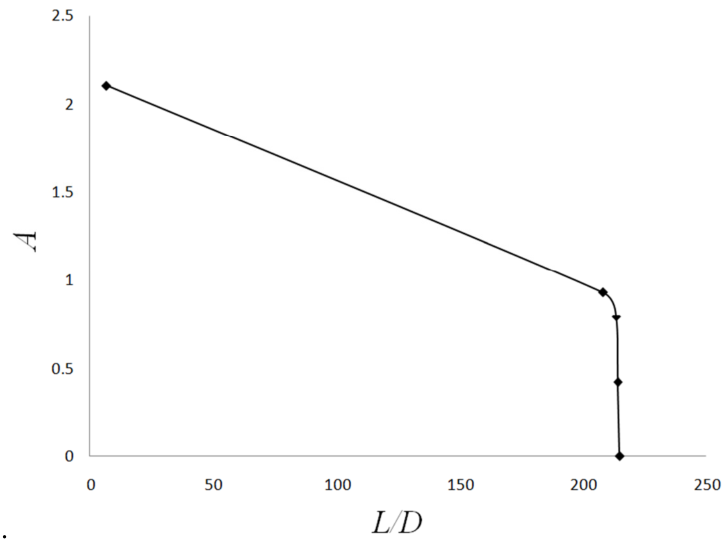


Fig.16 Variation of A with L / D

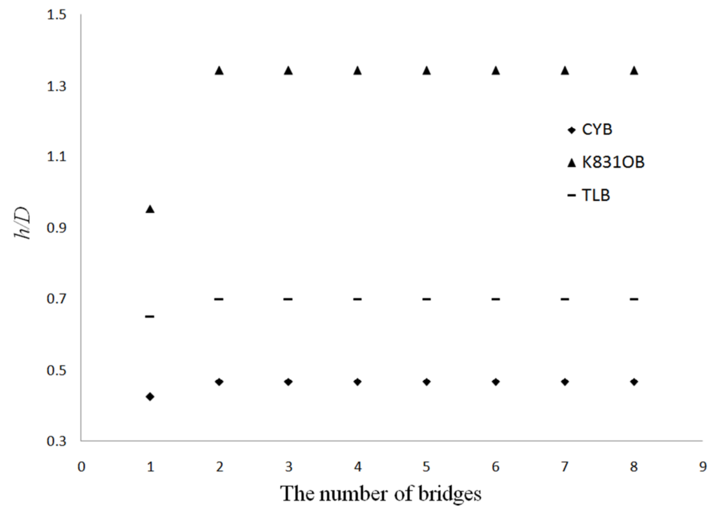


Fig.17 Variation of h / D with the number of bridges

5. Conclusions

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

(1) When the discharge is less than bankfull discharge, Eqs. (3), (5), and (6) can describe the parameters of the backwater of a single bridge. When the discharge exceeds the bankfull discharge, Eqs. (4), (7), and (8) can describe those parameters.

(2) The cumulative effect of two bridges on backwater exists for certain discharges and distances. The cumulative effect becomes obvious at large water depths and least evident on drop length.

(3) By analyzing the discrepancy between 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.

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 on the river, and the present results provide technical support as to how it can be avoided. Given the limitations of model scales, assumptions, and distances of the bridges in real rivers, further studies will be required to investigate further the cumulative effect on the backwater of groups of bridges.

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