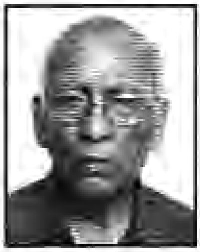
HYDRAULIC CONSIDERATIONS FOR PLANNING AND DESIGN OF GUIDE BANKS



S.K. Mazumder[1]

ABSTRACT

Guide banks are meant to confine and guide the river flow through the bridge without causing excessive afflux and damage to the approach embankments with the objective of reducing overall cost of the bridge and approach embankments. They help ensuring bridge safety and considerable reduction of scour in abutments. The current design practice (IRC-89:1997) has several draw backs. The paper intends to improve the current design practice introducing design consideration for deciding bridge length which in turn determine length of guide bank. Lagasse's elliptical guide bank and length of guide bank as per Lagasse's curves have been recommended. An illustrative example is worked out at the end.

1. INTRODUCTION

Rivers in flood plains are often shallow and flow in a wide alluvial belt, with meandering and/or braiding characteristics (Mazumder, 2017, 2016; Garde, 2006). To construct a bridge or barrage across such rivers and to reduce their overall cost, it is sometimes necessary to flume the normal waterway by construction of heavy embankments, called 'Guide Banks'. Guide banks are meant to confine and guide the river flow through the bridge/barrage without causing excessive afflux and damage to the approach embankments. Their alignment should be such that the pattern of flow is uniform with minimum return currents which may cause outflanking of the bridge/barrage. Their alignment and layout are best decided by physical and mathematical model studies to ensure that the entire waterway under the bridge is utilized and the depth of scour in the vicinity of the abutments and the piers adjacent to abutments is reduced. With the provision of guide bank, scour moves upstream away from the bridge and a substantial amount of money needed for bridge foundation and annual recurring cost of maintenance of bridge and its approach embankments can be saved. Thus, it is imperative that Guide banks which form one of the major and vital constituent of river training works must be planned and designed so that it is economic and hydraulically efficient.

Principal considerations involved in the design of a guide bank are geometry, height and length of guide bank. Laboratory studies show that a guide bank shaped in the form of a quarter of an ellipse, with ratio of major (length) to minor (offset) axes of 2.5:1 performs better than any other shape tested. Axial length of guide bank, measured from its head to tail end, as prescribed in all Indian codes, vary from 1.1L to 1.5L, where L is the effective waterway under the bridge. The height of guide bank is based on anticipated high water level (including afflux) during the passage of design flood. There should be sufficient height and freeboard to avoid overtopping and damage due to wave action.

Another important aspect of hydraulic design is scour which can be considerably reduced by providing guide bank. In an earlier paper, author (Mazumder et al, 2014) has discussed about scour in both fine and coarse soil. A lot of research study have been made on bridge scour by gifted research workers viz. Melville and Coleman (2000), Richardson and Davis (1995), Breussers and Raudkivi (1991) from abroad and Kothyari et al (1992), Dey (2005-06) from India.

The concept of guide bank was first introduced by its inventor, a british engineer, 'Bell' and that is why guide bank is often called Bell bund. Planning and design principles of guide bank is well documented

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in the CBIP (1989) publication 'River Behavior, Management and Training. IS:10751 (1994) and IRC-89 (1997) provide detailed guidelines for planning and design of guide banks in alluvial rivers in India. These standards have been prepared on the basis of researches carried out by British engineers in India, namely, Spring (1903), Lacey (1930), Gales' (1938), Inglis (1949) and others. Knowledge and experience gained from the existing guide banks constructed in the past as well as researches conducted at CWPRS, Pune (1945, 1938,), Indian Engineers e.g. Inglis & Joglekar (1936), Gole and Chitale (1967), Sharma et al (1976) evolved rational criteria of design of guide banks.

One of the primary objectives of writing this paper is to introduce some of the recent developments in hydraulics in planning and design of guide bank with a view to make it more economic and hydraulically more efficient. As regard scour, readers may refer to papers cited above.

2. GENERAL DESIGN FEATURES

As already mentioned, Planning and design of guide bank have been outlined in details in IS-10751(1994) and IRC-89 (1997) as well as annual reports of CWPRS, Pune and the Ministry of Railways Govt. of India (1963). CBIP (1989) has an excellent publication covering all aspects of planning and design of guide banks with illustrative examples.

2.1 Classification of Guide Banks

Guide banks can be classified according to their form in plan and their geometrical shape. They can be divergent,convergent or parallel (**Fig. 1A, 1B & 1C**) in the direction of flow upstream. Convergent and divergent guide banks require a longer length in comparison to parallel guide banks for the same degree of protection to the bridge and approach embankments. Parallel guide banks give better distribution of flow across the waterway. Parallel guide banks with suitably curved heads (IS:10751-1994 & IRC-89-1997) have been found to give uniform flow from the head of the guide banks to the axis of the structure. As shown in **Fig. 1A & 1B** flow is likely to separate in divergent and convergent guide bank resulting in formation of shoal within the guide bank which is hydraulically not desirable. Angle of divergence should not exceed 60 to avoid any flow separation in a diverging guide bank. No separation takes place in parallel guide banks., except near the head which is far away from bridge axis.

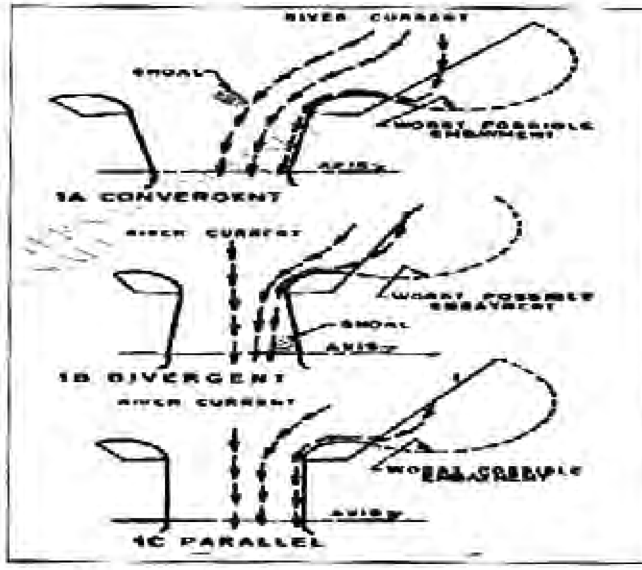

Fig. 1 Showing Different Forms Of Guide Banks in Plan (1A-Convergent, 1B-Divergent & 1C-Parallel) (IS:10751)

The guide banks can be elliptical with a circular or multi-radii curved head (**Fig. 2**). Elliptical guide banks have been found more suitable in case of wide flood plain rivers for better hydraulic performance. In case of elliptical guide banks, the ratio of major axis to the minor axis is generally in the range of 2 to 3. Due to gradual change in curvature in elliptical guide banks, the flow hugs the guide banks all along its length as against separation of flow occurring in case of straight guide banks near the curved head which leads to obliquity and non-uniformity of flow downstream.

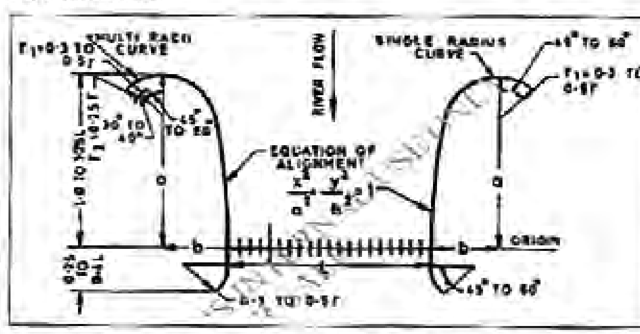

Fig. 2 Elliptical Guide Banks With Multi Radii Curved Head (Taken from IS:10751)

2.2 Waterway and Guide Bank Length

Length of a guide bank (L_g) is dependent on the waterway (W) under the bridge. In case a bridge spans the main channel as well as the flood plains fully i.e. from bank to bank, there is no need of any guide bank. It is needed only when the normal waterway (W = L in **Fig. 2**) is restricted by fluming (Mazumder, 2002) the river. Greater is the restriction/fluming, longer should be the guide bank length from hydraulic point of view.

Indian codes, however, prescribe just the opposite as it states that length of guide bank (L_g) should be varying from 1.1W to 1.5W. This means more is the fluming/restriction or in other words smaller is W (= L in **Fig. 2**), less will be length of guide bank and more is W, longer will be guide bank length. This is hydraulically incorrect as explained afterwards. IRC-5-2015 and other Indian codes recommend that waterway under a bridge should be equal to Lacey's (1930) regime width

$$P = W = 4.8\,Q^{0.5} \qquad \ldots (1)$$

Where, P is the wetted perimeter (P=W) in meter and Q is the design flood discharge in cumec.

In the meandering and braiding flood plains where guide bank is required, the above equation may not be applicable (Mazumder, 2017; 2010.2009).

3. HYDRALIC CONSIDERATIONS IN FIXING WATERWAY

As stated under section 2.2 the length of guide bank (Lg) is dependent on waterway (L in **Fig. 2**) provided under a bridge. Higher is the restriction of flow or greater is the fluming, more should be guide bank length .for improved hydraulic performance. It is, therefore, necessary to understand the basic hydraulic criteria governing waterway, shape and length of guide bank discussed in following paragraphs.

3.1 Permissible Fluming of a Channel with Sub-critical Flow

In a mild sloping channel where the flow is at sub-critical stage, the normal waterway in the channel can be contracted to an extent so that the flow under the bridge is not choked. If B_1 = (W) is the normal waterway and B_0 (= L) is the contracted waterway under the bridge, contraction/fluming ratio (B_0/B_1) can be derived from the fundamental relation given by equation (2).

$$B_0/B_1 = (F_1 / F_0)\,[\,(2 + F_0^2) / (2 + F_1^2)]^{3/2} \qquad \ldots (2)$$

where F_1 and F_0 are the Froude's number of flow at the normal and the contracted sections respectively. **Fig. 3** shows the functional relation between B_0/B_1(=L/W) and F_0 for different values of F_1 for approaching normal flow. Flow is choked (also called critical flow) when F_0 = 1. It may be seen (**Fig. 3**) that higher is the F_1-value, less is the opportunity of contracting/fluming. It also shows that there is hardly any advantage/economy if contraction/fluming is made such that F_0 exceeds approximately 0.70. Flow surface becomes wavy when F_0 > 0.70, with highest degree of wave amplitude at critical flow when F_0 = 1. Excessive contraction of sub-critical flow causes high loss in head due to higher velocity of flow velocity at the contracted section resulting in higher afflux. Any contraction beyond a critical limit (at F_0 = 1) will result in the formation of hydraulic jump downstream and there will be excessive afflux upstream. To be on safe side, it will be wise not to contract a channel for F_0-value higher than 0.50.

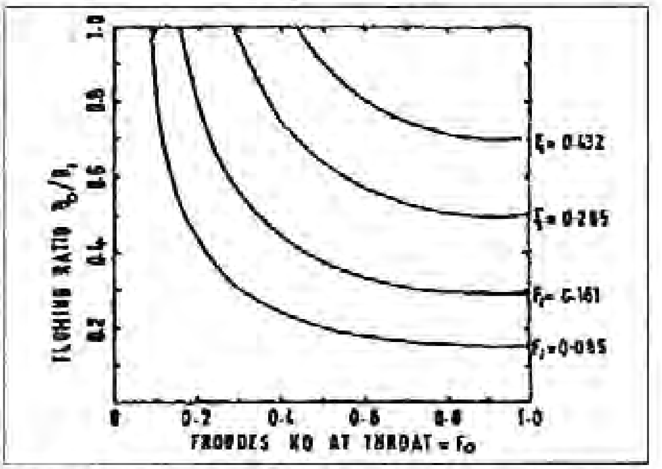

Fig. 3 Variation of B_0/B_1 with F_0 for Different F_1-Values

3.2 Permissible Afflux

Construction/Fluming of natural waterway will always result in afflux (Mazumder, 2003). In case of a straight channel with uniform flow and firm bank without any flood plain, Molesworth formula prescribed by IRC:5-2015 may be adopted to compute afflux given by Eq. (3) below,

$$h_1^* = [V^2 / 17.88 - 0.015]\,[(A/A_0)^2 - 1\,] \qquad \ldots (3)$$

where,

h_1* is the afflux in m, V is the mean velocity of flow in the river prior to bridge construction in m/s, A_0 and A_1 are the areas (in m²) of flow section at design HFL in the approach section and under the bridge respectively. Molesworth equation (3) is not applicable for rivers with wide flood plains and non-uniform approach flow for which Bradley (1970) suggested equation (4) for finding an approximate value of afflux.

$$h_1^* = 3\,(1 - M)\,V_n^2/2\,g \qquad \ldots (4)$$

where,

$M = A_0/A_1$, and Vn is the mean velocity of flow under the bridge at design HFL. Eq.4 shows that with increase in contraction, M will decrease and Vnwill increase thereby increasing afflux. Too high afflux will result in submergence of flood plain of the river causing damage to life and properties upstream.

Excessive afflux may cause overtopping and washing out of the bridge. Due to loss of freeboard, debris will accumulate near the piers and abutments leading to increase in scour near piers and abutments and consequent failure of the bridge. IRC-5 (2015) recommends that permissible maximum afflux due to bridge should not exceed 15 cm. As per FHWA (2012), afflux should be limited to a maximum value of 30 cm where submergence of flood plain will not result in any substantial damage upstream.

3.3 Instability of Flow and Outflanking of a Bridge

Too much restriction of flow may cause river instability both upstream and downstream of a bridge (Mazumder, 2004) resulting in decrease of hydraulic gradient ($S_w = dy/dx$). In the absence of bridge, the bed slope (S_0) is the same as water surface slope (S_w) and energy slope (S_e) i.e. $S_0 = S_w = S_e$ as the flow is normal. With afflux, both S w and Se reduces which causes reduction of stream power (Ω) expressed as

$$\Omega = \gamma Q S_e \quad \dots (5)$$

Higher the afflux , lower will be the hydraulic gradient (S_w) and energy slope (S_e) and lower will be stream power (Ω) causing loss of sediment carrying capacity of river. Sediments start depositing upstream resulting in reduction in bed slope (S_0). As propounded by Kennedy (1895), Lacey (1930) and Bharat Singh (1983), regime width of a channel increases with fall in bed slope. Maximum increase in stream width occurs upstream of the bridge where the magnitude of afflux is the highest and bed slope is minimum. Highly restricted waterway, often provided to reduce cost of a bridge, results in local widening of the river upstream and downstream of the bridge. Development of eddies and silting in the flood plain of the river causes flow instability and shifting of its main channel either left or right of the bridge. As shown in **Fig. 4** a bridge on NH-6 is likely to be outflanked due to high restriction of flood plain.

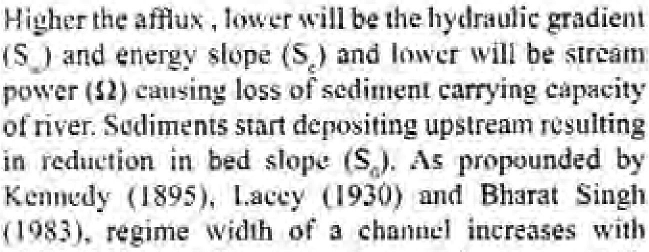

Fig. 4 Showing Widening of a River Upstream of a Bridge on NH-6 in M.P.

Similar instability may occur downstream of the bridge also when there is too much restriction of flood plain width. The difference between the high kinetic energy (K.E.) of flow ($V_0^2/2g$) at the contracted section and the normal K.E. of flow ($V_2^2/2g$) in tail channel i.e.($V_0^2/2g$-$V_2^2/2g$) does not get converted to potential energy unless the jet flow coming out from the contracted section is provided with a very long expanding transition downstream with a total angle not exceeding about 10° to 12° (Mazumder, 1993).

The only way a stream with a given flow, given tail water depth (y_2) and a given mean velocity in the tail channel (V_2) can contain the unconverted excess kinetic energy. ($V_0^2/2g$- $V_2^2/2g$) is through distortion of flow resulting in flow non-uniformityand jet type flow downstream. (Mazumder, 2010).

Experimental investigations were carried out (Mazumder and kumar, 2001) to determine flow regimes, hydraulic efficiency and flow stability in sub-critical straight expansion. It was noticed that flow stability downstream of expansion is governed by both the parameters expansion ratio (B_1/B_0) and rate of expansion $1/2(B_1-B_2)/Le$. Here B1 is the normal width of channel and B_0 is the contracted width of channel at bridge site, Le is the length of expansion. Since there is an abrupt expansion of flow downstream of all bridges (Le = 0), expansion ratio (B_1/B_0) alone governs stability of flow downstream of a bridge. Higher the expansion/fluming ratio, higher is the instability. The flow was found to be stable with symmetric eddies on either side up to a critical value of expansion ratio (B_1/B_0) of about 1.5 to 2. When B_1/B_0 exceeded 1.5 to 2, the side eddies became asymmetric and central jet flow was found to be unstable. Such unstable jet type flow downstream of a bridge or barrage can attack river banks causing unprecedented erosion requiring costly protection/ training works.

4. IDEAL SHAPE OF GUIDE BANK IN SUB-CRITICAL FLOW

Guide banks are similar to a transition structure. Upstream guide bank connecting normal waterway (B_1) to the flumed section of width (B_0 = L) under the bridge is similar to a contracting transition. In sub-critical flow, performance of contracting transitions of different shapes and lengths have been tested by several workers (Hinds,1928, Vittal et al,1983 and others). Mazumder and Ahuja (1978), tested contracting transition developed by Jaeger (1956) to minimize head loss, prevent separation of flow and achieve uniform distribution of flow at the throat section i.e. at the abutment of the bridge. Hydraulic efficiency and head loss coefficient (C_i) of Jaeger (1956) type transition of different axial lengths as measured is illustrated in **Fig. 5**.

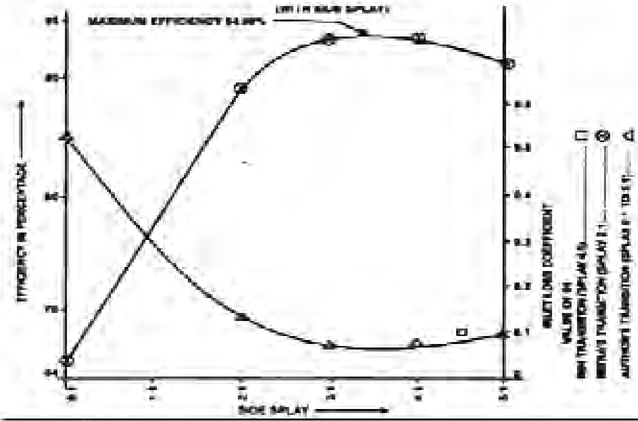

Fig. 5 Variation of Overall Efficiency and Inlet Loss Co-fficient (C_i)

It may be seen that maximum hydraulic efficiency and minimum head loss occurs when the axial length of transition (Governed by average side splay) is about 3:1 which means the length should be three times the offset on either side of the bridge. Providing more length will result in fall in efficiency and rise in head loss.

5. LENGTH OF ELLIPTICAL GUIDE BANK AS PER LAGASSE

Lagasse (1995) made exhaustive study of elliptical shaped guide bank (**Fig. 2**) and developed criteria to determine the length of guide bank. He suggested that the length of major axis (a) should be 2.5 times the length of minor axis (b) which is almost the same as results obtained by the author in case of Jaeger type shape. From his experimental study, he further recommended that keeping the ratio b/a (**Fig. 2**) as 0.4, the length of guide bank will be governed by the following two parameters:

(i) Q_r i.e. Ratio of return flow from flood plain (Q_f) and the main flow from a distance 30 m length adjacent to the abutment Q_{30} i.e. $Q_r = Q_f/Q_{30}$ and

(ii) Maximum mean velocity of flow under the bridge, V_{n2} which can be found from the relation $V_{n2} = Q/A_2$, where Q is the total flood discharge passing under the bridge including flood plain flow (unless there is relief culvert in the flood plain), A_2 is the cross sectional area of flow under the bridge.

Laggasse's (1995) design curve for finding length of guide bank is shown in **Fig. 6**. An example is worked out in **Appendix-I**.

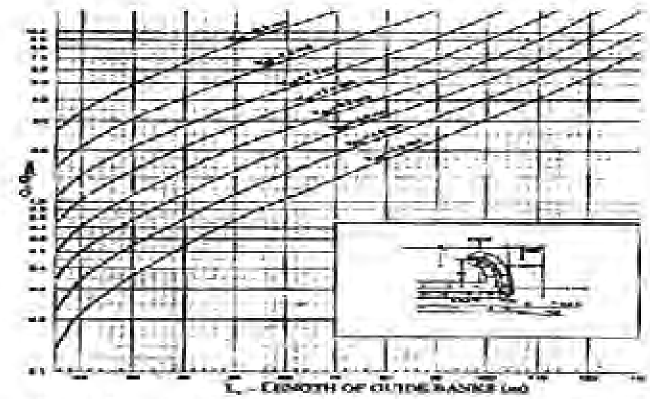

Fig. 6 Lagasse's Design Curves for Finding Length of Guide Bank

It is apparent from **Fig. 6**, that the length increases with increase in Q_r and V_{n2}, which is quite rational. If fluming is high, there will be more return flow and guide bank length will be more. In case return flow is negligible i.e. there is no fluming/restriction of waterway, no guide bank is needed.

Even where no guide bank is needed, author is of the view that short guide banks should be provided on either side for safety of abutment and reduction in abutment scour. Guide banks with railing and flower plants will be an attractive spot for recreation of people living nearby.

6. ILLUSTRATRIVE EXAMPLE

As discussed under sections 3, 4 and 5, an example has been worked out (Appendix-I) to illustrate the hydraulic design procedure of guide bank for an existing bridge on Yamuna river flood plain. Waterway for the bridge appears to be inadequate and the afflux is more than permissible limit. As per the IRC and BIS codes, guide bank lengths are too high compared to Lagasse's design for an elliptical type guide bank.

7. CONCLUSION

Guide banks are used when the width of flood plain of a river is very large compared to Lacey's regime waterway. They are useful to achieve uniform flow under the bridge, reduction of afflux and prevention of outflanking of the bridge . Scour in abutments and adjoining piers can be substantially reduced by providing guide banks. Overall cost of bridge and approach embankments are reduced.

Conventional design practice of guide banks as per Indian codes need revision in the light of recent development in hydraulics. Author recommends elliptical or Jaeger type guide bank replacing the parallel types. Lengths of guide bank can be considerably reduced as per Lagasse. An illustrative example has been worked out to illustrate the hydraulic design principle.

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Illustrative Example:

Appendix-1

It is propsed to design guide banks for a bridge on river Yamuna (**Fig. 7**) with following data:

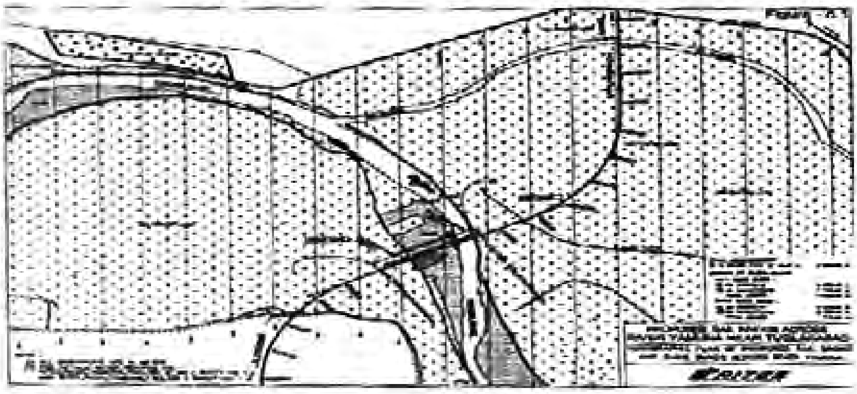

Fig. 7 Existing Bridge on Meandering Flood Plain of River Yamuna

Design Flood Discharge - 11,000 cumec

Design HFL- 216.50

Lowest Bed Level in main Channel-208 m

Lowest Bed Leven in Flood plain-212 m

Width of main channel-600 m

Flood plain width: Left-600 m , Right-1200 m

Estimated discharge distribution: Main Channel-6,800 cumec, Left bank-1400 cumec, Right bank-2800 cumec

Waterway

1. **Lacey method: (Eq-1)**

   P = 4.8Q0.5 = 4.8 (11,000)0.5 = 503 m

2. **Permissible Fluming approach :Eq.-2)**

   $B_o/B_1 = (F_1 / F_o) [(2+F^2_o) / (2+F_1^2)]^{3/2}$

Here, Depth of flow in main Channel Y_m =216.5-208 = 8.5 m, Area of main channel = 600 x 8.5 = 5,100 m^2

Depth of flow in flood plain = y_f =216.5-212 = 4.5 m, Area of Flood plains = A_f = (600 + 1200) x 4.5 = 8,100 m^2

Total Area upstream of Bridge $A_1 = A_m + A_f$ = 5,100 + 8,100 = 13,200 m^2

Average Velocity of flow upstream of bridge – $V_1 = Q/A_1$ = 11,000/13,200 = 0.83 m/s
Total width of flow upstream of bridge – B_1 – 600 + 600 + 1200 = 2400 m
Mean depth of flow upstream $y_1 = A_1/B_1$ – 13,200/2400 = 5.5 m
F1 – Froude's No. of Approaching flow upstream = $V_1/(gy_1)^{0.5}$ = 0.83/(9.8X5.5)$^{0.5}$ = 0.113
Taking maximum permissible value of Froude's no. under bridge as F_0 = 0.5 From Eq. 2
$B_0/B_1 = B_0/B_1 = (F_1/F_0)[(2+F^2_0)/(2+F_1^2)]^{3/2}$ – [(0.113/0.5)] [(2 + 0.25)]/([2 + 0.013))]^{3}/2 = 0.226 x 1.18 – 0.2655
B_0 = Permissible waterway under bridge = 0.2655 x B_1 – 0.2655 x 2400 = 637.2 m
Actual bridge length provided is 600 m i.e. the same as main channel width.

3. Permissible Afflux Approach

By Molesworth Eq.

h_1^* – [V12 /17.88 + 0.015] [(A_1/A_0) 2 –1] = [(0.832/17.88) + .015] [(13,200/5.100)^2-1] = [0.0535] [4.075] = 0.218 m – 21.8 cm

By Bradley formula

$h_1^* = 3(1-M)V_2^2/2g$ = 3[1-(5,100/13,200) x (2.152/2 x 9.8)] = 3[(1-0.386) (0.236)] = 0.435 m = 43.5 cm
Since the bridge is located on wide flood plain, Bradley formula is applicable. An afflux of 43.5 cm exceed the permissible value and hence waterway should have been increased to keep the maximum afflux as h_1^* = 0.3 m. Waterway required for h_1^* = 0.3 m is found to be 900 m

4. Stability Consideration

With elliptical type guide bank on either side on flood plain (for guide bank length computation is given below), Width of opening at the head of guide bank = 600 + 30 = 630 m. So, the ratio of opening – 2400/630 = 3.8 which is greater than 1.75. Hence flow is unstable.

For stability of flow, B_0/B_1 should not be greater than 1.5 to 2. Taking the mean value as 1.75, the minimum length of waterway required is 2400/1.75 = 1370 at head of guide bank i.e. 1370-30 = 1340 at bridge axis which is more than two times the bridge length actually provided i.e. 600 m.

This implies that the existing bridge length should have been increased to avoid flow instability both upstream and downstream. Ignoring stability criteria 900 m length of bridge should have been provided to keep the afflux to a maximum value of 30 cm.

Length of guide Bank

1. IRC Method:

Assuming Length of Bridge same as main channel width i.e. 600 m
Minimum axial length = 1.10 x L – 1.1 x 600 = 660 m

2. As per Lagasse:

(a) Left Guide Bank:

Flood discharge in left flood plain Q_f = 1400 cumec.
Q_{30} = Dischrage in 30 m length adjacent to pier – (11,000/600) x 30 = 550 cumec
Q_f/Q_{30} = 1400/550 = 2.5
V_{n2} = Mean velocity of flow under the bridge – Q/A2 = 11,000/(600 x 8.5) = 2.15 m/s

Entering the values of Q_f/Q_{30} = 2.5 and V_{n2} = 2.15 in **Fig. 6**, Left Guide Bank Length = 50 m
Guide bank length provided on Left side is 350 m which is 7 times more than Lagasse.
As per Lagasse, Offset of right guide bank = 0.25 x 50 = 12.5 m say 15 m

(b) Right Guide Bank:

Q_f = 2800, Q30 = 550 cumec
Q_f/Q_{30} = 2800/550 = 5.09
V_{n2} = 2.15 m/s

Entering the values of Q_f/Q_{30} – 5.09 and V_{n2} = 2.15 in **Fig. 6**, Left Guide Bank Length = 70 m
Length of guide bank provided on right side is 300 m which is 4.26 times more than that by Lagasse.
As per Lagasse's design, Offset of left guide banks are 0.25 (70) = 17.5 m- say 20 m