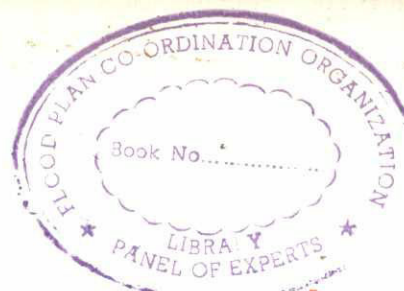


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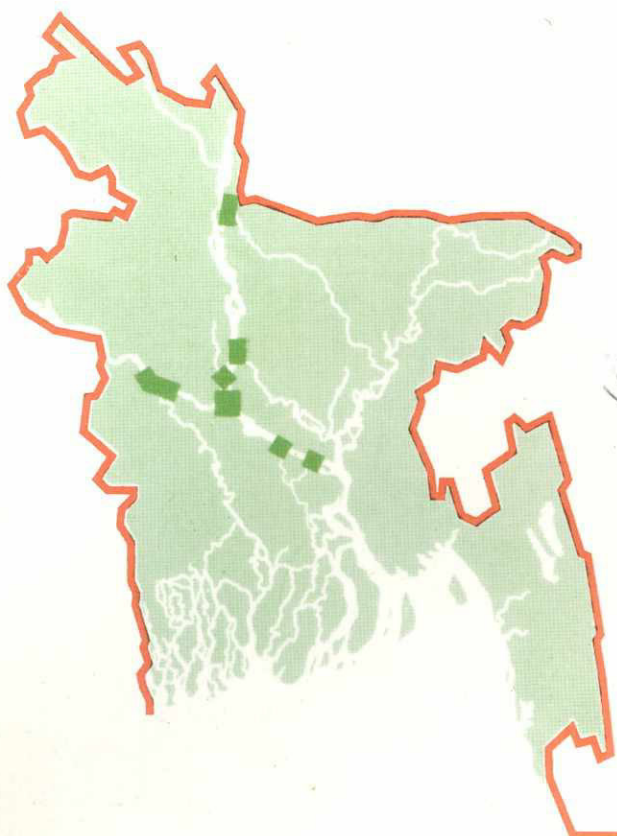


# GOVERNMENT OF BANGLADESH FLOOD PLAN COORDINATION ORGANIZATION

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## FAP 24 RIVER SURVEY PROJECT

Morphological Processes  
in  
the Bangladesh River System  
a Compilation of Papers



DELFT HYDRAULICS  
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Project ALA/90/04 — Commission of the European Communities



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Morphological Processes  
in  
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## Preface

For several decades now, international cooperation continues between Bangladesh and other countries, in the efforts of Bangladesh to develop its water resources. During this effort, very interesting investigations were made on the morphological processes of the river system of Bangladesh. Some of these works were presented in international conferences or are published in international journals, some are published locally in Bangladesh journals, and some others are reported in several project related technical reports. Although information exists in such a diversified form, no attempt was made so far to compile these works. The purpose of this volume is to bring together several papers published recently. Holding of two international workshops in Dhaka, Bangladesh, prompted completion of this volume. The first one is concerned with an experts' discussion on the preliminary findings of the River Training and Active Flood Plain Management Pilot Project (FAP 21/22). The other workshop, focusing on the 'Morphological Behaviour of Major Rivers in Bangladesh', will review the proposed study and survey programme for phase 2 of the River Survey Project (FAP 24) and will indicate the future needs in studying and data collection for the morphological impact assessment on the major rivers in Bangladesh. Both the workshops will be held in November 1993.

In the compilation process, care was taken to include most recent works. Some papers which could not be included are listed at the end of the compilation together with a list of unpublished reports. In addition, an index is presented to facilitate searching of certain key subjects. Any omission or mistake that might have crept in during the compilation, is the responsibility of the editor.

The editor would like to thank several of his office colleagues and members of the office-staff who worked hard to compile and complete this volume in time, especially, Prof. G.J. Klaassen, who initiated and made the first framework of this volume. Thanks are also due to Mr. F.G. Koch for making valuable comments.

Dhaka, Bangladesh  
25 October 1993  
River Survey Project

Dilip K. Barua, Ph.D.  
Editor



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(1) CLIENTS:

- BIWTA (BANGLADESH INLAND WATER TRANSPORT AUTHORITY)
- BWDB (BANGLADESH WATER DEVELOPMENT BOARD)
- FPCO (FLOOD PLAN COORDINATION ORGANIZATION), BANGLADESH
- JAMUNA MULTI-PURPOSE BRIDGE AUTHORITY, BANGLADESH

(2) FUNDING AGENCIES:

- ASIAN DEVELOPMENT BANK
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- COMMISSION OF THE EUROPEAN COMMUNITIES (CEC)
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- WORLD BANK



(3) AGENCIES WITH WHICH COOPERATION WAS ESTABLISHED:

- BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY, BANGLADESH
- RIVER RESEARCH INSTITUTE, BANGLADESH
- SURFACE WATER MODELLING CENTER, BANGLADESH

(4) CONSULTANCY FIRMS:

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- DELFT HYDRAULICS, THE NETHERLANDS
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- RHEIN-RUHR ING.-Ges.MBH, DORTMUND, GERMANY
- RPT (RENDELL, PALMER AND TRITTON), U.K.
- SIR WILLIAM HALCROW & PARTNERS LTD., U.K.

AND THE VARIOUS PUBLISHING COMPANIES AND PROFESSIONAL ASSOCIATIONS WHOSE PAPERS ARE REPRODUCED. WE THANK ALSO THE MEMBERS OF THE PANEL OF EXPERT OF FPCO, BOTH LOCAL AND EXPATRIATE, WHO HAVE FOLLOWED THE DIFFERENT STUDIES CLOSELY.

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## 1 Bangladesh and its Rivers: an Introduction

Bangladesh has many problems. Perhaps, the most relevant problem in the context of this volume is the utilization of its water and sediment resources. Its water resources in the form of surface and ground water, supply drinking water and support agriculture, fisheries, navigation, and industries. Its sediment resource, which is not so appreciated as a resource, but nevertheless is very important for Bangladesh because it builds new land, offsets subsidence, and possibly adds to the soil fertility. While these resources are abundant during one part of the year, there is scarcity of them during the rest of the year, creating the so-called 'high-flow' and 'low-flow' problems as pointed out by Thijsse (1964). During the southwest monsoon, the whole drainage basin, 92% of which lies outside Bangladesh, receives heavy rainfall and swells with overland flow. During the northeast monsoon, on the other hand, rainfall is inadequate for the country's need.

Two of the largest rivers in the world, the Ganges and the Brahmaputra joined by another large river the Meghna, together with their numerous tributaries and distributaries, criss-cross the whole country and drain into the Bay of Bengal. Together, their number is some 230, making a colossal length of some 24,000 km which is roughly equal to twice the diameter of the earth at the equator. Despite such a large number of rivers, the flood flows are not efficiently drained by the river system; consequently, flooding occurs almost every year. The reasons of the flooding can be several: the flat terrain of the country, and confluences creating backwater effect on smaller rivers, are some of the causes often mentioned. Moreover, the rivers are completely unconfined, with very high channel migration rates displacing population and destroying river-front structures. While such a big problem exists either with flood, drought or river migration, finding a solution to it is not easy and offhand. But, efforts continue through international cooperation to see a better understanding of the river system - its hydrology and its morphology.

The first systematic study on flood problems perhaps dates back to 1927, when the then Irrigation Department executed a study to establish the reason of the 1922 flood in north Bengal (Mahalanabish, 1927). Following that came the Wilcox report on the irrigation system in Bengal (Wilcox, 1930). After the emergence of East Pakistan devastating floods occurred for three consecutive years in 1954, 1955 and 1956. This prompted the government to



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request the technical assistance of the United Nations which sent a mission to study the flood problem in East Pakistan. This report published in 1957 is known as 'Krug Mission Report' and laid the foundation of forming the East Pakistan Water and Power Development Authority (EPWAPDA). In 1959, the EPWAPDA was created by the Government and in the same year International Engineering Company (IECO) was appointed as the general consultant of the EPWAPDA. The IECO master plan was published in 1964 (IECO, 1964). Side by side with this, East Pakistan Inland Water Transport Authority made the first survey of the country's rivers for navigation (NEDECO, 1967). Other notable studies during this period are made by Hardin (1963), Thijsse (1964), and LDL (1968).

Along with this efforts of Bangladesh Water Development Board (BWDB) or EPWAPDA engineers, and several consulting organizations, efforts continued by individual researchers to make scientific investigations on the river morphology. These researches are either university staff members or staff members of different consultancy organizations. The pioneering work in this regard was made by Coleman (1969) who laid the foundation of scientific investigations of river morphology in the country. It was followed by a paper on the alluvial roughness by Stevens and Simons (1973). From then on the story is more or less reflected here by the selection of papers, or by the bibliography provided in this volume.

Several papers arranged chronologically in this volume cover various aspects of river morphology dating from 1975 (Khan, 1975) to a forthcoming paper by van der Knaap (1994). In a total number of 24 papers, major subjects covered are: one paper on the thresholds on braiding and meandering (Struiksma and Klaassen, 1988), eight papers on the river characteristics (Khan, 1975; Bari and Alam, 1979; Rahman and Alam, 1980; Goswami, 1985; Klaassen and Vermeer, 1988a; Klaassen et al, 1988; Alam and Hossain, 1988; and Hossain, 1992), eight papers on planform changes (Galay, 1980; Bristow, 1987; Burger et al, 1988; Klaassen and van Zanten, 1988; Klaassen and Messelink, 1992; Klaassen et al, 1993; Mosselman et al, 1993; and Thorne and Russel, 1993), two papers on scour (Klaassen and Struiksma, 1988b; and Klaassen and Vermeer, 1988b), three papers on morphological modelling (Klaassen, 1990; Olesen, 1992; and Klaassen, 1992), and two papers on river training (van der Wal and Brühl, 1993 and van der Knaap, 1994). Investigations in these works have enhanced the level of understanding of different rivers. New or improved empirical relations are presented, and new ideas regarding approaches towards the modelling of braided river systems are proposed. Also, some novel analytical methods and

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physical modelling techniques were developed under different projects and are reported in the papers. In most of these investigations, use was made of historical data collected on the river system, mainly by BWDB and Bangladesh Inland Water Transport Authority (BIWTA). In addition, interpretations of satellite images were made of LANDSAT and MOS images. As one can see most of these works are on the Jamuna River, its hydraulic behaviour and planform changes. Works related to the Jamuna Bridge Project and other projects are responsible for this wealth of information on the Jamuna River.

Where do we go from here ? Obviously, the answer will come from a coordinated effort of different quarters under the River Survey Project (RSP), and for that purpose several study topics (RSP, 1993) are proposed. It can be understood, however, considering the duration and the areal coverage, the RSP can not answer all the key questions related to the understanding of the morphological behaviour of the river system. Moreover, understanding is not an one-time affair, rather it is a continuous exercise. However, it is certain that with RSP, the insight and the knowledge base of the rivers and their behaviour will be greatly enhanced making a definite contribution to the river management planning and to the optimization of designs for river training and flood protection structures.

Dilip K. Barua  
Gerrit J. Klaassen

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## 2 Braiding and Meandering

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## Paper 2.1

STRUIKSMA, N. and G.J.KLAASSEN (1988), On the threshold between meandering and braiding, *Proc. Intern. Conf. on River Regime, Wallingford, England*, 107-120pp.



International Conference on  
**RIVER REGIME**  
18-20 May, 1988

PAPER C3

ON THE THRESHOLD BETWEEN  
MEANDERING AND BRAIDING

by

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Summary

A prediction method for planform classification is developed by extending on the work by Struiksmā et al (1985) on bed deformation in curved alluvial channels. By comparing the results of the classification method with natural rivers and laboratory channels it is shown that the method yields fair results. The relation with the classic Leopold and Wolman (1957) classification method is shown by introducing Lacey's regime equations into the proposed criterion. Several implications of the proposed method are discussed.

1. Introduction

Traditionally, channel patterns of alluvial rivers have been classified as straight, meandering or braided (Leopold and Wolman, 1957), although such a classification is slightly obscured by changes in channel characteristics over the years (see e.g. Klaassen and Vermeer (1988) data for the braided Jamuna River, Bangladesh). Initially empirical classification graphs were derived, the first one by Leopold and Wolman (1957) who plotted bankfull discharge against channel slope. A separation between meandering and braiding could be made (see Figure 1). The derived equation for the separation line reads:

$$i = 0.0116 Q^{-0.44} \quad (1.1)$$

where  $Q$  is the bankfull discharge in  $m^3/s$  and  $i$  the channel slope. Later studies provided similar separation criteria, although the scatter involved "seems to make the distinction less satisfactory" (Bettess and White, 1983). Recently, Ferguson (1984) proposed to include the bed material size of the alluvial channels as additional parameter to reduce the scatter in the predictions.

From a theoretical point of view the problem of the transition from meandering to braiding has been studied by various authors using stability theory. Bettess and White (1983) provide a short review of this approach, but according to them "none of the theories adequately reproduce the sort of behaviour observed by Leopold and Wolman (1957) and others".



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In the present paper the recent theoretical and experimental work of Struiksma et al (1985) on the bed deformation in river bends is extended to a predictive method to distinguish between meandering and braiding. It is shown that this approach results in a classification diagram that allows to make such a distinction, provided that the channel dimensions are known in advance. By introducing Lacey's regime equations a threshold criterion similar to Leopold and Wolman (1957) is obtained. Instead of plotting  $i$  versus  $Q$  it is proposed to use the bed material size  $D$  as an additional parameter either via  $i/D$  or via  $i^3/D$  (conform the suggestions by Ferguson, 1984; see also Figure 2). Furthermore it is suggested that these parameters also govern the number of parallel braids (conform the empirical expressions derived by Howard et al, 1970).

## 2. Development of a criterion for planform classification

### 2.1 General

To arrive at a predictive discrimination method between meandering and braiding use will be made of the bed topography analysis presented by Struiksma et al (1985). They studied the equilibrium bed topography in curved channels and developed criteria for the spatial amplification of steady bed level perturbations. These results are used to derive a predictor for the threshold between meandering and braiding. In this chapter first a summary of the analysis carried out by Struiksma et al (1985) is given (Section 2.2) and subsequently the results are extended to arrive at the prediction method (Section 2.3).

### 2.2 Linear perturbation analysis

For their study of the bed deformation in curved alluvial channels Struiksma et al (1985) developed a mathematical model under the assumption that time-averaged flow velocities and overall sediment transport relations can be used. In addition to running a comprehensive numerical model of the relevant phenomena they carried out a linear perturbation analysis of the equilibrium bed topography according to this model. The main difference between this linear analysis and previous stability analyses (see Bettess and White (1983) for a review) is that, a non-uniform but steady perturbation is considered, whereas a uniform but unsteady perturbation is considered in the stability analyses. In view of the relatively large propagation velocity of the bed perturbations, as predicted by the latter and the generally low erodibility of the banks, the stability analyses cannot provide an adequate explanation for the development of meanders but only an explanation for the development of alternate bars (see Olesen, 1983 and Blondeaux and Seminara, 1985).

The steady perturbation analysis formally applies to mildly curved channels. Although in strongly curved channels this mild curvature approximation may give rise to some quantitative influence on the results, it is postulated that the general character of the solutions remains unaltered. Under a number of other simplifications the complete non-linear mathematical model is linearised. The analysis considers harmonic bed level perturbations of the form:

$$h' = \hat{h} \exp i(kx + k_y y) \quad (2.1)$$

where  $\hat{h}$  is a complex number in which the absolute value gives the amplitude and the modulus the phase of the perturbation,  $i = \sqrt{-1}$ ,  $k_y$  is the wave number in transverse  $y$ -direction and  $k$  is a complex wave number in downstream  $x$ -direction. The imaginary part of  $k$  describes the development of the amplitude

of the perturbation in the flow direction. The solution of the remaining variables is in a similar form. Impermeable side walls put constraints onto the transverse wave number, namely:

$$k_w = m \frac{\pi}{B}, \quad m = 1, 2, 3 \dots \quad (2.2)$$

For a meandering channel  $m = 1$ . Higher values of  $m$  pertain to braiding channels. Introducing Equation (2.1) into the linearized model yields non-trivial results if  $k$  satisfies a certain condition, in the form of a complex polynomial equation. The solution of this equation is generally dominated by two roots which have mostly the form:

$$k = \pm k_r + i k_i = \pm \frac{2\pi}{L_p} + i \frac{1}{L_D} \quad (2.3)$$

where:  $L_p$  = wave length and  $L_D$  = damping length.

The roots mainly depend on the ratio of the adaptation length  $\lambda_s$  of the bed topography development and the adaptation length  $\lambda_w$  of the main flow. These adaptation lengths are defined as (Struiksma, 1983; de Vriend and Struiksma, 1983):

$$\lambda_s = \frac{1}{\pi^2} \left(\frac{B}{h}\right)^2 f(\theta) h \quad (2.4)$$

$$\lambda_w = \frac{C^2}{2g} h \quad (2.5)$$

where  $B$  = channel width,  $h$  = channel depth,  $C$  = Chezy coefficient,  $g$  = acceleration of gravity and  $f(\theta)$  = weighing function of influence of sloping bed,  $\theta$  = Shields parameter defined via  $\theta = hi/\Delta D$ ,  $\Delta$  = relative density defined via  $\Delta = (\rho_s - \rho)/\rho$ ,  $\rho_s$  = specific density of sediment,  $\rho$  = specific density of water. The expression for the ratio  $\lambda_s/\lambda_w$  reads:

$$\frac{\lambda_s}{\lambda_w} = \frac{2}{\pi^2} \cdot \frac{g}{C^2} \cdot \left(\frac{B}{h}\right)^2 \cdot f(\theta) \quad (2.6)$$

In Struiksma et al (1985) for  $f(\theta)$  the following expression is used:

$$f(\theta) = 1.5 \theta \quad (2.7)$$

but there is evidence (van Mierlo, 1985; Odgaard, 1981) that for natural channels an improved expression reads:

$$f(\theta) = 0.85 \theta^{0.5} \quad (2.8)$$

The dependence of  $L_D$  and  $L_p$  in Equation (2.3) on the ratio  $\lambda_s/\lambda_w$  is shown in Figure 3. This figure presents the results of a strongly simplified model suggested by Struiksma (1983), which very well represents the behaviour of the roots according to the complete linearised model. Figure 3 also shows that the roots depend also on the value of a coefficient  $b$ . This coefficient  $b$  relates the sediment transport  $s$  per unit width to the velocity  $u$  via:

$$s \propto u^b \quad (2.9)$$



It is assumed that  $b$  is a constant. This implies that the sediment transport formula is approximated by a power law of the flow velocity. The parameter  $b$  depends on the Shields parameter. If the value of the Shields parameter lies close to the threshold value the parameter  $b$  is large and strongly variable. For larger transport rates, lower values for  $b$  are found. Figure 4 provides the approximate relationship between  $b$  and the Shield parameter  $\theta'$  (corrected for form roughness) as derived from the Meyer-Peter and Müller transport formula. The applied expression for  $\theta'$  reads:

$$\theta' = \frac{u^2}{C_{90}^2 \cdot \Delta \cdot D} \quad (2.10)$$

where  $C_{90} = 18 \log(12h/D_{90})$ . Equation (2.10) allows to estimate  $b$  for given  $u$ ,  $C_{90}$ ,  $\Delta$  and  $D$ . Another sediment transport formule results in a different relationship between  $b$  and  $\theta'$  (e.g. for the Engelund-Hansen formula  $b = 5$ ).

### 2.3 Criterion for planform classification

The foregoing summary of the linear perturbation analysis shows that the damping length  $L_D$  (Figure 3(a)) and wave length  $L_p$  (Figure 3(b)) are related to the ratio  $\lambda_s/\lambda_w$ . Now Figure 3 is considered in some more detail to derive a criterion for planform classification. According to Figure 3(a) the perturbations are damped in the downstream direction for  $\lambda_w/L_D > 0$ . If  $\lambda_w/L_D < 0$ , the perturbations will grow in downstream direction. Figure 3(b) shows that only for a restricted range of  $\lambda_s/\lambda_w$  (harmonic range) the wave number  $2\pi\lambda_w/L_p$  can be defined. Beyond that range the present approach based on  $m = 1$  (one single channel) is not appropriate any more. Probably braiding will occur, more channels will develop and the value of  $\lambda_s/\lambda_w$  is relatively large. Thus tentatively the following criterion can be formulated for the transition from meandering to braiding:

$$\lambda_w/L_D > 0 \quad \text{meandering}$$

within harmonic range for  $m = 1$

$$\lambda_w/L_D < 0 \quad \text{transition}$$

within harmonic range for  $m = 1$

$$\lambda_w/L_D < 0 \quad \text{braiding}$$

beyond harmonic range for  $m = 1$ .

For the time being the value of  $\lambda_s/\lambda_w$  where  $\lambda_w/L_D = 0$  is indicated as  $\lambda_s/\lambda_w^*$  and where  $2\pi\lambda_w/L_p = 0$  as  $\lambda_s/\lambda_w^{**}$  (upper limit of harmonic range). Thus the above criterion can also be formulated as:

$$\lambda_s/\lambda_w < \lambda_s/\lambda_w^* \quad \text{meandering}$$

$$\lambda_s/\lambda_w^* < \lambda_s/\lambda_w < \lambda_s/\lambda_w^{**} \quad \text{transition}$$

$$\lambda_s/\lambda_w^{**} < \lambda_s/\lambda_w \quad \text{braided}$$



The following remarks are made:

- (i) The values of  $\lambda_s/\lambda_w^*$  and  $\lambda_s/\lambda_w^{**}$  depend on the value of b (see Figure 3), according to the following relationship:

$$\lambda_s/\lambda_w^* = 2/(b - 3) \quad (2.11)$$

$$\lambda_s/\lambda_w^{**} = \left\{ \frac{1}{2} (b + 1) - \frac{1}{2} [(b + 1)^2 - 2 (b - 3)]^{0.5} \right\}^{-1} \quad (2.12)$$

- (ii) It can be shown that always  $\lambda_s/\lambda_w^{**} > \lambda_s/\lambda_w^*$ .

### 3. Verification of proposed criterion

For the verification of the proposed criterion data are required from alluvial channels, of which both the planform and channel characteristics should be available. The required channel characteristics are B, u (velocity), i, h and D, D<sub>90</sub>. For the verification data from both natural rivers and laboratory channels are available. Only data readily available in the literature are used here; no special tests or measurements were done.

Data was selected from the following sources:

- natural rivers (see Table 1):
  - Leopold and Wolman (1957)
  - Chitale (1970, 1973)
  - Mosley (1976)
  - RPT/Nedeco/BCL (1987)
- Laboratory channels (see Table 2):
  - Fujita and Muramoto (1981) (Items II-1 ... II-7)
  - Hong and Davies (1979) (Items III-1)
  - Schumm, and Khan (1972) (Items IV-1 ... IV-3)
  - Leopold and Wolman (1957) (Items V-1)
  - Ashmore (1980) (Items VIII-1)

The following remarks are made regarding the selected data:

- (i) Other river and channel data included in the above literature was not used because of one of the following reasons:
- the data was not complete,
  - the (test)conditions did not deviate substantially from other sets of data already included.
- (ii) Information from other sources (Ashmore and Parker, 1983; Parker and Anderson, 1975; Khan, 1978; Southard et al, 1981 could not be used for the same reasons.

From the basic data obtained from the literature the following parameters were derived (see Tables 1 and 2):  $\lambda_s/\lambda_w$ , C<sub>90</sub> (whereby it was assumed that D<sub>90</sub> = 2.5 D<sub>50</sub>, in case D<sub>90</sub> was not known),  $\theta'$ ,  $\lambda_s/\lambda_w^*$  and  $\lambda_s/\lambda_w^{**}$ . For the transformation of  $\theta'$  to b Figure 4 was used. By comparing the computed value of  $\lambda_s/\lambda_w$  with the threshold values  $\lambda_s/\lambda_w^*$  and  $\lambda_s/\lambda_w^{**}$ , the proposed criterion for planform classification can be evaluated. The results are given in the last columns of the Tables 1 and 2. From an inspection of these results it follows that the proposed criterion results in a fair prediction of channel pattern, although in the case of laboratory channels the predictions are less good than for natural rivers. The latter may be due to e.g. the limited accuracy in measuring the average water depth in small laboratory channels the applied relation between  $\theta'$  and b, etc.

The proposed criterion merely consists of computing the values of  $\lambda_s/\lambda_w$  and  $b$  comparing these with the appropriate values of  $\lambda_s/\lambda_w^*$  and  $\lambda_s/\lambda_w^{**}$ . The latter can also be done graphically as is shown in Figure 5. In this Figure points have been introduced that correspond to the data sets listed in the Tables 1 and 2. The index B and M indicate whether the channel pattern is braided or meandering.

Considering the results of the verification as presented in the Tables 1 and 2 and Figure 5 it may be concluded that starting from the results of the linear analysis by Struiksma et al (1985) a criterion was developed that results in a fair prediction of the channel pattern. Still, however, certain channels are not correctly classified, which may be caused by e.g. less correct data or a less distinct threshold boundary than assumed here, but also because the present theory is based on a linear analysis that does not necessarily reflect all characteristics of the complete phenomenon.

#### 4. Discussion

A major disadvantage of the proposed classification criterion is that it requires a priori knowledge on channel characteristics like widths and average depths during bankfull discharge. One would prefer a method that starts from more basic parameters like the bankfull discharge and the valley slope (or alternatively the yearly amount of sediment the river has to carry). Only by combining the present method with a method that predicts the channel characteristics a more universal classification method can be obtained.

It is of interest to combine the method proposed here with Lacey's regime equations. From Equation (2.6), the following relationship should hold for the transition from meandering to braiding:

$$\frac{2}{\pi^2} \cdot \frac{g}{C^2} \cdot \left(\frac{B}{h}\right)^2 \cdot f(\theta) = \text{constant} \quad (4.1)$$

or:

$$\left(\frac{B}{h}\right)^2 \cdot f(\theta) \cdot h^3 : : u^2 \quad (4.2)$$

Lacey's regime equations read:

$$h : : Q^{1/3} \quad (4.3)$$

$$B : : Q^{1/2} \quad (4.4)$$

Introducing these expressions into Equation (4.2) and using  $Q = B \cdot h \cdot u$  it is possible to find a relationship between  $Q$  and  $i$ . This relationship depends on the assumption made as to how  $f(\theta)$  reads. If  $f(\theta) : : \theta$ , then one obtains:

$$i : : Q^{-1/3} (\Delta D)^{1/2} \quad (4.5a)$$

For  $f(\theta) : : \theta^{0.5}$  a slightly different expression is found, notably:

$$i : : Q^{-1/3} (\Delta D)^{1/3} \quad (4.5b)$$

The exponent of  $Q$  in both expressions should be compared to e.g. Equation (1.1), proposed by Leopold and Wolman (1957). Although the exponent  $-1/3$  is closer to  $-0.44$  than the value of  $-1/6$  found by Ferguson (1984) by combining Parker's (1976) prediction method with Lacey's regime equation, there is still



a considerable discrepancy. Tentatively, however, it can be stated that the classification method proposed here can be shown to be similar to some extent to the empirical method proposed by Leopold and Wolman (1957).

Next the influence of the bed material size is considered. From Equation (4.5a) and (4.5b) it follows that the threshold is also affected by  $\Delta$  and by  $D$ . Figure 2, taken from Ferguson (1984), provides empirical evidence for this. From the data plotted in this figure it may be concluded which of the two expressions is more appropriate. Until now, the authors have not yet carried out an analysis to verify this. Anyhow it seems more appropriate to plot  $i/(\Delta D)^{1/2}$  or  $i/(\Delta D)^{1/3}$  against the bankfull discharge  $Q$  than only  $i$  versus  $Q$ .

Finally, one more implication of the proposed classification method is discussed here. It was proposed that if  $\lambda_s/\lambda_w$  exceeds  $\lambda_s/\lambda_w^{**}$  that in that case braiding would occur resulting in multiple channels. To study this in some more detail it is for the time being assumed that when the total bankfull discharge  $Q_t$  exceeds the threshold value for that particular valley slope a number of parallel channels will develop. All these individual channels should have such channel dimensions that  $\lambda_s/\lambda_w$  is smaller than  $\lambda_s/\lambda_w^{**}$  and anyhow smaller than  $\lambda_s/\lambda_w^{**}$ . Equation (4.5) can be written as:

$$i : : \left(\frac{Q_t}{n}\right)^{-1/3} (\Delta D)^\alpha \quad (4.6)$$

where  $n$  = number of parallel braids and  $\alpha$  = exponent depending as which expression for  $f(\theta)$  is used. Equation (4.6) can also be written as:

$$i : : Q_t^{-1/3} n^{1/3} (\Delta D)^\alpha \quad (4.7)$$

The only study known to the authors that provides information on the number of braids is the work by Howard et al (1970). Based on observations on natural rivers they derived the following relationship:

$$n = 0.58 R_q^{-0.19} i^{0.41} Q^{0.25} \quad (4.8)$$

where  $R_q$  = ratio of mean annual flood to the mean annual discharge. Neglecting the influence of  $R_q$  for the time being, Equation (4.8) can be written as:

$$i : : n^{2.4} Q^{-0.60} \quad (4.9)$$

The exponent of  $n$  is quite different for the Equations (4.7) and (4.9), but the predicted behaviour is to some extent similar: an increase of the slope results in an increase of the number of braids. In fact this behaviour was predicted by Bettess and White (1983) and it was used recently for the scaling of a model of a braided river (Klaassen, 1987). The large deviation between the exponent of  $n$  in the Equations (4.7) and (4.9) could be due to the limited data set used by Howard et al (1970). It is worthwhile to study the number of braids as a function of  $i$  and  $Q$  in more detail in the future.

## 5. Conclusions

Based on the study presented here the following conclusions can be drawn:

- (i) It is possible to extend the work of Struiksma et al (1985) on bed deformation in curved alluvial channels to arrive at a classification method for channel patterns. The proposed method predicts observed channel patterns fairly well.
- (ii) It can be shown that the proposed classification method in combination with Lacey's regime equations results in a more universal method that

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is comparably to the classic Leopold and Wolman (1957) classification diagram.

- (iii) It is preferable to plot  $1/\sqrt{(\Delta D)^{1/2}}$  or  $1/\sqrt{(\Delta D)^{1/3}}$  against bankful discharge  $Q$  than only  $i$  versus  $Q$  as was done by Leopold and Wolman (1957).
- (iv) Increase of slope or of discharge will probably result in an increase of the number of braids in a braiding river system.
- (v) More research is needed to substantiate in particular the conclusions (iii) and (iv).

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| Name of river          | B(m) | h(m)   | I (-)                  | u (m/s) | D (m)                 | $\frac{\lambda_B}{\lambda_W}$<br>(-) | C <sub>90</sub><br>(-) | O'<br>(-)             | $\frac{\lambda_B}{\lambda_W}$<br>(-) | $\frac{\lambda_B}{\lambda_W}$<br>(-) | channel pattern |          | +/- |
|------------------------|------|--------|------------------------|---------|-----------------------|--------------------------------------|------------------------|-----------------------|--------------------------------------|--------------------------------------|-----------------|----------|-----|
|                        |      |        |                        |         |                       |                                      |                        |                       |                                      |                                      | predicted       | observed |     |
| 1. Jamuna              | 5000 | 6.0    | $6 \times 10^{-5}$     | 1.33    | $0.2 \times 10^{-3}$  | 251                                  | 86.0                   | 0.725                 | 10                                   | 42                                   | B               | B        | +   |
| 2. Brahmaputra         | 9449 | 1.52   | $2.52 \times 10^{-4}$  | 1.73    | $0.3 \times 10^{-3}$  | 7396                                 | 72.1                   | 1.16                  | 13                                   | 55                                   | B               | B        | +   |
| 3. Saraswati           | 320  | 2.54   | $12.5 \times 10^{-4}$  | 2.50    | $0.3 \times 10^{-3}$  | 34                                   | 76.3                   | 2.17                  | 13                                   | 55                                   | B/M             | M        | +   |
| 4. Ranoli              | 287  | 0.89   | $7.8 \times 10^{-4}$   | 1.57    | $0.11 \times 10^{-3}$ | 97                                   | 75.8                   | 2.36                  | 13                                   | 55                                   | B               | B        | +   |
| 5. Yamuna              | 366  | 3.46   | $3.1 \times 10^{-4}$   | 1.38    | $0.32 \times 10^{-3}$ | 14                                   | 79.1                   | 0.576                 | 10                                   | 42                                   | B/M             | M        | +   |
| 6. Ganga               | 4023 | 5.18   | $1.6 \times 10^{-4}$   | 2.53    | $0.43 \times 10^{-3}$ | 144                                  | 78.9                   | 1.45                  | 13                                   | 55                                   | B               | M        | -   |
| 7. Yashwantpur         | 122  | 1.68   | $4.3 \times 10^{-4}$   | 1.40    | $2.05 \times 10^{-3}$ | 48                                   | 57.9                   | 0.273                 | 5                                    | 22                                   | B               | B        | +   |
| 8. Indus               | 983  | 4.88   | $1 \times 10^{-4}$     | 1.49    | $0.18 \times 10^{-3}$ | 19                                   | 85.2                   | 1.03                  | 13                                   | 55                                   | B/M             | M        | +   |
| 9. Tonle Sap           | 1402 | 6.10   | $0.167 \times 10^{-4}$ | 0.84    | $0.01 \times 10^{-3}$ | 32                                   | 109.6                  | 3.56                  | 13                                   | 55                                   | B/M             | B        | +   |
| 10. Savannah           | 107  | 5.18   | $1.1 \times 10^{-4}$   | 1.55    | $0.8 \times 10^{-3}$  | 0.175                                | 74.0                   | 0.332                 | 10                                   | 42                                   | M               | M        | +   |
| 11. Mississippi        | 1381 | 20.12  | $0.473 \times 10^{-4}$ | 1.54    | $0.5 \times 10^{-3}$  | 3.5                                  | 88.3                   | 0.37                  | 10                                   | 42                                   | M               | M        | +   |
| 12. Mississippi        | 2144 | 16.76  | $0.815 \times 10^{-4}$ | 1.19    | $0.3 \times 10^{-3}$  | 45                                   | 90.9                   | 0.346                 | 10                                   | 42                                   | B               | M        | -   |
| 13. Beaver Creek       | 1280 | 0.27   | $6.6 \times 10^{-5}$   | 0.26    | $2.96 \times 10^{-2}$ | 61                                   | 22.7                   | $2.69 \times 10^{-3}$ | 0.28                                 | 3                                    | B               | B        | +   |
| 14. Cottonwood Creek   | 1310 | 0.24   | $5 \times 10^{-5}$     | 0.55    | $1.8 \times 10^{-2}$  | 2.9                                  | 25.7                   | $1.55 \times 10^{-2}$ | 0.28                                 | 3                                    | B/M             | B        | +   |
| 15. Yellowstone River  | 1280 | 1.40   | $1.52 \times 10^{-3}$  | 0.48    | $8.23 \times 10^{-2}$ | 3.9                                  | 27.6                   | $2.2 \times 10^{-3}$  | 0.28                                 | 3                                    | B               | B        | +   |
| 16. Watta Branch       | 399  | 0.1518 | $3.1 \times 10^{-3}$   | 0.189   | $2.50 \times 10^{-2}$ | 1.6                                  | 19.8                   | $2.2 \times 10^{-3}$  | 0.28                                 | 3                                    | B/M             | B        | +   |
| 17. Middle Piney Creek | 698  | 0.21   | $9.1 \times 10^{-3}$   | 0.61    | $9.4 \times 10^{-2}$  | 1.1                                  | 18.5                   | $7.0 \times 10^{-3}$  | 0.28                                 | 3                                    | B/M             | M        | +   |
| 18. Beaver             | 55   | 2.74   | $2.4 \times 10^{-4}$   | 0.95    | $0.5 \times 10^{-3}$  | 0.45                                 | 72.7                   | 0.207                 | 4                                    | 28                                   | M               | M        | +   |
| 19. Ohau River         | 450  | 0.25   | $6.5 \times 10^{-3}$   | 0.4     | $20 \times 10^{-3}$   | 12414                                | 25.2                   | $7.66 \times 10^{-3}$ | 0.28                                 | 3                                    | B               | B        | +   |
| 20. Ohua River         | 450  | 0.56   | $6.5 \times 10^{-3}$   | 1.5     | $20 \times 10^{-3}$   | 590                                  | 31.5                   | $9.60 \times 10^{-3}$ | 0.28                                 | 3                                    | B               | B        | +   |
| 21. Rio Parana         | 1600 | 10.0   | $5 \times 10^{-5}$     | 1.0     | $0.3 \times 10^{-3}$  | 22                                   | 86.8                   | 0.268                 | 5                                    | 22                                   | B               | B        | +   |

Table 1 Comparison of predicted and observed channel pattern of natural rivers



| Test number | B (m)  | h (m)                 | f (-)  | u (m/s) | D (m)                 | $\frac{\lambda_B}{\lambda_W}$<br>(-) | C <sub>90</sub><br>(-) | $\theta'$<br>(-)      | $\frac{\lambda_B}{\lambda_W}$<br>(-) | $\frac{\lambda_B}{\lambda_W}$<br>(-) | channel pattern |          | +/- |
|-------------|--------|-----------------------|--------|---------|-----------------------|--------------------------------------|------------------------|-----------------------|--------------------------------------|--------------------------------------|-----------------|----------|-----|
|             |        |                       |        |         |                       |                                      |                        |                       |                                      |                                      | predicted       | observed |     |
| II-1        | 1.55   | 2.13x10 <sup>-2</sup> | 0.005  | 0.458   | 0.88x10 <sup>-3</sup> | 1.24                                 | 33.2                   | 1.89x10 <sup>-1</sup> | 3.3                                  | 15                                   | M               | B        | -   |
| -2          | 2.18   | 1.90x10 <sup>-2</sup> | 0.005  | 0.374   | 0.88x10 <sup>-3</sup> | 3.74                                 | 32.3                   | 1.37x10 <sup>-1</sup> | 2                                    | 4.8                                  | B/M             | B        | +   |
| -3          | 3.10   | 1.34x10 <sup>-2</sup> | 0.005  | 0.388   | 0.88x10 <sup>-3</sup> | 8.70                                 | 29.6                   | 1.71x10 <sup>-1</sup> | 2.85                                 | 13                                   | B/M             | B        | +   |
| -4          | 2.131  | 2.69x10 <sup>-2</sup> | 0.005  | 0.286   | 0.88x10 <sup>-3</sup> | 1.39                                 | 35.0                   | 6.44x10 <sup>-2</sup> | 0.4                                  | 2.5                                  | B/M             | M        | +   |
| -5          | 2.77   | 1.97x10 <sup>-2</sup> | 0.002  | 0.292   | 0.88x10 <sup>-3</sup> | 2.56                                 | 32.6                   | 7.47x10 <sup>-2</sup> | 0.83                                 | 5.2                                  | B/M             | M        | +   |
| -6          | 1.8725 | 2.19x10 <sup>-2</sup> | 0.005  | 0.380   | 0.88x10 <sup>-3</sup> | 2.59                                 | 33.4                   | 1.29x10 <sup>-1</sup> | 1.8                                  | 4.1                                  | B/M             | M        | +   |
| -7          | 2.25   | 1.54x10 <sup>-2</sup> | 0.005  | 0.380   | 0.88x10 <sup>-3</sup> | 7.15                                 | 30.7                   | 9.51x10 <sup>-2</sup> | 1.1                                  | 6.3                                  | B               | M        | +   |
| III-1       | 0.1    | 2.00x10 <sup>-2</sup> | 0.07   | 0.052   | 0.17x10 <sup>-3</sup> | 4.82                                 | 42.7                   | 5.39x10 <sup>-3</sup> | 0.28                                 | 3.0                                  | B               | B        | +   |
| IV-1        | 1.55   | 1.8x10 <sup>-2</sup>  | 0.015  | 0.52    | 0.70x10 <sup>-3</sup> | 6.09                                 | 30.8                   | 2.47x10 <sup>-1</sup> | 4                                    | 18.0                                 | B/M             | B        | +   |
| -2          | 0.56   | 4.42x10 <sup>-2</sup> | 0.0064 | 0.387   | 0.70x10 <sup>-3</sup> | 0.26                                 | 37.8                   | 9.08x10 <sup>-2</sup> | 1                                    | 5.8                                  | B/M             | M        | +   |
| -3          | 0.427  | 4.57x10 <sup>-2</sup> | 0.0026 | 0.296   | 0.70x10 <sup>-3</sup> | 0.065                                | 38.1                   | 5.22x10 <sup>-2</sup> | 0.28                                 | 3.0                                  | M               | M        | +   |
| V-1         | 0.49   | 3.6x10 <sup>-2</sup>  | 0.013  | 0.38    | 0.84x10 <sup>-3</sup> | 0.62                                 | 34.8                   | 8.62x10 <sup>-2</sup> | 1                                    | 5.8                                  | M               | B        | -   |
| VII-1       | 0.175  | 3.5x10 <sup>-2</sup>  | 0.013  | 0.163   | 1.2x10 <sup>-3</sup>  | 0.35                                 | 31.8                   | 8.15x10 <sup>-2</sup> | 0.83                                 | 5.2                                  | M               | B        | -   |

Table 2 Comparison of predicted and observed channel pattern of laboratory channels

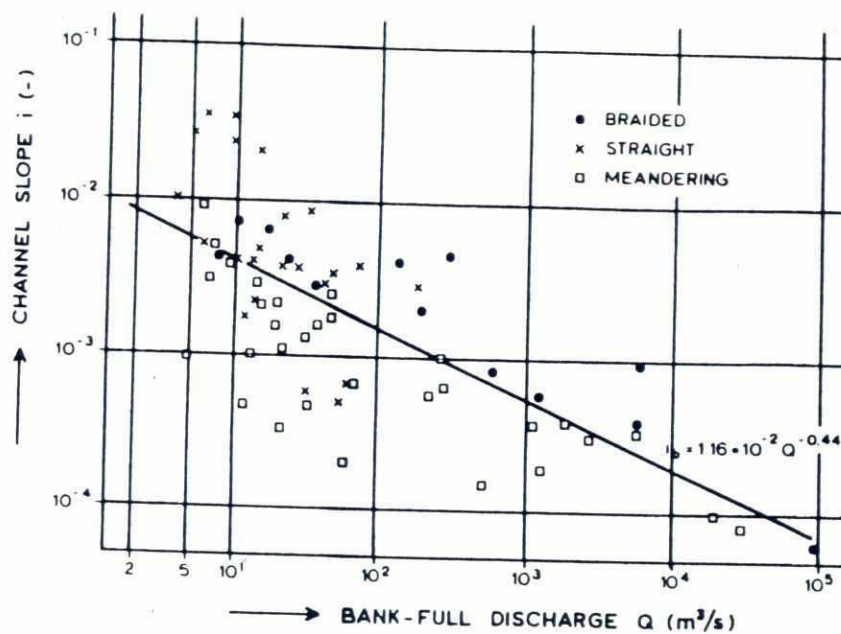


Figure 1 Classification diagram proposed by Leopold and Wolman.

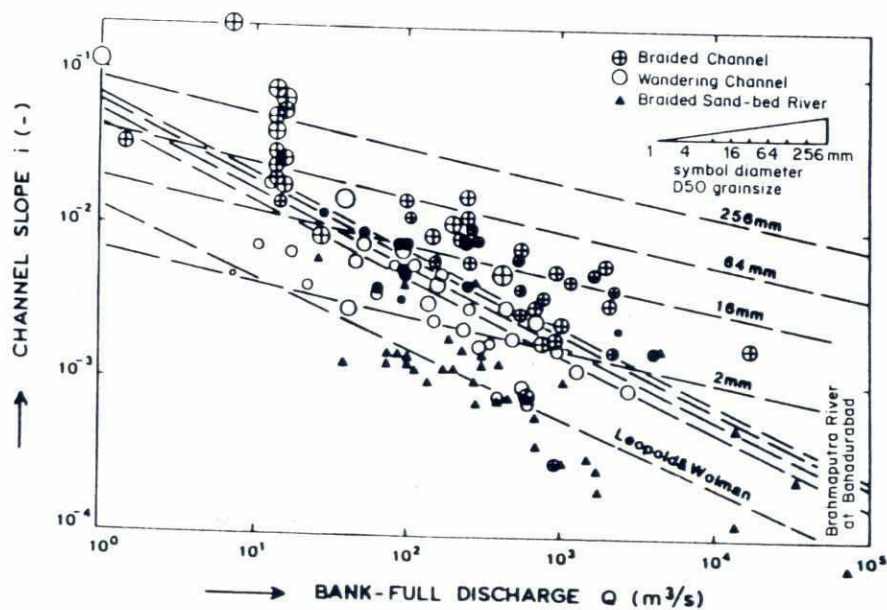
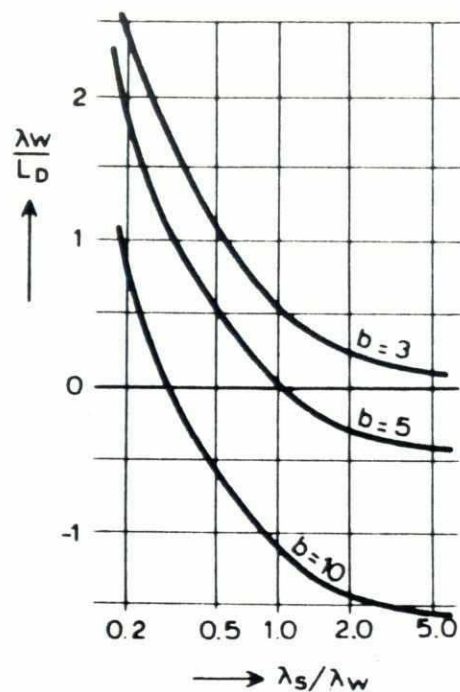


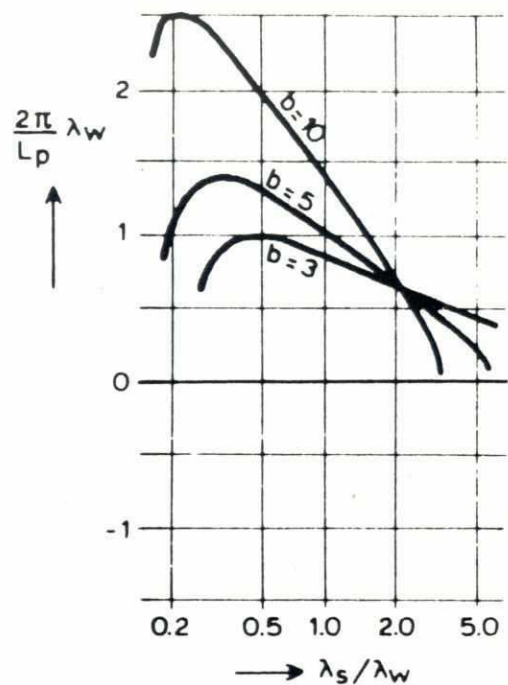
Figure 2 Discharge-gradient-grainsize plot of braided rivers (after Ferguson, 1984).

Note: the drawn lines are not relevant here.





(a) damping



(b) wave length

Figure 3 Damping and wavelength of perturbations in the linear analysis versus  $\lambda_s/\lambda_w$ .

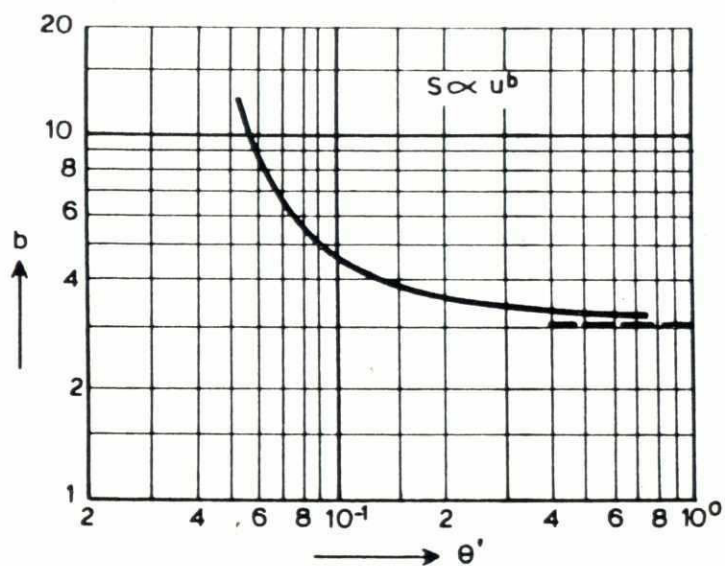


Figure 4 Approximate relationship between  $b$  and  $\theta'$ .

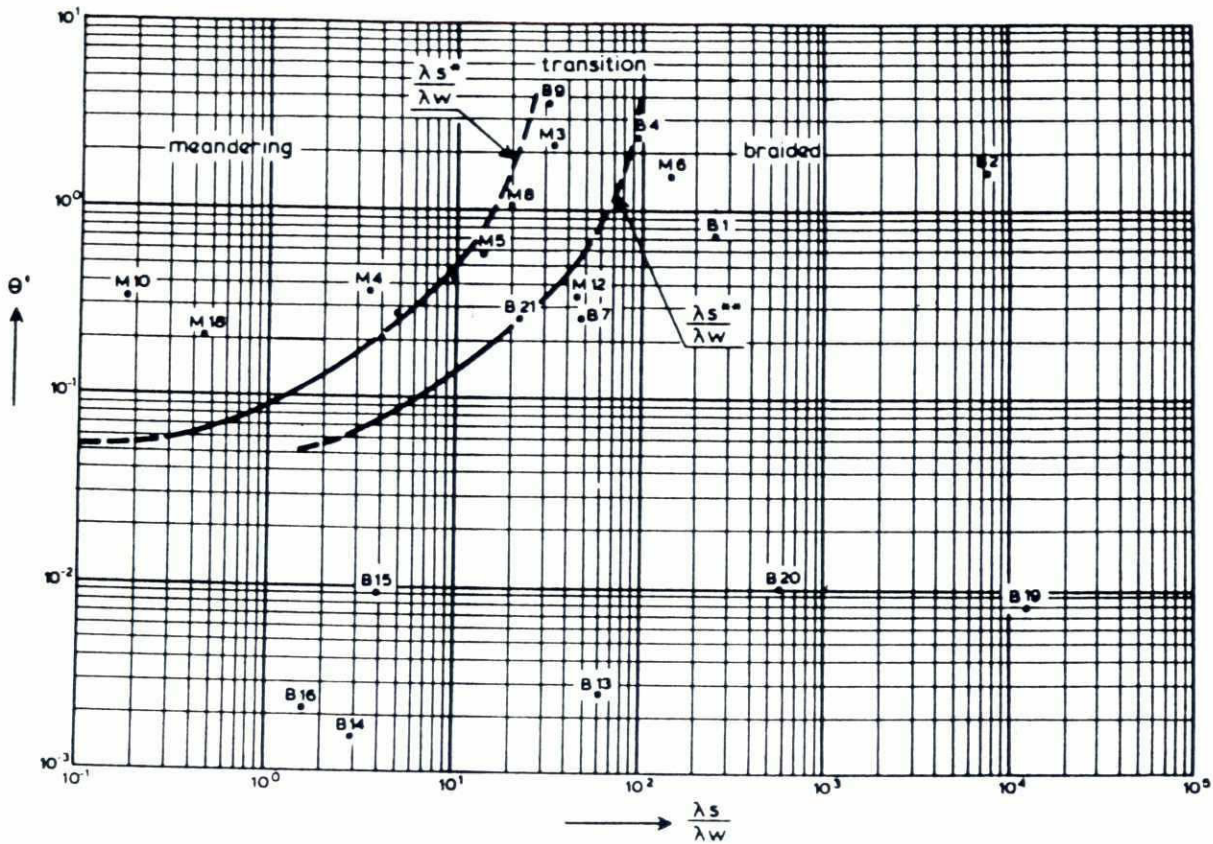


Figure 5 Classification diagram based on  $\theta'$  and  $\lambda_s/\lambda_w$ .



### 3 River Characteristics

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## Paper 3.1

KHAN, H.R. (1975), Roughness coefficient for some rivers in Bangladesh, *Jr. Institution of Engineers, Bangladesh*, vol. 2(4):105-112.



## Roughness Coefficient for some Rivers in Bangladesh<sup>†</sup>

Hamidur Rahman Khan \*

### Abstract :

The Manning's  $n$  was developed empirically as a coefficient which remained a constant for a given boundary condition regardless of slope of channel, size of channel or depth of flow. But however, each of these factors causes  $n$  to vary to some extent.

A study of the roughness characteristics of four rivers in Bangladesh revealed that the roughness coefficient  $n$  varied significantly with stage. The rivers are the Kumar at Lakhmika, Gumti at Comilla, Halda at South Sunderpur, and Sangu at Bandarban. For Kumar at Lakhmika the maximum value of  $n$  was about 10 times the minimum value.

The magnitude of  $n$  was greatest at low flow and its magnitude reduced sharply as the depth increased. Again its value increased as the depth of flow increased from intermediate to higher stages.

Manning's  $n$  accounts for the boundary resistance besides other losses. Boundary roughness includes the grain roughness and form roughness. Form roughness depends on the type of bedform. One of the major reasons for the above mentioned variation of roughness coefficient  $n$  with depth is due to the changes in the bedforms caused by changing stream power, shear stress, and relative roughness associated with stage variation.

Great care must be taken before selecting the value of  $n$  for natural rivers because sometimes standard tables for roughness coefficients can be misleading.

### Introduction :

In an effort to correlate and systematize existing data from natural and artificial channels, Robert Manning in 1889 proposed an equation which was developed into

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

In which

$V$  is the mean velocity in fps,  $R$  is the hydraulic radius, in feet,  $S$  is the slope of energy line, and  $n$  is the coefficient of roughness specifically known as Manning's  $n$ . The Manning  $n$  was developed empirically as a coefficient which remained a constant for a given boundary condition, regardless of slope of channel, size of channel, or depth of flow. As a matter of fact, however, each of these factors causes  $n$  to vary to some extent. In other words, the Reynolds

<sup>†</sup> This paper was selected for presentation in the 19th Annual Convention of the IEB, 1974.

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number, the depth of flow, and the relative roughness have an influence on the magnitude of Mannings  $n$ .

Initially Mannings equation was presented in the form

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

and the unit of length used was meter. In 1911 Buckley converted this equation to foot second units by inserting the factor  $(3.28)^{2/3}$  or 1.486 (there being 3.28 feet in a meter). The above equation is known in the English-speaking world as the Manning equation, although on the continent of Europe it is sometimes known also as Strickler's equation. The coefficient 1.486, although arising from a perfectly reasonable conversion process, is far more precise than is necessary; in fact the precision with which  $n$  is known hardly warrants writing the coefficient more precisely than as 1.49 or even 1.5. The Manning equation has proved most reliable in practice and extremely popular in most western countries.

#### Roughness Coefficient $n$ :

Many engineers think of a channel as having a single value of  $n$  for all discharges and stages. Actually the value of  $n$  is highly variable and dependent on many factors.

The  $n$  in Manning's formula normally accounts for the channel roughness and for normal internal distortion. Other losses in natural channels result from velocity disturbances caused by curvature of the channel, bank irregularities, expansion or contraction of the channel, obstructions in the channel. Before applying additional corrections for such losses, the selected  $n$  value should be carefully reviewed to determine whether or not it already accounts for these losses.

#### Variation of $n$ with Stage:

Figures 1, 2, 3, and 4 show that the value of  $n$  varies with stage for the rivers Kumar at Lakshmikanda, Halda at South Sunderpur, Gumti at Comilla, and Sangu at Banderban respectively. For the first three rivers the value of  $n$  and hence roughness of the channel is greatest at very shallow depth and it reduces sharply as the depth increases. Again the  $n$  value increases with depth at greater stages.

For Sangu river at Banderban the Manning's  $n$  does not change appreciably with stage. The data for this paper have been taken from the Bangladesh WAPDA (presently Bangladesh Water Development Board) Water Supply Paper no. 338.

The following equation was used for the computation of  $n$

$$n = \frac{984.1}{Q} A R^{2/3} S^{1/2}$$

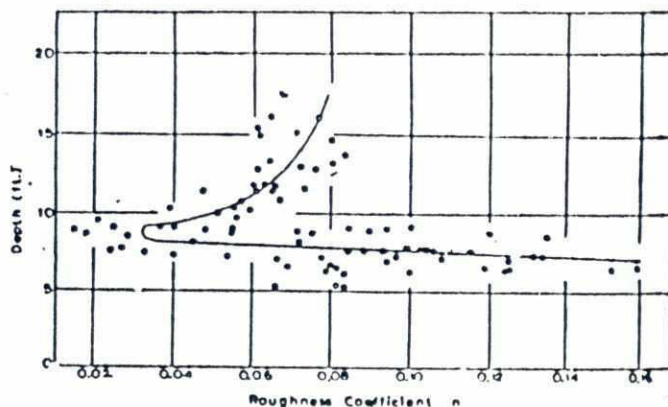


Figure-1: Variation of roughness coefficient with stage for Kumar at Lakshmikanda.

where  $Q$  was measured by 'velocity-area' method.  $A$  and  $R$  were obtained from the survey of the river cross section,  $S$  was taken as the instantaneous water surface slope between the two ends of a selected reach of the river  $S = \frac{h_w}{L}$ , where  $h_w$  was the water surface

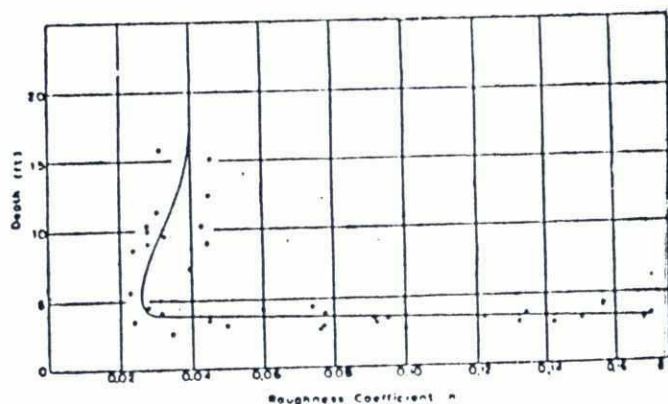


Figure-2: Variation of roughness coefficient with stage for Halda at South Sunderpur.



fall between the two ends of the reach and  $L$  is the length of the reach.

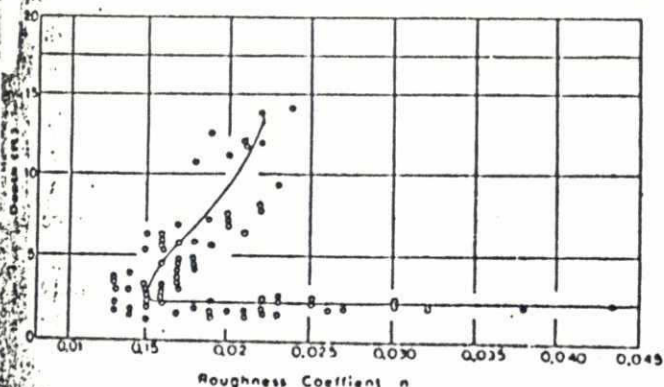


Figure-3 : Variation of roughness coefficient with stage for Gunti at Comilla.

In order to understand the variation of  $n$  with stage, one should try to get some idea about how the resistance characteristics of a channel changes with stage. It has already been stated that the Manning's  $n$  accounts for the boundary resistance

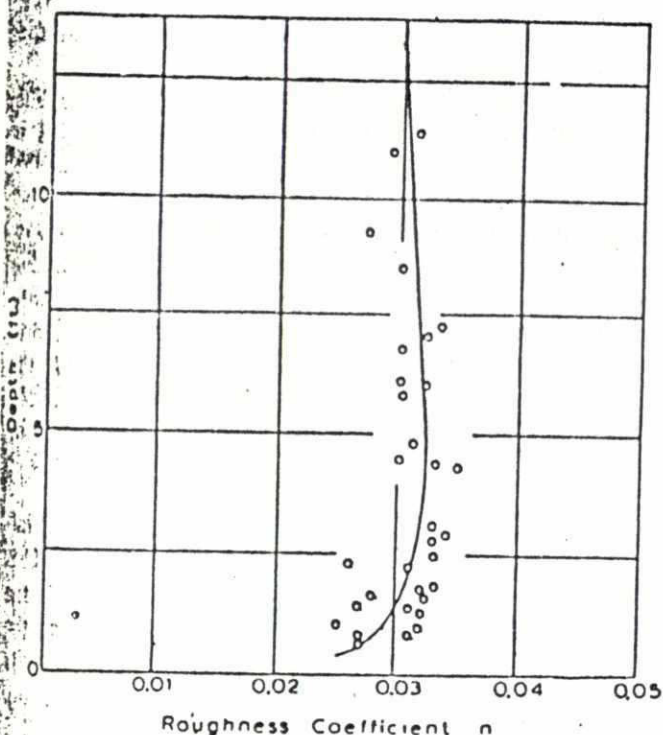


Figure-4 : Variation of roughness coefficient with stage for Sangu at Banderban.

JANUARY, 1975

besides other sources of energy losses. Boundary roughness includes the grain roughness (surface roughness) and the roughness caused by bedforms (form roughness). Grain roughness depends on the size of bed and bank materials and is more or less constant for a particular river cross section. Form roughness depends on the bedforms and it changes significantly as the bedforms change. It will be shown in the next few sections that bedforms are dependent on shear stress and stream power and hence on stage which accounts for the fact that form roughness depends on stage, discharge, slope, size of the bed material and magnitude of sediment discharge.

#### Bed Forms :

At extremely low velocities the critical shear stress of the bed is not exceeded and sediment motion does not occur. An increase in the velocity may result in sediment movement and formation of bedforms. The bedforms that may form in an alluvial channel are plane bed without sediment movement, ripples, ripples on dunes, dunes, plane bed with sediment movement,

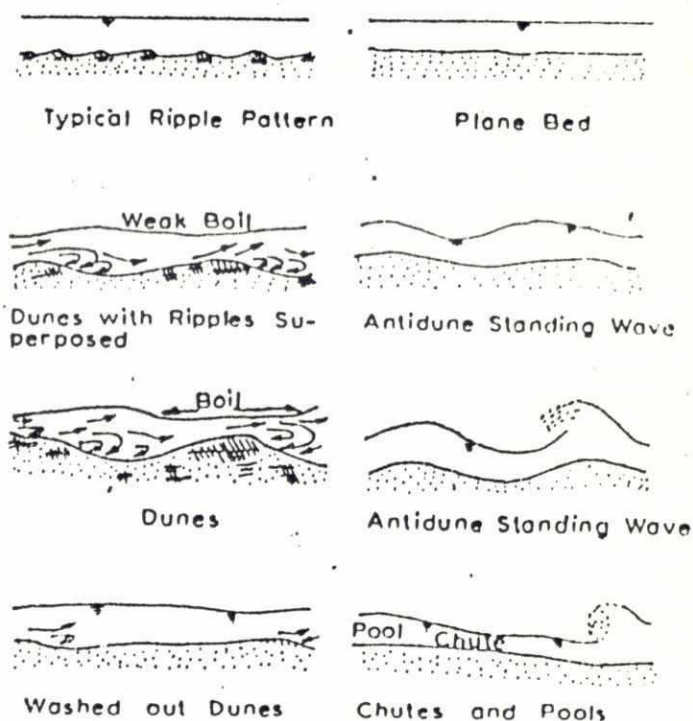


Figure-5 : Forms of bed roughness in alluvial channel (after Simons and Richardson 1961).



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 antidunes, and chutes and pools. These bedforms are listed in their order of occurrence with increasing values of stream power for bed materials having median size less than 0.6 mm. For bed materials coarser than 0.6 mm, dunes form instead of ripples after beginning of motion at small values of stream power. The typical form of each bedform is shown in figure 5. These bed-roughness elements are not mutually exclusive occurrences in time and space in a flume or natural river. They may form side by side in a cross section or reach of a natural stream giving a multiple roughness; or they may form in sequence, in time, producing variable roughness. Multiple roughness is specially related to variation in shear stress, stream power, or alluvial bed material.

A brief description of different bedforms are given below :

**Ripples :** Ripples are small triangle-shaped elements having a gentle upstream slopes and steep downstream slopes. Length ranges from 0.4 foot to 2 feet and height from 0.02 foot to 0.2 foot. Resistance to flow is large, and the resulting discharge coefficient ( $C/g^{1/2}$ ) ranges from 7 to 12. As depth increased, resistance to flow due to roughness decreased. Thus, there is a relative roughness effect produced by the ripple bed. Resistance to flow is independent of sand size when the bed configuration is one of ripples because ripple shape is independent of sand size and the effect of grain roughness is small relative to the form roughness. The separation zone downstream from a ripple causes very little, if any, disturbance on the water surface, and the flow contains little suspended bed material.

**Dunes :** When the shear stress or the stream power is gradually increased for flow over a bed having ripples or, if the bed material is coarser than 0.6 mm, over a plane bed, a rate of bed material transport, a magnitude of velocity, and a degree of turbulence will soon be achieved that causes sand waves called dunes to form. At smaller shear-stress values, the dunes will have ripples superimposed on their backs.

Dunes are large triangle-shaped elements similar to ripples. Their lengths range from 2 feet to many feet, depending on the scale of the flow systems. The amplitude of dunes can increase with increasing depth, so that the relative roughness can remain essentially constant or even increase with increase in

depth of flow. Field observations by Simons and Richardson (1970) indicated that dunes form in any channel, irrespective of the size of bed material, if the stream power is sufficiently large to cause general transport of the bed material without exceeding a Froude number of unity. For dunes, the discharge coefficient ( $C/g^{1/2}$ ) varied from 8 to 12.

**Plane Bed with Sediment Movement :** A plane bed is a bed without elevations or depressions larger than the largest grains of the bed materials. The resistance to flow for flow over a plane bed results from grain roughness and ( $C/g^{1/2}$ ) is large, ranging from 14 to 23. Grain roughness is not of the usual type, however, because grains roll, hop, and slide along the bed. For flow over a plane bed with sediment movement, the resistance to flow is slightly less than for flow over a static plane bed, which is essentially an artificial rigid-boundary condition that exists after screening when stream power is insufficient to cause significant transport of the bed material.

**Antidunes :** Antidunes form as a series or train of inphase symmetrical sand and water waves. Antidunes do not exist as a continuous train of wave that never change shape, rather, they form as trains of waves that gradually build up from plane bed and a plane water surface. The waves may grow in height until they become unstable and break like the sea surf, or they may gradually subside. The former have been called breaking antidunes; and the latter, standing waves. As the antidunes form and increase in height, they may move upstream, downstream, or remain stationary. Resistance to flow due to antidunes depends on how often the antidunes form, the area of the reach that they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as for a plane bed, and  $C/g^{1/2}$  ranges from 14 to 23.

**Chutes and Pools :** At very steep slopes, alluvial channel flow changes to chutes and pools. This type of flow consisted of a long chute in which the flow accelerated rapidly, a hydraulic jump at the end of the chute, and then a long pool in which the flow was tranquil but was accelerating. Resistance to flow was large, and  $C/g^{1/2}$  ranged from 9 to 16.

The above description of bedforms is the nomenclature proposed by Simons and Richardson (1961).



The most extensive experimental studies on bedforms were made by the U.S. Geological Survey at the Colorado State University, Fort Collins, Colorado,

U.S.A. Simons and Richardson's (1961) classification of bedforms based on these tests are presented in Table—1.

Table 1 : Classification of bedforms and other informations (after Simons and Richardson, 1961)

| Flow regime  | Bedform          | Bed materials concentration ppm | Mode of Sediment transport | Type of roughness            | Roughness $C/g^{1/2}$ |
|--------------|------------------|---------------------------------|----------------------------|------------------------------|-----------------------|
| Lower regime | Ripples          | 10-200                          | Discrete steps             | Form roughness predominates  | 7-12                  |
|              | Ripples on dunes | 100-1,200                       |                            |                              | —                     |
|              | Dunes            | 200-2,000                       |                            |                              | 8-12                  |
| Transition   | Washed-out dunes | 1,000-3,000                     |                            | Variable                     | 7-20                  |
| Upper regime | Plane bed        | 2,000-6,000                     | Continuous                 | Grain roughness predominates | 15-23                 |
|              | Antidunes        | 2,000                           |                            |                              | 14-23                 |
|              | Chutes and Pools | 2,000                           |                            |                              | 9-16                  |

#### Stream Power and Bedform :

Stream Power, as the term was used by Cook (1935) and Bagnold (1960) is the rate at which a stream loses energy per unit area of the boundary. The power for a whole channel can be defined as the product of the weight of fluid (in pounds) flowing per second and the energy per unit weight of the fluid (in foot pound per pound). This product can be written as  $wQx$  energy. The dimensions are  $FLT^{-1}$ . When the flow is uniform the energy loss per foot of channel length is given by the slope  $S$ , so that the stream power per unit length of channel becomes  $wQS$  (dimensions  $FT^{-1}$ ). The stream power per unit bed area

$$= \frac{wQS}{W}$$

$$= \frac{wVWDS}{W}$$

$$= wDSV$$

$$= TV$$

where

$w$  = specific weight of water-sediment mixture

$W$  = channel width

$T$  = shear stress =  $wDS$

$Q$  = discharge =  $VWD$  (wide channel)

A simple relation, Figure 6, was developed by Simons and Richardson (1964) to relate stream power, median fall diameter of bed material, and form roughness. This relation gives an indication of the form of bed roughness one can anticipate if the depth, slope, velocity, and fall diameter of bed material are known. Both flume and river data were utilized for preparing this diagram.

This figure shows that the stream power is greatest for bedforms in the upper flow regime and is smallest for ripples. Dunes, washed-out dunes, plane bed with sediment motion occur for intermediate and gradually increasing values of stream power.

#### Why Roughness Coefficient $n$ Varies with Stage ?

For rivers Kumar at Lakshmikanda, Gumti at Comilla, and Halda at South Sunderpur, the value of roughness coefficient was greatest at low flow. The magnitude of  $n$  reduced sharply for slight increase in depth. For Kumar the roughness coefficient was

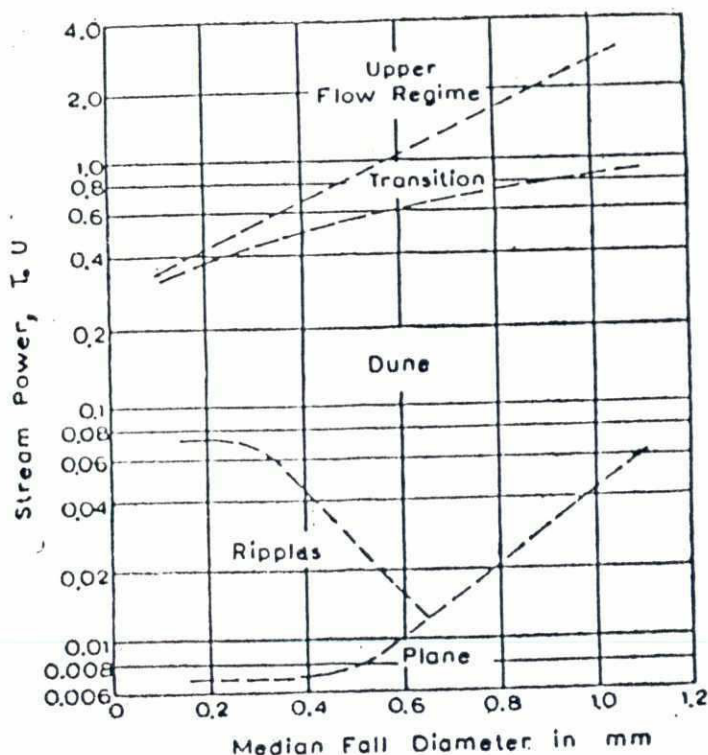


Figure-6: Relation between stream power, median fall diameter, and bed configuration (after Simons and Richardson, 1964).

about 0.16 for a depth of about 6.5 feet and its value reduced to about 0.017 for a depth of flow of about 9 feet. Again the Manning's roughness coefficient increased to about 0.078 for a depth of about 16 feet. This shows that there is significant reduction of  $n$  (about 10 times reduction) at low flow as the depth increases by about 2.5 feet. The magnitude of  $n$  increases as the depth of flow increases from intermediate to high stage. In this case the magnitude of  $n$  increased by about  $4\frac{1}{2}$  times.

A similar trend for variation of  $n$  with stage was observed for river Gumti at Comilla, and Halda at South Sunderpur.

The resistance to flow in an alluvial channel results from both grain roughness and form roughness. The grain roughness primarily depends on the bed and bank material sizes and it does not vary significantly with stage. The form roughness is a function of bed and bank irregularities significantly

larger in size than grains forming the channel perimeter..

Most probably greater resistance to flow at low flow is due to higher relative roughness. At lower stages, it is expected that the bedforms will be mostly dunes and ripples. Moreover larger dunes are formed at greater discharges and those do not undergo appreciable reduction in size as the flow decreases. As a result the relative roughness of the channel becomes very great at low flow which means a larger value for roughness coefficient  $n$ . Vegetation and aquatic growth protruding through the water surface at low flow also add to resistance to flow.

As the depth of flow increases, the vegetation goes under water and increased depth above dunes causes the relative roughness of the channel to decrease. So the roughness of the channel decreases.

The magnitude of  $n$  increases at higher stages. This could be either due to increased size of the dune or added area of the flood plain that goes under water as the water level increases. The roughness characteristics on the flood plain are complicated by the presence of vegetation, natural and artificial irregularities, lack of definite direction of flow, varying slopes and other complexities. Normally the roughness of flood plain is always greater than the roughness of the channel.

The effect of depth on bedform and hence on resistance to flow is great. Just as changes in slope can change the bed configuration when depth and size of bed material are constant, so will changes in depth change the bed configuration when slope and bed material are constant. With a constant slope and bed material, an increase in depth can change a plane bed without movement to ripples, and a ripple-bed configuration to dunes. An increase in depth, without varying slope and bed material, may cause a dune bed to change to a plane bed or antidunes, and that a decrease in depth may cause a plane bed or antidunes to change to dune-bed configuration.

Figures 1, 2, 3, and 4 show that for a particular stage, the roughness coefficient  $n$  varies over a wide range. This is probably because of varying magnitudes of stream power associated with different discharges at a given stage. When the discharge increases rapidly during flood season (rising stage) the slope of the energy line is greater because of congestion of flow and as a result the stream power



is greater. On the other hand the stream power is comparatively smaller for the same discharge during the falling stage. As a result the resistance to flow for a given discharge is greater during rising stage than for the same flow during the falling stage. With greater stream power the flow would be in the upper regime and it has already been discussed that the resistance to flow is smaller for bedforms associated with upper flow regime. Figure 7 shows such a break in a depth discharge relation caused by a change in bedform from dunes to plane bed or from plane bed to dunes.

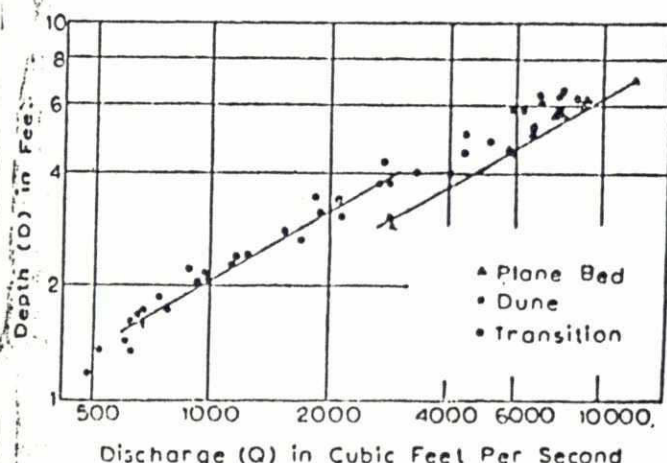


Figure-7: Relation between depth and discharge for Elkhorn river near Waterloo, Nebraska, USA.

For Sangu river at Banderban, the magnitude of  $n$  does not change much with stage. If the bedforms are mostly in the transition zone or in the upper flow regime, the resulting  $n$  values do not change much with stage. The relative roughness effects are not noticeable for these bedforms. Mean velocity in this river is much greater than other rivers and hence it can be expected that the bedforms will be those which are found in the transition zone or in the upper flow regime.

#### Conclusion :

Though the Manning  $n$  was developed empirically as a coefficient which remained constant for a given boundary condition, regardless of slope of channel,

size of channel, or depth of flow. As a matter of fact, however, each of these factors causes  $n$  to vary to some extent. In other words, the Reynolds number, the depth of flow, and the relative roughness have an influence on the magnitude of roughness coefficient  $n$ .

It has been shown in this paper that the magnitude of  $n$  for some rivers in Bangladesh varies over a wide range with stage. For river Kumar at Lakshmikanda the maximum value of  $n$  is about 10 times the minimum value of  $n$ . This shows that the idea of a single value of roughness coefficient for a channel for all occasions can be very much misleading.

Manning's  $n$  was greatest at low flow and its magnitude reduced sharply as the depth of flow increased. Again the magnitude of  $n$  increased as the depth of flow increased from intermediate to higher stages.

It has been proposed that one of the reasons for such great variation of  $n$  with stage is due to change in the bedforms caused by changing stream power, shear stress, and relative roughness associated with stage variation.

Manning's  $n$  accounts for the boundary resistance besides other losses. Boundary roughness includes the grain roughness (surface roughness) and the roughness caused by the bedforms (form roughness). Grain roughness depends on the size of bed and bank material and is more or less constant for a particular reach of a river. Form roughness depends on the type of bedform and it changes significantly as the bedform changes. It has been shown that the bedforms are dependent on shear stress and stream power (and hence on stage) which accounts for the fact that form roughness depends on stage, discharge, slope, size of bed and bank material, and the volume of sediment discharge.

#### Need for Additional Data and Research :

Informations on bedforms in rivers are needed to get better idea about the variation of  $n$  with stage and other factors.

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## Paper 3.2

BARI, M.F. and M.K. ALAM, (1979), Sediment transport in the Ganges, *Jr. Institution of Engineers, Bangladesh*, vol. 7(4):13-19.

# Sediment Transport in the Ganges

M. F. Bari\* and M. K. Alam\*\*

## Abstract :

This study is concerned with the sediment transport aspect of the Ganges. Field data collected by Bangladesh Water Development Board have been analysed and studied. Comparison of five recent recommended formulas have been done and presented. No attempt has been made to obtain a new formula but correction factors for suitable formulas have been found out. The study has revealed that the Colby relations and the Engelund—Hansen formula may be applicable with appropriate correction factors.

Attempt has also been made to separate the various types of sediment load for the river. This is, however, mostly in predictive form and gives mere an idea in absence of any information.

Rating curve has been developed and power-law relationship has been established for practical use.

Finally suggestions have been put forward to bridge the gaps of the existing data collection procedure of Bangladesh Water Development Board.

## Introduction :

Computation of sediment discharge is important in river regulation and design and operation of canal systems. Many formulas are available in literature but the computed transport rates vary drastically from theory to theory and choosing a suitable one in a specific situation is not easy. The ASCE Task Committee<sup>8</sup> studied thirteen well-known formulas and recommended three formulas. White et al.<sup>10</sup> evaluated all the commonly used theories against 1000 flume and 260 field measurements. The results as adopted is shown in Table I.

In the context of the present research and analysis it is found that the one time popular formula of Einstein lacks confidence in its application. It appears that the traditional application of Einstein formula for the rivers in Bangladesh needs serious reconsideration.

Table—I

Sediment Transport Theories : A Review  
by W.R. White, H. Milli, A.D. Crabbe

### Group A :

Ackers and White (1973)  
Engelund and Hanson (1967)  
Rottner (1959)

### Group B :

Einstein (Total Load, 1950)  
Bishop et al. (1965)  
Tofaleti (1968)

### Group C :

Bagnold (Total Load, 1966)  
Meyer—Peter and Muller (1948)

Group A: equations with highest percentage of data with mean discrepancy ratios in the range  $\frac{1}{2}$ —2 (say about 50%) and with little scatter within the sets.

Group B: equations with 35-50% of the data with mean discrepancy ratios in the range  $\frac{1}{2}$ —2.

Group C: Less generally applicable methods.

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82.

In the derivation and evaluation of sediment transport formulas, data of natural and artificial channel collected mostly in U.S.A. and U.K., are widely used. Evaluation of any of these relations has rarely been done using the data of the rivers in Bangladesh. The primary objective of the present study is to evaluate the reliability of five of these formulations: those of Colby (1964), Englund—Hansen (1967), Ackers—White (1973), Inglis—Lacey (1968) and Yang (1976) as predictors for the Ganges. The first and the second formula had been recommended by the ASCE Task Committee<sup>8</sup> while the second and the third one by White et al.<sup>10</sup>. The fourth formula had been developed based on data of canals and rivers of this sub-continent. The last formula is the most recent one and is claimed to be one of the best. A concomitant objective is to establish sediment-rating curve and formulate simplified empirical equation for practical use. A third objective is to make attempts to separate the various types of sediment load of the river.

#### Data analysis :

The observations analysed herein were taken from the Ganges at Kalikapur (Figure 1) during 1970, 1972, 1973<sup>5,6</sup>.

The discharge hydrograph and sediment concentration of the Ganges is presented in Figure 2 which illustrates that the peak sediment discharge generally occurs in advance of the peak of the water discharge. For large rivers, it is often reported that the peak sediment discharge precedes the peak water discharge. The observations by Einstein et al. (1940) for the Enoree river, by Jarocki (1963) for the Vistula and Volga, and by Nordin et al. (1963) for Rio Grande, all exhibit this trend.

The distribution of sediment concentration also varies with lateral position in a cross-section as shown in Figure 3.

Sediment discharges have been calculated by the five selected formulas. The unmeasured sediment discharge passing through the unsampled zone of the suspended sediment sampler has been estimated by Colby (1957) method.

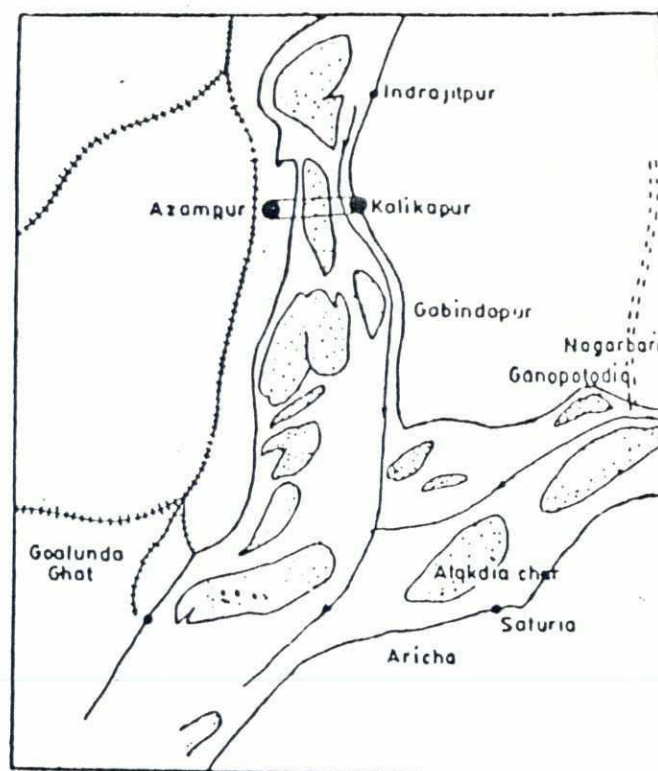


Fig. 1 Site Plan Showing Gauging Stations at Kalikapur

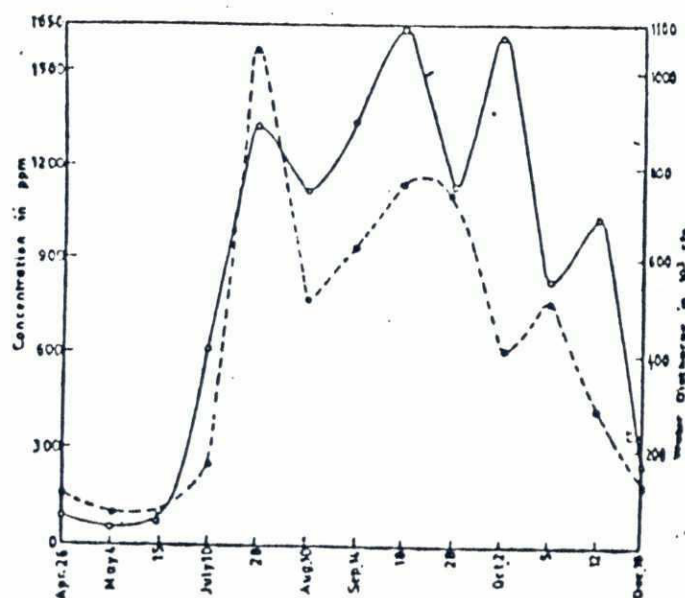


Fig. 2 Water Discharge and Sediment Concentration of the Ganges at Kalikapur, 1972

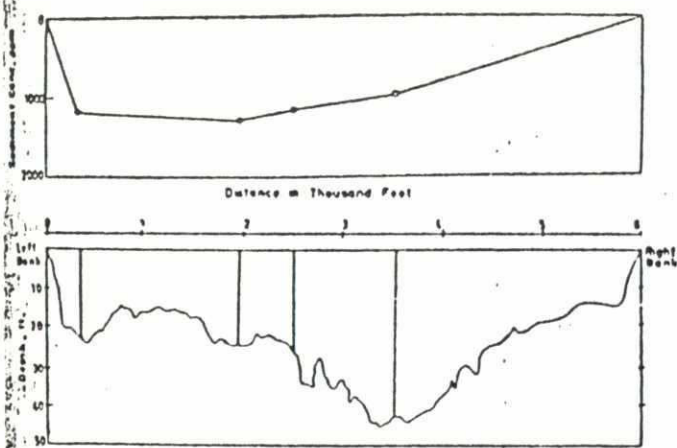


Fig. 3 Lateral variation of Sediment Concentration in the Ganges at Kalikapur on 10.3.77

### Performance of the formulas :

The comparison of the observed and predicted transport rates by Colby method is presented in Fig. 4. The mean line and the line of perfect agreement have been shown. The scatter of points about the mean line may be attributed to the presence of wash load in the measured sediment load. About one-

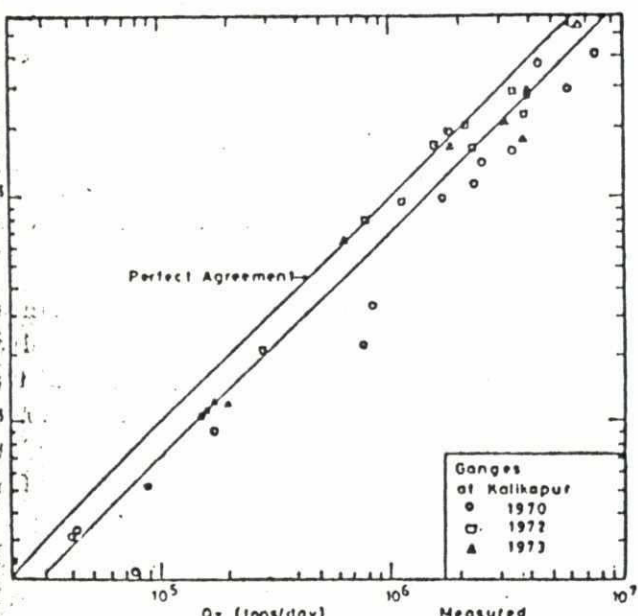


Figure 4 Plot of Measured Vs. Calculated Sediment Discharge by Colby Relations (1957, 1964)

fourth of the calculated values of sediment discharge deviate only 20 per cent from the observed amounts and about three-fourths fall within 0.5 and 2.0 times the observed amounts. Of the five formulas examined herein the Colby method predicts transport rates which are comparatively close to the measured quantities. This appears to be reasonable because Colby relations are primarily based on observations taken from natural streams. The ratio of the calculated and the observed sediment discharge varies between 0.26 and 1.12 with a mean value of 0.7 implying that appropriate correction factor is needed in order to make the formula valid. The correction factor (multiplier) has been evaluated to be 1.43 for the Ganges.

Though Engelund-Hansen formula underestimates the transport rates, the results are consistently good as shown in Fig. 5. The scatter in the figure may again be due to the appreciable quantity of wash load present in the measured load. The range of the maximum and the minimum transport rates is the east for this formula and tentatively suggests a consistent trend of errors. The ratio of sediment discharge (calculated/observed) varies from 0.10 to 0.50 with a mean value of 0.17. Thus the equation can be conveniently used as a transport formula with a suitable correction factor (multiplier) which has been evaluated to be 5.9.

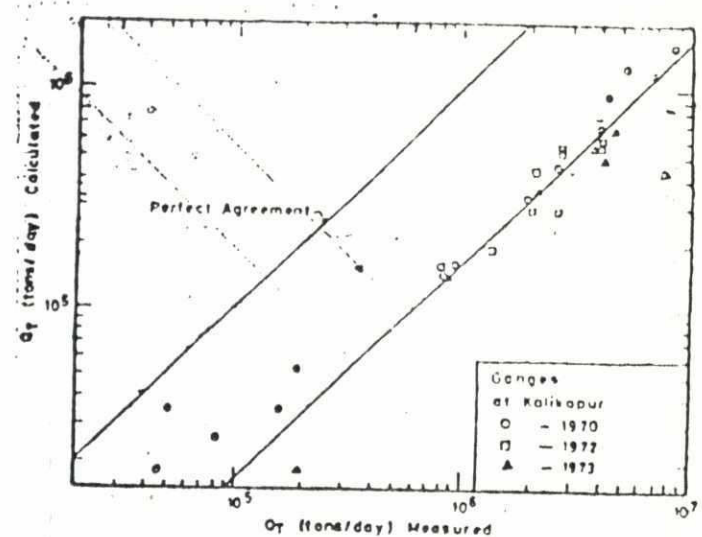


Figure 5 Plot of Measured Vs. Calculated Sediment Discharge by Engelund-Hansen Formula (1967)



The authenticity of Ackers-White formula in providing satisfactory results of sediment discharge is demonstrated in Fig. 6 and 7. As the calculated transport rates depend on the assumed significant particle size, two analyses of the present set of data, one based on  $D_{50}$  size and the other on  $D_{35}$  size of the bed material, have been done and presented. For  $D_{50}$  size the ratio (calculated/observed) varies from 0.10 to 1.80 with a mean value

of 0.25 and with  $D_{35}$  size these ratios are 0.12 and 1.84 with a mean value of 0.35. Looking to Figures 6 and 7 it is evident that there does not exist any consistent trend of variation of predicted transport rates. At times the formula underestimates and abruptly overestimates transport rates and thus the fluctuation of the error function is random and the formula lacks confidence.

The performance of Inglis-Lacey formula is disappointing as shown in Fig. 8. The predicted sediment discharge fluctuate from about 16 times lower to 3.4 times higher than the measured quantities. In case of Inglis-Lacey formula the inconsistency is the greatest and the confidence limit is minimum.

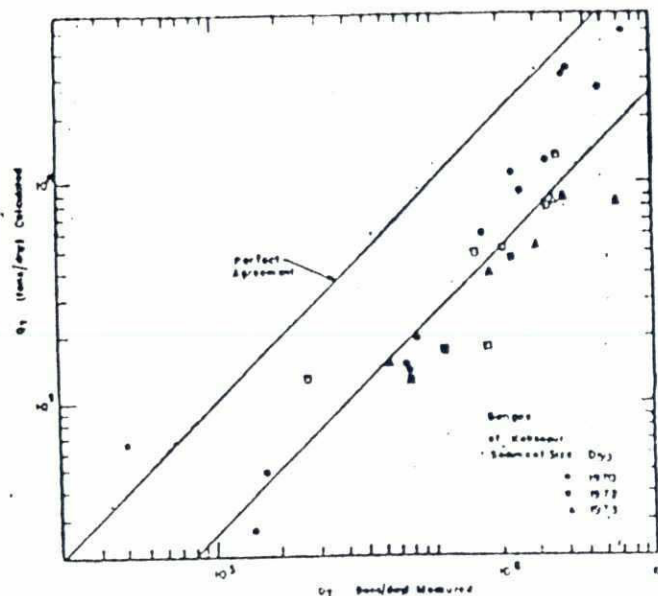


Fig. 6. Plot of Measured vs. Calculated Sediment Discharge by Ackers-White Formula (1933)

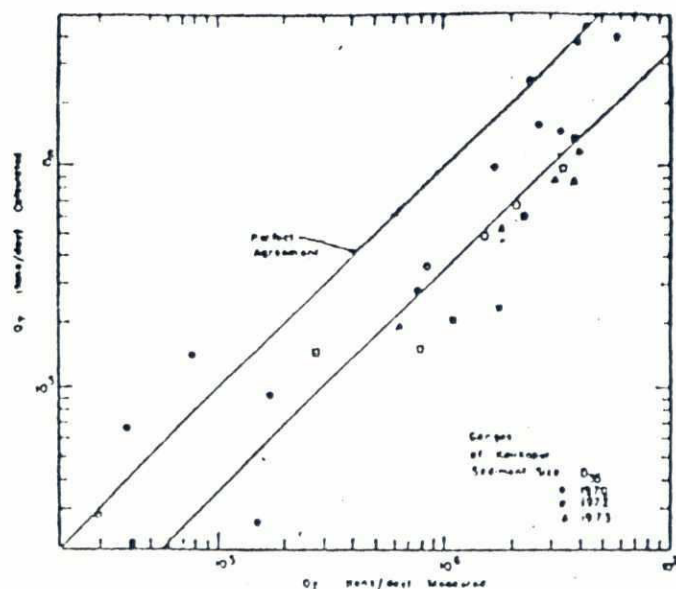


Fig. 7. Plot of Measured vs. Calculated Sediment Discharge by Ackers-White Formula (1973)

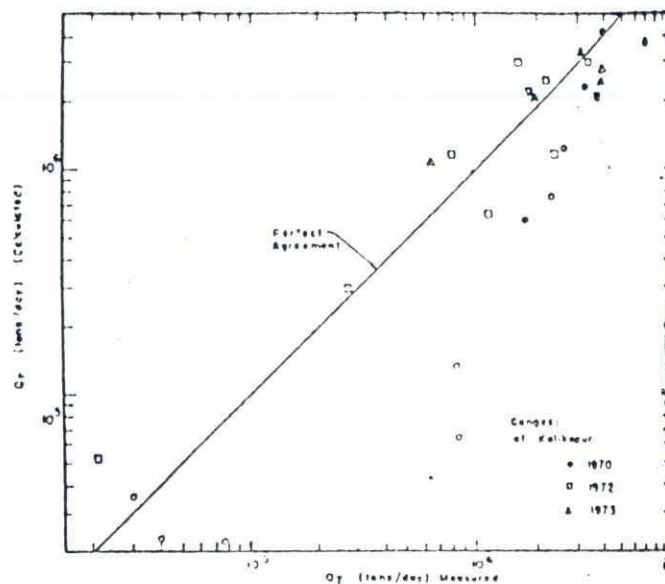


Fig. 8. Plot of Measured vs. Calculated Sediment Discharge by Inglis-Lacey Formula (1948)

Figure 9 shows that Yang formula underestimates transport rates over the whole range of data. The underestimation goes even to the extent of 25 times. In terms of mean value the predicted transport rate is 22 times lower than the observed value. However, in terms of stability criteria the performance of Yang formula is better than Ackers-White formula and yields a consistent trend of errors.

Overall, therefore, the Colby and the Engelund-Hansen formula give consistently better agreement with the field data than the others

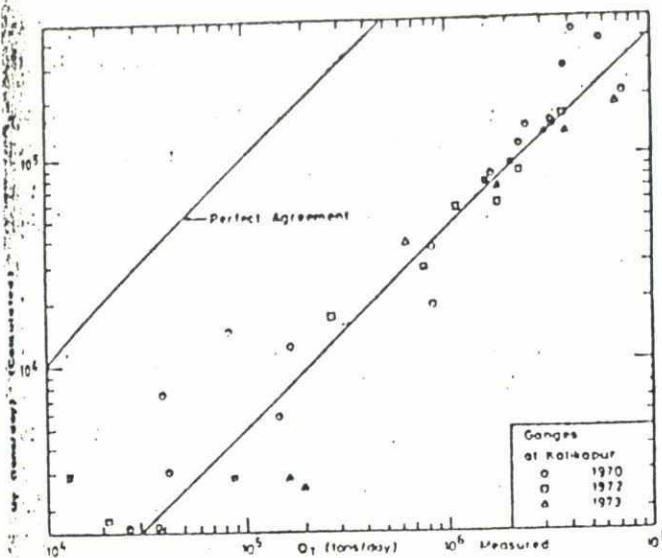


Figure 9 Plot of Measured vs. Calculated Sediment Discharge by Colby Formula (1976)

#### Sediment rating curve :

Figure 10 depicts the relationship between the water discharge and the measured suspended sediment discharge. The data have been fitted roughly by power relations of the form  $Q_s = K Q^n$  where  $Q_s$  = measured suspended sediment discharge, tons/day,  $Q$  = water discharge, cfs. The exponent  $n$  and the coefficient  $K$  have been evaluated to be 2.18 and  $4.33 \times 10^{-7}$  respectively.

#### Evaluation of wash load :

It is believed that mechanical analysis of suspended load samples could be refined to distinguish between wash load and suspended bed material load. In the present study precise determination of wash load has not been possible as size distributions of suspended load samples were lacking. However, a rough estimation of wash load is made by subtracting the calculated values of suspended load by Colby relations from the measured suspended load.

Wash load is usually caused by land erosion and not by channel erosion; we may thus expect a certain relationship with stream flow. Colby (1963) plotted wash load against water discharge for Powder River which gave an indication that the wash load was small if the stream flow was low, and the average concentration was much lower at low flows than at high ones.

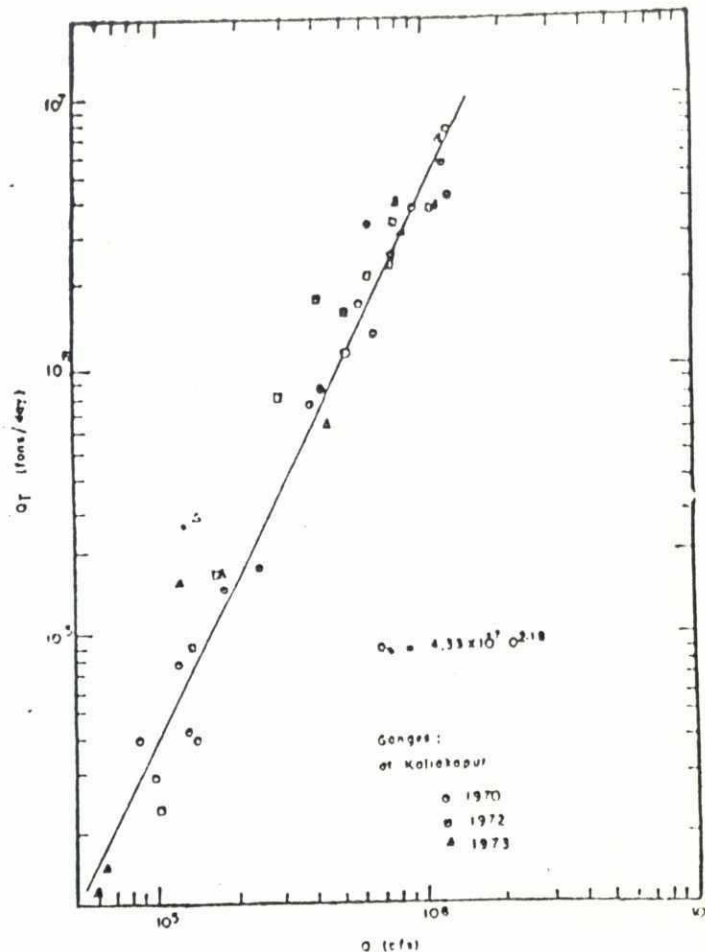


Fig. 10 Relationship Between  $Q$  and  $Q_s$

Fig. 11 shows the plot of wash load against water discharge. The following equation has been obtained correlating the wash load and water discharge.

$$Q_w = 9.61 \times 10^{-7} Q^{2.06}$$

where  $Q_w$  = wash load discharge, tons/day.

#### Conclusion :

1. Transport rates predicted by Colby relations are in closer agreement with the measured values while Engelund-Hansen formula gives consistent results in terms of error fluctuation. The correction factors (multiplier) are 1.43 and 5.9 for Colby and Engelund-Hansen formula respectively.

2. Unmeasured sediment load and wash load are, on the average 18 per cent and 33 per cent respectively of the measured suspended load.



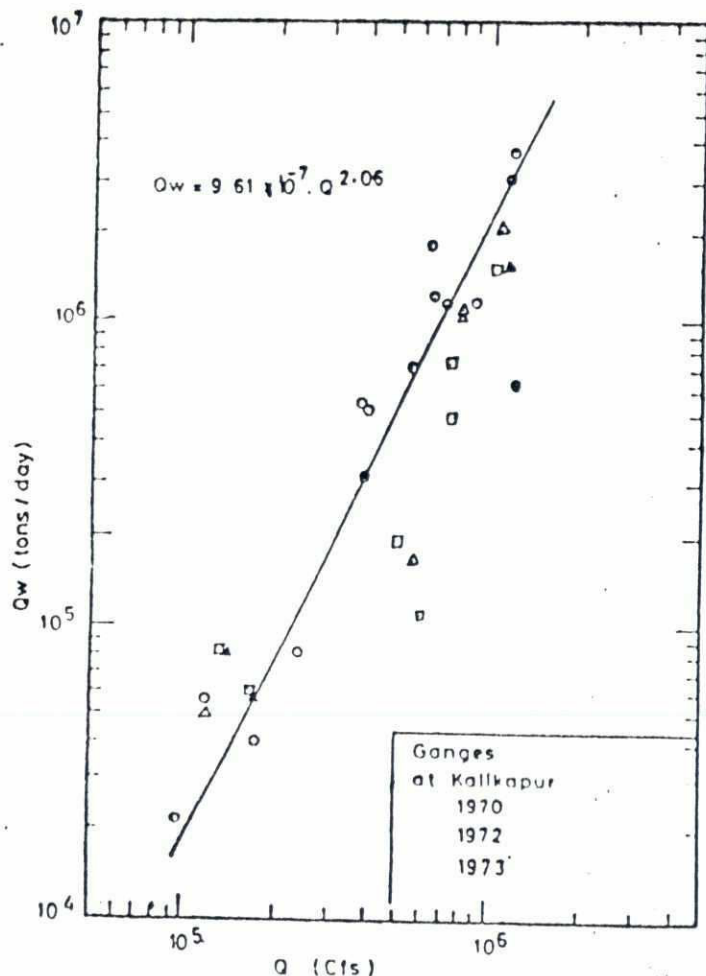


Figure 11 Relationship between Q and Qw

3. Rating curve developed herein is similar to those obtained for Missouri river by Straub (1936) and for Colorado and Niobrara river by the ASCE Task Committee (1971). The exponent is in reasonable agreement. There is variation in the value of the coefficient only.

4. The nationwide data collection procedure by Bangladesh Water Development Board needs modification. Valuable data have been collected but many of them are not in usable form for rational analysis due to lack of one or more simple data.

5. For studying sediment transport characteristics of a river, data from a number of sections should be available. There should be at least one more data collec-

tion station on the Ganges upstream of Kalikapur and another below the confluence of the Ganges and the Jamuna.

6. Measurement of bed load should be undertaken. However, very good bed load sampling technique still does not exist. Possibility of determination of bed load by modern method like the tracer technique may be explored.

#### Acknowledgements :

The authors acknowledge the help and cooperation of Mr. Serajuddin, Director, Hydraulic Research Laboratory (HRL), Bangladesh Water Development Board (BWDB), Mr. Zahurul Islam, Deputy Director, HRL, BWDB, and Mr. Shahidullah, Deputy Director, River Morphology, Research and Training, BWDB during data collection for this study.

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### Paper 3.3

RAHMAN, K.S. AND M.K. ALAM, (1980), Instability of River Padma from Goalundo to Chandpur, *Jr. Institution of Engineers, Bangladesh*, vol. 8(1):17-21.

## Instability of River Padma from Goalundo to Chandpur

Ezi Sayedur Rahman\* and Md. Khorshed Alam\*\*

### Abstract :

The probable behaviour of the river reach of the Padma for a length of 65 miles down the confluence of the Ganges-Brahmaputra at Goalundo to the upstream of the confluence of the Padma-Mehna near Chandpur is studied. Field data collected by Bangladesh Water Development Board have been studied and analysed. It has been observed that the sinuosity of the reach is gradually decreasing from year to year, the reach concerned thus has a tendency of approaching towards braiding characteristics. The bed slopes are getting steeper, the thalweg meander shapes,  $ML/MB$ , are increasing and the thalweg meander sizes,  $MB/W$ , are decreasing. The frequent wandering of the thalweg from one position to another is due to the great magnitude of discharge during floods and movement of sand bars and mid-channel islands.

The shifting of the bank line is erratic and due to aggradation of the channel bed and the movement of the thalweg. The left bank is predominantly under erosional activities while the right bank experiences both erosion and deposition. The maximum rate of erosion is about 1100 feet per year and the maximum rate of deposition is about 850 feet per year both occurring in the right bank.

### Introduction :

Padma is the combined flow of the Ganges and Brahmaputra down from Goalundo to the upstream of Chandpur, Figure 1. Since 1956 Bangladesh Water Development Board has been engaged for its hydrographic survey and data collection programme. Many valuable informations have been collected but unfortunately complete set of data were available only for the year 1967, 1969 and 1977 for the present study. The data

for 1956 has been taken as the base data for the analysis. Instability in a river can be rectified by adjusting the stream width, slope, meander shape and size. The phenomena, are of mutual interadjustment and needs careful quantitative study<sup>3</sup>. Qualitative aspects of the river Padma has been much studied and discussed<sup>4</sup>. But to the author's knowledge quantitative evaluation of the river geometric parameters have not yet been done. This paper is thus an attempt in that direction.

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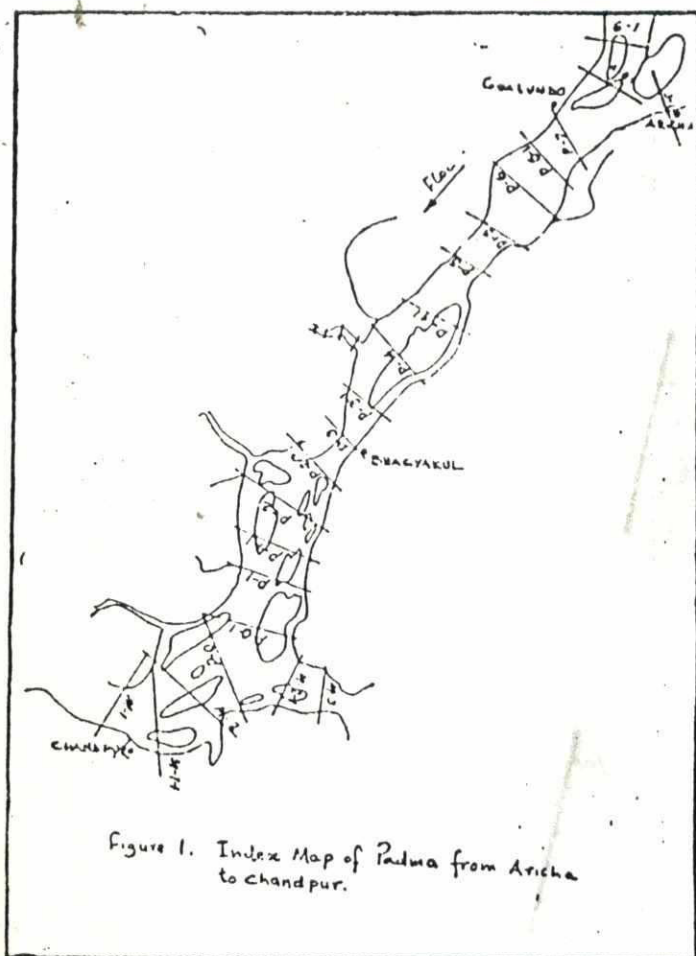


Figure 1. Index Map of Padma from Aricha to Chandpur.

Shifting of deep water channel, lateral movements of the banks and sinuosity (tortuosity) ratio ;

Looking to Figure 2 it is evident that deep water channel is continuously shifting towards left at sections P-6, P-7, P-4-1 and P-1. The movement is maximum at section P-7 and P-1 with average values being 2700 feet and 2250 feet per year respectively. Comparing bank lines of 1977 with those of 1956, it is found that the movement varies from section to section. The river is shifting towards left at sections P-7, P-5, P-4-1, P-2-1, P-2 and P-1 while it is migrating towards right at sections P-6-1, P-5-1 and P-0 as shown in Figure 2. The overall situation shows that the river is oscillating left and right within a band of nearly 80000 feet. The right bank has the maximum rate of erosion of 1090 feet per year at section P-6-1 and maximum rate of deposition of 853 feet per year at section P-2.

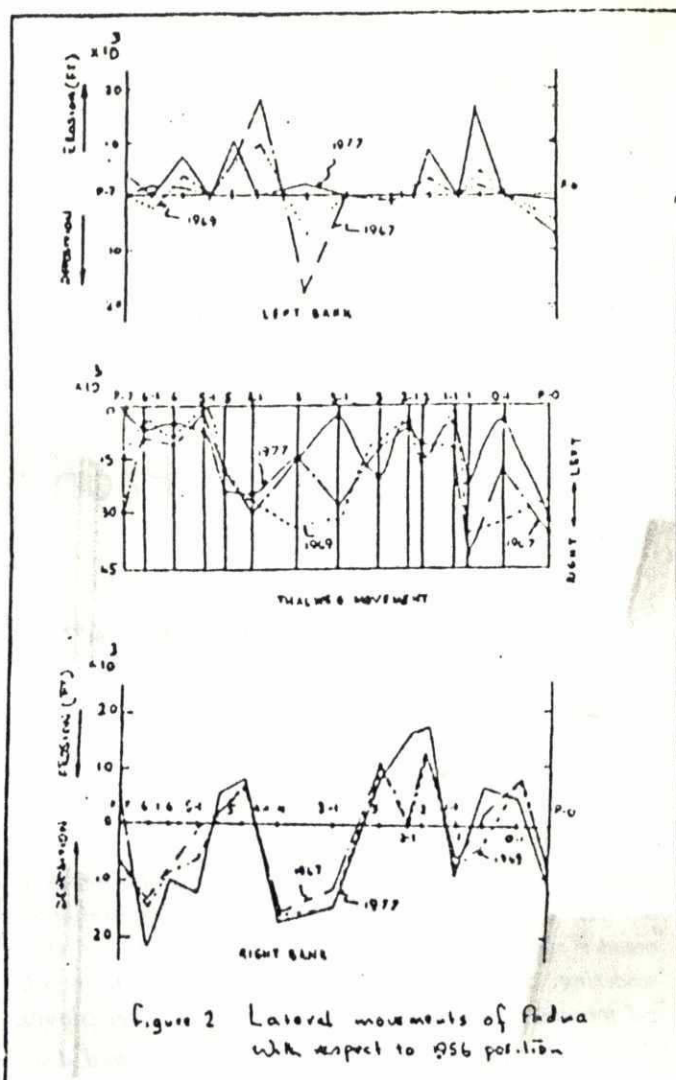


Figure 2 Lateral movements of Padma with respect to 1956 position

On the left bank, these values are 732 feet per year and 21 feet per year respectively at section P-1 and P-0. In most of the sections while one bank is eroding, the other is depositing. But at sections P-6, P-4 and P-1-1, the channel is getting wider, signifying erosion on both the banks.

Various factors are responsible for the lateral migration of the river. The major factor is the aggradation of the channel bed due to heavy sediment loads from the upper catchment area. Sudden flood events, lack of bank stability and improper use of flood plain may also be responsible for the shifting of the course.

Figure 3 is a plot of thalweg sinuosities against time in normal scale. A decreasing trend of the sinuosity

with time is noticed indicating the development of braiding tendency of the channel. The equation of the fit line can be written as

$$T_s = 1.3 - 0.018 t$$

where

$T_s$  = thalweg sinuosity

$t$  = time in years with reference to 1967.

A close inspection of Table 1 also reveals the fact of the braiding development. There is no systematic relation

Table 1. River Padma

| Year | Sections     | Bed slope<br>( $10^{-3}$ ) | Thalweg sinuosity<br>(LR/LV) |
|------|--------------|----------------------------|------------------------------|
| 1977 | P-7 to P-6   | .29                        | 1.05                         |
|      | P-6 to P-4-1 | .13                        | 1.15                         |
|      | P-3 to P-2-1 | .14                        | 1.14                         |
|      | P-2 to P-1   | .40                        | 1.05                         |
|      | P-1 to P-0   | .2                         | 1.25                         |
| 1969 | P-5 to P-4-1 | 1.7                        | 1.15                         |
|      | P-4-1 to P-3 | .31                        | 1.31                         |
|      | P-3-1 to P-2 | .50                        | 1.07                         |
|      | P-2 to P-1   | .10                        | 1.33                         |
|      | P-7 to P-5   | .36                        | 1.24                         |
| 1967 | P-4 to P-3   | 1.1                        | 1.25                         |
|      | P-3 to P-2   | .2                         | 1.08                         |
|      | P-2 to P-0-1 | .012                       | 1.48                         |

between sinuosity and bed slope but on the general basis bed slope is getting steeper and the river has a tendency to follow more or less shorter course forming multiple channels. To understand the phenomenon more clearly quantitative evaluation of sediment flow is very important in this reach but unfortunately no sediment measuring station exists.

The relationship between thalweg sinuosity and channel shape ( $W/D$ ) is shown in Figure 4. It is seen from the curves that with the lowering of thalweg sinuosity, the value of  $W/D$  (the ratio of surface width to average depth) is increasing i.e., the channels are becoming wider and shallower.

In figure 5 it is seen that an increase of thalweg meander shape,  $ML/MB$  is associated with decrease of thalweg sinuosity. In other words, the thalweg

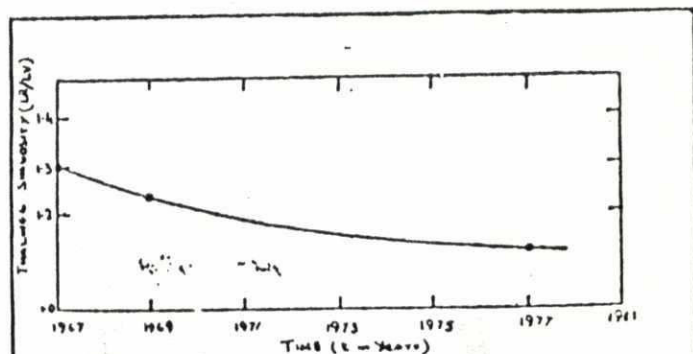


Figure 3. Plot of Thalweg Sinuosity Versus time

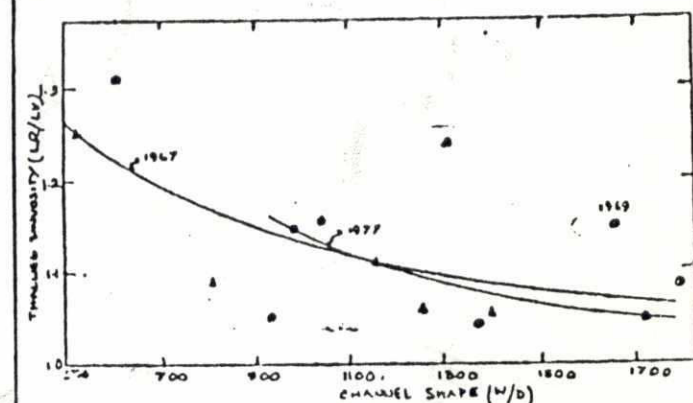


Figure 4. Plot of Thalweg Sinuosity Versus Channel Shape.

of Padma is getting flatter from year to year, the meander length,  $ML$  is increasing, and the width of meander belt,  $MB$  is decreasing. The observation seems to be similar to that of Chittale<sup>3</sup>.

Plot of thalweg sinuosity,  $LR/LV$  against thalweg meander size  $MB/W$  is shown in Figure 6. On general basis the figure represents that decrease of thalweg sinuosity causes decrease of thalweg size  $MB/W$ . From the figure it can also be said that thalweg meander size may remain same from year to year but this does not mean that sinuosity will also have a constant value, rather the value is on decreasing trend. The rate of decrease of thalweg sinuosity is higher between 1967 and 1969 while it is much lower during 1969-1977.

Table 2 verifies the American observation<sup>2</sup> that meander length varies directly with the width of the channel. Small increase in the average



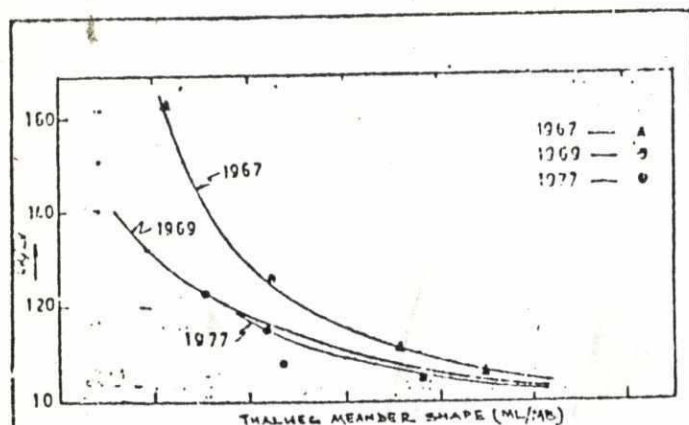


Figure 5. Plot of Thalweg Sinuosity Versus Thalweg Meander Shape.

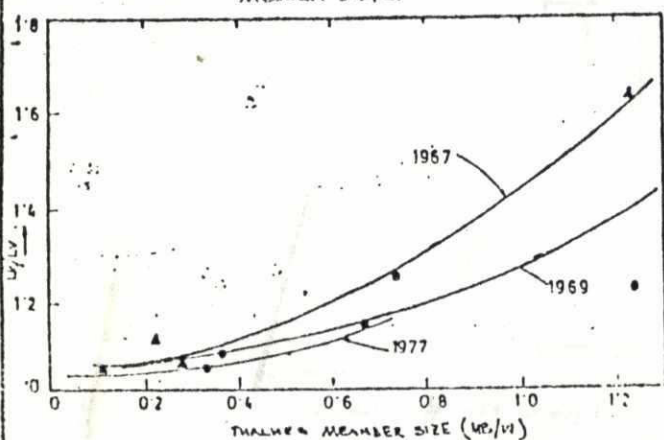


Figure 6. Plot of Thalweg Sinuosity Versus Thalweg Meander Size.

Table 2. River Padma

| Year | Sections       | Width (feet) | Meander length ML (feet) |
|------|----------------|--------------|--------------------------|
| 1967 | P-6 to P-4-1   | 6054         | 54482                    |
|      | P-3 to P-1-1   | 5922         | 42112                    |
|      | P-2 to P-0     | 77644        | 77644                    |
| 1977 | P-7 to P-6     | 6975         | 47508                    |
|      | P-6-1 to P-4-1 | 16845        | 73696                    |
|      | P-2 to P-1     | 2630         | 35786                    |
| 1969 | P-5-1 to P-3-1 | 28162        | 85540                    |
|      | P-3-1 to P-2-1 | 11186        | 53430                    |
|      | P-2 to P-0     | 18424        | 84224                    |

width of the channel causes a very rapid increase in meander length. However, the rate of increase of thalweg meander length is different for different years. In 1967 the rate of increase in thalweg meander length is about 6.0 times to that of the average width of the channel. For 1969 the river does not show any systematic trend. Data of this period may be considered of more or less to that of transition type. As the sinuosity of the thalweg goes on reducing, the rate of increase of meander length is also increasing from year to year as shown in the table for 1977. The rate of increase of meander length for this year is about 10 times to that of the average width and similar to Leopold regime relationship.

### Conclusions :

The following conclusions can be drawn from the present study :

1. The thalweg sinuosity of the reach is gradually decreasing from 1.30 in 1967 to 1.12 in 1977 indicating that the river has a tendency to develop braiding.
2. The shifting of banklines is erratic and it is due to aggradation of the channel bed and frequent movement of the thalweg within the river banklines. The maximum rates of erosion and deposition are 1100 and 850 feet per year respectively, both on the right bank.
3. An increase of thalweg meander shape,  $ML/W$  is associated with decrease of thalweg sinuosity. This observation seems to be similar to that of Chittale.
4. A decrease of thalweg sinuosity causes decrease of thalweg meander size,  $ML/W$  confirming Chittale's general observation on natural channels.
5. With small increase in the average width of the channel, there is a great increase of thalweg meander length,  $ML$ . The rate of increase varies from 6 to 10 times of the width.
6. A sediment measuring station is highly necessary at or near the Padma-Meghna, confluence for useful quantitative analysis of the morphological behaviour of the river.

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## Paper 3.4



GOSWAMI, D.C.(1985), Brahmaputra River, Assam, India: basin denudation and channel aggradation, *Water Resources Research*, vol. 21(7):959-978.





# Brahmaputra River, Assam, India: Physiography, Basin Denudation, and Channel Aggradation

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The Brahmaputra River in Assam, India, characterized by high seasonal variability in flow, sediment transport, and channel configuration, experienced a secular period of aggradation from 1971 to 1979. The suspended load budget indicates an overall aggradation of the 607-km Assam reach of the Brahmaputra by about 16 cm during that period, with about 70% of the suspended sediment inflow into the reach being retained in the channel. Expressed as a percentage of the change in storage for the different reaches, computed errors due to sampling variability in sediment discharge generally lie within about 5-15% and do not appear to be large enough to affect the conclusions drawn from the suspended load budget. For a 145-km reach of the Brahmaputra, an alternative method based on measurement of channel cross sections suggests 21 cm of aggradation, somewhat more than estimated by the suspended load budget. Based on the suspended load carried by trans-Himalayan rivers, the present rate of denudation of the eastern Himalayas is estimated to be 73-157 cm/10<sup>3</sup> years. The average rate for the last 2-3 million years, estimated from the volume of alluvial fill in the Brahmaputra valley in Assam, the sediment yield of the Himalayan rivers, and assuming a total yield to deposition ratio of 1.4 (present study), is 3 cm/10<sup>3</sup> years. The current high rate of denudation of the Himalayas may be attributed mainly to the rapid uplift of the mountain system, recent earthquake activity, and high susceptibility of geologic formations to erosion by running water coupled with the effectiveness of the monsoon rainfall regime.

## INTRODUCTION

The relation of the yield of sediment from a drainage basin to the behavior of the river channel itself is a fundamental area of inquiry in fluvial studies. Recent studies have examined storage and removal of sediments from watersheds [Leopold and Miller, 1954; Wolman, 1967; Dietrich and Dunne, 1978; Kelsey, 1980], but only a few have attempted to estimate the change in sediment storage for an individual reach of channel using the concept of continuity [Colby, 1964; Hansen, 1970; Megahan and Nowhin, 1976]. With the exception of studies related to the impact of large dams, most analyses of storage of sediment in river channels have been restricted to relatively small watersheds in the temperate regions of the western world.

This study attempts to present an integrated account of basin denudation, sediment transport and channel bed changes in the Brahmaputra River in Assam (India), and to evaluate quantitatively the rates of basin denudation and channel aggradation. The Brahmaputra, one of the major river systems of the world, is characterized by very high rates of basin erosion, sediment yield, and channel change. The only major published research to date on the Brahmaputra was done by Coleman [1969] in the lower Bangladesh portion of the river. This study deals with the river in Assam downstream from the crest of the Himalayas and above the Bangladesh border (Figure 1).

Based on available suspended sediment records, a sediment balance equation is used to estimate rates of channel bottom elevation change in the river. These results are compared with those derived from the end area method based on measured channel cross sections. The present rate of erosion of the Hi-

malayan catchment is estimated from the sediment yield in rivers, alluvial fill of the Brahmaputra valley in Assam, and the ratio of yield to deposition obtained from the sediment balance model. Because the unique combination of tectonics, geology, and climate of the basin appears to be directly related to high erosion rates and channel change in the Brahmaputra, a brief description of the geologic setting and relevant features of the channel environment of the river is given here.

## STUDY AREA

This study focuses on the Brahmaputra river system in Assam, India, with limited reference to the adjoining highlands of conterminous India and those of China and Bhutan. The catchment of the Brahmaputra occupies an area of 580,000 km<sup>2</sup>. Surrounded by an almost continuous chain of high hills and plateaus on the north, east, and south, the Brahmaputra valley in Assam represents a tectono-sedimentary province 720 km long and 80-90 km wide, with elevations ranging from 120 m at Kobo in the extreme east through 50.5 m at Gauhati to 28.45 m at Dhubri in the extreme west (Figure 1). The channel of the river itself, with an average width of 8 km, occupies about one tenth of the valley. With over 40% of its area under cultivation, the Brahmaputra valley in Assam is the home of more than 15 million people.

## Geology and Tectonics

The Himalayan watershed of the Brahmaputra comprises four topographic units that rise progressively to the north (Figure 2). The lowermost ranges called sub-Himalayas (average elevation 1000 m) consist mainly of Tertiary sandstones and are conspicuous by the presence of many raised, relatively young terraces [Gansser, 1964]. The Middle Himalayas (average elevation 4000 m) are underlain by lower Gondwana (Palaeozoic) deposits comprising shales, slates, and phyllites overlain by a thick horizon of basaltic rocks. The Greater Himalayas (average elevation 6000 m) consist primarily of granites and gneisses. Further to the north the Trans-Himalayas of Tibet (average elevation 4500 m) are

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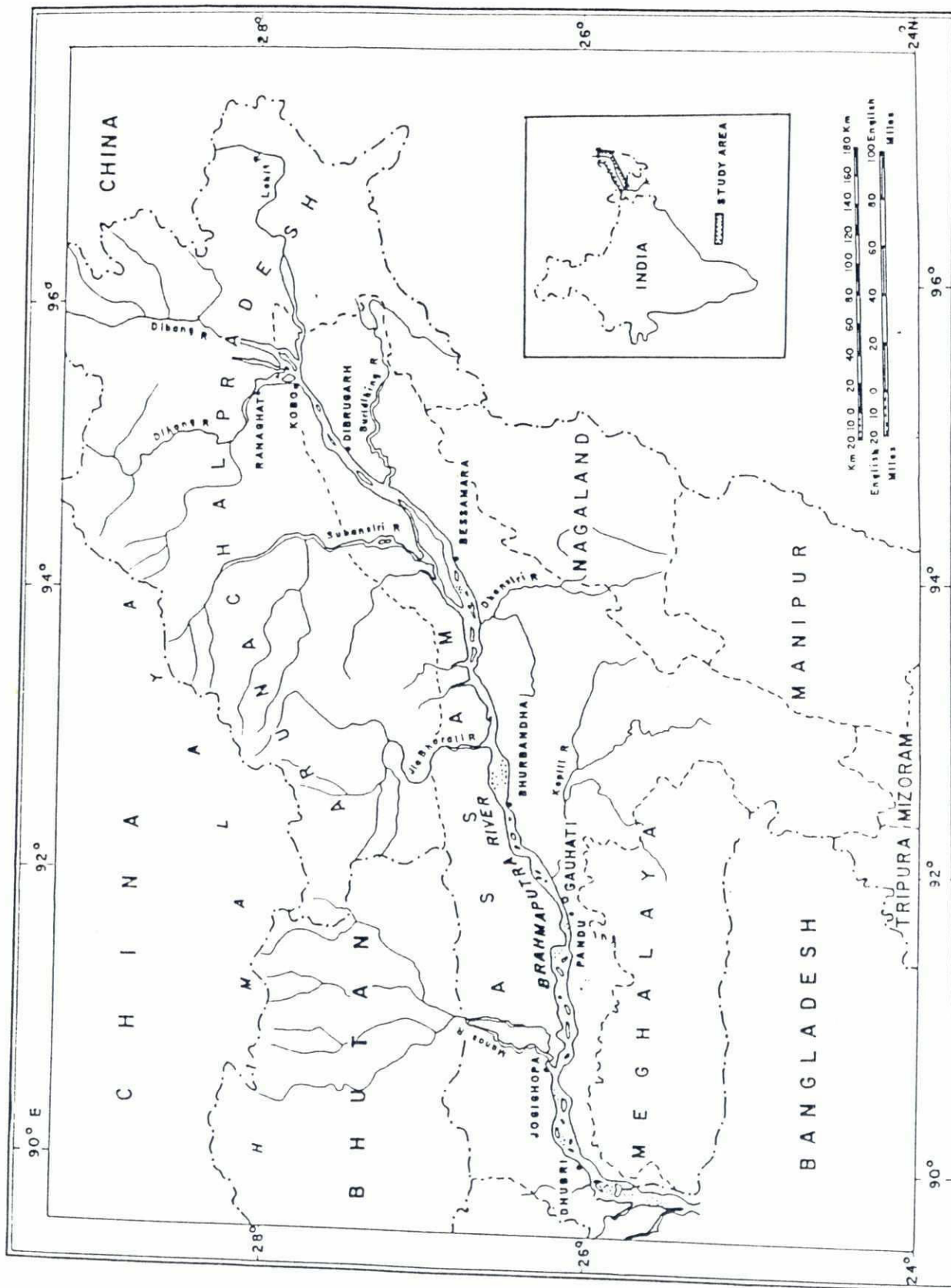


Fig. 1. Brahmaputra River System, Assam, India.

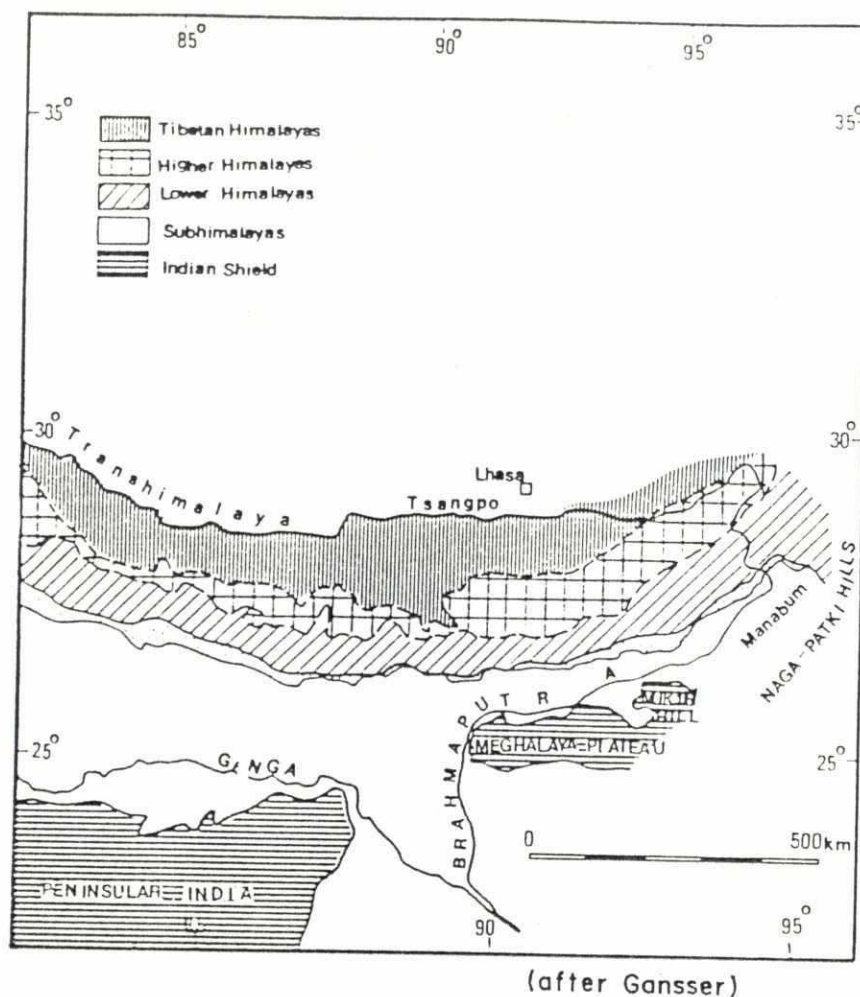


Fig. 2. Geology of the Eastern Himalayas and the Brahmaputra Basin.

made up of sedimentary formations of Palaeozoic to Eocene age [Wadia, 1968].

The Patkai-Naga ranges bordering the Brahmaputra valley on the east and southeast (average elevation 1000 m) consist of Tertiary formations criss crossed by a large number of active faults. The highland to the south of the valley, comprising the Meghalaya plateau and Mikir hills (elevation 600–1800 m) made up primarily of gneisses and schists, forms a part of the stable Indian Peninsular block of Precambrian age.

The Brahmaputra valley in Assam is underlain by recent alluvium approximately 200–300 m thick, consisting of clay, silt, sand, and pebbles [Geological Survey of India, 1974]. Its present configuration evolved during 2 million years of Pleistocene and recent time [Murthy, 1968; Rao, 1979]. Tectonic origin of the Brahmaputra valley and Assam Himalayas is suggested by a number of authors [Holmes, 1964; Evans, 1964; Mittal and Srivastava, 1959; Tapponnier and Molnar, 1977].

The Brahmaputra valley and its adjoining highlands constitute an extremely unstable seismic region. The earthquakes of 1897 and 1950, each of Richter magnitude 8.7, are among the most severe in recorded history. Both caused extensive landslides and rockfalls on hillslopes, subsidence and fissuring of ground in the valley, and changes in the course and morphology of several tributary rivers [Oldham, 1898; Poddar, 1952]. Major earthquake episodes in Assam appear to be separated by quiescent periods of about 30 years [Khatti and Wyss, 1978]. As a result of the 1950 earthquake, extensive landslides

occurred on the Himalayan slopes, temporarily blocking the courses of the Subansiri, the Dibang, and the Dihang (Figure 1). Bursting of these dams after several days released an enormous amount of ponded water, producing devastating floods downstream. Poddar [1952] noted that a large amount of the sediment generated by the landslides was brought down into the rivers, raising their beds considerably. At several places channels of the Brahmaputra were choked with sediments released from fissuring and squeezing of soft levee areas.

#### *Climate, Vegetation, and Soil*

The catchment of the Brahmaputra, excluding the Tibetan portion, forms an integral part of the monsoonal regime of southeast Asia. Rainfall averages 230 cm annually, with a variability of 15–20%. There is a marked spatial variation in the distribution of precipitation over the catchment (Figure 3). The Himalayan sector receives 500 cm of rainfall per year, the lower ranges receiving more than the higher areas. Monsoonal rains from June to September account for 60–70% of the annual rainfall.

Natural vegetation in the Brahmaputra basin varies with altitude from tropical evergreen and mixed deciduous forests within the valley and foothills, to alpine meadows and steppes in the higher ranges and, in Tibet, about 20% of the Brahmaputra valley is forested.

Soils in the sub-Himalayan region developed on the Tertiary sandstones are shallow and consist primarily of sands



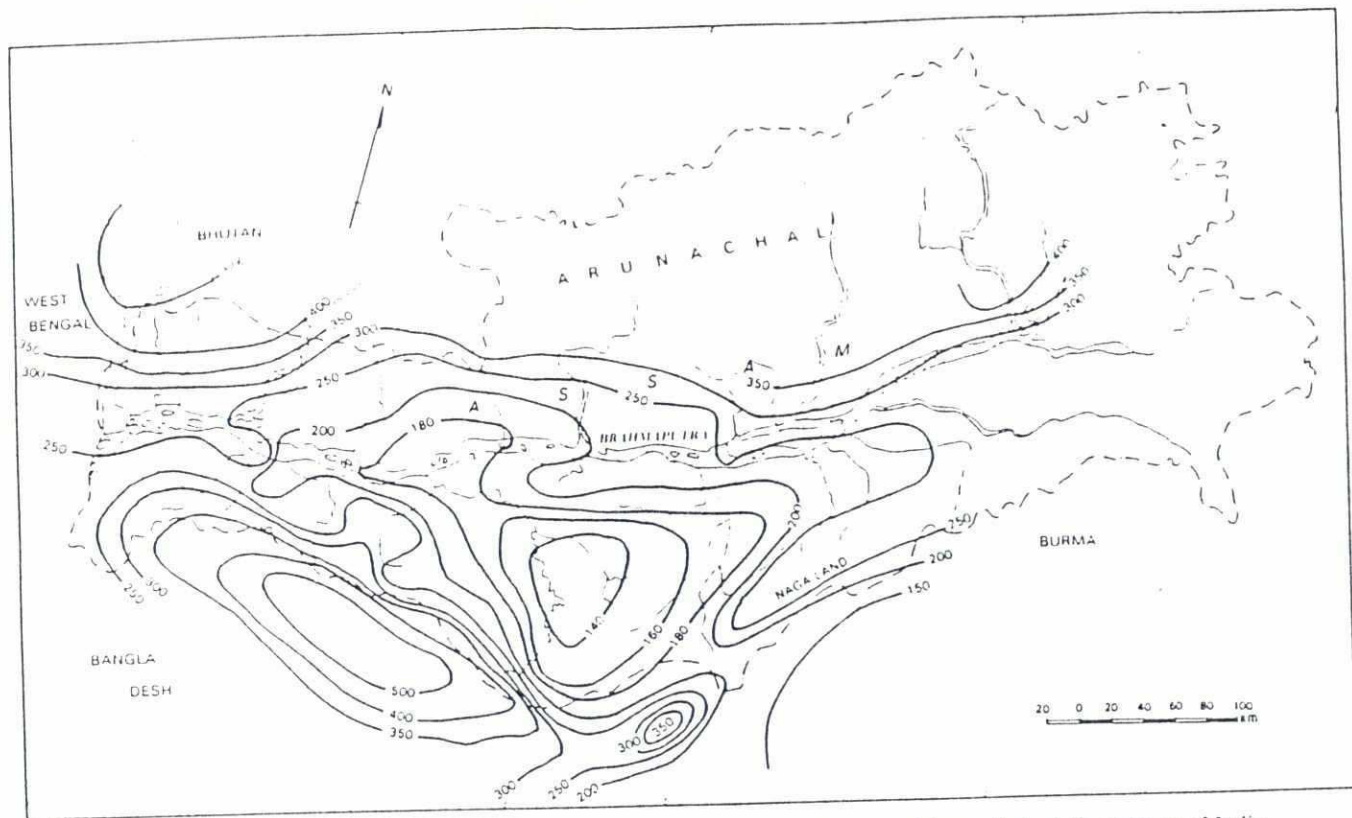


Fig. 3. Isohyetal map of Brahmaputra Valley and adjoining highlands (based on Meteorological Department of India data).

with admixtures of cobbles and boulders. Alluvial soils, formed on recent river deposits, occur in the Brahmaputra valley. A few isolated pockets of deeply weathered older alluvium occur in upland areas within the valley and in the piedmont region. Borings in the Quaternary sediments of the Brahmaputra valley extending down to more than 100 m [Geological Survey of India, 1977] show repeated sequences of clay, fine sand, coarse sand, coarse sand with cobbles, pebbles, and boulders.

#### THE RIVER

##### *Course and Morphology*

The Brahmaputra river originates in a great glacier mass in the Kailas range of the Himalayas south of Lake Gunkyud in southwest Tibet (elevation 5300 m) and flows through China, India, and Bangladesh for a total distance of 2880 km before emptying into the Bay of Bengal through a joint channel with the Ganga (Figure 1). In Tibet, where it is called Tsangpo, the

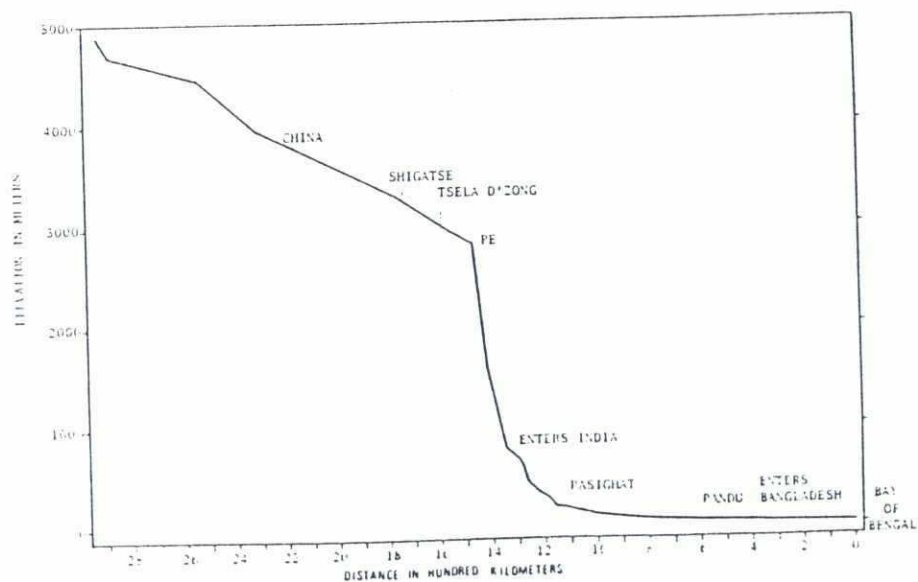


Fig. 4. Longitudinal bank profile of the Brahmaputra River.

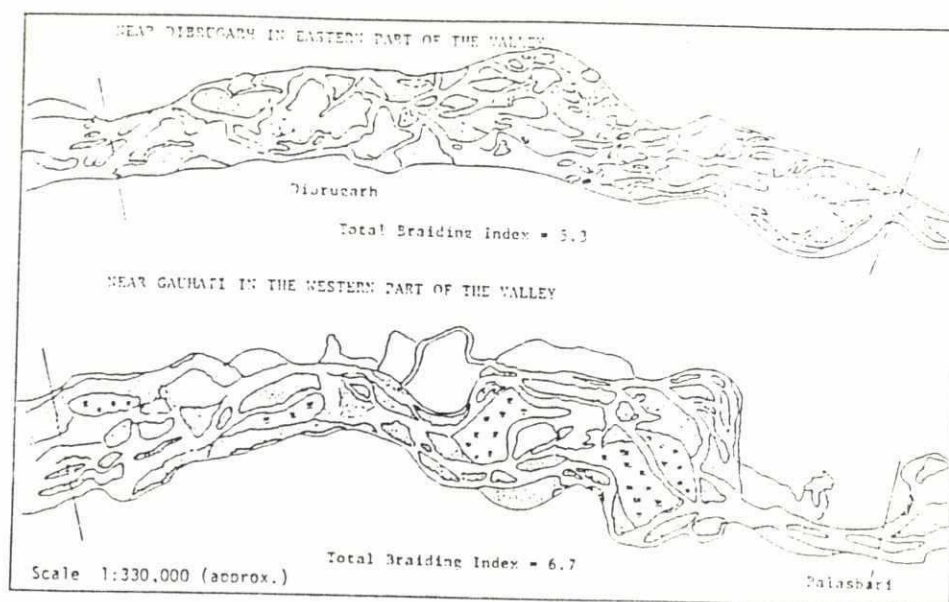


Fig. 5. Braiding in the Brahmaputra River, Assam.

Brahmaputra flows eastward for 1100 km along the bottom of a longitudinal graben parallel to and about 160 km north of the Himalayas (Figure 4). At the extreme eastern end of its course in Tibet, the Tsangpo suddenly enters a deep narrow gorge at Pe (3500 m) which skirts around the Namcha Barwa Peak (7755 m) and continues southward across the Himalayan ranges. The gradient of the river in the gorge section ranges from about 4.3 to 16.8 m/km. On entering India, the Tsangpo, now called Dihang, traverses 226 km of mountainous course before debouching onto the Assam plain near Pasighat (elevation 155 m). At the exit of the gorge the slope of the river is only 0.27 m/km. Near Kobo, 52 km south of Pasighat, two rivers (Dibang and Lohit) meet the Dihang, and the combined flow, called the Brahmaputra, moves westward through Assam

for 720 km until near Dhubri, where it swerves to the south and enters Bangladesh. The Brahmaputra has a gradient of 0.09–0.17 m/km near Dibrugarh at the head of the valley and is further reduced to about 0.1 m/km near Gauhati.

In Assam the Brahmaputra flows in a highly braided channel characterized by the presence of numerous lateral as well as midchannel bars and islands (Figure 5). Brice [1964] in his study of the Loup rivers in Nebraska defined a braiding index as the ratio of twice the sum of lengths of bars and islands in the reach to the length of the reach measured midway between the banks. For the Loup rivers a total braiding index of 1.5 was selected to differentiate braided from nonbraided reaches. Braiding indices of the two reaches of the Brahmaputra shown in Figure 5 are 5.3 and 6.7. Most of the

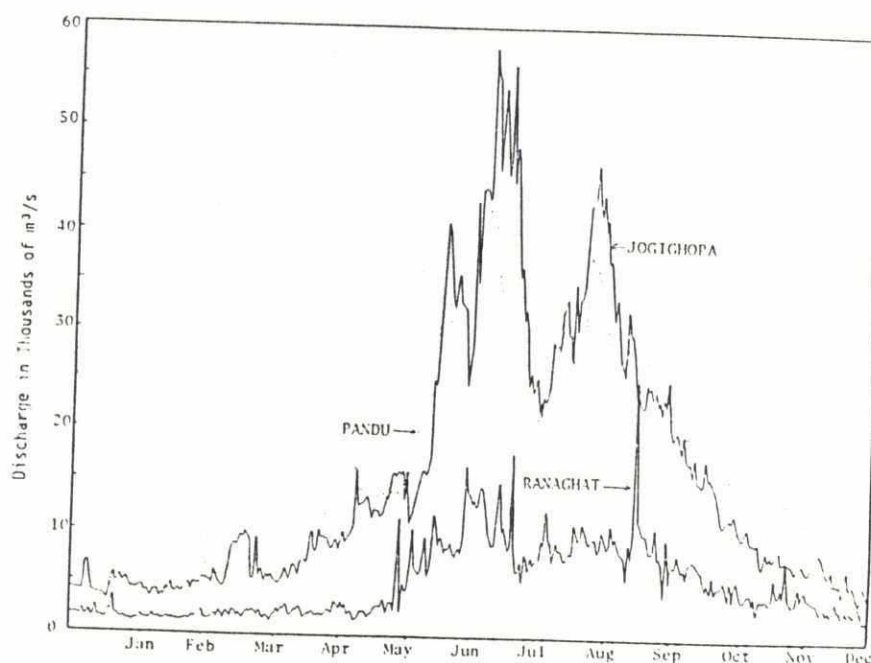


Fig. 6. Discharge hydrographs of the Brahmaputra River at Ranaghat, Pandu, and Jogighopa for 1976.



TABLE 1. Major Sediment Transporting Rivers of the World

| River                        | Drainage Area, $10^3/\text{km}^2$ | Average Annual Suspended Load |                            |
|------------------------------|-----------------------------------|-------------------------------|----------------------------|
|                              |                                   | Metric Tons, $\times 10^6$    | Metric Tons, $\text{km}^2$ |
| Ganges                       | 1480                              | 1670                          | 1128                       |
| Brahmaputra                  |                                   |                               |                            |
| Yellow                       | 770                               | 1080                          | 1403                       |
| Amazon                       | 6150                              | 900                           | 146                        |
| Yangtze                      | 1942                              | 478                           | 246                        |
| Brahmaputra at Pandu, Assam* | 500                               | 402                           | 804                        |
| Irrawady                     | 430                               | 265                           | 616                        |
| Magdalena                    | 240                               | 220                           | 916                        |
| Mississippi                  | 3270                              | 210                           | 64                         |
| Orinoco                      | 990                               | 210                           | 212                        |
| Indus                        | 970                               | 100                           | 103                        |

Data after Milliman and Meade [1983].

\*Data after this study.

channel bars in the Brahmaputra are transient in nature, being merged during summer high flows and changing drastically their geometry and location.

#### Hydrology and Sediment Transport

The Brahmaputra is the fourth largest river in the world in terms of average discharge at mouth, with a flow of  $19,830 \text{ m}^3 \text{ s}^{-1}$  (D. C. Goswami, unpublished manuscript, 1982). The hydrologic regime of the river responds to the seasonal rhythm of the monsoons and to the freeze-thaw cycle of the Himalayan snow. The rainy season (May through October) accounts for 82% of the mean annual flow at Pandu. The hydrographs of the Brahmaputra (Figure 6) show the high fluctuations in daily discharge of the river. Discharge per unit drainage area in the Brahmaputra basin rivers is among the highest of major

ivers of the world. The Jia Bharali river, a major northbank tributary, for example, carries a mean annual discharge of  $0.0858 \text{ m}^3 \text{ s}^{-1} \text{ km}^{-2}$ . At Pandu the Brahmaputra yields  $0.0306 \text{ m}^3 \text{ s}^{-1} \text{ km}^{-2}$ , and the mean annual flood discharge is  $51,156 \text{ m}^3 \text{ s}^{-1}$ . The latter has a recurrence interval of 2.1 years.

The Brahmaputra is one of the most sediment-charged large rivers of the world (Table 1). Among the largest rivers of the world, it is second only to the Yellow river in China in the amount of sediment transported per unit of drainage area. At Pandu, Assam, the river transported an average annual suspended load of  $15,200 \text{ ha m}$  (402 million metric tons) during the period 1955–1979. During the rainy season (May through October) the river transports more than 95% of the annual suspended load at Pandu at an average daily rate of  $80 \text{ ha m}$  (2.12 million metric tons). Flows that recur at least once every 2 years carry more than 95% of the load. Thus moderate events of relatively frequent occurrence are responsible for the transport of the bulk of the sediments for the period of record [Wolman and Miller, 1960]. Sediment-rating curves used to estimate total suspended load of the main river and the tributaries (Figure 7) show that suspended load increases more rapidly than water discharge, indicating increasing concentration at higher flow. (Depth-integrated suspended load samples are collected daily at the locations on the Brahmaputra and near the mouths of the tributaries referred to in the text. Tabulated mean monthly values in many instances were the only ones available to the author and are used here as described.) The slope of the rating curve for the Brahmaputra at Pandu was apparently higher during the period of high sediment loads. The rate of change of sediment load with discharge for the Manas river at Mathanguri (the coefficient 4.1 in Figure 7c) is very much higher than any the author has seen for rivers in the U.S. and elsewhere. Manas is a north-bank tributary, and Mathanguri is at the point where the river debouches onto the plains from the Himalayas having traversed a course in which the gradient is as steep as  $19 \text{ m/km}$ .

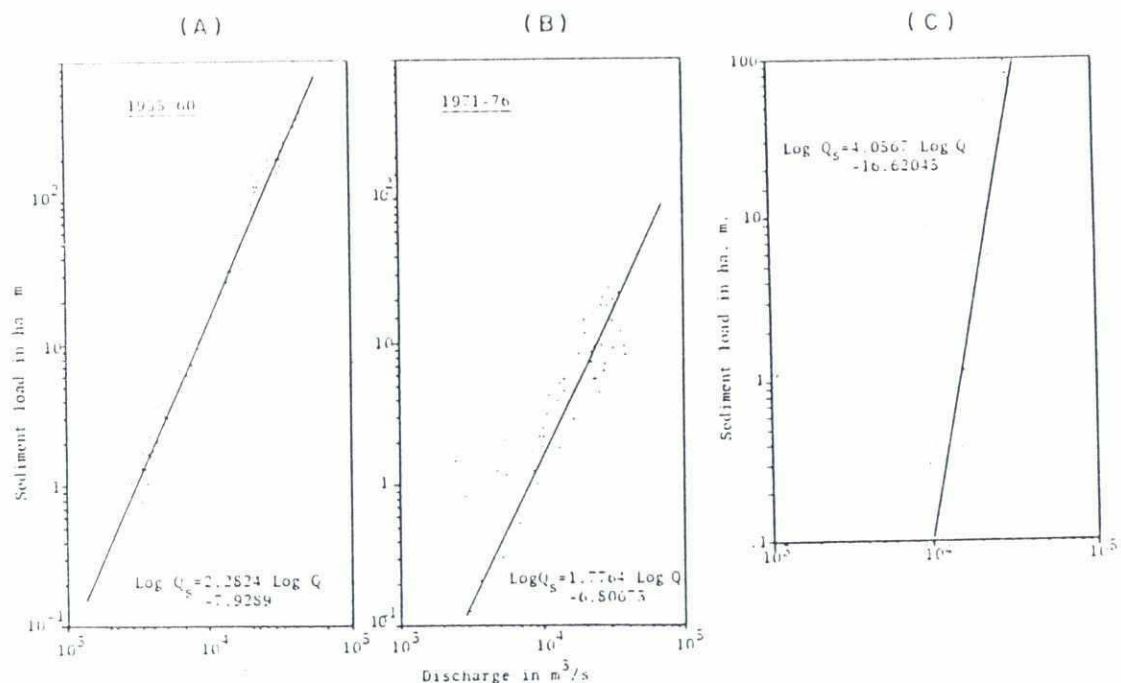


Fig. 7. Sediment rating curves for (a) Brahmaputra at Pandu (1955 and 1960), (b) Brahmaputra at Pandu (1971–1976), and (c) Manas River at Mathanguri (1972–1978).

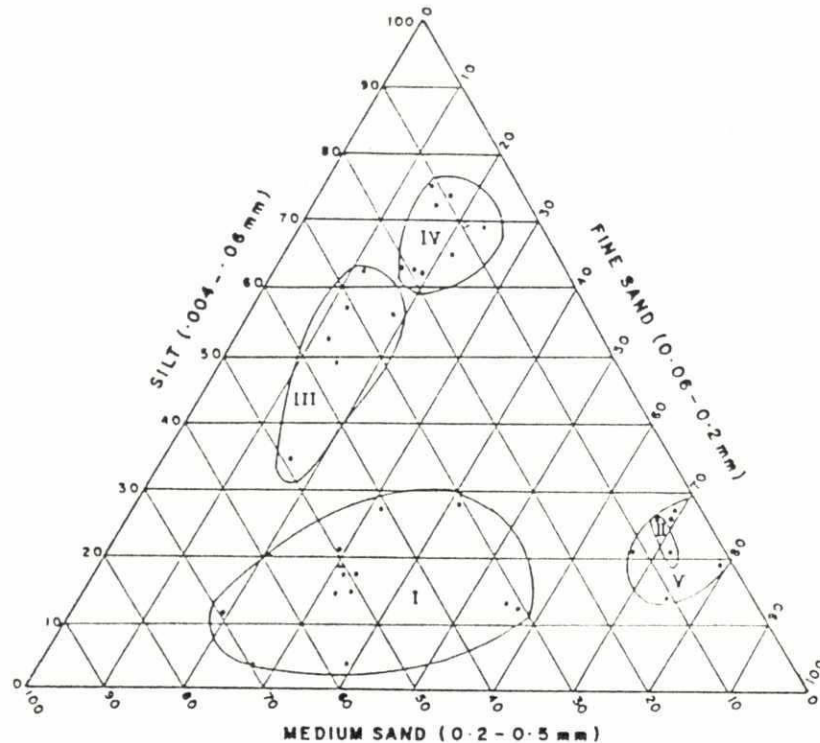


Fig. 8. Textural characteristics of suspended sediments of the Brahmaputra basin rivers. Group 1, trans- and sub-Himalayan tributaries that are mostly braided in pattern; group 2, the Pithimari and Pagladiya rivers originating in the sub-Himalayas of Bhutan; group 3, the rivers of the Manas system originating in the outer Himalayas; group 4, south bank rivers, mostly meandering, originating in the Naga and Patkai hills; group 5, south bank rivers flowing from the Meghalaya plateau.

Moreover, the Himalayan catchment in Bhutan that receives about 500 cm of rainfall in the wet outer hills seems to be highly productive of sediment due to the combined effect of damaging land use practice (mainly deforestation) and friable soils.

Predominant sizes of suspended sediments and bed materials of the Brahmaputra at Pandu are 0.06 and 0.09 mm, respectively. Textural characteristics of suspended sediments of the Brahmaputra basin rivers, illustrated in the ternary diagram (Figure 8) covering the flood season (June through August 1977), indicate five different clusters of particle sizes representing the various rivers. For example, group 1 includes the trans- and sub-Himalayan tributaries that are mostly braided and carry a relatively high proportion of the coarser fractions. During the monsoon high-flow season (June through August 1977) the ratio of medium sand (0.2–0.5 mm), fine sand (0.06–0.2 mm), and silt (0.004–0.06 mm) was on the order of 1:2:3 at Pandu, as against 2:3.5:1 at Ranaghat. The great range of concentrations for the same discharge shown in Figure 9 suggests that the peak sediment concentration in relation to discharge depends strongly on the rate of sediment supply rather than on the hydraulics of flow. Graphical presentation of daily sediment concentration against discharge at Pandu for July 12–31, 1977, and at Ranaghat for July 1–14, 1977, (Figure 10) shows hysteresis in the relationship between these two parameters [Guy, 1964; Walling, 1974]. At both places, concentration is generally higher on the rising than on the falling stage of the hydrograph, although considerable variations exist. Bed materials in the Brahmaputra river are mostly in the fine sand-silt range. The clay fraction is less than

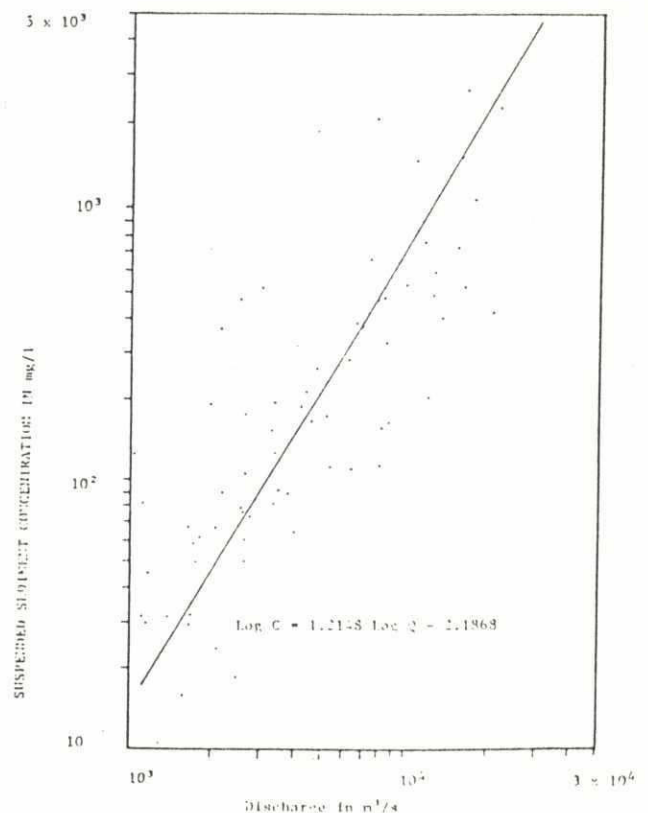


Fig. 9. Concentration in relation to discharge, Brahmaputra river at Ranaghat.



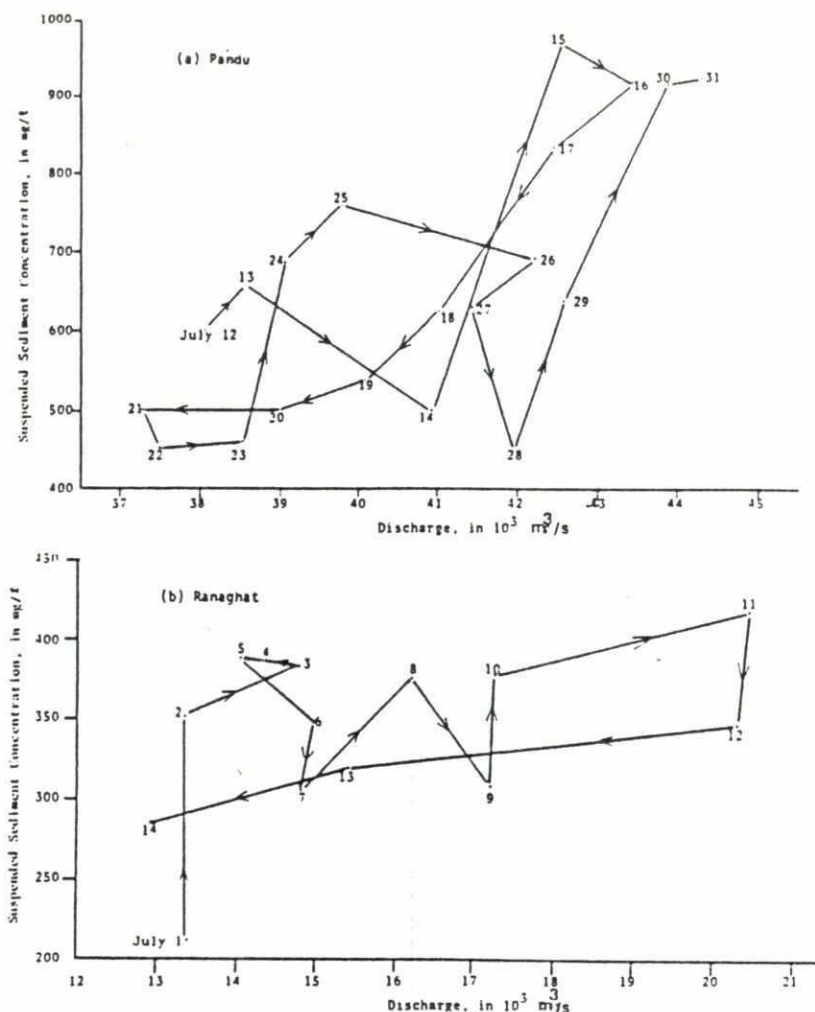


Fig. 10. Relation of sediment concentration and discharge at successive flows, Brahmaputra River at (a) Pandu (July 12-31, 1977) and (b) Ranaghat (July 1-14, 1977).

5%. Median diameter ( $d_{50}$ ) of the bed materials ranges from 0.03 mm at Dibrugarh to 0.17 mm at Salmara (Figure 11). A particle size distribution of bar materials at Pandu (curve x in Figure 13) indicates that bar materials ( $d_{50}$  at Pandu = 0.03 mm) are coarser than the bed materials. Rates of bed material transport in the river computed using different sediment discharge formulas differ significantly. Analysis (D. C. Goswami, unpublished manuscript, 1982) suggests that the estimate of 5-15% (of total load) derived from Laursen's [1958] formula appears reasonable.

Daily flows as well as sediment discharge for the Brahmaputra at Pandu during the monsoon season, 1960, are illustrated in Figure 12. It is interesting to note not only that the sediment load is very closely related to the discharge, but that even on this large river major fluctuations occur in both flow and sediment load during the monsoon period. The peak of the sediment load precedes the discharge peak by one to several days, a fact generally attributable to the high concentration of relatively fine sediments. Even more striking are the enormous fluctuations in the discharge of some of the major tributaries such as the Jia Bharali or the Subansiri, which may be among the flashiest very large rivers of the world (D. C. Goswami, unpublished manuscript, 1982). Sediment transport in the Brahmaputra is more variable than flow. During 1971-1979, while the coefficient of variation of

the mean annual flow is 0.15, that of sediment discharge is as high as 0.85 (Figure 13). The repetitive pattern of rise and fall of flow and sediment load in the river is associated with the seasonal pattern of monsoonal precipitation and freeze-thaw cycle of Himalayan snows. Study of the mean monthly flow and sediment discharge at Pandu for 1971-1979 indicates a modest increase in flow and a very much larger increase in sediment discharge, particularly since 1977.

#### Bank Materials and Processes

Bank materials of the Brahmaputra river are mostly composed of varying proportions of fine sand and silt, with only the occasional presence of minor amounts of clay (generally less than 5%), as is shown in vertical sections at three different locations on the river in Figure 14. The profiles generally include two distinct parts: a relatively fine-grained topstratum and a coarser substratum. The coarser sediments presumably represent channel bars and islands accreted laterally through wandering of the channel, and the finer sediments represent vertical accretion from overbank flows [Fisk et al., 1954].

The bank line of the Brahmaputra, for the most part, is extremely unstable. Bank failures are rampant and seem to be a function of the hydraulic character of the flow and the engineering properties of the bank materials. As is suggested by Thorne et al. [1979], bank form and rate of recession are

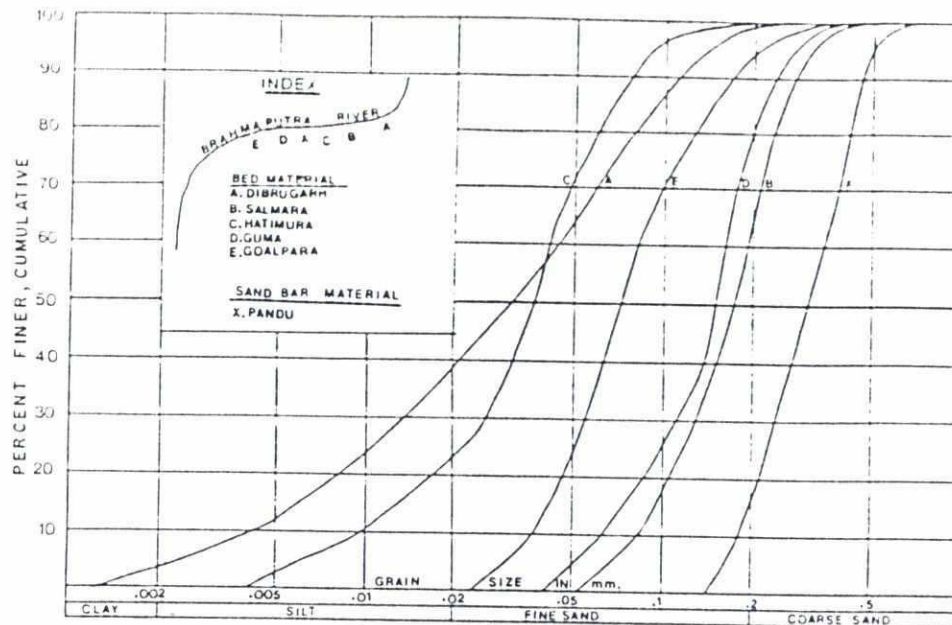


Fig. 11. Particle size distribution of bed and bar materials, Brahmaputra River.

controlled mainly by the fluvial processes, whereas the mechanisms of failures are determined by the engineering properties of the soils. Shear failures in the upper bank materials seem to be by far the most widespread mode of bank failure. This may be caused either by undercutting of the upper bank materials by currents during high flows, producing an overhanging cantilevered block which eventually fails, or by oversteepening of the bank materials due to migration of the thalweg closer to the bank during falling stages [Coleman, 1969]. Large-scale slumping of banks observed during the falling stages of the

river may be associated with return flows in the permeable alluvium. High moisture content, low proportion of clay, and good sorting of the bank materials in the Brahmaputra make them highly susceptible to erosion by the river.

#### Channel Behavior and River Stage

The channel configuration of the Brahmaputra undergoes large changes in response to variations in the flow regime and to patterns of sediment transport in the river. During the low-flow season (November through March) the river flows in a

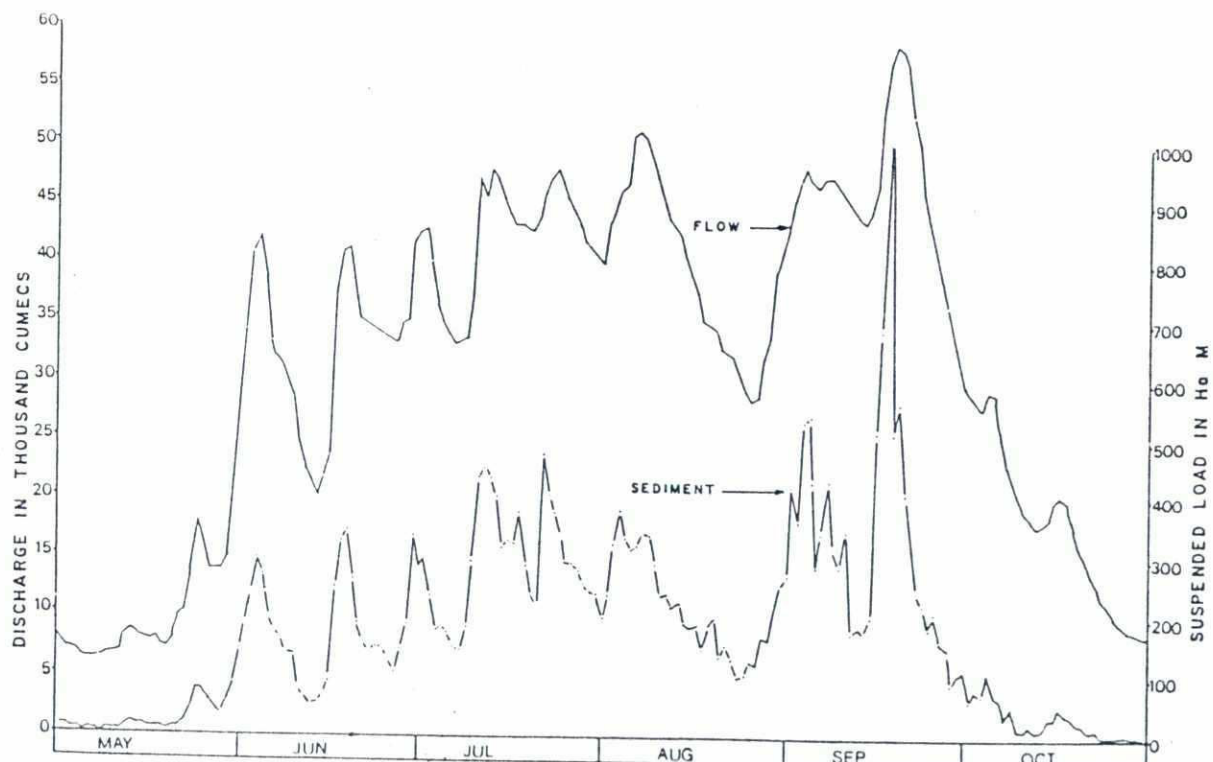


Fig. 12. Daily flow and sediment discharge during the rainy season (May to October 1960), Brahmaputra River at Pandu.



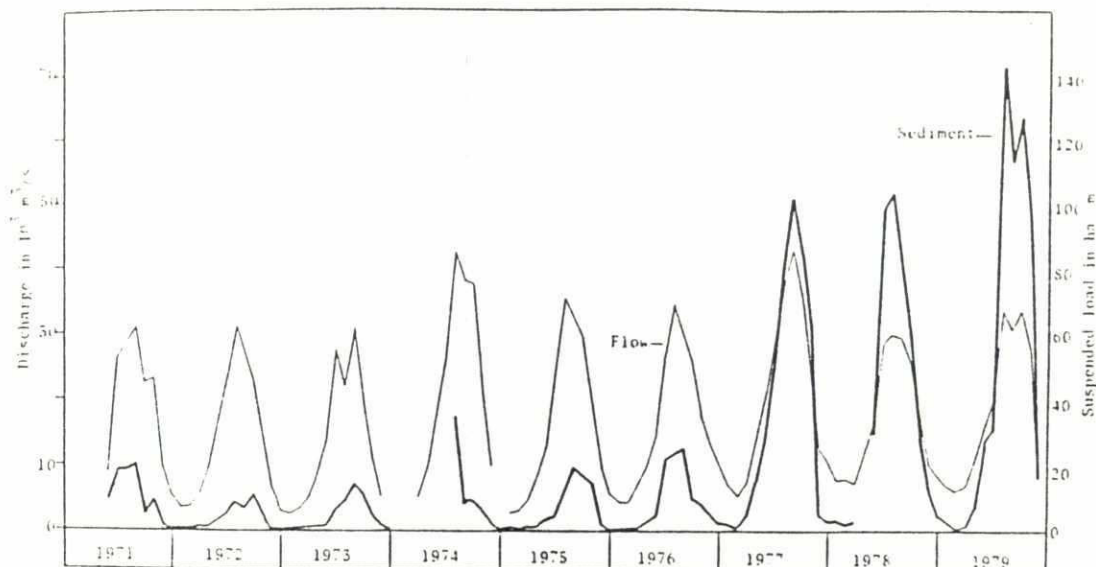


Fig. 13. Mean monthly flow and sediment discharge, Brahmaputra River at Pandu (1971–1979).

highly braided channel characterized by the presence of numerous sandbars and islands. As the flow begins to rise with the onset of the monsoon in May, most of the bars and islands are submerged, and the river then flows in a more or less straight channel. The most striking feature of the change in channel configuration of the Brahmaputra noted by Coleman [1969] is the continuous shift of the thalweg from one location to another within the banklines of the river. The movement of the thalweg is high during the rising stages (May through June), relatively less during the peak of the flood (July through August), most erratic during the falling stage (September through October), and very little during the low flow stage (November through March). A similar seasonal pattern in the movement of the channels was observed in the Yellow river of China [Chien, 1961].

The stage at selected discharges (Figure 15) shows a rising trend between 1969–1976. Blench [1972] termed the stages corresponding to a specified discharge "specific gages" and suggested that trends in these relationships indicate changes in the regime of the river over a period of years. The mean

annual suspended load and water discharges from 1955 to 1963 were 74.85 ha m and  $16,530 \text{ m}^3 \text{ s}^{-1}$  as against 12.96 ha m and  $14,850 \text{ m}^3 \text{ s}^{-1}$  from 1969 to 1976, respectively. The rising trend from 1955–1963 (Figure 15), characterized by high sediment transport in the river, indicates aggradation in the reach, while the falling trend from 1969–1976, marked by low sediment transport, suggests degradation of the channel. However, the gaging site at Pandu is in a reach constricted by granitic hills and has a narrower than average cross section, about 1.5 km wide. The observed aggradation or degradation in this cross section may not be representative of long reaches of the river. As is shown later (Table 2), from 1971 to 1979 the reach immediately upstream from Pandu suffered some degradation, whereas the reach immediately downstream experienced significant aggradation. Degradation indicated by the specific gage versus time relationship for 1969–1976 at Pandu may not be indicative of changes in the longer reaches.

Changes in the stage-discharge relationships for monthly minimum flow in the Brahmaputra at Pandu (Figure 16) indicate that the stage is low for a given discharge on the rising

limb of the annual hydrograph and high on the falling limb. The higher stage on the falling limb may be due to filling on the declining discharge or to lower sediment concentrations carried at higher depths and lower velocities [Maddock, 1970]. In Figure 16c the stage-discharge relationship is represented by discontinuous curves. Such discontinuities (as well as the loops mentioned above) in the stage-discharge relationship may be related to large changes in resistance to flow associated with transformation of the sand bed in dune stage at low or moderate flow to a plane bed or standing wave stage at higher flow. Figure 15 shows similar differences on rising and falling stages.

Coleman [1969] observed an array of bedforms in the Brahmaputra in Bangladesh ranging in wave-height from a few centimeters to 15 m. In the Assam reach of the river data on hydraulic parameters and measurements of bed configuration are not available. However, preliminary evaluation based on some measurements of velocity, depth, average slope, and bed material indicates the following relationships of flow and bed form regime.

Stream power was calculated for the Brahmaputra at Pandu to relate it to various bed forms and grain sizes as given by Allen [1968]. For high flow, July 31, 1977, when velocity and depth were measured, the stream power of the Brahmaputra at Pandu is estimated as follows:

$$\begin{aligned} \text{stream power} &= V\tau_0 = \gamma VDS \\ &= (2.19 \times 1000)(13.4)(0.00013) \\ &= 3.815 \text{ kg m}^{-1} \text{ s}^{-1} \end{aligned}$$

where

- $V$  average velocity of flow,  $\text{m s}^{-1}$ ;
- $\tau_0$  shear stress,  $\text{kg m}^{-2}$ ;
- $\gamma$  specific weight of water,  $\text{kg m}^{-3}$ ;
- $D$  average flow depth, m;
- $S$  slope of energy gradient.

The estimated value of stream power places the Brahma-

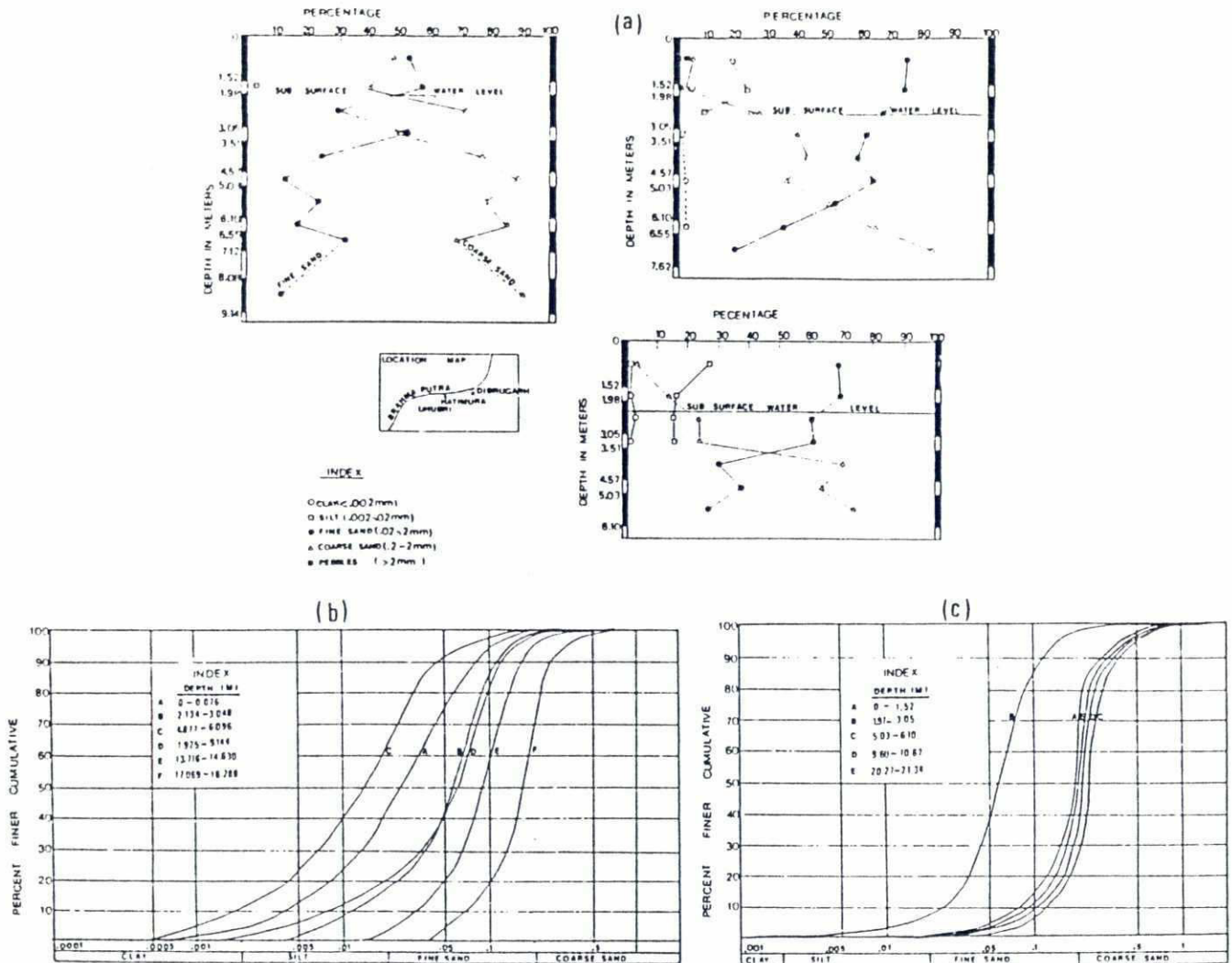


Fig. 14. Particle size distribution of bank materials, Brahmaputra River at (a) Dibrugarh, (b) Hatimura, and (c) Dhubri. Small inset map shows approximate locations.

putra in the neighborhood of antidune, plane bed region for the smallest grain size 0.1 mm, comparable to bed material sizes in the Brahmaputra in Allen's [1968] graph relating stream power to bed forms and grain sizes.

Middleton and Southard [1977] plotted bed forms in relation to depth, velocity, and sediment size using data from recent works by Boothroyd *et al.* [1975], Jackson [1975], and Dalrymple [1977] for both laboratory flumes and natural

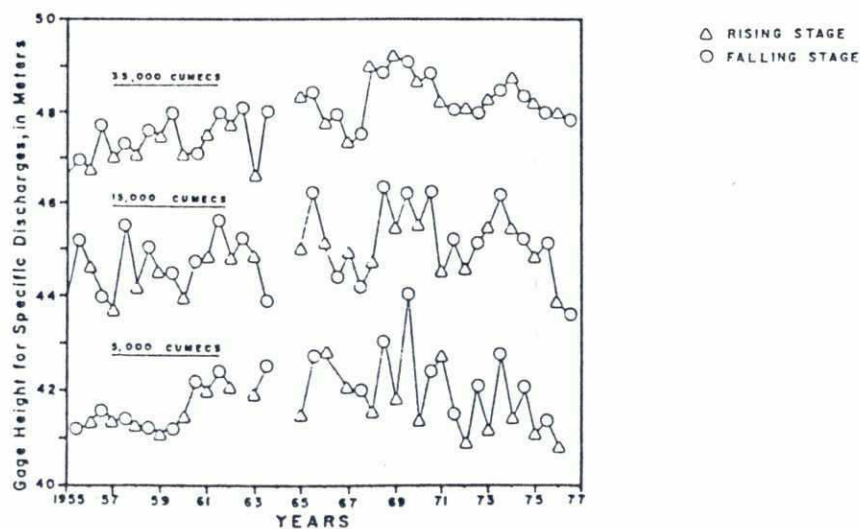


Fig. 15. Stage for selected discharges, Brahmaputra River at Pandu (1955-1976).



TABLE 2. Suspended Sediment Budget for the Period 1971-1979, Brahmaputra River, Assam Reach

| Gaging Station on the Brahmaputra (1) | North Bank Tributary (2) | South Bank Tributary (3) | Mainstream Inflow, $M_i$ (4) | Tributary Inflow, $T_{ij}$ (5) | $\sum T_{ij}$ (6) | $M_i + \sum T_{ij}$ (7) | Outflow From the Reach, $O_i$ (8) | Difference in Storage, $\pm \Delta s$ (7-8) (9) | Aggradation or Degradation of Channel Bed, cm (10) | Percent Error for Different Reaches* (11) |
|---------------------------------------|--------------------------|--------------------------|------------------------------|--------------------------------|-------------------|-------------------------|-----------------------------------|-------------------------------------------------|----------------------------------------------------|-------------------------------------------|
| Ranaghat                              |                          |                          | 28,139                       |                                |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Dibang                   |                          |                              | 15,500                         |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Lohit                    |                          |                              | 14,700                         |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Burisuti                 |                          |                              | 18                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Buridihing               |                              | 1,800                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Desang                   |                              | 840                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Dikhow                   |                              | 308                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Jhanji                   |                              | 128                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          |                          |                              | 50                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Others                   |                              |                                | 33,344            | 61,483                  | 93,065                            | -31,582                                         | -19.50                                             | 13.8                                      |
| Bessamara                             |                          |                          | 93,065                       |                                |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Subansiri                |                          |                              | 7,118                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Ranganadi                |                          |                              | 1,380                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Buroi                    |                          |                              | 108                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Borgong                  |                          |                              | 122                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Jiabharali               |                          |                              | 17,096                         |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Bhogdoi                  |                              | 132                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Dhansiri(S)              |                              | 1,325                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          |                          |                              | 50                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Others                   |                              |                                | 27,331            | 120,396                 | 52,439                            | +67,957                                         | +57.90                                             | 6.4                                       |
| Bhurbandha                            |                          |                          | 52,439                       |                                |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Gabharu                  |                          |                              | 102                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Belsiri                  |                          |                              | 81                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Pachnoi                  |                          |                              | 99                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Dhansiri(N)              |                          |                              | 180                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Noanadi                  |                          |                              | 35                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Nanoi                    |                          |                              | 41                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Barnadi                  |                          |                              | 94                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Kopili                   |                              | 1,040                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Digarua                  |                              | 73                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          |                          |                              | 50                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Others                   |                              |                                | 1,795             | 54,234                  | 57,370                            | +3,136                                          | -2.55                                              | 139                                       |
| Pandu                                 |                          |                          | 57,370                       |                                |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Puthimari                |                          |                              | 1,594                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Mutanga                  |                          |                              | 201                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Darranga                 |                          |                              | 18                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Baralia                  |                          |                              | 146                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Nona                     |                          |                              | 14                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Pagladiya                |                          |                              | 202                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Mora Pagladiya           |                          |                              | 71                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Buradiya                 |                          |                              | 27                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Tihu                     |                          |                              | 81                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Kaldiya                  |                          |                              | 67                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Pahumara                 |                          |                              | 118                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Beki                     |                          |                              | 17,540                         |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Bhalukdoba               |                          |                              | 447                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Manas                    |                          |                              | 253                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       | Buriaie                  |                          |                              | 1,388                          |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Kulsi                    |                              | 5.3                            |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Deosila                  |                              | 12.4                           |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Dudhnoi                  |                              | 19                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Krishnai                 |                              | 67                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Jinari                   |                              | 19                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          |                          |                              | 50                             |                   |                         |                                   |                                                 |                                                    |                                           |
|                                       |                          | Others                   |                              |                                | 22,340            | 79,710                  | 33,651                            | +46,059                                         | +51.59                                             | 9.5                                       |
| Jogighopa                             |                          |                          | 33,651                       |                                |                   |                         |                                   |                                                 |                                                    |                                           |

Sediment load is given in hectare meters. Difference in storage of suspended sediment in the Ranaghat-Jogighopa reach =  $\sum_{i=1}^n M_i + \sum_{i=1}^n \sum_{j=1}^n T_{ij} - \sum_{i=1}^n O_i = 231,013 + 84,810 - 236,525 = +79,298$  ha m (notations explained in the text). On the basis of storage or removal of sediment from the reaches (shown in the budget), rates of aggradation or degradation of the channel bed are estimated as follows. Ranaghat-Jogighopa Reach as a whole: area =  $607 \times 8.1 \times 10^6 \text{ m}^2$ ; volume of sediment deposited =  $79,298 \times 10^4 \text{ m}^3$ ; amount of aggradation =  $79,298 \times 10^4 \div 607 \times 8.1 \times 10^6 \text{ m} = 16.13 \text{ cm}$ . Ranaghat-Bessamara Reach: area =  $197 \times 8.2 \times 10^6 \text{ m}^2$ ; volume of sediment removed =  $31,582 \times 10^4 \text{ m}^3$ ; amount of degradation =  $31,582 \times 10^4 \div 197 \times 8.2 \times 10^6 \text{ m} = 19.50 \text{ cm}$ . Bessamara-Bhurbandha Reach: area =  $145 \times 8.1 \times 10^6 \text{ m}^2$ ; volume of sediment deposited =  $67,957 \times 10^4 \text{ m}^3$ ; amount of aggradation =  $67,957 \times 10^4 \div 145 \times 8.1 \times 10^6 \text{ m} = 57.9 \text{ cm}$ . Bhurbandha-Pandu Reach: area =  $152 \times 8.1 \times 10^6 \text{ m}^2$ ; volume of sediment removed =  $3,136 \times 10^4 \text{ m}^3$ ; amount of degradation of the reach =  $3,136 \times 10^4 \div 152 \times 8.1 \times 10^6 \text{ m} = 2.55 \text{ cm}$ . Pandu-Jogighopa Reach: area =  $113 \times 7.9 \times 10^6 \text{ m}^2$ ; volume of sediment deposited =  $46,059 \times 10^4 \text{ m}^3$ ; amount of aggradation of the reach =  $46,059 \times 10^4 \div 113 \times 7.9 \times 10^6 \text{ m} = 51.59 \text{ cm}$ .

\*Based on calculations detailed in the work by D. C. Goswami (unpublished manuscript, 1982).

ivers. Plotted on Middleton and Southard's graph, the Brahmaputra at high flow would be in the dune phase. This result appears to be more plausible inasmuch as the Froude number for the depth and velocity used in the computation would be 0.19. In the absence of observations, however, no firm conclusion can be drawn. Conceivably, the bed configuration of the Brahmaputra does enter the antidune or plane bed phase during peak flows, but the dune phase seems to prevail over much of the range and at lower stages. As in many other alluvial rivers, variations in stage corresponding to specific discharges are likely to be related to bed forms, and bed forms may influence sediment transport as suggested by the scatter on the sediment rating curves for the Brahmaputra (Figure 7).

With the progress of the flood cycle, sediment transport in the Brahmaputra increases, the thalweg starts to change position, and the geometry and location of bars change. As the flow begins to recede deposition over the bed takes place as bars and islands. During the low-flow season the river flows in several sinuous channels between these sandbars and islands. With the onset of the next flood season, the sequence of events is repeated.

#### AGGRADATION AND DEGRADATION OF THE CHANNEL

Two independent methods, one based on the sediment budget, the other on measured cross sections, provide a basis for estimating the amounts of aggradation or degradation in the Brahmaputra river in Assam over several decades; each is considered separately below.

##### Suspended Sediment Budget

Changes in storage of sediments along successive reaches of the Brahmaputra river have been estimated using a sediment budget approach, and from these estimates some measures of aggradation and degradation of the river bed are deduced. Based on the principle of continuity, the following simple model was developed. The total inflow of suspended load into any reach represents the sum of the inflow at the upstream end of the reach and the inflow from all tributaries entering the reach. By comparing the total inflow with the outflow of suspended sediments at the downstream end of the reach, an estimate of storage or removal of sediments within the reach is obtained determined by the sign of the difference.

For a given reach  $i$ , let  $M_i$  denote the inflow of sediments from the mainstream, and  $\sum_{j=1}^{n_i} T_{ij}$  the sum of inflows from  $n_i$  tributaries flowing into the reach. If measured outflow of suspended sediments from the reach is  $O_i$ , then the difference in storage ( $\pm \Delta s$ ) of sediments in the reach is given by

$$M_i + \sum_{j=1}^{n_i} T_{ij} - O_i = \pm \Delta s$$

Taking the outflow of the preceding reach as the inflow of the immediate downstream reach, the iteration is carried out for all reaches.

If the number of reaches in the river is  $m$ , then the difference in sediment storage in the entire section between the first and the last gaging station is given by the equation

$$\sum_{i=1}^m M_i + \sum_{i=1}^m \sum_{j=1}^{n_i} T_{ij} - \sum_{i=1}^m O_i = \pm \sum_{i=1}^m \Delta s_i$$

The results of the computations of loss and gain of sediments in the successive reaches on the Brahmaputra river are presented in Table 2. These calculations assume a bulk density

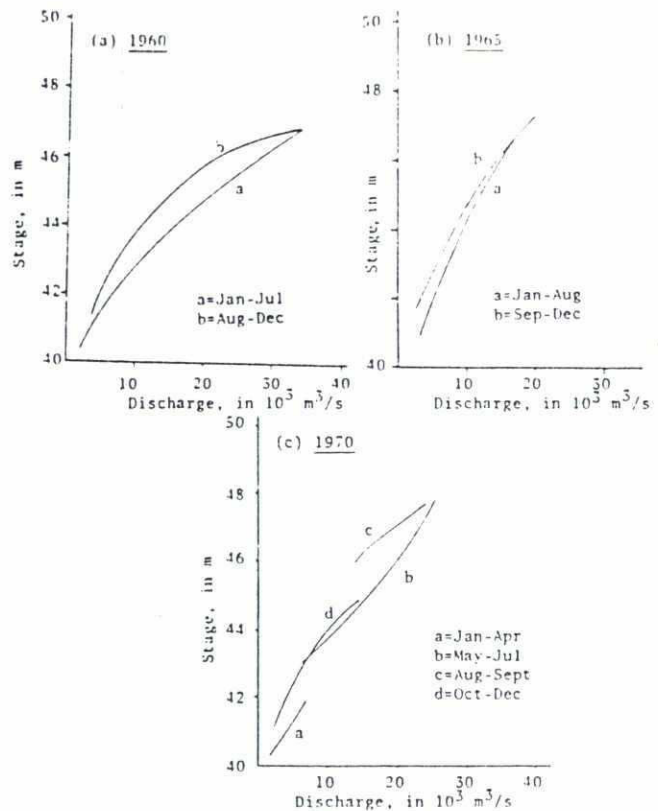


Fig. 16. Stage-discharge relationships for monthly minimum flows, Brahmaputra River at Pandu during (a) 1960, (b) 1965, and (c) 1970.

of  $2.65 \text{ T/m}^3$  with no allowance for greater porosity and lower bulk density of deposited sediment. A porosity of 40% is equivalent to a bulk density of  $1.6 \text{ T/m}^3$ . Estimated depths of degradation are affected by the bulk density assumed and the influence of these assumptions is discussed in the conclusions. Subject to possible errors discussed below, these data show that during the period of study (1971–1979), the Bessamara-Bhurbandha and the Pandu-Jogighopa reaches have undergone considerable aggradation, while the Ranaghat-Bessamara and the Bhurbandha-Pandu reaches have experienced some degradation (see Figure 1 for location of the reaches). Taken as a whole, the Assam section of the river, Ranaghat to Jogighopa, registers a net gain of about 79,000 ha m of sediment. This is 70% of the total suspended load contributed by the tributaries and the mainstream flowing into the reach. If distributed uniformly over the channel, assuming an average channel width of 8.1 km, this volume of sediment would raise the bed by about 16 cm (for the computations, see Table 2).

Given the profile of the river (Figure 4), one would expect aggradation in the most upstream reach from Ranaghat to Bessamara because of the abrupt flattening of slope near Pasi-ghat. However, as is discussed more fully below, degradation in the reach during 1971–1979 may simply represent partial removal of sediments deposited during the previous period (1955–1971). More sediment was removed from the Ranaghat-Bessamara reach than was received from suspended load inflows from 1971 to 1979. The excess that moved out of the reach must have come from either erosion of the banks or from scour of the river bed, or both. Although no reliable estimates of bank line recession are available for the period



TABLE 3. Some Measures of Rate of Recession of Bank Lines in the Brahmaputra River

| Cross Section* | Width of Cross Section,*<br>m |        |        | Change in Width of<br>Cross Section, m |           |           | Average Annual<br>Rate of Change in<br>Cross Sections, m |
|----------------|-------------------------------|--------|--------|----------------------------------------|-----------|-----------|----------------------------------------------------------|
|                | 1971                          | 1977   | 1981   | 1971-1977                              | 1977-1981 | 1971-1981 |                                                          |
| 1              | 9.233                         | 9.711  | 10.020 | 478                                    | 309       | 787       | 71.5                                                     |
| 2              | 9.244                         | 9.600  | 9.578  | 356                                    | -22       | 334       | 30.4                                                     |
| 3              | 9.131                         | 11.500 | 10.900 | 2,369                                  | -600      | 1,769     | 160.8                                                    |
| 4              | 10.597                        | 12.700 | 12.700 | 2,103                                  | 0         | 2,103     | 191.2                                                    |
| 5              | 11.720                        | 13.860 | 13.845 | 2,140                                  | -15       | 2,125     | 193.2                                                    |
| 6              | 9.527                         | 11.760 | 11.610 | 2,233                                  | -150      | 2,083     | 189.4                                                    |
| 7              | 11.875                        | 13.800 | 15.280 | 1,925                                  | 1,480     | 3,405     | 309.5                                                    |
| 8              | 9.690                         | 12.440 | 13.010 | 2,750                                  | 570       | 3,320     | 301.8                                                    |
| 9              | 10.096                        | 10.980 | 10.910 | 884                                    | -70       | 814       | 74.0                                                     |
| 10             | 6.640                         | 6.670  | 6.680  | 30                                     | 10        | 40        | 3.6                                                      |

\*Cross sections are located between Dibrugarh and Bessamara (see Figure 1).

under review, the order of magnitude involved can be gathered from the variations in channel width presented in Table 3. From 1971 through 1977, bank lines receded more than 2 m at some locations. Taking a conservative figure of 100 m (the median value in Table 10 is 175 m) as the minimum lateral recession of the bank lines for the entire reach, and assuming an average bank height of 3 m, bank erosion would have contributed at least 5910 ha m of sediment during the period 1971-1979, about 19% of the estimated loss of 31582 ha m in the reach (Table 2). Therefore degradation of the channel bed represents 81% of the estimated loss from the reach, or 25,580 ha m. If this amount of sediment were removed uniformly from the channel, the channel bed would be lowered by about 16 cm. A higher contribution from bank erosion would, in turn, reduce the estimated lowering of the bed.

Additional information on aggradation and degradation of a reach from Kobo (52 km downstream from Ranaghat) to Bessamara based on a series of sequential channel cross sections (end area method) for 1971 and 1977 is given below. The Kobo-Bessamara reach, 145 km long, is treated here as roughly equivalent to the 197-km-long Ranaghat-Bessamara reach. To permit comparison of the budget and end area results over the same time period, an additional suspended load budget for the Ranaghat-Bessamara reach is presented in Table 4 for the period 1971-1977 covered by the cross-sectional surveys. The budget shows removal of about 22,000 ha m of sediment from the reach, approximately 70% of the estimated removal of 31,582 ha m for the 1971-1979 period (Table 2).

A suspended load budget of the type used here involves errors in measurement of discharge as well as sediment concentration. D. C. Goswami (unpublished manuscript, 1982) described in detail the estimates of errors in sediment discharge measurements and their effects on suspended load budget for the Brahmaputra river. Errors in both flow discharge and sediment concentration measurement should be less than 25% for a difference in storage of suspended sediments in the Brahmaputra to be detectable. In other words, the mean error due to measurement of suspended sediment discharge should be less than about 50%. In an example where reasonably high percent errors in sediment concentration and water discharge measurements at Pandu are assumed, the computed error due to sampling variability is 4370 ha m. This value represents a confidence interval of one standard deviation on either side of the expected total mea-

sured sediment discharge. It is assumed that errors in measurement at other gaging stations on the Brahmaputra are similar to the one computed for Pandu. Expressed as a fraction (in percentage) of the storage or removal of sediments ( $\pm \Delta s$ ) for the different reaches of the Brahmaputra, the errors (due to sampling variability) lie within about 14%, except in the case of the Bhurbandhu-Pandu reach, where the high (139%, Table 2, last column) error is due to the extremely small value of storage,  $\Delta s$ , in the reach. The storage value is relatively large for the Assam reach as a whole, Ranaghat to Jogighopa, and the error is estimated to be 5.5%. This error is not large enough to nullify the conclusions regarding storage of sediments within the river. As long as the error due to variability of sampling remains low compared to the difference in storage ( $\Delta s$ ),  $\Delta s$  will be detectable. If the same uncertainty affects measurements at both ends of a reach, the computed difference in storage will be unaffected. Only when the upstream gage has a particular bias and the downstream gage does not, or vice versa, or when the errors have different signs would the bias affect the storage computation. In the light of the estimates of the tolerance limit of the error discussed above, the magnitude of the estimated errors for the different reaches of the Brahmaputra (Table 2) should not affect the general conclusions regarding change in storage of sediments within successive reaches, except in the Bhurbandhu-Pandu reach noted earlier.

Of the four reaches of the Brahmaputra in Assam over a total length of 607 km, two reaches (258 km) have experienced an average aggradation of about 55 cm and the other two reaches (349 km) have suffered a degradation of about 11 cm during the period 1971 through 1979. Taken as a whole, the Assam section of the river between Ranaghat and Jogighopa is estimated to have experienced an aggradation of about 16 cm. About 70% of the suspended sediment inflow into the reach is apparently retained in the channel. There is, in addition, some contribution of sediment from bank erosion. The high rate of aggradation of the channel seems to be a major cause of the braiding which characterizes this reach of the river at low and moderate flows.

#### AGGRADATION AND DEGRADATION OF REACHES FROM CHANNEL CROSS SECTIONS

Changes in sediment storage between 1971 and 1977 in 15 different reaches of the Brahmaputra river between Kobo and Bessamara, over a total length of 145 km, have been calcu-

TABLE 4. Sediment-Budget for the Ranaghat-Bessamara Reach, 1971-1977

| Gaging Station on the Brahmaputra | North Bank Tributary | South Bank Tributary | Mainstream Inflow, $M_i$ (1) | Tributary Inflow, $T_{ij}$ (2) | Sum of Tributary Inflow, $\sum T_{ij}$ (3) | Total Inflow Into the Reach, $M_i - \sum T_{ij}$ (4) | Outflow From the Reach, $O_i$ (5) | Differences in Channel Storage, $\pm \Delta s$ (4)-(5) (6) |
|-----------------------------------|----------------------|----------------------|------------------------------|--------------------------------|--------------------------------------------|------------------------------------------------------|-----------------------------------|------------------------------------------------------------|
| Ranaghat                          |                      |                      | 19.860                       |                                |                                            |                                                      |                                   |                                                            |
|                                   | Dibang               |                      |                              | 12,000*                        |                                            |                                                      |                                   |                                                            |
|                                   | Lohit                |                      |                              | 11,500*                        |                                            |                                                      |                                   |                                                            |
|                                   | Burisuti             |                      |                              | 13                             |                                            |                                                      |                                   |                                                            |
|                                   |                      | Buridihing           |                              | 1,266                          |                                            |                                                      |                                   |                                                            |
|                                   |                      | Desang               |                              | 779                            |                                            |                                                      |                                   |                                                            |
|                                   |                      | Dikhow               |                              | 268                            |                                            |                                                      |                                   |                                                            |
|                                   |                      | Jhanji               |                              | 111                            |                                            |                                                      |                                   |                                                            |
|                                   |                      | Others               |                              | 50                             |                                            |                                                      |                                   |                                                            |
| Bessamara                         |                      |                      |                              |                                | 25.987                                     | 45.847                                               | 67.914                            | -22.067                                                    |

Sediment load is given in hectare meters. Length of the reach = 197 km; average width of the reach = 8.2 km; mean degradation in the reach =  $22.067 \times 10^4 = 197 \times 8.2 \times 10^6 \text{ m} = 13.66 \text{ cm}$ .

\* Estimated figures.

lated from measurements of sequential cross sections spaced about 9 km apart (Figure 1).

For the few reaches in which three sets of cross sections were available, changes in storage of sediment for the periods 1957-1971 and 1977-1981 were also computed. These values provide a background against which to evaluate computations for the period 1971-1977.

Figure 17 shows two sets of sequential cross sections of the Brahmaputra between Kobo and Bessamara for the years 1957, 1971, and 1977 (cross sections 1 and 3 in Table 5). Cross sections in Figure 17a show considerable fill during 1957-1971, followed by scour during 1971-1977 which removed about one third of the volume of materials deposited during 1957-1971. The position of the channel changed drastically from one time period to another. Close to both the north and south banks considerable accumulation of sediment took place between 1957-1977. Cross sections in Figure 17b show considerable fill occurred during 1957-1971, followed by a lesser amount during 1971-1977. The cross section for 1977

shows that the northern bank line had extended about 3 km beyond the limit of the 1957 and 1971 bank lines. Aggradation of the channel seems to be highest in the center during 1957-1977, and the deepest channels are closer to the south bank.

Estimates of aggradation or degradation for various reaches in the Kobo-Bessamara section of the Brahmaputra river are given in Table 5. The surface area of the reach between two adjacent cross sections was calculated from the average width and the length of the reach. Mean aggradation or degradation of the reach was then obtained by dividing the change in storage of sediments in the reach by the bed area of the reach.

The width of the Brahmaputra is highly variable, both temporally and spatially. Examination of the widths of the various cross sections given in Table 5 suggests variations in channel width up to 5 km for reaches varying in length from 7 to 14 km. There may be considerable local variations in channel width as well. Little reliable data are available regarding the intensity and rate of temporal variation in channel width of the Brahmaputra. However, the order of magnitude involved

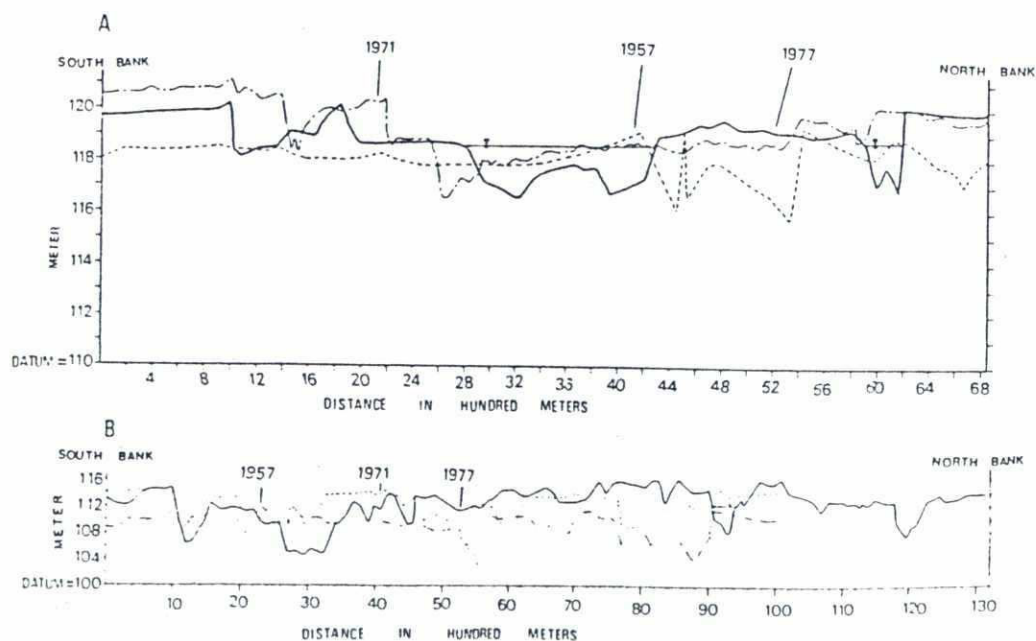


Fig. 17. Cross sections of the Brahmaputra River for 1957, 1971, and 1977.



9

TABLE 5. Estimation of Rates of Channel Aggradation and Degradation From Scour and Fill in Successive Cross Sections in the Kobo-Bessamara Reach of the Brahmaputra River

| Cross Section | Width of Cross Section, km | Average Width of the Reach, km | Length of the Reach, km | Years Between | Cross Sectional Fill (+) and Scour (-), m <sup>2</sup> | Storage or Removal of Sediment From the Reach ( $\pm \Delta s$ ), ha m | Rates of Aggradation (+) or Degradation (-) of Channel, m |
|---------------|----------------------------|--------------------------------|-------------------------|---------------|--------------------------------------------------------|------------------------------------------------------------------------|-----------------------------------------------------------|
| 1             | 6.93                       |                                |                         | 1957-1971     | +16.300                                                |                                                                        |                                                           |
|               |                            | 8.67                           | 7.00                    | 1971-1977     | -6.100                                                 |                                                                        |                                                           |
| 2             | 10.425                     |                                |                         | 1957-1971     | +26.120                                                | +14.847                                                                | +2.446                                                    |
|               |                            | 11.82                          | 14.00                   | 1971-1977     | -5.300                                                 | -3.990                                                                 | -0.657                                                    |
| 3             | 13.22                      |                                |                         | 1957-1971     | +18.000                                                | +30.884                                                                | +1.866                                                    |
|               |                            | 14.09                          | 11.00                   | 1971-1977     | +1.950                                                 | -2.345                                                                 | -0.142                                                    |
| 4             | 14.97                      |                                |                         | 1957-1971     | +33.868                                                | +28.527                                                                | +1.841                                                    |
|               |                            | 14.58                          | 9.00                    | 1971-1977     | +24.100                                                | +14.327                                                                | -0.924                                                    |
| 5             | 14.20                      |                                |                         | 1957-1971     | -19.000                                                | +6.691                                                                 | +0.510                                                    |
|               |                            | 11.77                          | 12.00                   | 1971-1977     | +6.560                                                 | +13.797                                                                | +1.051                                                    |
| 6             | 9.35                       |                                |                         | 1957-1971     | +1.180                                                 | -10.692                                                                | -0.757                                                    |
|               |                            | 9.41                           | 9.60                    | 1971-1977     | -4.470                                                 | +1.254                                                                 | +0.089                                                    |
| 7             | 9.47                       |                                |                         | 1977-1981     | +6.966                                                 |                                                                        |                                                           |
|               |                            | 9.99                           | 8.62                    | 1971-1977     | -4.360                                                 | -4.328                                                                 | -0.469                                                    |
| 8             | 10.51                      |                                |                         | 1977-1981     | +6.373                                                 | +6.403                                                                 | +0.709                                                    |
|               |                            | 11.26                          | 13.90                   | 1971-1977     | -20.460                                                | -10.697                                                                | -1.242                                                    |
| 9             | 12.00                      |                                |                         | 1977-1981     | +15.093                                                | +9.252                                                                 | +1.074                                                    |
|               |                            | 12.57                          | 8.90                    | 1971-1977     | -27.038                                                | -33.011                                                                | -2.109                                                    |
| 10            | 13.14                      |                                |                         | 1977-1981     | -1.754                                                 | +9.270                                                                 | +0.592                                                    |
|               |                            | 12.06                          | 8.10                    | 1971-1977     | -1.625                                                 | -12.755                                                                | -1.140                                                    |
| 11            | 10.97                      |                                |                         | 1977-1981     | +8.77                                                  | -3.70                                                                  | -0.035                                                    |
|               |                            | 12.31                          | 9.90                    | 1971-1977     | +6.837                                                 | +2.111                                                                 | +0.216                                                    |
| 12            | 13.65                      |                                |                         | 1977-1981     | -16.522                                                | -6.336                                                                 | -0.649                                                    |
|               |                            | 12.68                          | 7.10                    | 1971-1977     | +2.710                                                 | +4.726                                                                 | +0.388                                                    |
| 13            | 11.71                      |                                |                         | 1977-1981     | -4.205                                                 | -10.260                                                                | -0.842                                                    |
|               |                            | 11.18                          | 10.10                   | 1971-1977     | +8.824                                                 | +4.095                                                                 | +0.455                                                    |
| 14            | 10.66                      |                                |                         | 1977-1981     | -5.212                                                 | -3.343                                                                 | -0.371                                                    |
|               |                            | 8.66                           | 7.60                    | 1971-1977     | -6.811                                                 | +1.017                                                                 | +0.090                                                    |
| 15            | 6.66                       |                                |                         | 1977-1981     | -6.759                                                 | -6.045                                                                 | -0.535                                                    |
|               |                            | 7.16                           | 8.00                    | 1957-1971     | +5.599                                                 |                                                                        |                                                           |
| 16            | 7.65                       |                                |                         | 1971-1977     | -1.834                                                 | -3.285                                                                 | -0.499                                                    |
|               |                            |                                |                         | 1977-1981     | -1.909                                                 | -3.294                                                                 | -0.500                                                    |
|               |                            |                                |                         | 1957-1971     | +5.822                                                 | +4.568                                                                 | +0.797                                                    |
|               |                            |                                |                         | 1971-1977     | -7.606                                                 | -3.774                                                                 | -0.659                                                    |

In the reach between cross sections 1 and 16 during 1971-1977, total volume of sediments gained = 41.327 ha m and total volume of sediments lost = 74.095 ha m  $\pm$  net removal ( $-\Delta s$ ) = 32.768 ha m. Length of the reach = 144.8 km; average width of the reach = 10.96 km; average degradation of the reach =  $32,768 \times 10^4 \div 10.96 \times 144.8 \times 10^6$  m = 20.65 cm.

can be obtained from the changes in cross-sectional width of the river in the reach between Dibrugarh and Bessamara during 1971-1977 (Table 3) where observed variations ranged from 30 m to 2.75 km.

Data presented in Table 5 suggest that the reach of the Brahmaputra between Kobo and Bessamara suffered a net loss of about 33,000 ha m of sediment during the 1971-1977 period. This figure is obtained through cumulation of the volumes of sediment gained or lost in 15 different reaches of the river between Kobo and Bessamara. If the above total volume of sediments was removed uniformly from the bed, the reach would have experienced an average degradation of about 21 cm. However, the rates of aggradation and degradation for a longer period (1957-1971) at six cross sections (sections 2, 3, 4, 5, 6, and 16 in Table 5) indicate considerable gain of sediments in all reaches except one. Aggradation in the Kobo-Bessamara reach ranged from 0.5 m to a maximum of 2.4 m during the 1957-1971 period. The Brahmaputra river had a much higher rate of sediment transport from 1957-1960 and a moderately high rate during 1966-1969, as against the lowest rate during 1971-1976, as shown by mean annual sediment transport rates at Pandu (D. C. Goswami, unpublished manuscript, 1982). Despite missing records from 1961 to 1966 and in 1970, sedi-

ment transport during the entire period 1956-1971 appears, on the average, to have been considerably higher than during 1971-1977. Thus in the Kobo-Bessamara section of the river, rates of sediment transport seem to be positively correlated with aggradation or degradation of the channel. Assuming that the estimated rates of aggradation averaging 1.25 m during 1957-1971 in the few reaches between Kobo and Bessamara for which a full record is available (Table 5) represent the average rate for the entire reach, degradation of 21 cm during 1971-1977 in the same reach seems to indicate removal of only a small fraction of the volume of sediments deposited during 1957-1971. Since flow in the river did not change appreciably, even during the period of low sediment transport, reduction in sediment inflow appears to have resulted in removal of sediments from the bed.

In the previous section on the suspended load budget, the Ranaghat-Bessamara reach was estimated to have lost about 22,000 ha m of sediment during 1971-1977 (Table 4). This reach is roughly comparable to the Kobo-Bessamara reach. Thus for the same reach (Ranaghat-Bessamara) and during the same period (1971-1977) the volume of sediment estimated to be removed from the reach is 33,000 ha m on the basis of cross-sectional studies and 22,000 ha m on the basis of the



suspended load budget. The difference of about 11,000 ha m between the two estimates could well be due to errors in sediment load measurements, to storage changes in the 52-km-long reach of the river between Ranaghat and Kobo which is not covered by cross-sectional studies, and to unknown exchange rates of sediments between the bed, the banks, and the flow. As was discussed earlier, a conservative figure of 100 m for the minimum bank line recession of the Brahmaputra in the Ranaghat-Bessamara reach would generate at least 5910 ha m of sediment, which is about 1.8% of the excess sediments removed from the reach. A maximum value of 300 m (Table 3) for bank recession would treble the estimated sediment derived from the banks, the remaining 46% to be derived from the bed. Given the range, it seems probable that from 1971 to 1977 the Kobo-Bessamara reach derived the bulk of the excess sediment that moved out of the reach by lowering the bed, that is, by degradation.

These estimates of channel change based on scour or fill of cross sections indicate a pattern broadly consistent with the one suggested by the suspended load budget discussed earlier. However, estimation of channel change from widely spaced cross sections involves uncertainty arising from the fact that scour or fill at a cross section may not be representative of conditions in a long reach. The problem may be more severe in a river like the Brahmaputra, where variability in bed and bank configuration is significant both in time and space.

Both the cross-sectional and sediment budget methods suggest moderate degradation in the Kobo-Bessamara reach, although estimates from the former method are about 50% higher than those from the latter. If the deposited sediment is assumed to have a much lower bulk density, then the estimates by both methods are about the same. Changes in the cross sections indicate that the degradation observed in the reach during 1971–1977 is a result of removal of a fraction of the volume deposited in the earlier period 1957–1971.

#### RATES OF DENUDATION IN THE BRAHMAPUTRA BASIN

Patterns of regional denudation have been examined on the basis of suspended load in rivers by many authors [Dole and Stabler, 1909; Fournier, 1960; Menard, 1961; Schumm, 1963; Judson and Ritter, 1964; Strakhov, 1967; Hogleman, 1968; Young, 1969; Trimble, 1975; Yoshikawa, 1974; Li, 1976; Campbell, 1977; Milliman and Meade, 1983]. The rate of denudation is estimated by assuming uniform removal of sediment over the basin equivalent to the average annual suspended load measured at the basin outlet. Although "sediment yield per unit drainage area" thus obtained provides the most commonly used index of basin erosion, it is seriously flawed by the assumption of uniform removal of sediment from the entire watershed. As is pointed out by Gregory and Walling [1973], a large proportion of the sediment yield may be derived from a relatively small proportion of the basin. For the Amazon river, Gibbs [1967] estimated that over 80% of the suspended load at the mouth is derived from the Andes mountains several thousand miles away.

Because of their unique topographic and climatic characteristics, despite the potential error of overgeneralization, the rates of suspended sediment yield per unit drainage area of a few selected rivers in the Brahmaputra basin are used here to estimate some rates of denudation for the region. The rivers include the three trans-Himalayan tributaries, the Subansiri, the Jia Bharali, and the Manas, together with the upper course of the Brahmaputra known as the Dihang river. Based on the suspended load in rivers during 1971–1979, the average denu-

ation rate of these four Himalayan catchments (drainage areas  $> 10,000 \text{ km}^2$ ) is  $0.73 \text{ mm/year}$ , or  $73 \text{ cm}/10^3 \text{ years}$ . As the 1971–1979 period (for which data are available for the four rivers) represents a phase of low sediment transport in the Brahmaputra, the denudation rate based on records for this period is probably lower than the long-term average. To provide what may be a more general estimate, average sediment yields for the 1955–1979 period are estimated for the four Himalayan tributaries by correlation with sediment load data for the Brahmaputra at Pandu for the same period. The mean annual sediment yield at Pandu during 1955–1979 (disregarding the gap in the data from 1961 to 1965 and in 1970) is 2.16 times higher than the mean sediment load for the period 1971–1979. Assuming that the ratio of approximately 2:1 also holds for the mean sediment load between the periods 1955–1979 and 1971–1979 for the four Himalayan rivers, the calculated average denudation rate of the Himalayan catchments for the 1955–1979 period would be  $157 \text{ cm}/10^3 \text{ years}$ .

Menard [1961] suggested a denudation rate of more than  $100 \text{ cm}/10^3 \text{ years}$  for the eastern Himalayas based on Lyell's [1893] estimate of the silt load of the Brahmaputra and Khosla's [1953] data on the average silt load of the Sapt Kosi river in Uttar Pradesh. Similarly, Schumm [1963] estimated a denudation rate of  $97.5 \text{ cm}/10^3 \text{ years}$  for the Himalayas based on Khosla's [1953] data on suspended sediment yield of the Kosi river at Barakshetra, Bihar, at a rate almost the same as Menard's [1961] for the whole Himalayan range. Although no measured sediment load figures for any east Himalayan river were used in either of the previous studies, these estimates of the erosional rate of the eastern Himalayas compare well with the rate of  $73$  to  $157 \text{ cm}/10^3 \text{ years}$  estimated here. Fairbridge [1968, p. 871] estimated a mean denudation rate of  $60 \text{ cm}/10^3 \text{ years}$  for the Himalayas on the basis of the volume of Ganges sediment cone on the floor of the Bay of Bengal, the catchment area, and the period of uplift.

As is illustrated in Figure 18, sediment yield per unit area of the Brahmaputra basin increases by about 240% between Tsela D'zong, China, and Ranaghat, India, whereas the increase in drainage area between these two locations is about 28%. Water yield per unit drainage area also increases by almost 120%. Since Tsela D'zong lies at the upper limb of the Himalayan slope and Ranaghat at its southern foothills, the increase in sediment yield per unit drainage area may be attributed mainly to the action of the monsoonal downpours on the slopes of the lower and middle Himalayas (for location, see Figure 4). As was mentioned earlier, monsoon rains exceed  $500 \text{ cm/year}$  in the Himalayan sector. The bulk of this rainfall occurs in the southern part of the sub-Himalaya which is characterized by steep slopes and friable soils. Moreover, this area is the scene of extensive landslides during the rainy season and, on a massive scale, during earthquakes. Between Ranaghat and Bessamara sediment yield per unit area increases by about 110%, while the increase in drainage area is about 55%. Moreover, water yield per unit area of the basin declines by about 3% (or is nearly constant). This implies that sediment load is nearly independent of flow in terms of capacity. The large increase in sediment yield between the two locations may be attributed mainly to removal of sediments generated by the recent earthquake and accumulated in various storage sites in the basin including the beds of river channels. Increased sediment may also derive, to some extent, from removal of sediments in lands under cultivation on steep slopes and in valley bottoms. Considering the documented importance of human influence on erosion rates in other



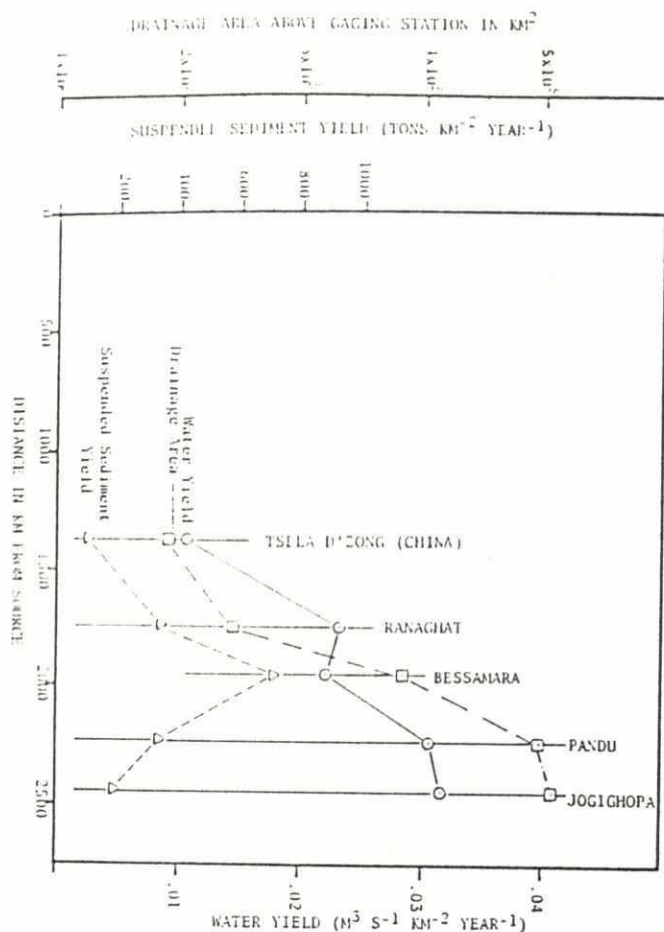


Fig. 18. Suspended sediment yield, water yield, and contributing drainage area at different gaging stations on the Brahmaputra River (1971-1979).

erosion-prone areas such as the loess regions of China or tectonically oversteepened slopes of New Zealand, sediment yields to the Brahmaputra may also be augmented by accelerated erosion. Data available here, however, cannot be used to evaluate this conjecture.

Sediment yield per unit drainage area shows a rapid decline the west of Bessamara. This is probably due to the increasing distance from the zone of maximum sediment supply between Ranaghat and Bessamara as both relief and precipitation decline.

From the known rates of transport and deposition of sediment in the Brahmaputra, the concomitant question of denudation in the Himalayas and alluviation of the valley in Assam can be examined. The Brahmaputra valley in Assam, 56,166  $km^2$  in area, received a mass of sediments 200-300 m thick during the last 2 million years [Rao, 1977]. Thus the volume of sediments stored during this period can be estimated at about  $140 \times 10^7$  ha m. By using the ratio of total yield from the drainage basin to storage of 1.4 found in this study, the volume of sediments eroded from the Himalayas during the last 2 million years to fill the valley to its present level would be about  $200 \times 10^4$  ha m. This gives a denudation of 59.3 m in the 2 million year period from the drainage area of 337,000  $km^2$  of the Himalayan watersheds of the north bank tributaries of the Brahmaputra. This average rate of denudation in the eastern Himalayas during the last 2 million years, about 3  $cm/10^3$  years, is about 2% of the rate measured in the last

three decades. Menard [1961] suggested an average erosion rate of 21  $cm/10^3$  years for the Himalayas during the  $40 \times 10^6$  years since the last Eocene uplift based on an assumption that post-Eocene sediments below the Indo-Gangetic plain are 3 km thick. Table 6 presents a summary of the rates of denudation of the Himalayas suggested in various studies including the present one.

Based on Gilluly's [1949] estimates of the rates of uplift for selected areas in California, Schumm [1963] proposed a present rate of orogeny of 7.6  $m/10^3$  years. Zeitler *et al.* [1981] estimated a similar rate of 7  $m/10^3$  years for the Nanga Parbat Massif and Douglas *et al.* [1982] estimated a rate of 4-10  $m/10^3$  years for the Kashmir Basin in the western Himalayas. If this rate, 7  $m/10^3$  years, is assumed for the eastern Himalayas as well, then the rate of uplift of the mountain is about six times greater than the present rate of denudation. Sharma *et al.* [1978] estimated an average uplift rate of the Himalayas of 55  $cm/10^3$  years based on fission track annealing of apatite from intrusive granite. This is roughly 20 times an average denudation rate of 3  $cm/10^3$  years. Powell and Conaghan [1973] suggested that the rising Himalayas did not attain their present elevation until late Pliocene or Pleistocene and may still be rising. However, no rates of uplift of the Himalayas were suggested in the study. The deep gorges of the trans-Himalayan rivers including the Brahmaputra and the Pleistocene terraces located at different heights in the Himalayas indicate the relatively rapid rise of this mountain system with episodic phases of even more accelerated uplift.

A number of studies including Schumm [1963], Yoshikawa [1974], and Li [1978] concluded that denudation rates increase rapidly with uplift. The higher a mountain block rises, the steeper will be the gradients of streams cutting into it, resulting in increased erosion of stream valleys. Yoshikawa [1974] reports that in Japan, where 50-66% of the present height of mountains may be attributed to orogenic rise during the Quaternary period, modern denudation rates in various catchment areas of reservoirs range from 0.1 to 6.3  $mm/years$  (1.3 to 630  $cm/10^3$  years). Similarly, Li [1976] ascribes the present high rate of denudation of the Central Range of Taiwan to the rapid uplift of the island since Pliocene times aided from time to time by isostatic adjustments to denudation. He estimated the average rate of physical denudation of the Central Range to be 1300  $mg cm^{-2} yr^{-1}$  (or 520  $cm/10^3$  years). This is an extremely high rate of denudation in the contemporary world. In contrast, the Mississippi region has been denuded almost at the present rate of 4.2  $cm/10^3$  years for the past  $150 \times 10^6$  years [Fisk *et al.*, 1954]. Further, the present rate of denudation of the Appalachian region is 0.8  $cm/10^3$  years, as against a rate of 6.2  $cm/10^3$  years during the last  $125 \times 10^6$  years [Dole and Stabler, 1909; Menard, 1961].

Clearly, caution is warranted in using records of short duration and inferring that over long periods of time the relationship between the sediment eroded and stored in a drainage basin remains constant. Nevertheless, the data, and comparisons from other regions, indicate that the high rate of denudation of the Himalayan catchment of the Brahmaputra is probably attributable to rapid uplift of the mountains, steep slopes, high monsoonal precipitation, and recurring earthquakes. The rising Himalayas directly influenced the occurrence and intensity of monsoonal precipitation because of their role as a barrier against the moisture-bearing winds. Furthermore, the intensity of rock disintegration through frost action may have increased as a result of the rise in elevation. Steeper slopes may be due to the nature of uplift and to excess



in the rate of uplift over denudation. Moreover, steepness of slope is one of the important variables controlling rapid mass wasting through landslides and rockfalls during earthquakes and torrential rains.

Denudation of the Himalayas and alluviation of the Brahmaputra valley in Assam are concomitant phenomena. The valley is being alluviated through deposition of sediments brought by the Brahmaputra and its numerous tributaries from the Himalayan source region. As is shown in the sediment budget (Table 2), at least 70% of the incoming suspended load is trapped in the Assam section of the river, resulting in considerable aggradation of the channel. The average rate of denudation of the drainage basin of the Yellow river in China, the only river that carries more sediment than does the Brahmaputra, is  $290 \text{ mg cm}^{-2} \text{ yr}^{-1}$  (or  $110 \text{ cm}/10^3 \text{ years}$ ) [Committee for Water Resources Control, 1973]. However, the principal source of sediment of the Yellow river is the loess of north central China and it is not related to the orogenic rise of mountain masses. The alluvial valley of the Brahmaputra in Assam is thus a unique feature whose growth is closely linked with the tectonics and denudation of the loftiest mountain system, the Himalayas, and also with the prevalence of the most potent rainfall regime, the monsoon.

#### CONCLUSION

The Brahmaputra is one of the most heavily sediment-laden large rivers of the world, carrying an average annual suspended load of  $15200 \text{ ha m}$  (402 million metric tons) at Pandu in Assam from 1955 through 1979. During the rainy season (May through October) the river transports more than 95% of the annual suspended load at Pandu at an average daily rate of  $80 \text{ ha m}$  (2.12 million metric tons). The Brahmaputra river in Assam, India, characterized by high seasonal variability in flow, sediment transport, and channel configuration, is currently experiencing a secular period of aggradation. A suspended load budget indicates an overall aggradation of the 607-km Assam reach of the Brahmaputra between Ranaghat and Jogighopa by about 16 cm during the period 1971 through 1979. The figure would be higher if allowance were made for the lower bulk density of the deposits. About 70% of the suspended sediment inflow delivered to the reach from the mainstream and tributaries is retained in the channel. For the Assam reach as a whole, the error due to sampling variability in sediment discharge, expressed as a percentage of the storage of sediments for the reach, is estimated to be small relative to the storage.

An alternative method of evaluating aggradation based on measurement of cross sections indicates removal of sediments from a 145-km reach between Kobo and Bessamara, an upstream segment of the river in Assam, during 1971–1977. Degradation of the channel in the reach was about 21 cm. The suspended load budget also suggests a degradation of this particular reach during 1971–1977 of the same order of magnitude. In contrast, limited measurements of cross sections for a longer period (1951–1971) indicate a net aggradation of about 1.25 m in the same reach. Thus degradation of 21 cm during 1971–1977 removed only a small fraction of the volume of sediments deposited earlier. A similar trend is indicated by the specific gage-discharge relationship at Pandu for the same two periods.

There appear to be phases of rapid aggradation associated with earthquakes followed by periods of relatively slower removal. Variations in gage height for specific discharges at Pandu may indicate such a pattern with aggradation during

TABLE 6. Some Estimates of Past and Present Rates of Himalayan Denudation

| Study             | Past Rate            |                         | Present Rate,<br>cm $10^3$ year | Past to Present Ratio |
|-------------------|----------------------|-------------------------|---------------------------------|-----------------------|
|                   | Time,<br>$10^6$ year | Rate,<br>cm $10^3$ year |                                 |                       |
| Menard [1961]     | 40                   | 21                      | 100                             | 0.2                   |
| Schumm [1963]     |                      |                         | 97.5                            |                       |
| Fairbridge [1966] | 20                   | 60                      | 100                             | 0.6                   |
| This study        | 2                    | 3                       | 73–157<br>(average 115)         | 0.02                  |

1955–1963. The period of rapid aggradation may be associated with the great earthquake of 1950, which generated a large debris load for the rivers. Thus recurring tectonic disturbances in the region seem to have pronounced effects on the channel configuration of the Brahmaputra in Assam.

Tectonics and denudation of the Himalayas and alluviation of the Brahmaputra valley in Assam are concomitant phenomena. Based on the suspended load carried in trans-Himalayan rivers, the present rate of denudation of the eastern Himalayas is estimated to be  $73\text{--}157 \text{ cm}/10^3 \text{ years}$ . The average rate for the last 2–3 million years, estimated from the volume of alluvial fill in the Brahmaputra valley in Assam, the sediment yield of the Himalayan rivers, and a ratio of basin yield to sediment deposition of 1.4 obtained from the present study, is  $3 \text{ cm}/10^3 \text{ years}$ . The present high rate of denudation of the Himalayas may be attributed mainly to the rapid uplift of the mountain system and to the high susceptibility to erosion of geologic formations within the intense monsoon rainfall regime.

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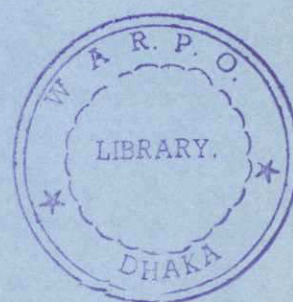
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## Paper 3.5



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CHANNEL CHARACTERISTICS OF THE  
BRAIDING JAMUNA RIVER, BANGLADESH

by

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SUMMARY

Channel characteristics of the braiding Jamuna River in Bangladesh were studied within the framework of a feasibility study for a bridge across this large braiding sand-bed river. Planform characteristics were studied on the basis of LANDSAT-images. The variation of some topological and geometrical braiding indices over the years was studied for in total six reaches. Also the planform of channel bends was studied. Cross-sectional characteristics were derived from bed level soundings made by the Bangladesh Water Development Board since 1966/67. From the analysis of the available data it is concluded that over the last decades any lateral shifting of the Jamuna River is not noticeable. This is in contrast with the behaviour of the river since the avulsion at the end of the 18th century, which was followed by a Western movement of up to 10 km in some reaches.

1. Introduction

The Jamuna River in Bangladesh is the lowest reach of the Brahmaputra River upstream of its confluence with the Ganges River (see Figure 1). The Jamuna River is a large, braiding sand-bed river. The average discharge during floods is about 50,000 m<sup>3</sup>/s, the maximum width during floods about 17 km, the bed material is quite uniform and varies from 0.25 mm near the Indian border to 0.16 mm at the confluence with Ganges River. The valley slope in Bangladesh decreases gradually from 0.10 to 0.06 m/km. More details on the river characteristics can be found in e.g. Coleman (1969) and, more recently, in RPT/NEDECO/BCL (1987), notably appendix C.

About two centuries ago the Brahmaputra River shifted from its original course (presently named the Old Brahmaputra River, see Figure 1) to its present course, now known as the Jamuna River. The reason for this avulsion is not discussed here, as it is outside the scope of this contribution. Readers interested in this phenomenon are referred to Morgan and McIntyre (1967) and Coleman (1969). Since its avulsion the Jamuna River has increased its total width and has gradually shifted in Western direction, in some reaches up to 10 km (see Coleman (1969) or IECO (1969)). Within the framework of a feasibility study for a bridge across the Jamuna river, to be located slightly downstream of Sirajganj (see Figure 1), an important issue is whether or not the river has reached an equilibrium condition in the centuries that have passed since

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the avulsion. This was especially important because the risk of outflanking of the bridge had to be evaluated as part of the feasibility study.

During this evaluation a detailed analysis was made of all the available data on the Jamuna River. In this process a number of interesting data on the characteristics of this large braiding sand-bed river became available. In this paper a summary of these results is given. Broadly speaking, a distinction can be made in planform data and cross-sectional data. They are discussed here separately in Chapters 2 and 3, respectively. Some preliminary conclusions on the river's present state are given in Chapter 4.

## 2. Planform characteristics

### 2.1 General

Planform characteristics of braided streams can be divided in:

- (i) characteristics of the network of channels and
- (ii) characteristics of the individual channels.

Howard et al (1970) have introduced a number of topological and geometrical indices for networks of braided channels. Their approach and definitions were used in the analysis of the Jamuna planform characteristics. Details are provided in Section 2.3. Planform characteristics of individual channels relate to radii of curvature of bends in relation to channel widths, the total arc length of bends and channel directions. These parameters and their correlation are discussed in Section 2.4. The data used in the analysis are described in Section 2.2.

### 2.2 Available data

For the analysis discussed here the main source of information are satellite images, notably LANDSAT-scenes. These images are available from 1972 onward. The smallest details that can be distinguished on these images have approximate dimensions of  $80 \times 80 \text{ m}^2$ . For this large river with channel widths in the order of kilometers this is quite sufficient. A selection of relevant LANDSAT-images was made. For the present study in particular the scenes 148-42 and 148-43 (for Landsat 4 and 5 138-42 and 138-43, respectively) are of relevance. Images from different years, covering the full period between 1973 and 1987, were used in the present analysis.

As the satellite images cover a rather short period only, attempts were made to use older data too. In particular old maps were consulted. Looking at the drawn channel configurations it had to be concluded, however, that most probably the maps were not based on quantitative information but should be characterized as a rough impression of what the river looked like from the banks. These maps were subsequently discarded. No older aerial photographs were obtained either, leaving the LANDSAT-images as the only source of reliable information. Figure 2 provides a print of a satellite image of the lower reaches of the Jamuna River in February 1973. Also the other selected satellite images are related to low flow conditions, allowing for a mutual comparison on realistic terms. A disadvantage of using low-flow images is that the width that can be observed on the scenes does not correspond to bankfull stages.



### 2.3 Braiding indices

Braiding indices are used to characterize channel networks of braided streams. Howard et al (1970) have proposed a number of parameters, partly from a topological and partly from a geometrical nature. In doing this, they made a distinction between channel segments, nodes where segments branch or join, and islands enclosed by segments. In the present paper the following parameters are used:

- E average number of segments bisected by the crosslines at the ends and interior of a river reach (-)
- $E_1$  excess segments index, defined via  $E_1 = E - 1$  (for a purely meandering river  $E_1 = 0$ ) (-)
- N total number of segments entirely within the river reach and entering the reach from a lower numbered reach (-)
- C average width of the stream between the outermost segments within the section (km).

Howard et al (1970) have proposed additional parameters (including average segment length, average wave length, etc.) and have also used derivative scale free parameters. Here these parameters are not used, because it is felt that in particular observed channel widths are subject to a considerable bias as they are related to low flow conditions.

For the present analysis the total length of the Brahmaputra (Jamuna) River in Bangladesh from the Indian border to the confluence with the Ganges River (see Figure 1) was divided in six reaches, each with a length of about 30 km. For each of these river reaches braiding indices were determined on the basis of LANDSAT-images. The derived indices for the parameters defined before are listed in Table 1 for four years (1973, 1978, 1981 and 1987). No data are available for the upper reaches in the years 1973 and 1978, as no satellite image was available.

The following observations can be made regarding the data provided in Table 1:

- The degree of braiding decreases in downstream direction. This may be in line with the observed decrease in valley slope (conform Leopold et al, 1967 and Struiksmā and Klaassen, 1988).
- After 1981 the degree of braiding has increased, while the total width of the braiding pattern has not changed substantially. In particular reach 4, just upstream of Sirajganj (see Figure 1), has experienced substantial changes. The 1986 data (not included in Table 1) support the change in braiding characteristics. At present there is no explanation for the change. Attempts are being made to correlate the braiding pattern to the flood discharges in previous periods.

### 2.4 Bend characteristics

The LANDSAT-images were also used to study the characteristics of bends in individual channels, being part of the braided network. Some results are provided in the Figures 3 and 4. Figure 3 related to the relation between radii of curvature, channel widths (B) and arc lengths ( $\psi$ ). A plot of radii of curvature versus channel widths for the year 1977 (see Figure 3 (a)) shows considerable scatter. In the figure also a relation derived from Leopold et al (1964) for meandering channels was plotted. This relation reads:

$$R = 2.35 B^{0.99} \quad (2.1)$$

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Leopold et al (1967) have not suggested this particular relation but it can be derived from their empirical relations between meander length, channel width and radius of curvature. The large deviation between Equation (2.1) and the present observations may be explained at least partly from the use of low flow data. As is shown in Section 3.5, the width of the channels varies with the discharge to the power of about 0.5. Average low flow discharges in the Jamuna River are a factor 10 lower than bankfull discharges (RPT/NEDECO/BCL, 1987), thus a consistent deviation of about 3 can be explained by the use of low flow data. In Figure 3(a) a corrected relation is plotted taking into account this factor 3. It may be concluded that for the present braided river the radii of curvature are on the average smaller than for meandering channels. This analysis does not take into account the possibility that the radii of curvature may change during the transition from flood to low flow stages.

A much better relationship is obtained when the arc length of the bends is plotted against the radii of curvature. For the low flow period of 1983/1984 this relationship is plotted in Figure 3 (b). From this figure it may be concluded that rather extended bends are only possible for small radii of curvature, and thus for smaller channels only (according to Equation (2.1), which does approximately hold for bankfull discharges). Thus the major channels are less curved than the smaller ones.

Channel directions for the year 1983/84 are presented in Figure 4. Here the observed channel directions (according to the definition shown in the figure) are plotted against the radii of curvature, as it was anticipated that the channel directions (expressed as the deviation from the overall valley slope) would decrease when the conveyance of the channels would increase. Taking into account the relationship between R and B (and thus the conveyance, see also Section 3.5), this is substantiated by Figure 4. This figure may have a significant bearing on the design of the river training works for the bridge. When near the bridge the different channels are combined to one major channel, the channel direction of this channel may differ only slightly from the direction of the valley slope.

### 3. Cross-sectional characteristics

#### 3.1 General

Apart from planform data, also cross-sectional characteristics of the Jamuna River were studied. This was done on the basis of soundings which have been carried out by the Bangladesh Water Development Board (BWDB) since 1966/67. The analysis carried out was aiming mainly at the identification of any consistent changes in channel location, but as side product also the variation in wet area and in conveyance in longitudinal direction and over the years was obtained. Furthermore regime equations were derived for the channels in the Jamuna River and a comparison was made with a regime equation derived for (smaller) stable channels with low sediment transport rates.

The structure of this Chapter is as follows. The available basic data (i.e. BWDB soundings) are described in Section 3.2. Data on channel conveyance and wet area and their variation in place and time are provided and discussed in Section 3.3. In Section 3.4 the horizontal and vertical stability are studied. Section 3.5 is considering the dimensions of individual channels (notably widths and average depths), and an attempt is made to correlate them to the bankfull discharge of the channels.



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The present analysis expands on recent work by Ullah (1987), who compared soundings of 1965/66 with more recent ones from 1983/83. By including soundings from other years, here a better insight is obtained into the fluctuations over the years in relation to any trends in bed levels and channel locations.

### 3.2 Available data

Cross-sectional soundings of the Jamuna River have been made since 1965/66. Initially these measurements were done every year, but they were interrupted in 1970 due to the Independence War. In 1975/76 the soundings were taken up again, although not every year the Jamuna River was included in the country-wide morphological measurements. In the period between 1965/66 and 1986/87 for most of the cross-sections in total some 10 soundings are available for analysis.

The soundings were carried out during the low flow season. Both the submerged areas and the dry islands (locally called "chars") in between the braids were sounded. The bed levels are related to the local Public Works Datum (PWD). The horizontal distance was measured from monuments. Sometimes the monuments got lost either by bank erosion or by other causes. Consequently for a number of cross-sections it is not possible to make a quantitative comparison of channel location in the horizontal direction.

Initially the distance between the cross-sections was 4 miles, but gradually more cross-sections have been introduced over the years. Figure 5 provides an overview of the sounded cross-sections.

In the present study twelve cross-sections were selected for detailed analysis considering:

- 1) conveyance
- 2) width (of channels and chars)
- 3) depth (average and maximum)
- 4) average cross-section level.

Cross-sections J5, J5-1, J6, J6-1, J8 were selected in view of their location close to the proposed bridge site (J6-1), the others to investigate possible changes in channel characteristics upstream and downstream of the bridge site. Figure 6 provides the results of soundings of two cross-sections (notably J13-1 and J6-1) for a number of years. Other cross-sections may have a total width of more than 15 km.

In a braided river the channel directions may deviate considerably from the direction of the valley slope (see Section 2.4), while furthermore this direction will change over the years. Consequently the channel width of individual channels may be too large when derived from the sounded cross-sections.

### 3.3 Processing of cross-sectional data

The cross-sectional data contain important information on channel characteristics and horizontal and vertical stability. To isolate this information, the cross-sectional data have to be processed further. A computer program was developed for this purpose. This program uses the sounded data as input, and subsequently the following processing is being done per cross-section:

- for three discharge levels (44,000 m<sup>3</sup>/s approximately corresponding to bankful stage, 12,000 m<sup>3</sup>/s and 4,000 m<sup>3</sup>/s representing average low flow conditions in the Jamuna River) stages are plotted in the sounded cross-

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- sections, and channels and sand bars (chars) are differentiated;
  - the total wet area for each of the three discharge levels is computed;
  - going from the left (Eastern) bank to the right (Western) bank monument the cumulative conveyance is computed as a function of the distance to the left bank monument; in a second step this relationship is made dimensionless by dividing it by the total conveyance for that particular cross-section and stage (for an example of the obtained results see Figure 8);
  - the width and the average levels of the individual channels in each cross-section is determined, together with the average char level, the average channel level averaged over all channels and finally the average bed level of the whole sounded cross-section, including both the parts above the water and the submerged parts.

It is of interest to study the results of the computation of the wet area and the conveyance of the river for bankfull conditions. In Figure 7 the computed values of these two parameters are plotted for some cross-section. From this figure it may be concluded that both the wet area and the conveyance do vary considerably over the years. There is no trend in the computed values, so apparently the variation is inherent to the river's behaviour. Including the direction of the different channels and thus reducing the computed values does not result in a reduction of the scatter.

It should be stressed here that the underlying assumption for the figures in Figure 7 is that the low flow cross-sections are representative for bankfull stages also. The scarce information available on changes of the cross-section through the year seems to support this assumption.

### 3.4 Horizontal and vertical stability

A major item is the stability of the Jamuna River in its present course. A distinction can be made between the horizontal and the vertical stability.

Horizontal stability is considered on the basis of the relative conveyances computed according to the method outlined in the previous section. The relative conveyances for a sequence of years were plotted for all cross-sections considered. Some examples are presented in Figure 9, notably the cross-sections J6 and J5-1. Any significant horizontal shift in river location would show up in the figure as a consistent shift in the relative conveyance in either direction. Such a trend is not detectable. The relative conveyance do show a substantial scatter, and this is due to the phenomenon that gradually minor channels become more important and vice versa. That this occurs gradually (see also Figure 6) is different from what is stated by Coleman (1969). Coleman's data relate to thalweg locations and these may vary considerably in one year, while the actual changes in depth of different channels in one cross-section may be rather small. Other analyzed cross-sections show a similar picture as Figure 9. It seems therefore that over the last two decades there is no significant shifting of the Jamuna River in either direction.

Vertical stability was studied by considering the average bed levels of the channels, the chars and of the total cross-section over the years. In Figure 10 the average bed level over the whole width is plotted for all years considered and for the different cross-sections analyzed. Substantial changes may occur over subsequent years. At least partly this may be due to the absence of a criterion as to which part of the chars (= flood plain) level should be included when computing the average bed level. However, considering Figure 10 it may be concluded that on the average there is no consistent trend in the level of the Jamuna River either. So it seems that the Jamuna River is stable in the



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vertical direction also. The changes in planform in recent years (see Section 2.3) can therefore not be explained by the advance of an alluvial fan. Summarizing, it may be stated that it seems that over the last decades the Jamuna River is stable both in the horizontal and in the lateral direction. This contradicts the findings of Ullah (1987), who based on a comparison of 1965/66 and 1983/84 data the conclusion that substantial changes have occurred. Apparently concluded is not substantiated when more years are included in the analysis.

### 3.5 Regime equations for Jamuna channels

Regime equations have originally been developed for stable canals on the Indian subcontinent. Lacey (1930, 1947) derived the following equations for the wetted perimeter  $p$  and the hydraulic radius  $R$  of a stable channel:

$$p = 2.67 Q^{1/2} \quad (3.1)$$

$$R = 0.47 \left(\frac{Q}{f}\right)^{1/3} \quad (3.2)$$

where  $Q$  = bankfull discharge and  $f$  = silt factor, defined via:

$$f = 1.59 D_{50}^{1/2} \quad (3.3)$$

where  $D_{50}$  = diameter of the bed sediment in mm. All the other parameters are expressed in imperial units. Simons and Albertson (1963) extended the regime equations to include the effect of the soil properties of the banks. For canals with sand banks and beds they found:

$$p = 3.3 Q^{0.512} \quad (3.4)$$

pointing at slightly larger widths for channels with sandy banks. Some others, e.g. Stevens (1986), have shown that the above equations hold for small sediment charges only. For larger charges the width may be larger.

It is of interest to attempt to compare the characteristics of the individual channels of the braiding Jamuna River to these regime equations. This was done for bankfull discharge (44,000 m<sup>3</sup>/s). When it is assumed that the roughness and the energy slope of each channel in a cross-section is about the same, the total discharge in a cross-section can be distributed over the channels according to their conveyance. Next, the width  $B$  and the average depth  $\bar{h}$  of the individual channels can be plotted against their discharge. The result is presented in Figure 11. The plotted data can be described by the following relations:

$$\bar{h} = 0.23 Q^{0.32} \quad (3.5)$$

$$B = 16.1 Q^{0.53} \quad (3.6)$$

Both relations are in S.I - units. The Lacey equations, expressed in these units and assuming  $p = B$  and  $R = \bar{h}$  for these very wide channels, read:

$$\bar{h} = 0.47 \left(\frac{Q}{f}\right)^{1/3} \quad (3.7)$$

$$B = 4.81 Q^{1/2} \quad (3.8)$$

As  $D_{50} \approx 0.2$  mm,  $f = 1.13$ , so Equation (3.8) can be written as:

$$\bar{h} = 0.45 Q^{1/3}$$

(3.9)

Comparison of the Equations (3.7) and (3.9) to the Jamuna Equations (3.5) and (3.6) yields the following conclusions:

- the exponent of the discharge is approximately the same in both comparable equations;
- the Jamuna channels are substantially wider and more shallow than the stable channels of Lacey; also the relatively small increase of the width indicated by Simons and Albertson (1963) is irrelevant for the Jamuna channels.

Apparently large sediment loads (in the Jamuna River up to 10,000 ppm for the total load, wash load included) result in very wide, shallow channels. The aspect ratio of the Jamuna River is about 750, which is extremely high.

Furthermore the following remarks are made:

- i The substantial scatter in Figure 11 is at least partly due to the channel direction not being perpendicular to the cross-section. However, also other factors will play a rôle.
- ii Lacey proposed a third equation for the slope of a stable channel. Introducing the relevant figures in this equation results in a slope which is 20% smaller than the actual slope of the Jamuna River.

Average depths and widths can also be derived for lower stages. For this, and also for the previous analysis, it is assumed that the cross-sections sounded during low flow conditions do not substantially differ from cross-sections for higher stages (see Figure 11). Under this assumption the average depth and width are described by:

$$\bar{h} = 0.56 Q^{0.23}$$

(3.10)

$$B = 18.9 Q^{0.51}$$

(3.11)

It is remarkable how similar these equations are to the equations which relate to bankfull discharge only (Equation (3.5) and (3.6)). Apparently the Jamuna channels can to some extent be considered as regime channels for lower stages also, possible indicating that morphological processes in the Jamuna River are relatively fast.

#### 4. Conclusions

Both planform characteristics and cross-sectional characteristics of the braiding Jamuna River in Bangladesh were studied. It seems that this river, which was subject to a major avulsion about 2 centuries ago, has stabilized in its present position. Recently, the braiding of the river seems to have increased, for which no explanation is available at present.

Radii of curvature and bend characteristics are approximately similar to meandering rivers, which may indicate that individual channels of a braiding river may to some extent be comparable to the single channel of meandering rivers.

Regime equations for individual channels of the Jamuna River deviate considerably from regime equations for sand-bed canals and rivers. A possible explanation for this may be the high sediment load of the Jamuna River.



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## 5. Acknowledgements

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| Date      |   | 21 Feb., 1973 |                       |          |           | 22 Feb., 1978 |                       |          |           | 27 Dec., 1981 |                       |          |           | 27 Feb., 1987 |                       |          |           |
|-----------|---|---------------|-----------------------|----------|-----------|---------------|-----------------------|----------|-----------|---------------|-----------------------|----------|-----------|---------------|-----------------------|----------|-----------|
| Parameter |   | E<br>(-)      | E <sub>1</sub><br>(-) | N<br>(-) | C<br>(km) | E<br>(-)      | E <sub>1</sub><br>(-) | N<br>(-) | C<br>(km) | E<br>(-)      | E <sub>1</sub><br>(-) | N<br>(-) | C<br>(km) | E<br>(-)      | E <sub>1</sub><br>(-) | N<br>(-) | C<br>(km) |
| Reach     | 1 | -             | -                     | -        | -         | -             | -                     | -        | -         | 3.2           | 2.2                   | 16       | 7.5       | 3.7           | 2.7                   | 21       | 8.0       |
|           | 2 | 3.2           | 2.2                   | 19       | 8.5       | 3.0           | 2.0                   | 20       | 8.0       | 3.3           | 2.3                   | 19       | 12.0      | 3.8           | 2.8                   | 21       | 8.0       |
|           | 3 | 3.3           | 2.3                   | 31       | 8.5       | 3.3           | 2.3                   | 19       | 8.0       | 3.5           | 2.5                   | 26       | 10.0      | 3.8           | 2.8                   | 27       | 10.0      |
|           | 4 | 2.8           | 1.8                   | 12       | 10.5      | 2.8           | 1.8                   | 16       | 8.0       | 2.9           | 1.9                   | 12       | 10.0      | 4.3           | 3.3                   | 25       | 10.0      |
|           | 5 | 1.9           | 0.9                   | 18       | 4.5       | 1.6           | 0.6                   | 9        | 5.0       | 1.7           | 0.7                   | 8        | 6.5       | 2.7           | 1.7                   | 22       | 6.0       |
|           | 6 | 1.9           | 0.9                   | 8        | 6.0       | 2.2           | 1.2                   | 14       | 6.5       | 1.8           | 0.8                   | 4        | 6.0       | 2.6           | 1.6                   | 9        | 8.0       |

Table 1 Braiding indices of the Jamuna River, Bangladesh over the period 1973-1987

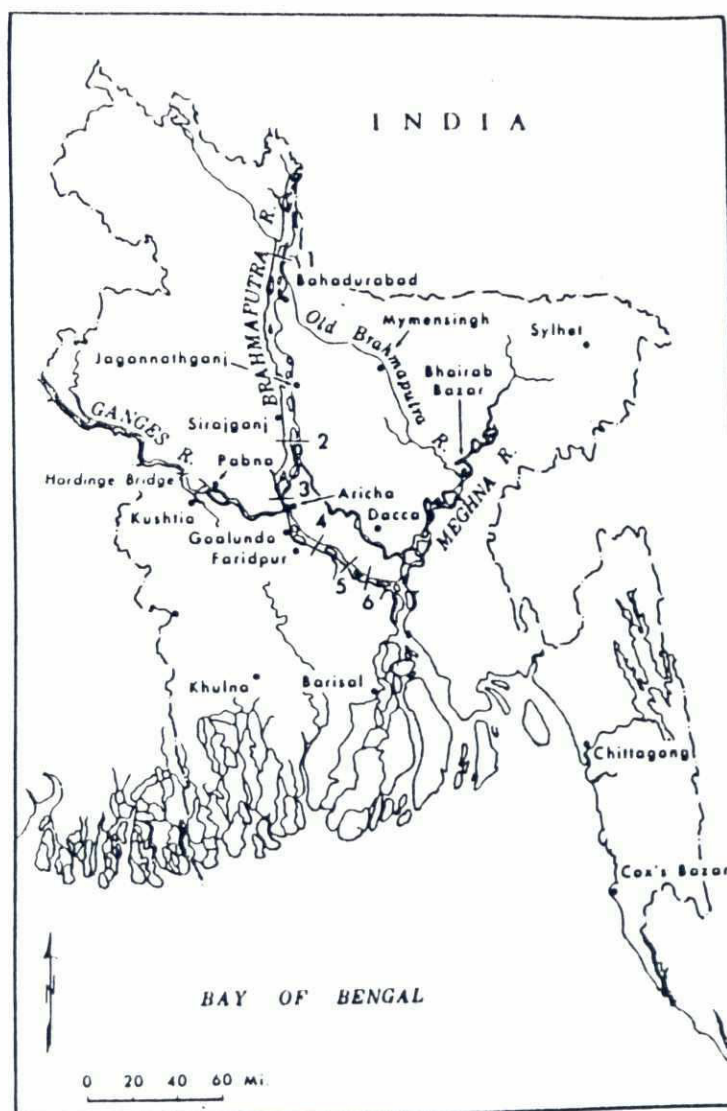


Figure 1 Index map of Bangladesh with major rivers (Coleman, 1969)



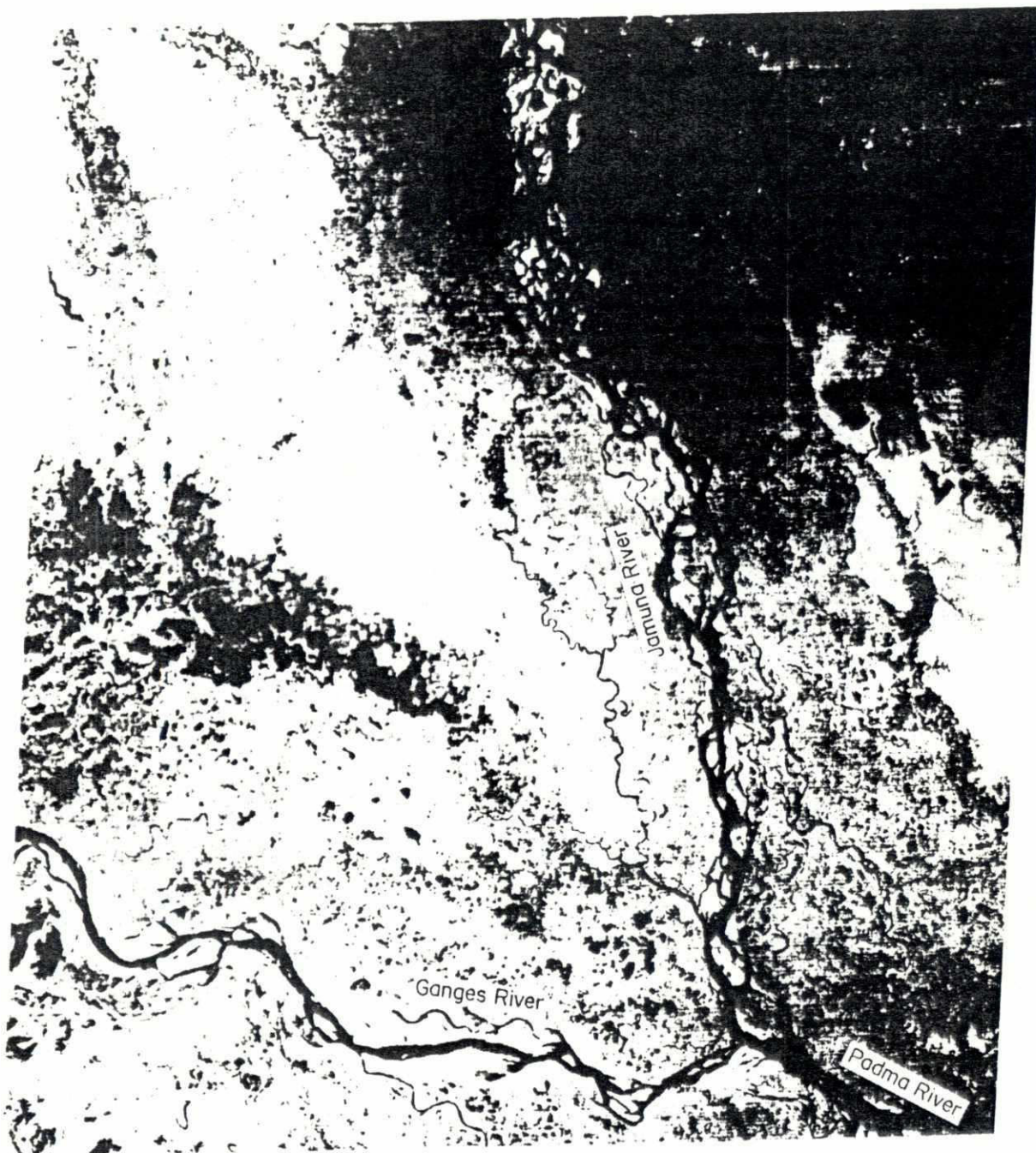
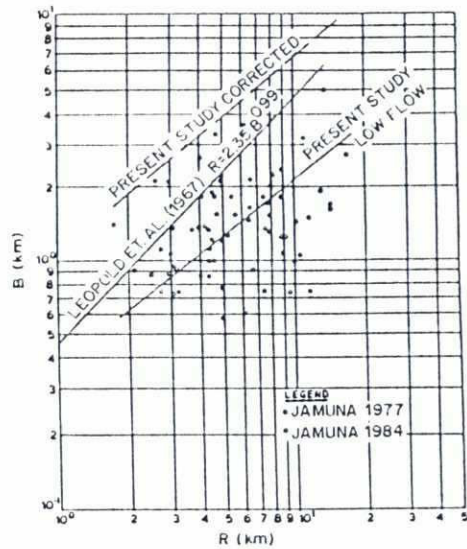
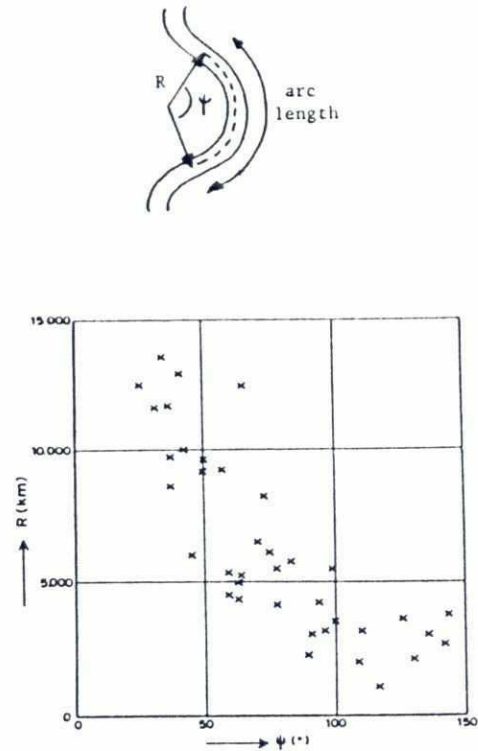


Figure 2 LANDSAT-image lower reach of the Jamuna River, date 27-2-1973  
(scale 1:1,000,000)



(a) Width versus radius of curvature for low flow data 1977



(b) Arc length versus radius of curvature for low flow data 1983/1984

Figure 3 Bend characteristics of the Jamuna River

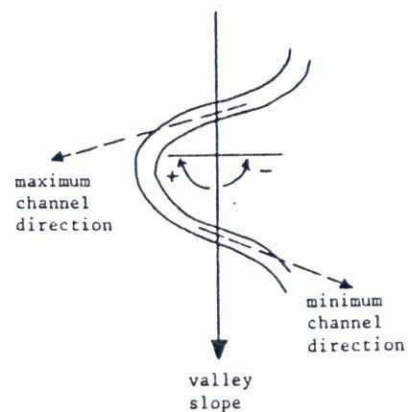
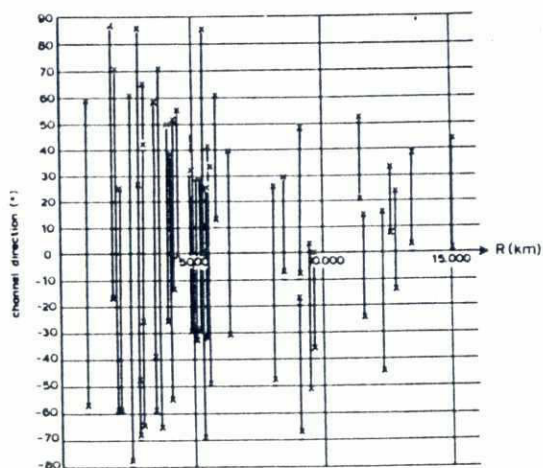


Figure 4 Channel direction versus width for the Jamuna River for the low flow season 1983/1984



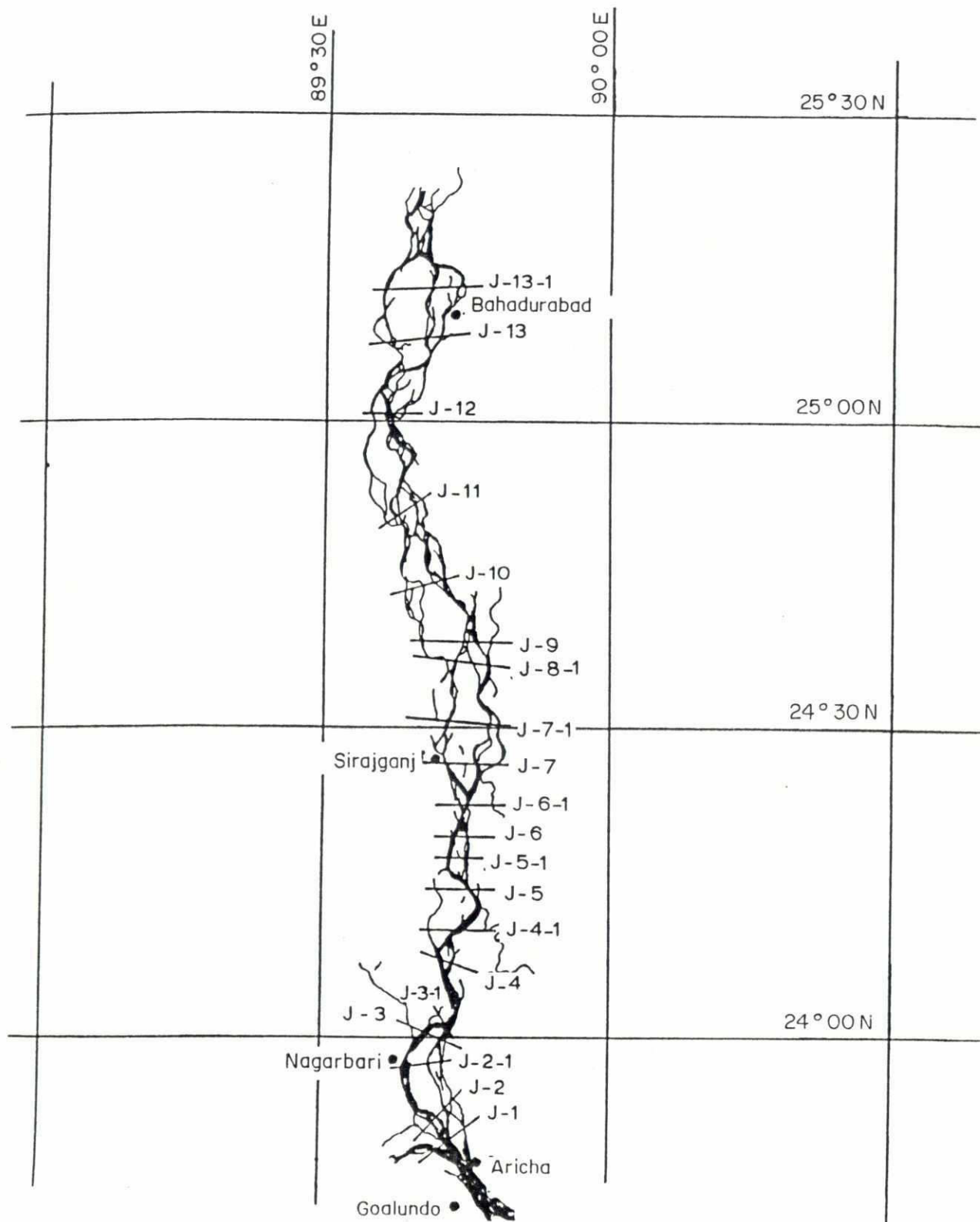
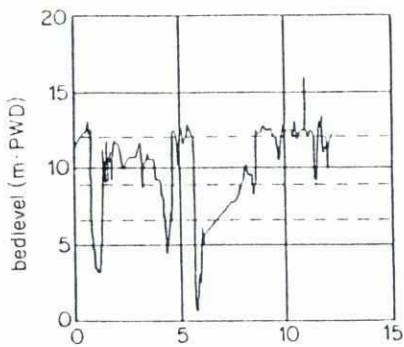
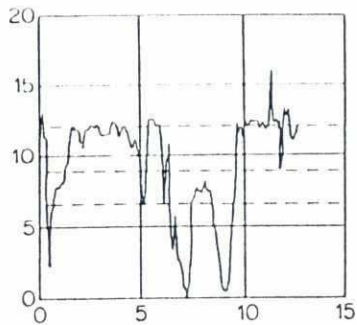


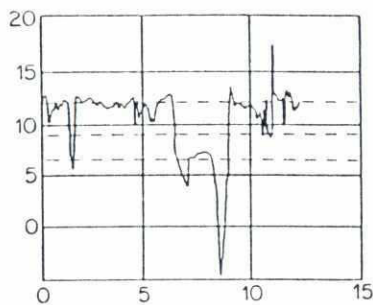
Figure 5 Cross sections measured by BWDB in the period 1966-1987



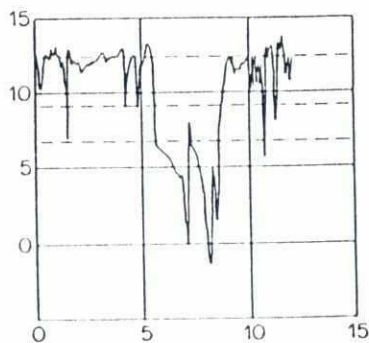
J6-1  
67/68



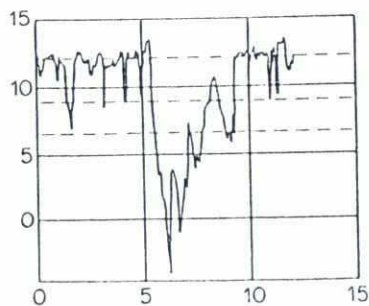
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69/70



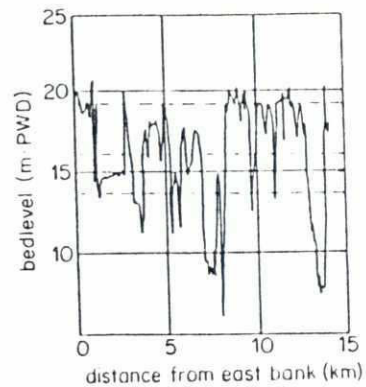
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76/77



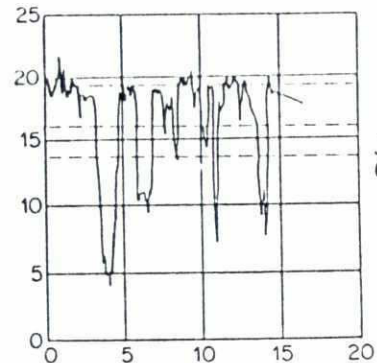
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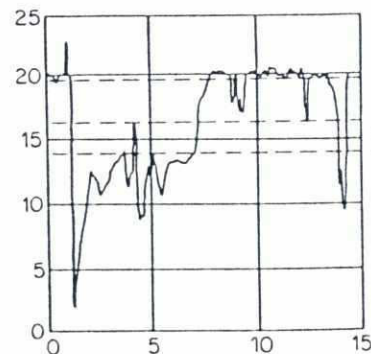
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86/87



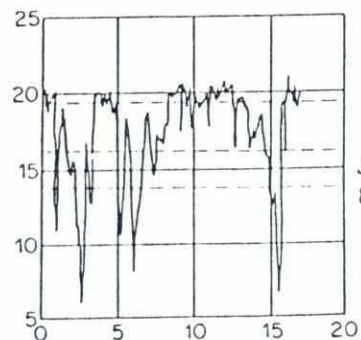
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66/67



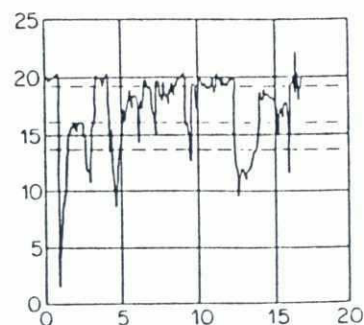
J13-1  
69/70



J13-1  
76/77



J13-1  
81/82



J13-1  
85/86

Figure 6 Two cross sections of the Jamuna River and their variation in time



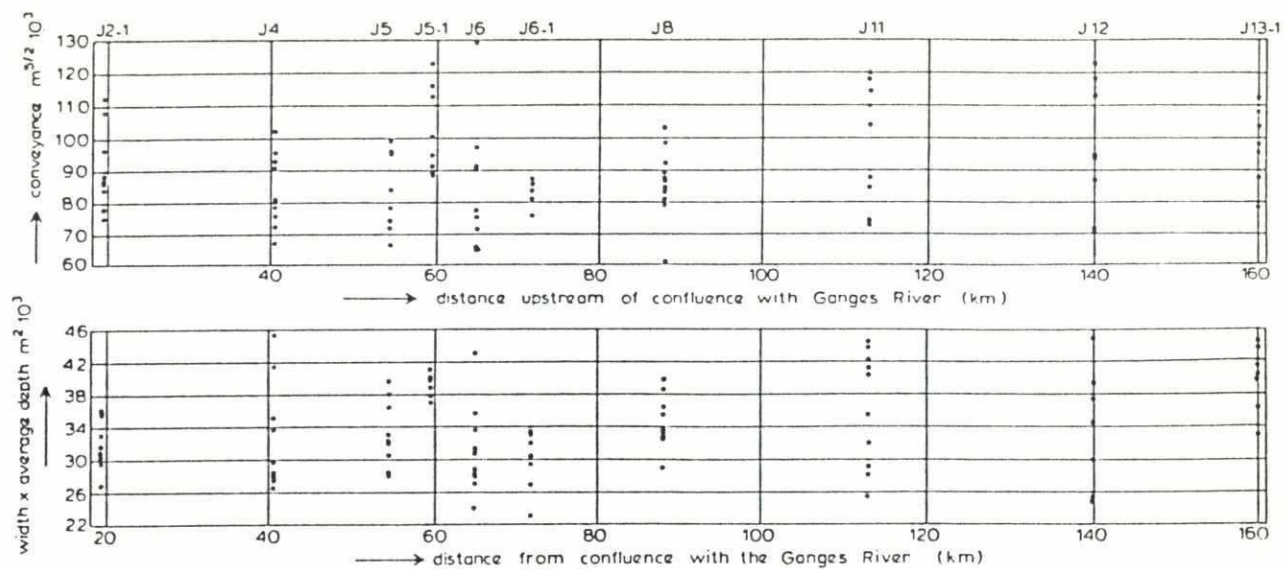


Figure 7 Variation of wet cross sectional area and conveyance along the Jamuna River.

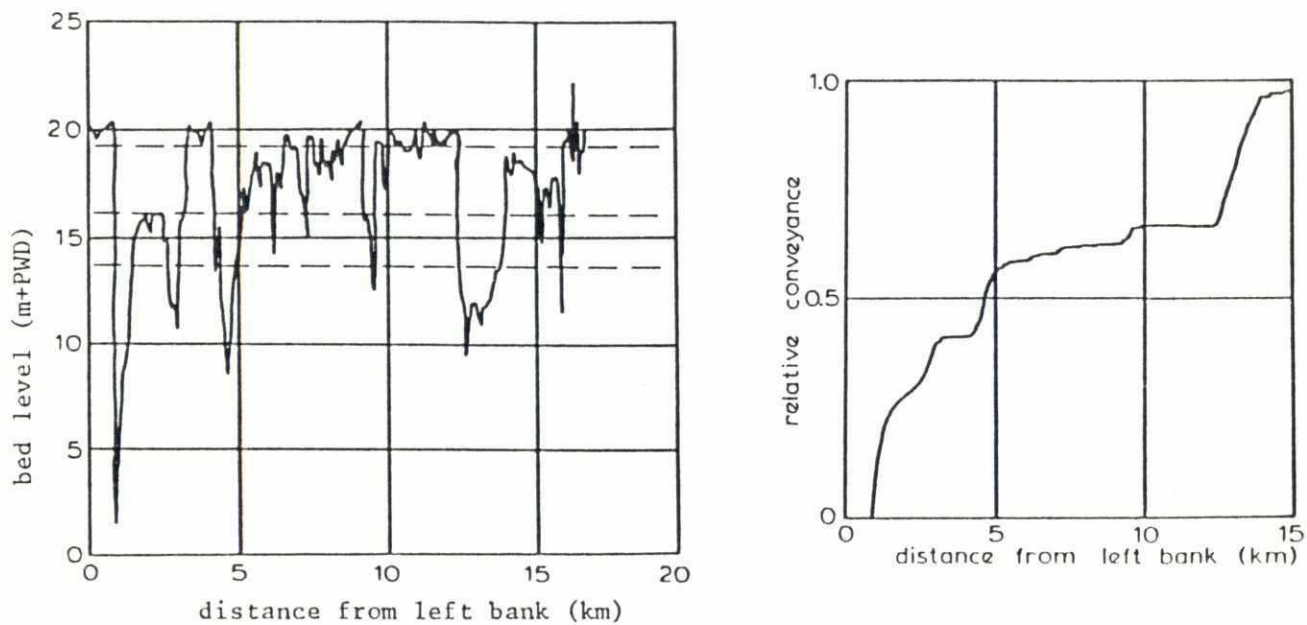


Figure 8 Cross section J13-1 in the year 1985/1986 and the relative conveyance as function of distance from the left bank.

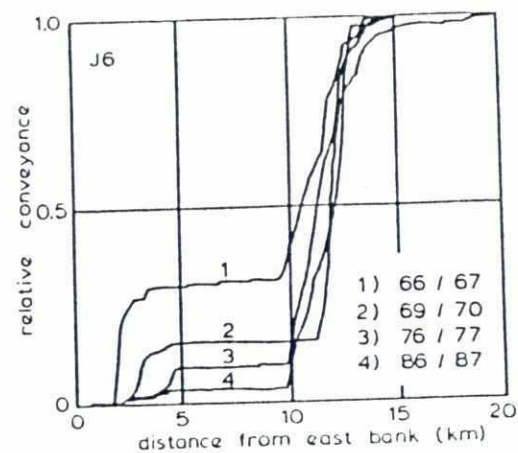
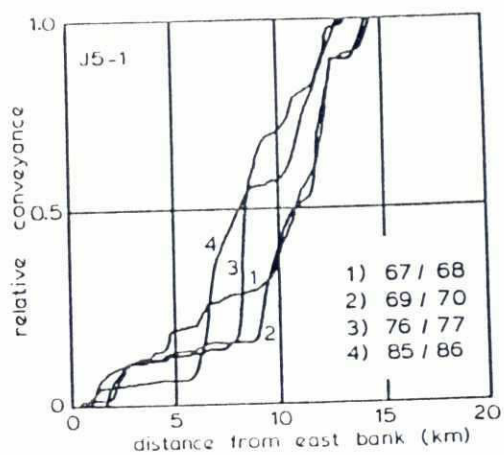


Figure 9 Two examples of the variation of the relative conveyance in time

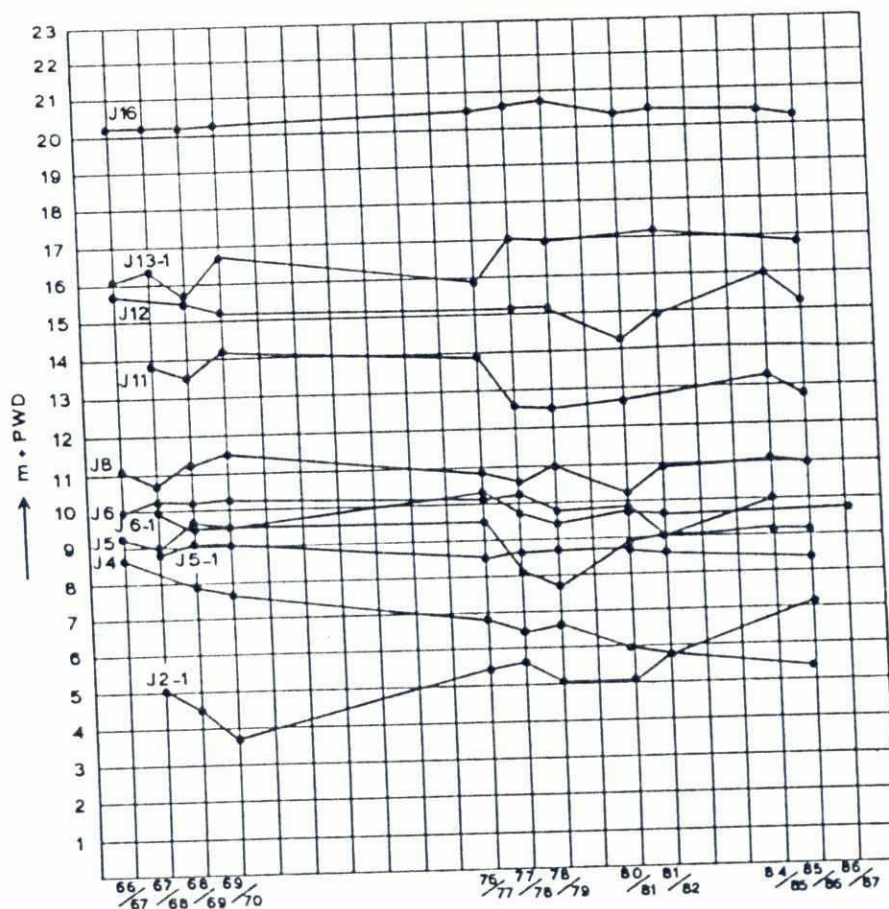
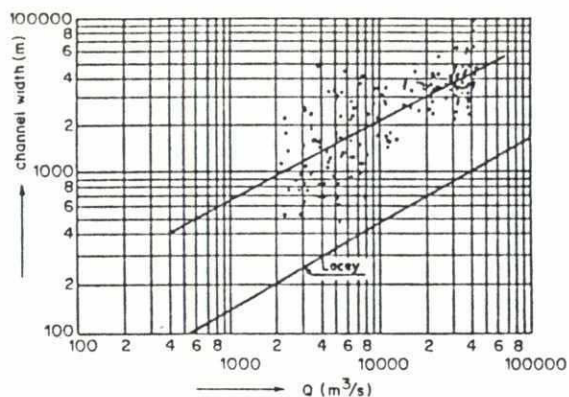
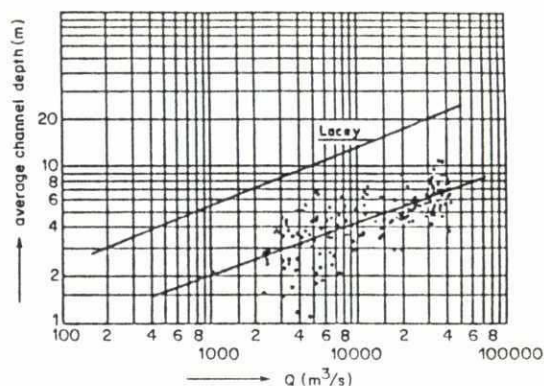


Figure 10 Average bed level of the Jamuna River cross-sections as function of time



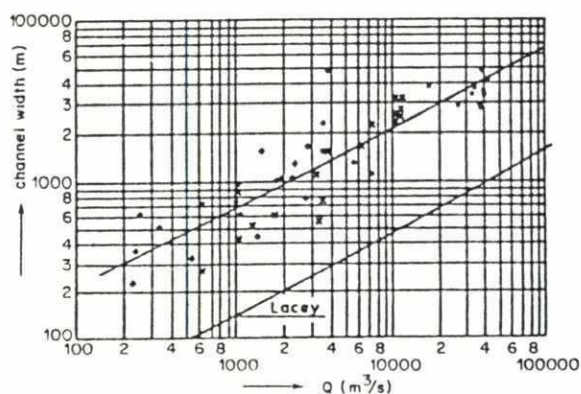


(a) width versus discharge

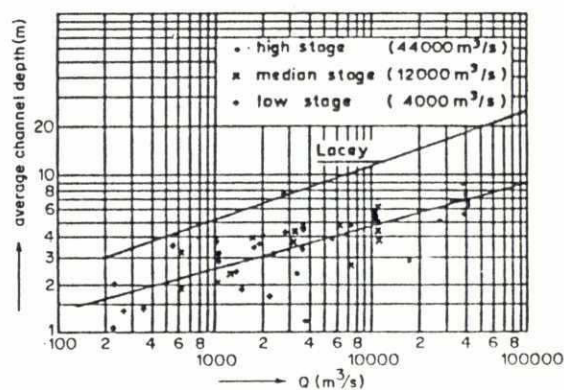


(b) average depth versus discharge

Figure 11 Width and depth of Jamuna River channels at bankful discharge (44000 m<sup>3</sup>/s)



(a) width versus discharge



(b) average depth versus discharge

Figure 12 Channel width and depth at cross section J6-1 at three defined stages

## Paper 3.6

KLAASSEN, G.J., K. VERMEER AND NAZIM UDDIN  
(1988), Sedimentological processes in the Jamuna (Lower  
Brahmaputra) River, Bangladesh, *Proc. Intern. Conf. on  
Fluvial Hydraulics, Budapest, Hungary*, 381-394pp.



1988

BUDAPEST

INTERNATIONAL CONFERENCE ON FLUVIAL HYDRAULICS

SEDIMENTOLOGICAL PROCESSES IN THE JAMUNA  
(LOWER BRAHMAPUTRA) RIVER, BANGLADESH

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**SUMMARY:** Bedform dimensions, resistance to flow of, and sediment transport in the Jamuna River in Bangladesh were studied within the framework of a feasibility study for a bridge across this large braiding sand-bed river. During this study use was made of historical data, while in 1987 also a special hydrographic survey was carried out. Results of the analysis of historical data and from this recent measuring campaign are presented here.

1. Introduction

The Government of Bangladesh has set as one of its highest priorities the integration of the Western and Eastern regions of the country by the construction of a 5 to 6 km long Multi-purpose Bridge across the Jamuna River, which bisects the country from North to South. The Jamuna Multi-purpose Bridge Authority (JMBA) has been created by the Government of Bangladesh to further the studies and construction of the Project and to assure the necessary funds for its implementation.

The World Bank as Executing Agency for the United Nations Development Programme has appointed Rendel Palmer & Tritton, NEDECO and Bangladesh Consultants Ltd. to undertake Phase II of an Appraisal Study into the proposed crossing. This study is being undertaken in 1987 and 1988. The Consultants will complete the engineering and geotechnical investigation, commenced in Phase I of the study, and will prepare the final design of the approaches and the river training works, together with designs of two river bridge alternatives. Within this frame-works studies were made of bedform dimensions resistance to flow of, and sediment transport in the Jamuna River.

The Jamuna River is the lowest reach of the Brahmaputra River in Bangladesh (see Figure 1). The Jamuna River is a large, braiding sand-bed river. The average discharge during floods is about 50,000 m<sup>3</sup>/s, the maximum width during

<sup>1), 2), 3)</sup> Senior morphologist, Morphologist, and River Engineer, respectively, in the Jamuna Bridge Phase II Study team of Rendel, Palmer and Tritton/NEDECO/Bangladesh Consultants Ltd. Joint Venture



floods is more than 15 km, the bed material is quite uniform and varies from 0.25 mm near the Indian border to 0.15 mm at the confluence with the Ganges River. The valley slope in Bangladesh decreases gradually from 0.10 to 0.06 m/km. More details on the river characteristics can be found in e.g. Coleman (1969) and more recently in RPT/NEDECO/BCL (1987), notably appendix C.

Within the framework of the feasibility study for the bridge, to be located some kilometres downstream of Sirajganj (see Figure 1), extensive river studies are being carried out. These river studies are aiming at the prediction of maximum scour depth in the Jamuna River and of the resistance to flow. The maximum scour depth is of importance to determine the foundation depth of the bridge piers, guide bunds and other river training works related to the proposed bridge. The resistance to flow, especially during flood conditions, is important for the prediction of backwater effects, resulting from the local constriction of the river at the bridge site. Constriction scour will develop during the flood season, because the span of the bridge will be considerably smaller than the normal river width during the flood. The sediment transport rate determines the rate at which constriction scour develops in time during the passage of floods, and this is, in addition to the channel's resistance to flow, a determining factor for the resulting backwater and also for the maximum scour that may occur. Therefore, sediment transport rates are also being studied.

The present paper discusses the resistance to flow, bedform dimensions and sediment transport. They are discussed here in Chapter 3, 4 and 5 respectively. The available historical data and the special field measurements carried out in 1987, are described in Chapter 2. Some preliminary conclusions are drawn in Chapter 6. When the collected data have been analyzed in more detail, more definitive conclusions can be formulated. Other main items during the total study are channel characteristics of the Jamuna River (Klaassen & Vermeer, 1988a), scour in bends, local scour and scour at confluences (for the latter see Klaassen & Vermeer, 1988b), changes in channel pattern and resulting bank erosion (see Burger et al, 1987) and constriction scour in braided rivers (Klaassen & Struiksma, 1988).

## 2. Data used

The data used in the present study can be divided in:

- historical data;
- data collected during a special field survey in 1987.

The historical data comprise the following items:

- Sounding maps from the Bangladesh Inland Water Transport Authority (BIWTA). The reason for preparing these maps is to identify the channels with maximum depths for navigational purpose (including ferrying). In the study for the Jamuna Bridge these maps were mainly used to determine local scour, bend scour and confluence scour (Klaassen & Vermeer 1988b).
- Cross-sectional soundings of the Jamuna River carried out by the Bangladesh Water Development Board (BWDB) since 1966. Originally cross-sections were sounded every 4 miles during low flow. The distance between cross-sections is now reduced to 1 mile. These soundings were used to analyse channel stability (Klaassen & Vermeer 1988a) and to derive equations for water depth/discharge and channel width/discharge relations.
- Sediment transport measurements carried out by BWDB at Bahadurabad since 1966. These data are used to determine the sediment transport as function of discharge.

Most of the time the soundings by BIWTA and the cross-sectional soundings by BWDB were carried out during low flow conditions. They may not be representative for flood conditions. Therefore, special measurements were carried out



during the flood season. Although some bedform data were collected already during the flood period in 1986, the bulk of the hydrographic measurements for the present study was done in 1987.

Figure 2 shows an overview of the river reaches where measurements were carried out. Initially it was intended that measurements would be done three times at each location, but owing to the extreme flooding in Bangladesh in 1987 the measuring programme had to be reduced. The bulk of the measurements consisted of soundings, both longitudinal and cross-sectional. The soundings were done with echosounders. For the determination of the position of the ships used and for establishing the track of the ships while sounding, Tri-sponder equipment was used in addition to the existing Decca chain in Bangladesh. The accuracy of the Tri-sponder equipment is in the order of 1 m. However, owing to the absence of a feedback system it was not possible to sail along exactly the same track during repeated measurements. This affects the accuracy and applicability of the dune-tracking method to determine the bed load transport. The results of the measurements were used to determine bedform dimensions, local scour, bend scour and confluence scour during flood conditions.

### 3. Resistance to flow

As stated in Chapter 1, in the present study the resistance to flow of the Jamuna River is important for the prediction of the backwater effects resulting from the river width constriction at the bridge site. In particular, more insight is required in the suspected reduction of resistance to flow during flood conditions. Such a reduction was found by Stevens & Simons (1973) for the Padma River, the continuation of the Jamuna river after the confluence with the Ganges River. This reduction of resistance to flow can be caused by the reduction of bedform dimensions and transition to flat bed situation. Bedform dimensions are discussed in Chapter 3. In this chapter, the resistance to flow of the Jamuna River is determined in two ways, using historical data on the Jamuna River:

- via discharge measurements (local average water depth and flow velocity in combination with measured slope over a larger distance) and,
- via average at-a-station-relationships between average depth and discharge, and between channel width and discharge.

For the first method, the resistance to flow, as represented by the Chézy coefficient:

$$C = Q/B (h)^{3/2} i^{1/2}, \quad (1)$$

in which  $Q$  = discharge [ $m^3/s$ ],  $B$  = stream width [ $m$ ],  $\bar{h}$  = average depth [ $m$ ],  $i$  = slope [-], is computed, using data presented by Uddin (1985). The data were derived from BWDB (Bangladesh Water Development Board) discharge measurements. The results for the station Sirajganj are given in Figure 3. From these data it is concluded that Chézy values vary between 40  $m^{1/2}/s$  for low flow and 100  $m^{1/2}/s$  for flood conditions. The very high Chézy values for flood conditions indicate a transition to flat bed for high discharge. However, during the (very high) flood of 1987 a flat bed situation was only observed some two months after the peak flood and not during the peak flood itself (see Chapter 4). There is of course the possibility that the large Chézy values are the result of a combination of local and general river data. Furthermore, it is interesting to note that a clear lagging behind can be observed: during the rising stage the river bed is smoother than during the recession of the flood. The second method is based on the analysis of cross-sectional data, measured by the Bangladesh Water Development Board (BWDB). This analysis aimed primarily at the derivation of regime and at-a-station-relationship equations for the Jamuna River, (see Klaassen & Vermeer, 1988a). With these average at-

a-station equations an estimation of the Chézy coefficient can be obtained in the following way:

1) the general expression for the equations reads:

$$B = a Q^b, \quad (2)$$

$$\bar{h} = c Q^d, \quad (3)$$

in which a, b, c and d are calibrated parameters.

2) Insert Equations (2) and (3) in Equation (1), yielding:

$$C = \frac{Q^{1-b-1.5d}}{a c^{3/2} i^{1/2}} \quad (4)$$

For the Jamuna River the following approximate values for the parameters were found (Klaassen & Vermeer, 1988a):  $a = 18.9$ ,  $b = 0.51$ ,  $c = 0.56$ ,  $d = 0.23$  yielding the following expression for the Chézy coefficient of the Jamuna River:

$$C = \frac{0.126 Q^{0.14}}{i^{1/2}} \quad (5)$$

For an assumed slope of  $i = 7.10^{-5}$ , this yields a Chézy coefficient varying between  $47 \text{ m}^2/\text{s}$  at a discharge of  $4000 \text{ m}^3/\text{s}$  and a Chézy coefficient of  $67 \text{ m}^2/\text{s}$  at a discharge of  $44,000 \text{ m}^3/\text{s}$ , (see also Figure 3) which is approximately bankfull discharge.

The results of the latter analysis seems to yield more realistic Chézy coefficients. In any case, also this approach points at a substantial increase in Chézy coefficients for higher discharges.

#### 4. Bedform dimensions

Bedform dimensions do to a large extent determine the resistance to flow of an alluvial river. Bedform roughness is caused by the energy losses of the flow behind the crest of a bedform. According to the literature the bedforms disappear for increasing Shields parameter. This is often attributed to a decrease of the heights of the bedforms. However, during recent experiments in flumes (see Klaassen et al, 1986) it was found that  $H/L \rightarrow 0$  ( $H$  = bedform height,  $L$  = bedform length) for increasing Shields parameter owing to increasing bedform length rather than decreasing bedform height.

Data on bedforms in the Jamuna River were first provided by Coleman (1969). Based on an extensive study he distinguished the following bedforms:

- 'ripples': typical heights 0.2 to 0.5 m (probably not relevant for the present study),
- 'mega ripples': typical height 1.0 m, typical propagation velocity 120 m/day,
- 'dunes': typical height 5 m, typical celerity 60 m/day, and
- 'sand waves': typical height 10 m, typical celerity 200 m/day.

More recently Bristow (1985) presented some data on bedform heights observed during low flow conditions. However, for high flow conditions only limited data on bedform dimensions in the Jamuna River are available. Therefore, special measurements were carried out for the present study.

During flood conditions in 1986 and 1987, bedform dimensions were measured in the Jamuna River. This was done for two reasons:



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- to study the relation between bedform dimensions and flow characteristics of the Jamuna River and, in particular, to check whether a reduction of bedform height occurs during flood conditions; and
- to determine a lowest bed level, owing to passing bedform dimensions and local scour for the design of the river training works and for the bridge piers of the Jamuna Bridge.

The hydrographic survey in 1987 was carried out between July and November. At some locations two measurements were carried out in this period. This means, that the change of the bedform dimensions could be analysed as a function of the stage of the flood season. In Table 1 the soundings used for the bedform analysis are given. The water levels on the dates of the soundings are given, too. The maximum water level was reached in the second half of August. During the last measurements the water level had already dropped 4 m compared to this maximum flood level.

The following observations could be made after studying the obtained bedform data:

- i) During the measurements obtained in July and August, relatively large dunes were abundantly present according to all soundings, some of them having small dunes superimposed on them, (see Figure 4.a)
- ii) In the following month (September), so after the peak discharge in the second half of August, the large dunes are not reduced in height and their lengths have not increased. However, the small, superimposed dunes have disappeared (see Figure 4.b).
- iii) In the second half of October and in November the large dunes have disappeared as well as flat bed situations although some can still be observed on nearly all soundings (see Figure 4.c).

So, when the peak discharge is reached, large dunes, having average height  $H \approx 3.0$  m and average length  $L \approx 200$  m (based on the sounding near Belkuchi on the 22 August 1987) are still present, with small dunes superimposed with average height  $H \approx 1.1$  m, and average length  $L \approx 24$  m. This seems in contradiction with the observation that resistance to flow of the Jamuna River is very low during the peak discharge. The washing out of the smaller superimposed dunes and the large dunes takes place in a later stage of the flood season only.

The bedform roughness is determined to a large extent by the slope of the lee side of the bedforms. It was investigated whether these slopes are so gentle that they hardly contribute to the resistance to flow reduction if the steepness could be identified. It appears, that the bedforms in the Jamuna River have a very gentle slope ( $\alpha > \pm 10$ ) for all measurements. This holds for the smaller superimposed dunes, too. It has been shown (Ogink, 1987) that gentle slopes reduce the influence of the bedform roughness on the resistance substantially (see also Klaassen et al, 1986). For slopes of 1:10 the bedform roughness is reduced to approximately 20%. In that case the Chézy value can be estimated with:

$$f = f' + 0.2 f'',$$

$$\text{or } \frac{1}{C^2} = \frac{1}{(C')^2} + \frac{0.2}{(C'')^2} \quad (6a)$$

where

$$C' = 18 \log \frac{12R}{k_s} \quad (6b)$$

with  $f$  = total friction coefficient (-),  $f'$  = friction due to particle roughness (-),  $f''$  = friction due to bedforms,  $C$  = Chézy value ( $m^{2/3}/s$ ),  $C'$  = Chézy value due to particle roughness ( $m^{2/3}/s$ ),  $C''$  = Chézy value due to bedforms ( $m^{2/3}/s$ ),  $R$  = hydraulic radius (m) and  $k_s$  = specific grain roughness (m). When (i) for  $R$  the average depth is taken, which is appropriate in this case of



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rivers with a large width/depth ratio, (ii)  $k_s$  is represented by  $3.D_{90}$  (Van Rijn 1984) (and  $3.D_{90}$  is approximated by  $2.D_{50}$ ) the Chézy value is about  $70 \text{ m}^{1/2}/\text{s}$ , taking  $0.2 \text{ mm}$  for  $D_{50}$ . This Chézy value is close to the Chézy values based on analysis of the cross-sections. Based on the present observations the conclusion may be drawn that bedform roughness plays only a limited role in the total resistance to flow of the Jamuna River during floods.

The other reason for the present study of bedforms is the determination of scour depth, related to bedforms. The trough of the bedform is below the average bed level. Therefore, the dune height was investigated in relation with the water depth. A graph was made to investigate the dune height versus water depth relation (see Figure 5). Maximum dune heights of  $6 \text{ m}$  were observed. This is not in line with observations by Coleman (1969) who discovered bedform heights up to  $15 \text{ m}$ . It seems possible that Coleman interpreted confluence scour erroneously as bedforms.

The relation  $H/\bar{h}$  versus  $\bar{h}$  was also investigated ( $\bar{h}$  = average waterdepth). For  $H$  the average dune heights of reaches with approximately equal dune heights were taken. It appears, that the  $H/\bar{h}$  ratio can be up to about  $0.35$  (see Figure 6), but no relation with the water depth was found. Dune height prediction models like the ones provided by Yalin (1964) and Allen (1968) seem not applicable for the Jamuna River, especially not for larger waterdepths. As a reduction of the  $H/\bar{h}$  ratio for increasing waterdepth could not be shown, a maximum dune height of  $0.35 \times$  average waterdepth should be taken into account for the maximum scour depth related to bedforms.

Finally the ratio  $L/\bar{h}$  versus  $\bar{h}$  was investigated. This ratio was always smaller than  $20$  and usually between  $5$  and  $10$  for waterdepths varying from  $9$  to  $17 \text{ m}$ . For the  $L/\bar{h}$  ratio a dependence of the average depth can not be shown either.

## 5. Sediment transport

Sediment transport by the Jamuna River is quite large. Coleman (1969) already gave some figures based on suspended load measurements at Bahadurabad during the period 1958-1962. Average yearly sediment transport is about  $600$  million tons. This is in line with figures based on the same data provided by Latif (1969). The preceding data relate to total load, both wash load and bed material load.

Here, the bed material load should be considered because only the bed material load is important for morphological changes. Data on suspended bed material load are obtained from BWDB measurements of sediment transport ("coarse sediment transport", whereby the wash load was excluded). Data referring to the period 1968-1970 are presented in Figure 7. According to the data of 1968-1970 the suspended load can be represented by the function:

$$S_J = 4.1 \cdot 10^{-6} Q^{1.38} \quad \text{m}^3/\text{s} \quad (7)$$

The magnitude of the bed load in the Jamuna River was estimated by doing dune tracking measurements during the 1987 flood season. During these measurements a Trisponder positioning system was used. The accuracy of the measurements was limited, but an estimate of the bed load transport could be obtained. Consequently, for the data obtained, the bed load is approximately  $10\%$  of the suspended bed material transport, indicating that suspension is the most important mode of transport. The value of  $10\%$  corresponds nicely to a prediction by Van Rijn (1987). Thus, the total bed material load of the Jamuna River can be approximated by:

$$S_J = 4.5 \cdot 10^{-6} Q^{1.38} \quad \text{m}^3/\text{s} \quad (8)$$



This transport formula is compared with Engelund/Hansen (1967) transport formula. To define total channel width and average depth, the derived average at-a-station equations are used (see Chapter 4). The Engelund/Hansen transport formula can be written as:

$$S_{EH} = \frac{0.01}{D_{50}} i^{3/2} \frac{1}{Bh^{1/2}} Q^2 \quad (9)$$

Using the equations given in Chapter 4 and taking  $D_{50} = 0.18$  mm and  $i = 7.10^{-5}$  it was found that the ratio  $S_{EH}/S_J$  is approximately 0.5.

This ratio appears not to be dependent on the discharge. Sediment transport measurements of the period 1968-1970 are therefore reasonably well represented by the Engelund/Hansen formula multiplied by 2, when channel dimensions are determined by analysis of the measured BWDB cross-sections.

## 6. Conclusions

Bedform, resistance to flow and sediment transport data of the Jamuna River in Bangladesh were analyzed.

Resistance to flow of the Jamuna River for low flow conditions is comparable to other rivers, while for flood condition a rather low resistance to flow was observed. This is in line with the conclusions of Stevens & Simons (1973) for the Padma River. Dune heights appear to be larger than current prediction models indicate, especially for larger water depths. During flood conditions, resistance to flow owing to particle roughness, seems to be in the same order of magnitude as the form roughness, the bedforms having such gentle lee side slopes that they hardly contribute to form roughness.

Sediment transport on the Jamuna River is quite large, especially at high discharge. The Engelund/Hansen transport formula multiplied by 2, gives a fair prediction of the bed material transport in the Jamuna River.

## 7. Acknowledgements

The present studies were carried out within the framework of the Jamuna Bridge Appraisal Study. The present study is carried out by Rendel, Palmer and Tritton, NEDECO and Bangladesh Consultants Ltd. Mr. T. Swanenberg assisted in the analysis of the bedform measurements and the dune tracking. Publication of this paper has been kindly permitted by both World Bank and Jamuna Multi-purpose Bridge Authority.

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| Location  | Data     | Water level<br>at Sirajganj | $\bar{h}$ | Large Dunes |           |                | Small<br>Superimposed |           |                | Remarks              |
|-----------|----------|-----------------------------|-----------|-------------|-----------|----------------|-----------------------|-----------|----------------|----------------------|
|           |          |                             |           | $\bar{H}$   | $\bar{L}$ | $\bar{\alpha}$ | $\bar{H}$             | $\bar{L}$ | $\bar{\alpha}$ |                      |
| Nagarbari | 27-07-87 | 13.48                       | 10.8      | 2.8         | 80        | 11             | N.A.                  | N.A.      | N.A.           | flat bed predominant |
| Nagarbari | 20-11-87 | 9.30                        | -         | N.A.        | N.A.      | N.A.           | N.A.                  | N.A.      | N.A.           |                      |
| Belkuchi  | 22-08-87 | 14.00                       | 18.1      | 3.0         | 220       | 32             | 1.1                   | 24        | 10             | flat bed predominant |
| Belkuchi  | 17-11-87 | 9.46                        | -         | 1.2         | 75        | 30             | N.A.                  | N.A.      | N.A.           |                      |
| Sirajganj | 11-09-87 | 13.82                       | 11.7      | 2.4         | 70        | 10             | N.A.                  | N.A.      | N.A.           | flat bed predominant |
| Sirajgang | 11-10-87 | 12.33                       | 12.7      | 2.4         | 50        | 7              | N.A.                  | N.A.      | N.A.           |                      |

$\alpha$  = slope of the dune lee side

Table 1 Bedforms of the Jamuna River

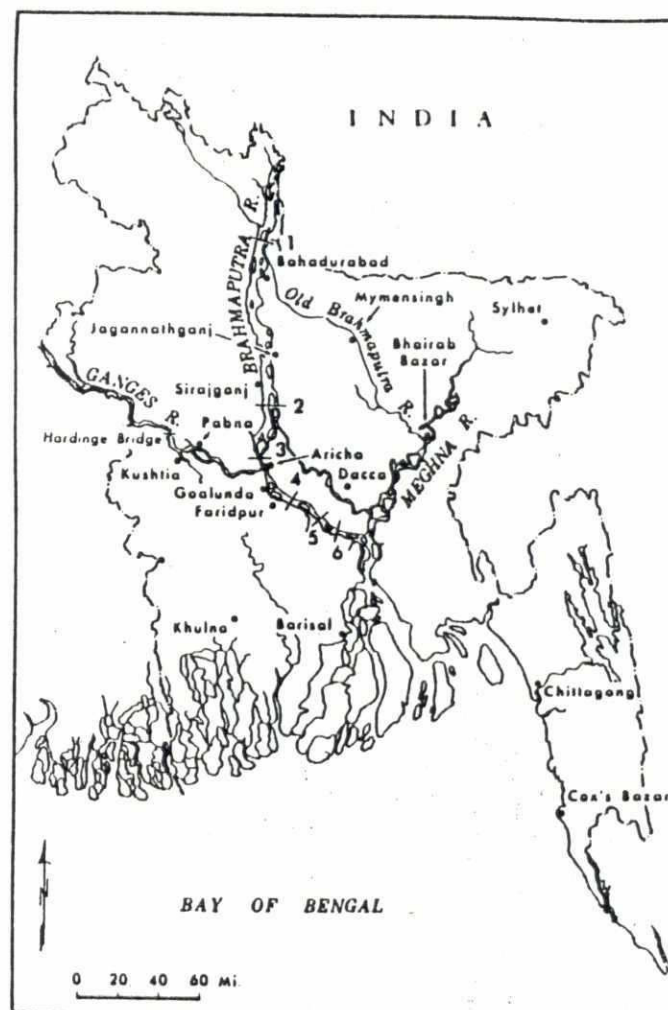


Figure 1 Index map of Bangladesh with major rivers (Coleman, 1969)

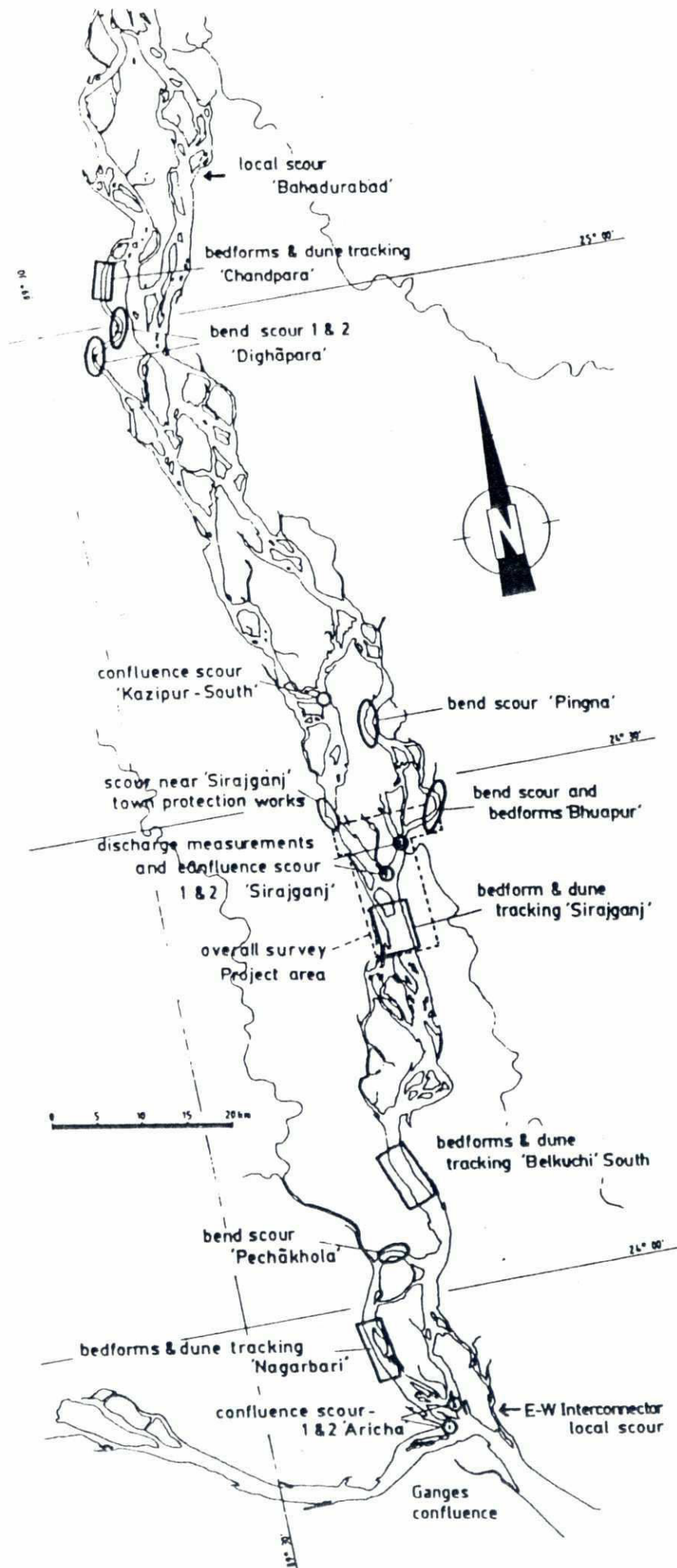


Figure 2 Location of special surveys 1987 on the Jamuna River



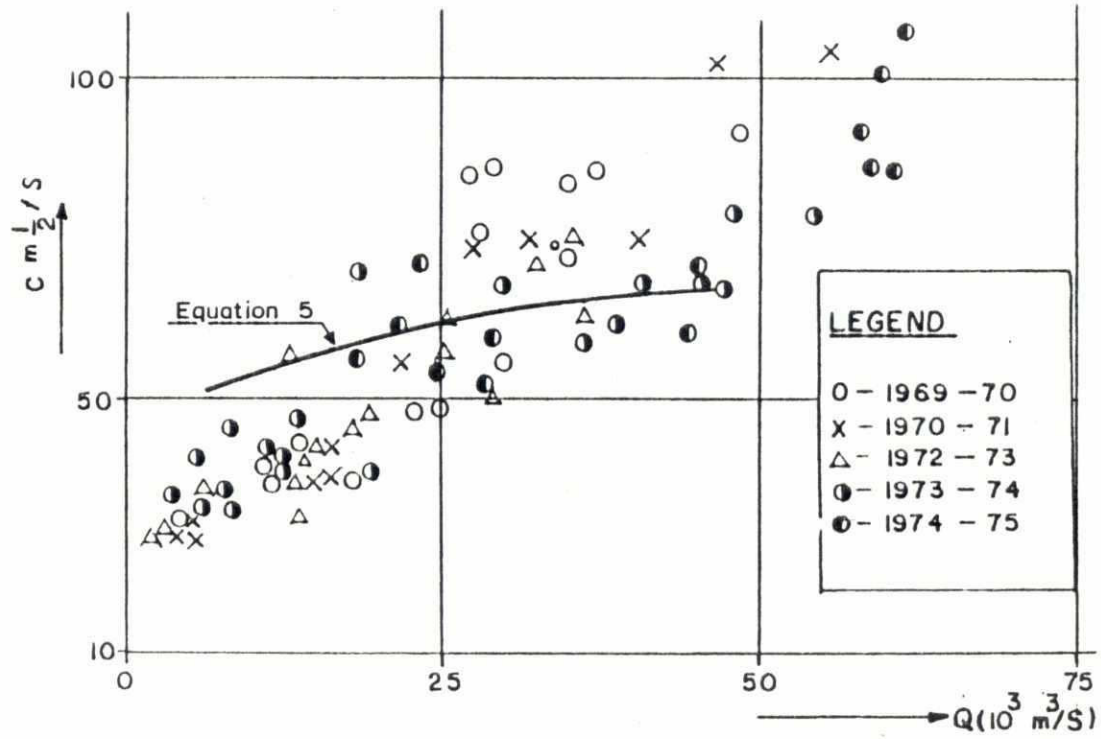
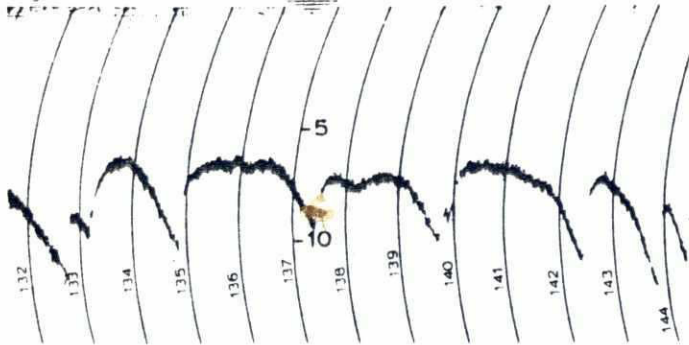
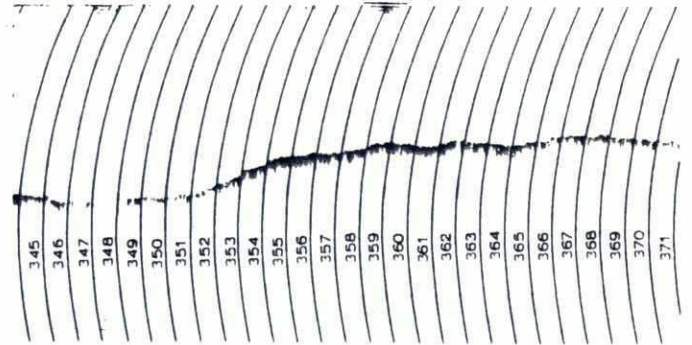


Figure 3 Resistance to flow Jamuna River at Sirajganj

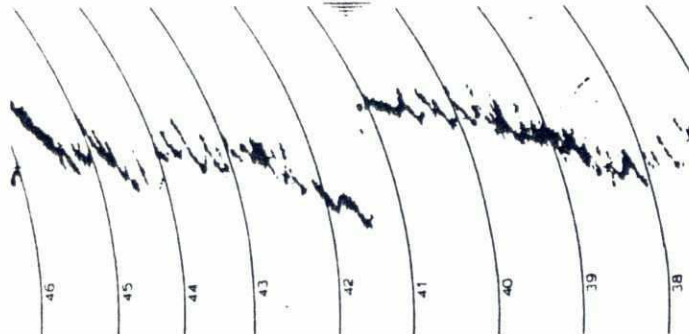
Nagarbari 27-7-87



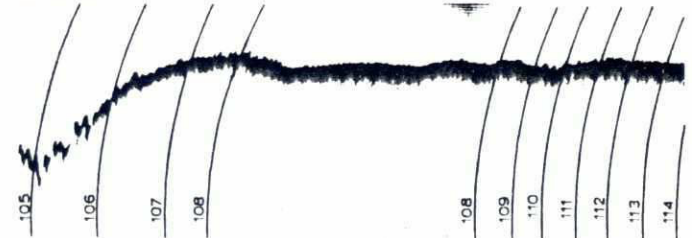
Sirajganj 11-10-87



Belkuchi 22-8-87

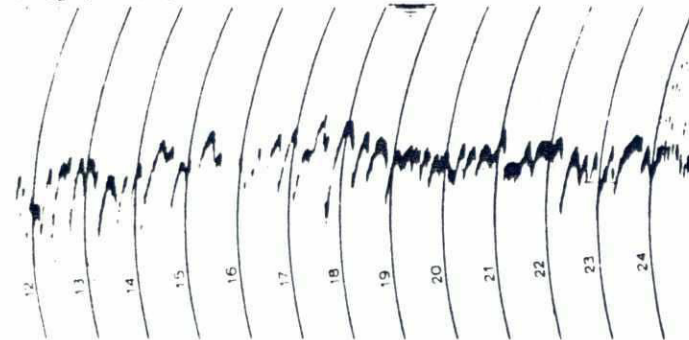


Belkuchi 17-11-87

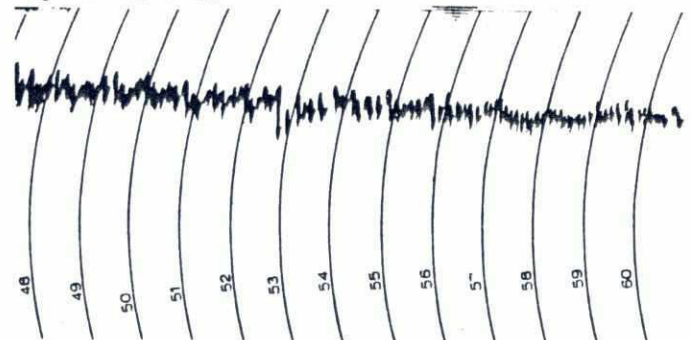


(a) July / August , 1987

Sirajganj 11-9-87



Nagarbari 20-11-87



(b) September , 1987

(c) October / November , 1987

Figure 4 Bedforms in the Jamuna River observed during special field survey in 1987 (vertical scale  $\pm 1:500$ )



70 y

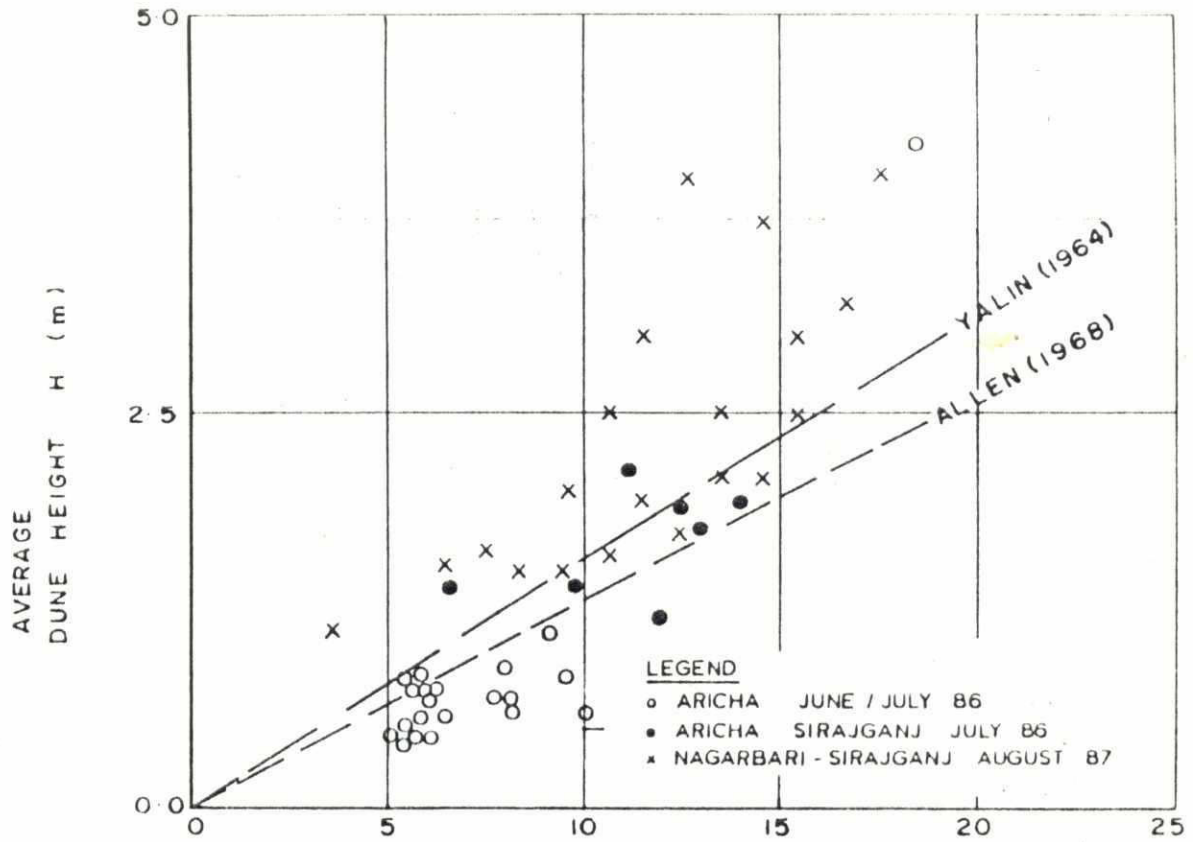


Figure 5 Average dune height versus waterdepth

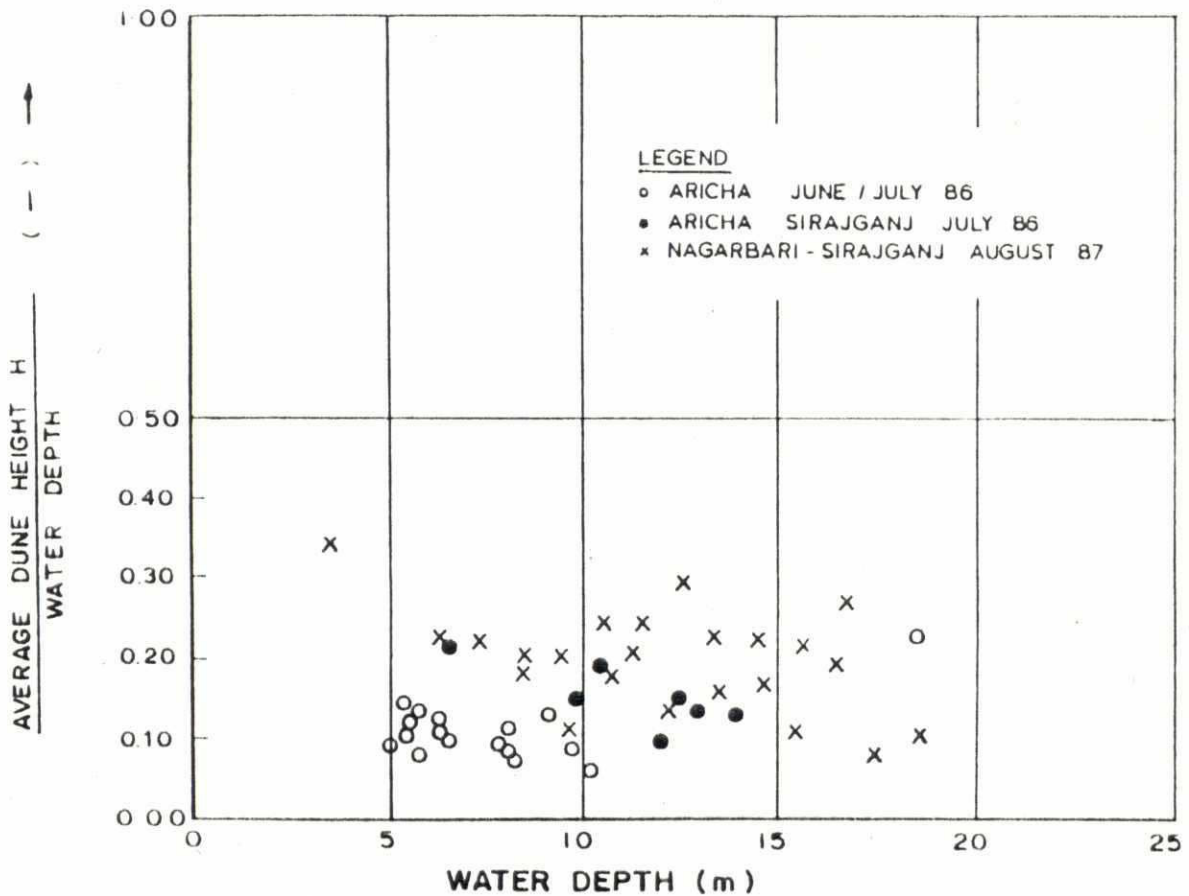


Figure 6 Relative dune height versus waterdepth

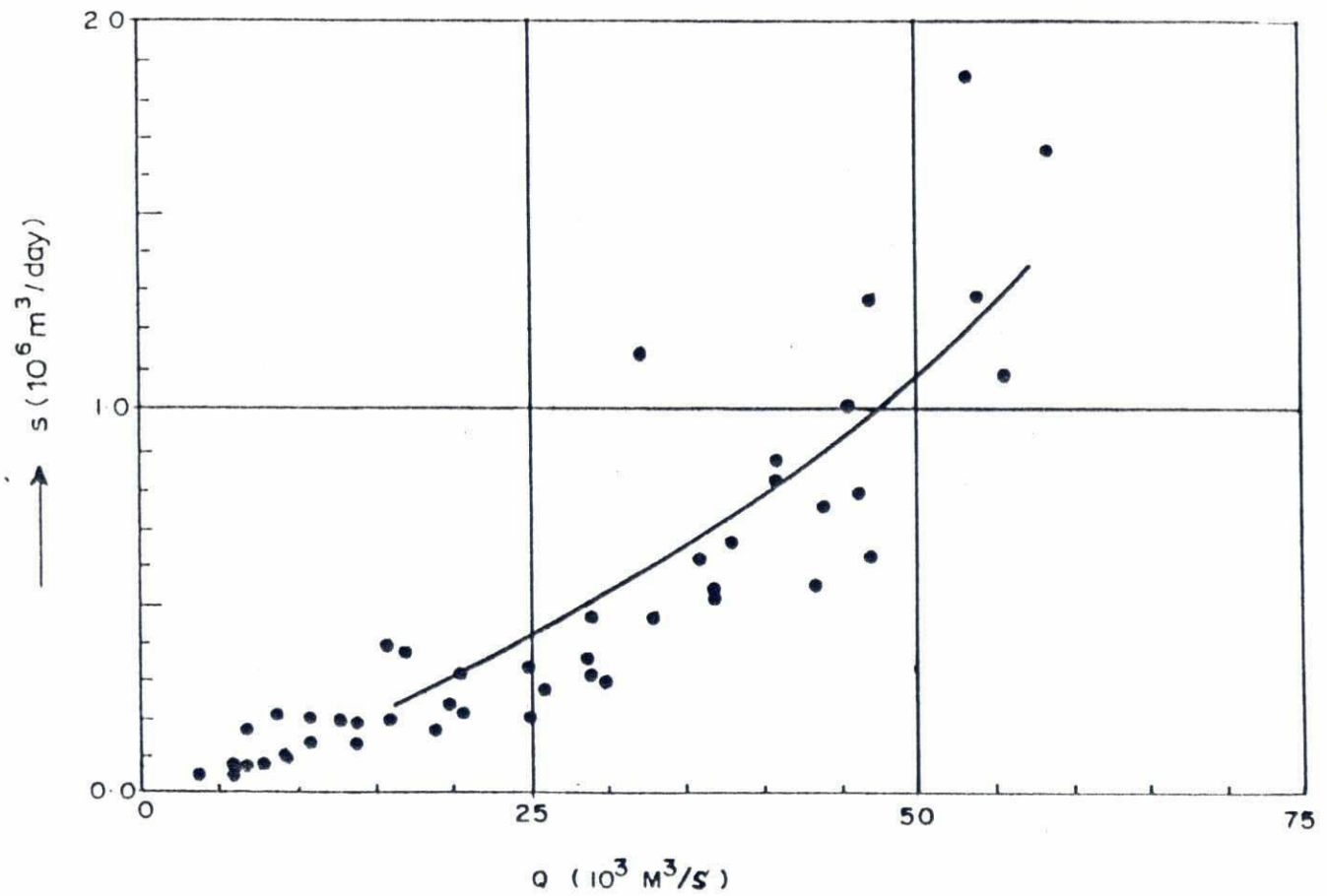


Figure 7 Measured suspended bed material load in Jamuna River at Bahadurabad (1968-1970)



## Paper 3.7

ALAM, M.K. AND M.M. HOSSAIN, (1988), Sediment transport in the River Jamuna, *Jr. Institution of Engineers, Bangladesh*, vol.16(2):17-26.

# Sediment Transport in the River Jamuna

M. Khorshed Alam\* and M. Monowar Hossain\*

## Abstract :

Bangladesh Water Development Board has collected sediment data on the Jamuna at Chilmari, Nakfatarchar and Nagarbari for several years. The data collection procedure involves collection of suspended sediment with the help of Binkley sediment samplers. This paper makes an attempt to analyse these data to have a quantitative evaluation of the transport rate of the river Jamuna. In doing so, a comparative treatment of the applicability of few well-known transport formulae are made. The transport formulae selected in the present analysis are based on a prior comparison by Bari and Alam<sup>2</sup> and also by Hossain<sup>7</sup>. This study considers the testing of efficiency of the (i) Colby<sup>4</sup> (ii) Engelund Hansen<sup>5</sup> (iii) Ackers-White<sup>1</sup> (iv) Inglis-Lacey<sup>8</sup> (v) Yang<sup>13</sup> and (vi) Hossain<sup>7</sup> equation. The performance of the various transport equations against these data are also discussed.

## Introduction :

Jamuna is one of the major rivers of Bangladesh. After flowing about 1600 miles in Nepal and India by the name of Brahmaputra it enters through northern border of Bangladesh at Rangpur and travels about 175 miles (270 Km) south to join the Ganges near Goalundo in Bangladesh. The combined flow known as the Padma then joins the Meghna 65 miles (105 Km) downstream and finally discharges into the Bay of Bengal. The total catchment area is about 234000 sq. miles (606,000 sq. Km.) of which about 72000 sq. miles (186400 sq. km) is in India and 28000 sq. miles (72500 sq. km) in Bangladesh<sup>10</sup>.

Bangladesh Water Development Board<sup>3,6</sup> had collected sediment data at various stations namely Bhadurabad, Chilmari, Sorajganj (Nakfatarchar) and Nagarbari for several years. This paper makes an attempt to analyse those data to have a quantitative evaluation of the sediment transport. The applicability of few well known transport formulae are tested. The selected formulae are Colby<sup>4</sup> Engelund-Hansen<sup>5</sup>, Ackers-White<sup>1</sup>, Inglis-Lacey<sup>8</sup>, Yang<sup>13</sup> and Hossain<sup>7</sup>. Efforts have also been made to compute the intensity of transport at various times of the year.

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## Data Collection :

Data collected by Bangladesh Water Development Board included measurement of suspended load by Binkley sediment samplers. The analysis made herein is based on the data of the Jamuna at Nagarbari during 1970 and 1972 as well as at Nakfatarchar during 1969 and 1970. Sediment data for the station at Chilmari and Bhadurabad were also available but they were incomplete in nature and hence could not be studied here. Field data as available were in a different form and brought into usable state according to the following criteria:

- i. Width of the active portion of the channel has been taken as the surface width and therefore width of the shoals has not been included in the breadth parameter.
- ii. Mean depth of the flow has been determined from the equivalent rectangular channel section whose top lateral dimensions equals the water surface width. Thus the water section area divided by the water surface width gives the mean depth.
- iii. Mean velocity has been obtained by dividing the water discharge by the area of cross-section.



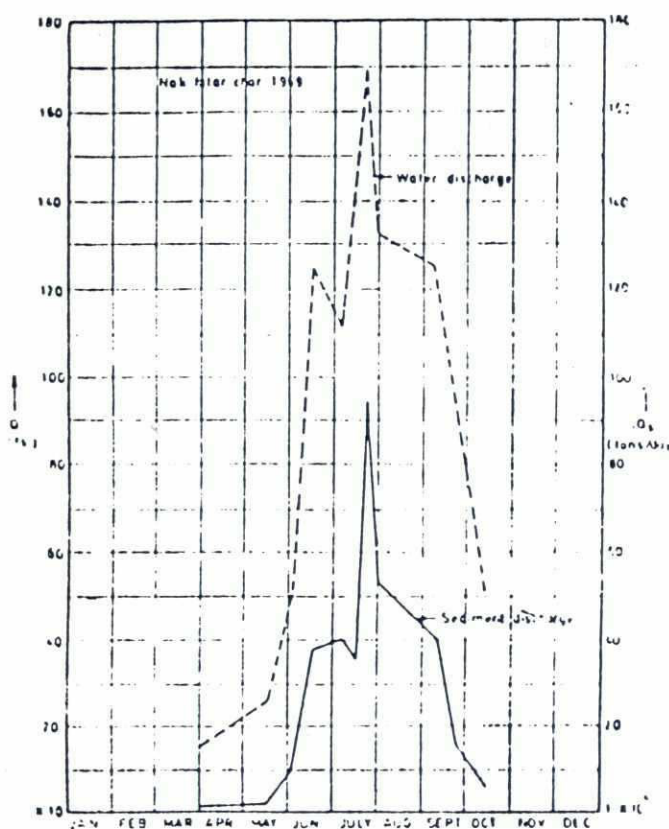


FIG. 1 WATER-SEDIMENT DISCHARGE HYDROGRAPH AT NAKFATARCHAR, 1969

### Water-Sediment Hydrographs :

#### Nakfatarchar:

Fig. 1 and 2 show water-sediment flow hydrographs for the station Nakfatarchar for the years 1969 and 1970. Both the water and sediment hydrographs have two peaks and occurred simultaneously. The first peak was observed around mid-June and the second peak which was the major peak occurred around late July. During the monsoon (May to October), the river stage was high, the maximum sediment movement was found to occur and the amount of suspended sediment transport was about 487 million tons in 1969 and 512 million tons in 1970 as shown in Table I. A. The table also shows the daily minimum and maximum rate which varied between 1.5 million to 9.4 million tons in 1969 and 0.8 to 6.08 million tons in 1970.

TABLE—I.A

#### Sediment flow at Nakfatarchar

|                        | 1969         |             | 1970         |             |
|------------------------|--------------|-------------|--------------|-------------|
| Quantity (million Ton) | Nov. to Apr. | May to Oct. | Nov. to Apr. | May to June |
| Daily minimum          | —            | 1.5         | 0.045        | 0.80        |
| Daily maximum          | —            | 9.4         | 0.585        | 6.18        |
| Total:                 | —            | 487.0       | 30.0         | 512.0       |

Data for the lean period (November to April) was available only for the year 1970 and during this period sediment passing the station amounted about 30 million tons which was about 5.8 percent of that total sediment flow in the monsoon. The daily sediment flow varied between 0.046 million tons to 0.58 million tons approximately.

#### Nagarbari :

Fig. 3 and 4 show the water-sediment flow hydrographs at Nagarbari station for the years 1970 and 1972 respectively. Fig. 3 shows that there are three peaks for both water and sediment discharges. The highest peak in water discharge occurred in mid-August while the highest peak for the sediment discharge occurred in early October. Possibly the local erosion during the subsiding stage of the river caused this rising of the peaks. Sediment flow was low during the lean period. The daily rate of sediment movement during this period varied between 0.065 m tons to 0.306 m tons. During monsoon (May to October) this rate was between 0.872 m tons to 5.88 m tons. The total sediment flow were 20 and 462 million tons during the lean and monsoon period respectively (Table I B).

In the year 1972 (fig.4) there was one major peak for water and sediment discharge during first half of August. There was another very minor peak during October. During May to October the minimum and maximum daily sediment flow was about 0.713 million tons and 3.922 million tons respectively. The total sediment flow was about 307.0 million tons. During the lean period (i.e. November to April) the total

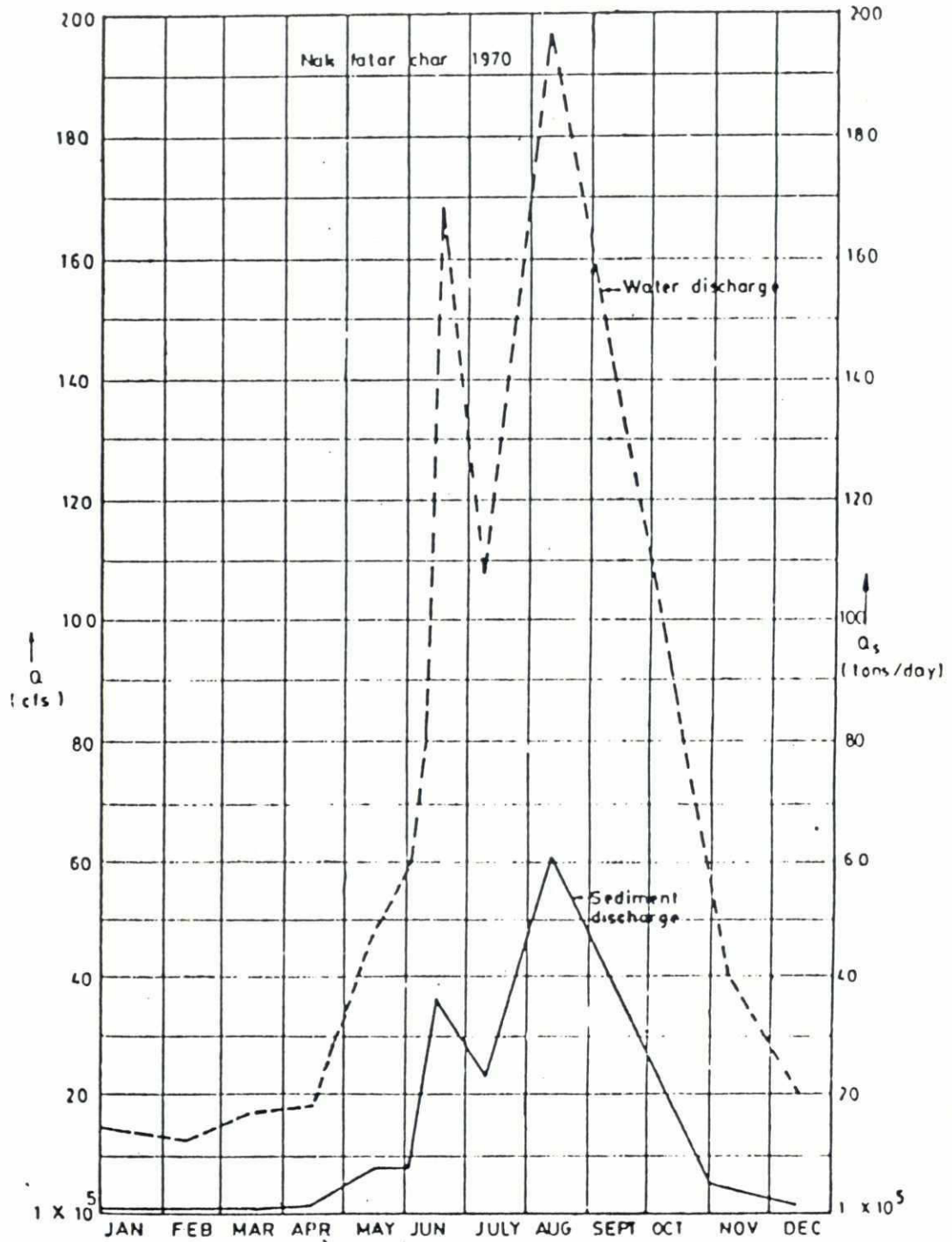


FIG. 2 WATER-SEDIMENT DISCHARGE HYDROGRAPH AT NAKFATARCHAR, 1970



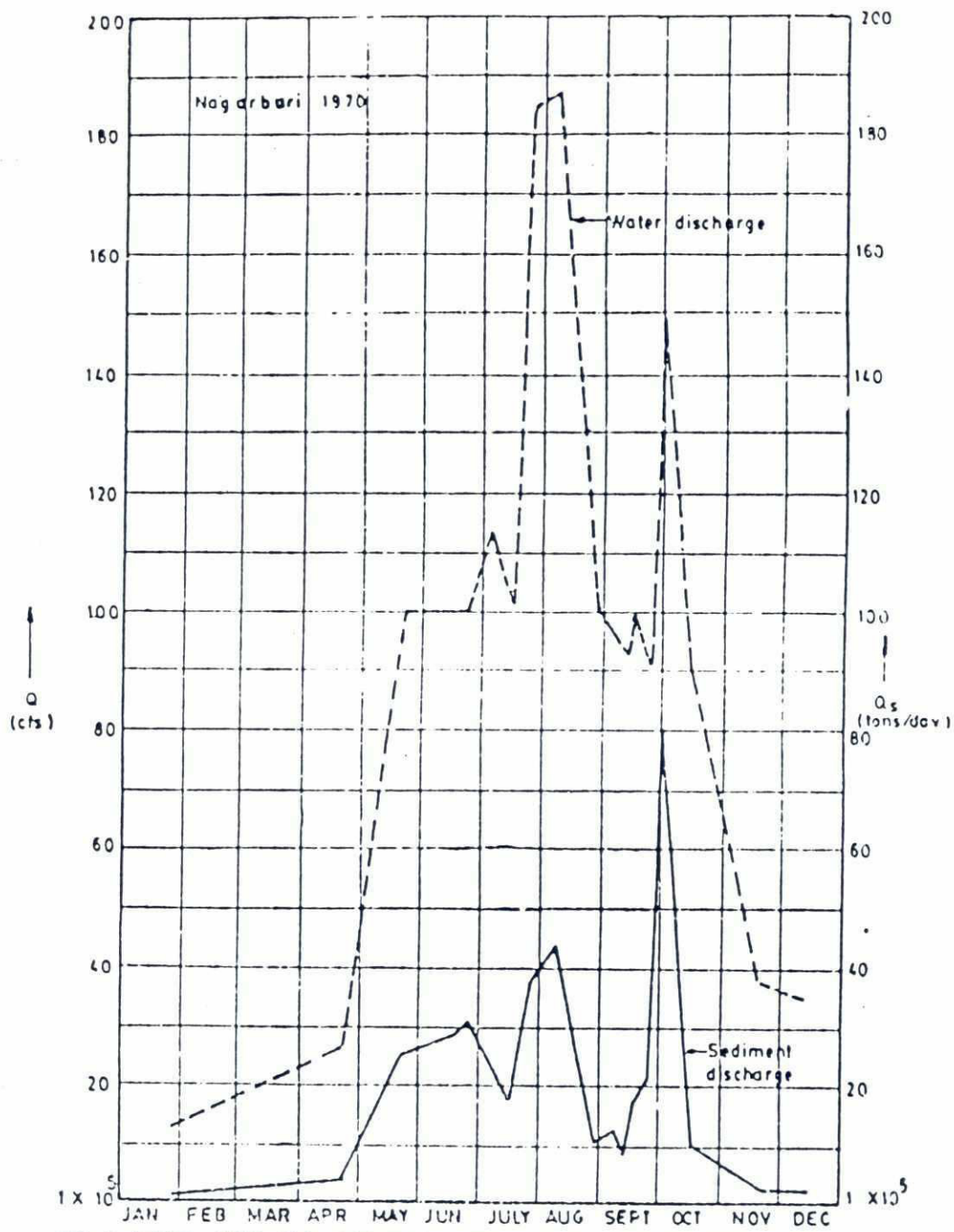


FIG. 3 WATER-SEDIMENT DISCHARGE HYDROGRAPH AT NAGARBARI, 1970

### Comparison of Transport Formulae and Evaluation of Transport Capacity:

Six well known sediment transport formulae have been selected to test their efficacy against the present set of data. The formulae selected as mentioned earlier are Colby<sup>4</sup> Engelund-Hansen<sup>5</sup> Inglis-Lacey<sup>6</sup>, Ackers-White<sup>1</sup> Yang<sup>11</sup> and Hossain<sup>7</sup>. These formulae have been tested against broad spectrum of world wide data and have been identified to provide the transport capacity of the river with a fair degree of certainty. The first and the second formulae had been recommended by ASCE Task committee<sup>11</sup>. The third formulae had been developed from the data of the canals and rivers of this subcontinent and thus corresponds to the regional characteristics of the present study. The fourth formulae have been analysed by White et al<sup>12</sup> and established as one of the best formulae. Introduced by Yang<sup>11</sup> the fifth formulae is based on the new concept of stream power and is claimed by its author as the most reliable formulae in the field of sediment transport and the sixth formulae is one of the recent formulae and has been analysed as one of the present day best by the second author<sup>7</sup>.

In studying these formulae a point worth mentioning is that they estimate total bed material load while the field data in the present case are concerned with suspended load. The amount of bed load transported by a large deep river is about 5 to 25 percent of the total suspended load<sup>9</sup>. The amount of bed load in the present case has been considered to be 25 per cent of the suspended load for the evaluation of total load.

Figures-5 and 6 and Tables 2 and 3 show the comparison of the field measurement and estimated values by the formulae for the station Nakfaterchar and the Nagarbari respectively. From the figures and tables it may be noticed that the estimated sediment load by various formulas are lower than the actual values. This leads to the connotation that the river Jamuna is over loaded all throughout the year and the magnitude varies according to discharge. The estimation by all formulae except by Hossain and partly by Engelund-Hansen show that over-loading intensity is greater at discharge range between 0.1 million cfs to 0.6 million cfs than at

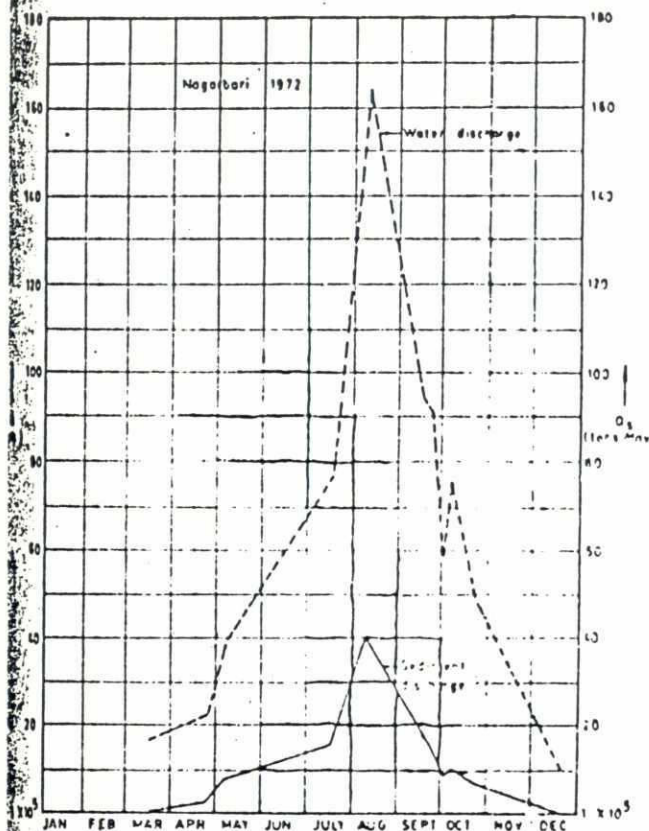


Fig. 4 WATER-SEDIMENT DISCHARGE HYDROGRAPH AT NAGARBARI, 1972

sediment discharge was 22.02 million tons, and minimum and maximum daily sediment flow are 0.042 million tons and 0.274 million tons respectively (Table 1. B).

TABLE 1.B

Sediment flow at Nagarbari

| Quantity<br>(million Ton) | 1970            |                | 1972            |                |
|---------------------------|-----------------|----------------|-----------------|----------------|
|                           | Nov. to<br>Apr. | May to<br>Oct. | Nov. to<br>Apr. | May to<br>Oct. |
| Daily minimum             | 0.015           | 0.872          | 0.042           | 0.713          |
| Daily Maximum             | 0.306           | 5.880          | 0.274           | 3.99           |
| Total:                    | 20.0            | 462.0          | 22.0            | 307.00         |



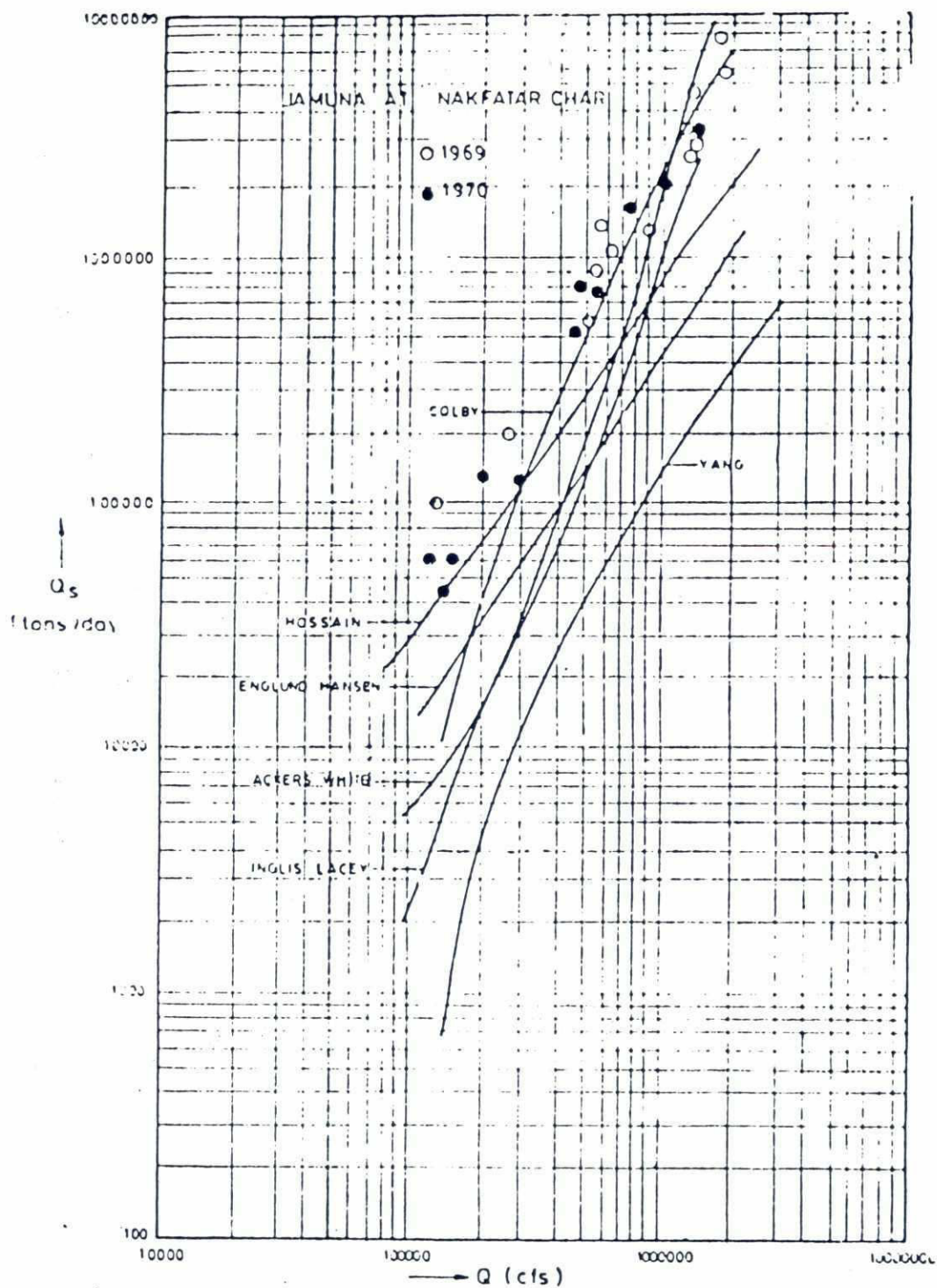


FIG. 5 COMPARISON OF TRANSPORT FORMULAS, JAMUNA AT NAKFATAR CHAR

TABLE-3

Jamuna at Nagarbari

Loading Calculation (measured/calculated)

(A)  $Q < 0.6$  million cfs (Flow velocity between 1.3 fps to 6.23 fps)

| Year | Description | Colby<br>(1964) | Engelund<br>(1967) | Inglis<br>(1968) | Ackers<br>(1973) | Yang<br>(1973) | Hossain<br>(1985) |
|------|-------------|-----------------|--------------------|------------------|------------------|----------------|-------------------|
| 1970 | Max         | 3.73            | 5.07               | 22.72            | 7.19             | 25.64          | 2.51              |
|      | Min         | 0.935           | 2.36               | 1.29             | 2.23             | 11.40          | 1.15              |
|      | Average     | 1.53            | 3.18               | 3.32             | 3.68             | 16.20          | 1.55              |
| 1972 | Max         | 2.10            | 11.23              | 16.70            | 10.64            | 43.50          | 3.33              |
|      | Min         | 1.76            | 2.26               | 2.74             | 4.35             | 4.50           | 0.72              |
|      | Average     | 1.92            | 5.14               | 4.38             | 6.70             | 14.74          | 1.618             |

(B)  $Q > 0.6$  million cfs, (Flow velocity between 3.63 fps to 6.23 fps)

|      |         |       |      |       |      |       |      |
|------|---------|-------|------|-------|------|-------|------|
| 1970 | Max     | 2.55  | 7.30 | 9.60  | 5.90 | 25.60 | 5.32 |
|      | Min     | 0.75  | 2.20 | 0.80  | 0.94 | 0.99  | 1.91 |
|      | Average | 1.01  | 3.30 | 2.30  | 2.50 | 7.43  | 2.78 |
| 1972 | Max     | 1.06  | 4.20 | 2.30  | 3.48 | 11.10 | 2.90 |
|      | Min     | 0.89  | 3.14 | 1.035 | 2.20 | 1.30  | 1.99 |
|      | Average | 0.975 | 3.50 | 1.70  | 2.50 | 5.56  | 2.50 |

The Colby formulae utilised data of natural streams of American rivers with median diameters between 0.2mm to 0.4mm. According to this formulae the Jamuna at Nagarbari (between October to April) was over loaded between 1.75 times to 2.75 times. At high flow (during May to October) sediment passing through the station was almost at its transport capacity and over loading was 1.2 times. Thus considering the data of this study the application of the formulae is satisfactory.

Result obtained by Hossain's formulae may be regarded consistent in prediction. The formulae has been developed using sands of median sizes 0.15mm to 0.30mm and taking into account the influence of settling velocity in its calculation. The formulae predicted that over loading magnitude increased as the flow discharge increased. This does not seem to be unreasonable. When the flow discharge increased the velocity (In the present case 4 fps to 7fps) also increased and definitely the overall transport capacity increased. At the same time the erosive power also increase causing greater sediment displacement from bed and banks and ultimately causing over loading. Field experience in the case of Jamuna, support this phenomena. The formulae predicted that during lean period (October to May  $Q < 0.6$  million cfs) the over loading at Nakfatarchar varied between 2.25 to 2.8 times. During monsoon this value increased

to 3.3 to 4.8. At Nagarbari the corresponding magnitude of loadings were 1.8 to 2.00 and 3.0 to 3.5 respectively.

From the analysis of the formulae it appears that the Colby's and Hossain's formulae display maximum confidence in predicting the transport capacity of the Jamuna.

#### Regression Equations :

Simple power type equations have been developed relating sediment transport ( $Q_s$ ) with water discharge ( $Q$ ) and velocity of flow ( $v$ ) employing regression technique. Fig. 7 shows the water sediment discharge relation

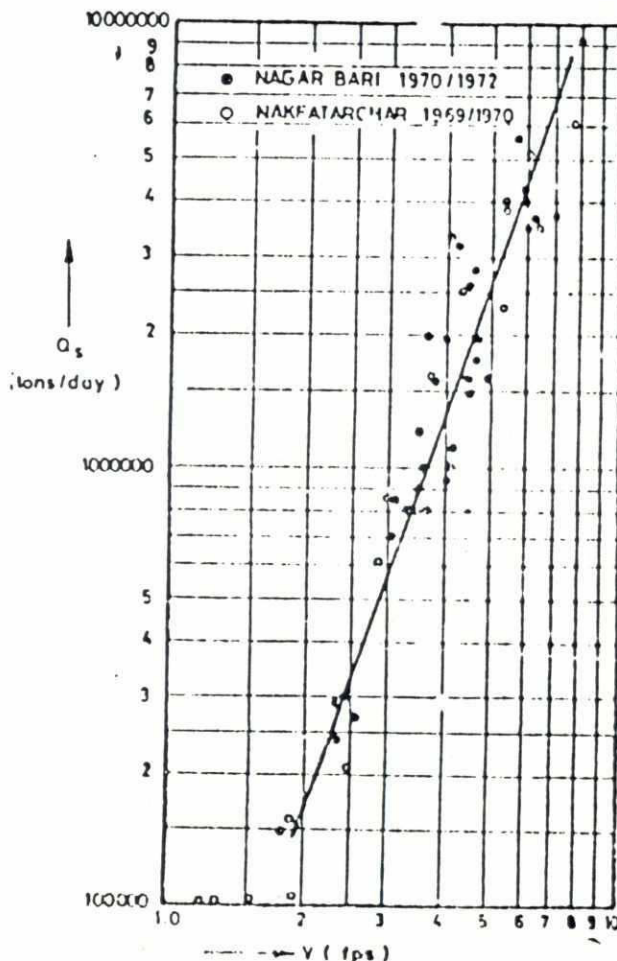


FIG. 7 VELOCITY-SEDIMENT DISCHARGE RELATIONSHIP FOR THE JAMUNA



TABLE--2

## Jamuna at Nakfatarchar

Loading Calculation (measured/calculated)

(A)  $Q < 0.6$  million cfs (Flow velocity between 1.05 fps to 5.62 fps)

| Year | Descrip-<br>tion | Colby<br>(1964) | Engelund<br>-Hanson<br>(1967) | Inglis<br>-Lacey<br>(1963) | Ackers<br>-White<br>(1973) | Yang<br>Hosain<br>(1985) |
|------|------------------|-----------------|-------------------------------|----------------------------|----------------------------|--------------------------|
| 1969 | Max              | 3.79            | 5.70                          | 17.24                      | 6.53                       | 20.83                    |
|      | Min              | 0.90            | 3.26                          | 0.368                      | 3.01                       | 16.70                    |
|      | Average          | 1.42            | 4.23                          | 1.05                       | 3.24                       | 18.45                    |
| 1970 | Max              | 14.20           | 6.71                          | 23.50                      | 10.00                      | 51.80                    |
|      | Min              | 1.13            | 2.22                          | 2.47                       | 0.83                       | 5.62                     |
|      | Average          | 2.22            | 3.41                          | 5.15                       | 2.44                       | 17.85                    |

(B)  $Q > .6$  million cfs (Flow velocity between 3.76 fps

|      |         |       |      |       |      |       |
|------|---------|-------|------|-------|------|-------|
| 1969 | Max     | 1.6   | 10.6 | 5.21  | 5.92 | 26.3  |
|      | Min     | 0.84  | 1.09 | 0.393 | 1.85 | 12.82 |
|      | Average | 1.13  | 6.25 | 1.07  | 2.85 | 19.20 |
| 1970 | Max     | 1.20  | 5.60 | 4.70  | 6.50 | 18.50 |
|      | Min     | 0.629 | 3.48 | 0.304 | 1.11 | 19.00 |
|      | Average | 1.20  | 3.50 | 0.60  | 2.12 | 13.25 |

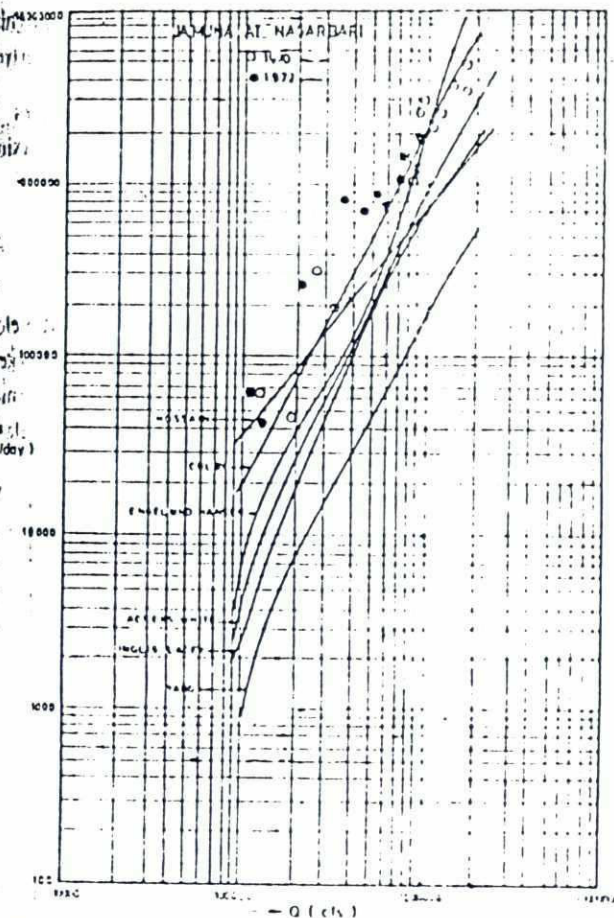


FIG. 6 COMPARISON OF TRANSPORT FORMULAS, JAMUNA AT NAKFATARCHAR

discharge range above 0.6 million cfs. The low flow normally occurred in the months of October to May and the velocity ranged from 1.3 fps to 4.5 fps. At flows greater than 0.6 million cfs the velocity varied between 3 fps to 7.0 fps.

So far the performance of the formulas are concerned the analysis reveal that Inglis-Lacey formulae estimate the sediment load in an inconsistent manner especially at high discharge ( $Q > 0.6$  million cfs). This formula as mentioned earlier had been derived from the data of canals and rivers of this subcontinent where median diameters of the sediment varied between to 0.04mm to 0.12mm. In the present case the median diameter varied between 0.1mm to 0.24mm. This may be the reason for inconsistency in the present case.

The estimation by Yang formulae also appears to be inconsistent and unreasonably low and cast doubts about its applicability in the present case. Formulae by Ackers-White covers quite a wide range of sediment data (0.03mm to 0.40 mm) and flow range but the estimation do not display satisfactory performance in the present case. According to this formulae Jamuna both at Nakfatarchar and Nagarhari were overloaded to the extent of about 3.50 times.

Engelund-Hansen formula estimated the transport rate quite consistently but prediction was lower and according to this formula the Jamuna on the average were overloaded from 5.0 times to 9.75 times, the average value being around 5.0. Engelund-Hansen formulae had been developed for sand and silt of medium diameter ( $d_{50}$ ) greater than 0.19mm and thus its application in the present case may be questioned.

for the data considered in this study. The regression equation may be written as follows:

$$Q_s = 4.06 \times 10^{-4} Q^{1.602}, \quad (r = 0.820)$$

Where,  $Q_s$  is in tons per day

$Q$  is in cubic foot per second

$r$  is the coefficient of correlation

Similar type of equation has also been developed relating sediment transport and velocity. Fig. 8 shows the velocity-sediment discharge graph for the data of both Nagarbari and Nakfatarchar. The regression equation for this case (Fig. 8) may be written as follows:

$$Q_s = 22811.8 V^{2.95} \quad (r = 0.878)$$

where  $V$  is the velocity of flow.

It may be noticed from the above two equations that sediment discharge may satisfactorily be expressed as a function of water discharge and flow velocity.

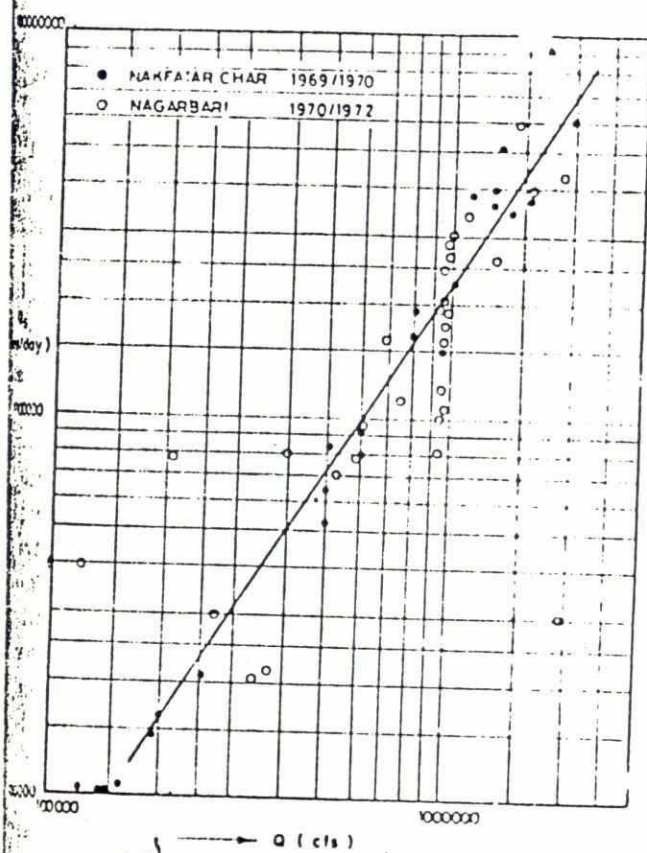


FIG. 8 WATER SEDIMENT DISCHARGE RELATIONSHIP FOR THE JAMUNA

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## Conclusions :

The following conclusions can be drawn from the present analysis :

- The total suspended sediment discharge in the Jamuna at Nakfatarchar may be taken between 439 million tons to 575 million tons and about 94 percent of the flow occurs in the monsoon (i.e. between late May to late October). At Nagarbari, which is about 32 miles downstream of Nakfatarchar the total suspended sediment discharge may be considered from 310 million tons to 500 million tons and about 92 percent of the total sediment discharge occurs between May to October.
- Of all the formulae considered for testing in this study the Colby's and Hossain's formulae may be used to predict the transport capacity of the river more successfully.
- Simple power type relations may be developed to predict sediment movement in the Jamuna.

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## Paper 3.8

HOSSAIN, M.M., (1992), Total sediment load in the Lower Ganges and Jamuna, *Jr. Institution of Engineers, Bangladesh*, vol. 20(1-2):1-8.



## Total Sediment Load in the Lower Ganges & Jamuna

M.M. Hossain\*

### Abstract :

A study was undertaken to estimate the total sediment flow of the Ganges and Jamuna. Hydromorphological data collected by Bangladesh Water Development Board were utilised for this purpose. For the Jamuna, data at Bahadurabad gauging station between 1982 and 1988 and for the Ganges data at Hardinge bridge station between 1980 and 1986 were used. Total annual sediment flow which is of general concern were estimated for various years during the above mentioned period using one of the recently developed sediment transport function based on dimensional considerations.

According to the estimates of this transport function, the total annual sediment flow of the Ganges at Hardinge bridge were found to vary between 350 million tons to about 600 million tons. The highest and lowest daily sediment flow was found to be of the order of 10 million tons and 9000 tons respectively. On the other hand, the total annual sediment flow of the Jamuna at Bahadurabad was found to vary between 405 million tons to 815 million tons with daily minimum of 20000 tons and maximum of 12 millions tons. The study revealed that total average sediment flow through the major rivers of Bangladesh was of the order of 1.2 billion tons with a maximum of 1.5 billion tons.

The estimated total sediment flow when compared with the findings of previous studies showed good compliance. Moreover, correlations were developed between water flow and sediment flow for use in practical design problems.

### Introduction :

The alluvial plains of Bangladesh is composed of the sediments brought down by the three major river systems namely: the Ganges, the Brahmaputra and the Meghna. The sediment carried down by these river system are supplied from the erosion of their enormous catchment area. The Meghna system has the smallest catchment area and provides the lowest sediments, practically insignificant compared to what is provided by the Ganges and Brahmaputra river systems.

The source of both the Ganges and Brahmaputra are from the Himalayas. After traversing through different routes, the Ganges enters Bangladesh at 18 km downstream of the Farakka from north west and the Jamuna through northern border at Rangpur. The Ganges travels further down for a distance of about 125 Km and Jamuna for about 270 km to meet at Goalundo. The river known as the Padma then travels further downstream to meet the Meghna near Chandpur and the combined flow finally discharges into the Bay of Bengal through a number of estuary channels.

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The total catchment area of the Jamuna upto Bahadurabad is about 606000 sq. km and that of the Ganges upto Goalundo confluence is about 977044 sq. km and that of the Meghna upto Bhairabbazar is about 65000 sq. km. It may be noted here that the total territorial boundary of Bangladesh is only 9 percent of combined catchment area of the Ganges - Jamuna.

#### Data :

Data required to carry out this study are water level, discharge, cross-sectional area, energy gradient, and sediment related data. These data were available from Bangladesh Water Development Board (BWDB). Daily discharge, water level, cross-sectional area and sediment related data at Hardinge Bridge and Bahadurabad stations were available on a weekly basis. For the Ganges hydraulic data from 1980 to 1986 were utilised to estimate the total sediment load. The measured suspended sediment data for this station were available for the years 1984 to 1986. On the other hand the hydraulic and geometric data from 1982 to '88 were utilised to estimate the total sediment load of the Jamuna at Bahadurabad. The measured suspended data used in this study for the Jamuna were from 1982 to '86. Older data have been used by the previous investigators and are reported elsewhere<sup>1,2,3,4</sup>. Findings to previous investigators were considered to determine the total sediment load in the Ganges-Jamuna system.

#### Review :

The first mentionable and pioneering work on the hydraulic behaviour including sediment transport aspect of the Jamuna was reported by Coleman<sup>5</sup>. He reported that combined daily suspended sediment flow of the three major rivers during the flood season was of the order of 13 million tons with Jamuna alone 7 millions tons and the Ganges some 6 million tons. The Meghna being the smallest contributor of sediment load to the delta formation process, no mention as to sediment movement was available. Coleman also reported that these rivers of the Bengal basin deliver some one billion ton of suspended sediment a year to the Bay of Bengal. His estimates were based on the measured data during the period 1958-'62. Studies related to sediment transport aspects of the Jamuna were also conducted by the Master plan Organisation<sup>6</sup> and Jamuna Multipurpose Bridge Authority<sup>7</sup>. But no

definite and detail studies were reported as to the quantum and rates of sediment movement.

Studies were conducted by Bari,<sup>1</sup> Alam and Bari,<sup>2</sup> Alam and Hossain,<sup>3</sup> Sultana<sup>4</sup> and Hossain<sup>8</sup>. These studies were mainly concentrated to assess the applicability of few well known sediment transport equations against the data of the Ganges and Jamuna. Some of these investigators however, while adjudging the efficacy of various transport equations also computed the total annual sediment flow<sup>3,4</sup>. The annual sediment flow estimated by various investigators at various times are shown in Table-1. It may be worthwhile to mention that the recommended sediment transport equations which were found to be the best against data of the Ganges and Jamuna were Hossain<sup>9,10</sup> Colby<sup>11</sup> and Engelund-Hansen<sup>12</sup>. Hossain's equation is considerably recent and was developed on the basis of dimensional considerations. Later this equation was applied against the world wide data along with other very well known sediment transport equations<sup>13</sup>. Hossain's equation was found to be superior and stable against those world wide data from large alluvial channels. Thus, this study is concerned with estimation of total sediment transport of the Ganges and Jamuna using Hossain's approach and utilising the most recent hydraulic data collected by BWDB.

TABLE -1.a

*Sediment transport of the Ganges at Hardinge Bridge in million tons<sup>4</sup>*

| Sediment load estimated by | No of available observations | Sediment transport during |                   |                 |
|----------------------------|------------------------------|---------------------------|-------------------|-----------------|
|                            |                              | 1984<br>Jun-Nov.          | 1985<br>Jul.-Dec. | 1986<br>Jul-Nov |
| Measured                   | 21                           | 183                       | 304               | 181             |
| Colby                      | 21                           | 234                       | 368               | 220             |
| Hossain                    | 20                           | 335                       | 367               | 324             |

TABLE - 1.b

*Sediment transport of the Jamuna in million tons<sup>3</sup>*

| Station       | 1969 | 1970 | 1972 |
|---------------|------|------|------|
| Nakfatar Char | 310  | 500  | -    |
| Nagarbari     |      | 439  | 575  |



TABLE - 1. c

*Sediment transport of the Ganges and Jamuna in million Tons<sup>17</sup>*

| Year | Jamuna at Bahadurabad | Ganges at Paksey | Ganges at Goalando |
|------|-----------------------|------------------|--------------------|
| 1967 | 675                   | 481              | 830                |
| 1968 | 527                   | 526              | 753                |
| 1969 | 421                   | 557              | 586                |

TABLE - 1.d

*Summary of Sediment transport in the Jamuna and Ganges in million tons<sup>5</sup>*

|                            | Max.  | Min.  | Mean  |
|----------------------------|-------|-------|-------|
| Ganges at Hardinge         | 735.9 | 257.3 | 478.9 |
| Brahmaputra at Bahadurabad | 696.5 | 530.9 | 607.7 |

### Hossain Approach :

It has already been mentioned that the present study uses Hossain approach<sup>10</sup> in determining the sediment flow of the Ganges and Jamuna. In deriving this approach it was argued that sediment concentration in a stream of steady water and sediment flow is a power function of,

- the product of Froude number and Slope of energy gradient
- the settling velocity ratio, and
- the discharge ratio.

The functional form of the equation could be expressed as follows:

$$C_t = A [X^a Y^b Z^c]$$

where,  $C_t$  = total average sediment concentration in parts per million

$$\begin{aligned} A &= 6.946 \times 100000 \text{ for } B/H < 500 \\ &= 6.946 \times 1000000 \text{ for } B/H > 500 \\ X &= VS/(gH)^{1/2} \text{ and } a = 0.745 \\ Y &= w_r/w \quad b = 0.633 \\ Z &= Q/Q_c \quad c = 0.500 \end{aligned}$$

$w_r$  = settling velocity for a representative sediment size for which  $D_{50} = 0.15\text{mm}$  at ambient temperature

$w$  = settling velocity of the sediment load

$Q$  = measured discharge

$Q_c$  = assessed discharge and obtained from the following equation<sup>14</sup>.

$$= [(2.15 + KB/H) H(gS)^{1/5} J^{5/2}]$$

$B$  and  $H$  are respectively normalised average width and depth. The value of  $K$  is proposed to be 0.055 when  $Q < 15000$  cumecs. and 0.17 when  $Q > 15000$  cumecs. Settling velocities of the sediment particles have been computed using Rubey's equation<sup>15</sup>.

Once the daily average concentrations are obtained from above equations, it was readily available to the author the daily sediment flow from which sediment transport in that particular station of the river was obtained. The present computation was then compared with the findings of the previous investigators and total annual average sediment transport of these two major rivers of Bangladesh was ascertained.

### Result & Discussion :

Total sediment load utilising the available hydraulic data for the Ganges at Hardinge bridge station were estimated. These were then plotted against respective discharges as shown in Fig. 1. This figure shows that the estimated lowest and highest daily sediment flow is of the order of 10000 tons and 10.3 million tons respectively. The estimated total annual sediment load of the Ganges and Jamuna in the present analysis is shown in Table-2. Table 2 shows that the total load of the Ganges varies from about 350 million tons to 600 millions tons annually, the average value being 435 million tons. Data on observed suspended load for the Ganges during flood season of the years 1984, '85, and '86 were also available. Suspended sediment load computed on the basis of these data are shown in Table 1.a. This table shows that the annual suspended load movement of the Ganges varies from 181 to 304 million tons during flood season, the average being 225 million tons. The geometric and

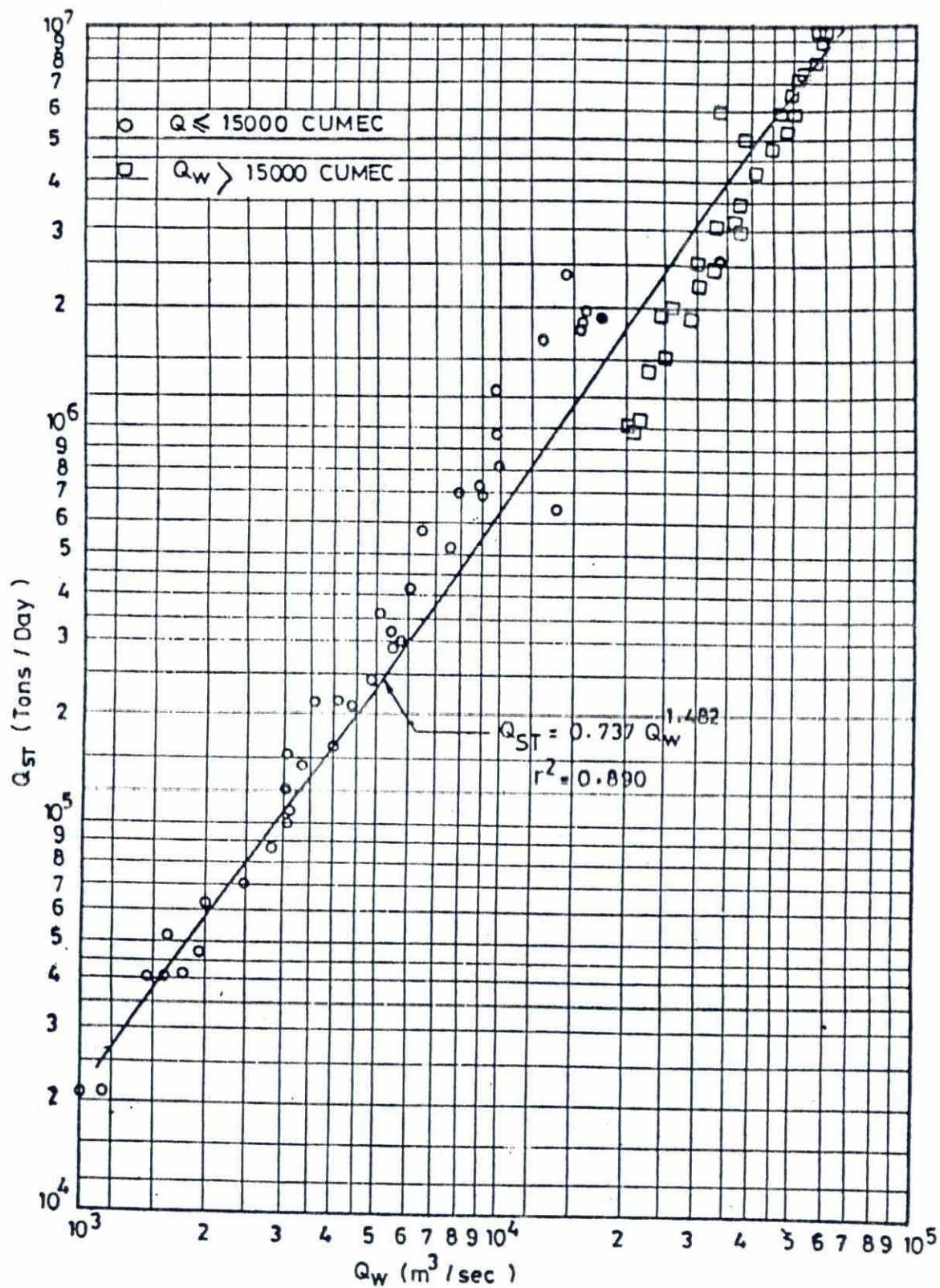


FIG. 1 WATER SEDIMENT DISCHARGE RELATIONSHIP FOR THE GANGES AT HARDINGE BRIDGE (1980 - 87)



TABLE - 2

Summary of Sediment Transport in millions tons in the Ganges and Jamuna.

| Year   | 1980 | '81 | '82 | '83 | '84 | '85 | '86 | '87 | '88 |
|--------|------|-----|-----|-----|-----|-----|-----|-----|-----|
| Ganges | 594  | -   | -   | 420 | 423 | 348 | 381 | -   | -   |
| Jamuna | -    | -   | 733 | 754 | 747 | -   | 403 | 815 | 840 |

hydraulic data corresponding to suspended sediment measurement during 1984, '85 and '86 are utilized to compute the total load using Hossain and Colby equations. It may be worth while to mention that about 85 percent of the total sediment movement occurs during the flood season, i.e., from May to November. According to this estimates as shown in table 1.a that about 342 million tons and 274 million tons of total sediment is transported through the Ganges and Jamuna during flood season annually estimated on the basis of Hossain and Colby equation respectively.

Fig. 2 shows the plot of estimated total sediment load by Hossain approach for the Jamuna at Bahadurabad during the period 1982-'88 against water flow. It is seen from this figure that the daily maximum sediment flow of the Jamuna is of the order of 12 million tons which is almost twice the amount as reported by Coleman. The average annual sediment flow during the study period was found to be varying from 405 to 840 million tons with an annual average of about 700 million tons. In the present analysis data needed for total load computation during the flood season of 1988 were not available. During this period, total sediment flow was estimated from the sediment rating curve prepared for other years. It is seen from table 2 that about 840 million tons of total sediment load movement occurred in 1988. It may be emphasised here that the correlation shown in Fig 1 and 2 are quite consistent and well correlated and may be used for quick estimate of sediment flow at various discharge levels for the Ganges and Jamuna respectively. Power type equations using regression technique have been developed and may be written as follows

$$Q_{st} = 0.737 Q_w^{1.482} \quad (r = 0.940) \quad (1)$$

$$Q_{st} = 4.020 Q_w^{1.300} \quad (r = 0.903) \quad (2)$$

where  $r$  is the coefficient of correlation,  $Q_{st}$  is the total sediment flow in tons per day and  $Q_w$  is the water flow in cumecs. Equation (1) and (2) are respectively valid for the Ganges and Jamuna.

Now the question arise: how much of sediments flow through these rivers and how much is really delivered to the Bay of Bengal. The answer for the first question may be obtained if we analyse the results of various estimates of sediment transport studies of the Ganges and Jamuna as reported in Tables -1.a to 1.d and 2. These results were obtained for data spreading for about 30 years period. The data used by Coleman<sup>5</sup> dates back to 1958-'62 period (table 1.d). According to him, the Jamuna carries some 608 million tons of suspended sediment and the Ganges 480 million tons annually on average. Therefore, the combined suspended sediment flow is about 1.1 billion tons which has already been mentioned. This table also shows that a maximum of 1.5 billion tons of sediment may flow through these rivers if the peak sediment flow in these two rivers occurs in the same year. The sediment transport values obtained from the present analysis utilizing more recent data and of course from a different approach as shown in table -2 agrees fairly well with those what is reported by Coleman.

Holeman<sup>16</sup> reported that the Ganges carries some 1.6 billion tons and the Jamuna carries some 0.8 billion tons of sediment annually. The total is thus 2.4 billion tons which is very often quoted in our country. Holeman however did not mention the methodology followed in arriving at these values but it appears that the basin erosion was the basis for his estimate. It should be noted that the basin erosion depends on many inter-related factors and its estimation is quite complicated. The result is often unreliable and the question of how much of the eroded material from the basin reach the stream course is a matter of investigation as yet. Therefore the values given by Holeman for both the Ganges and Jamuna and the total as 2.4 billion tons are highly questionable. Moreover this total sediment load does not comply with any of the contemporary or subsequent findings, especially the sediment load of the Ganges.

The annual sediment flow of the Ganges and Jamuna was also estimated by BWDB utilising



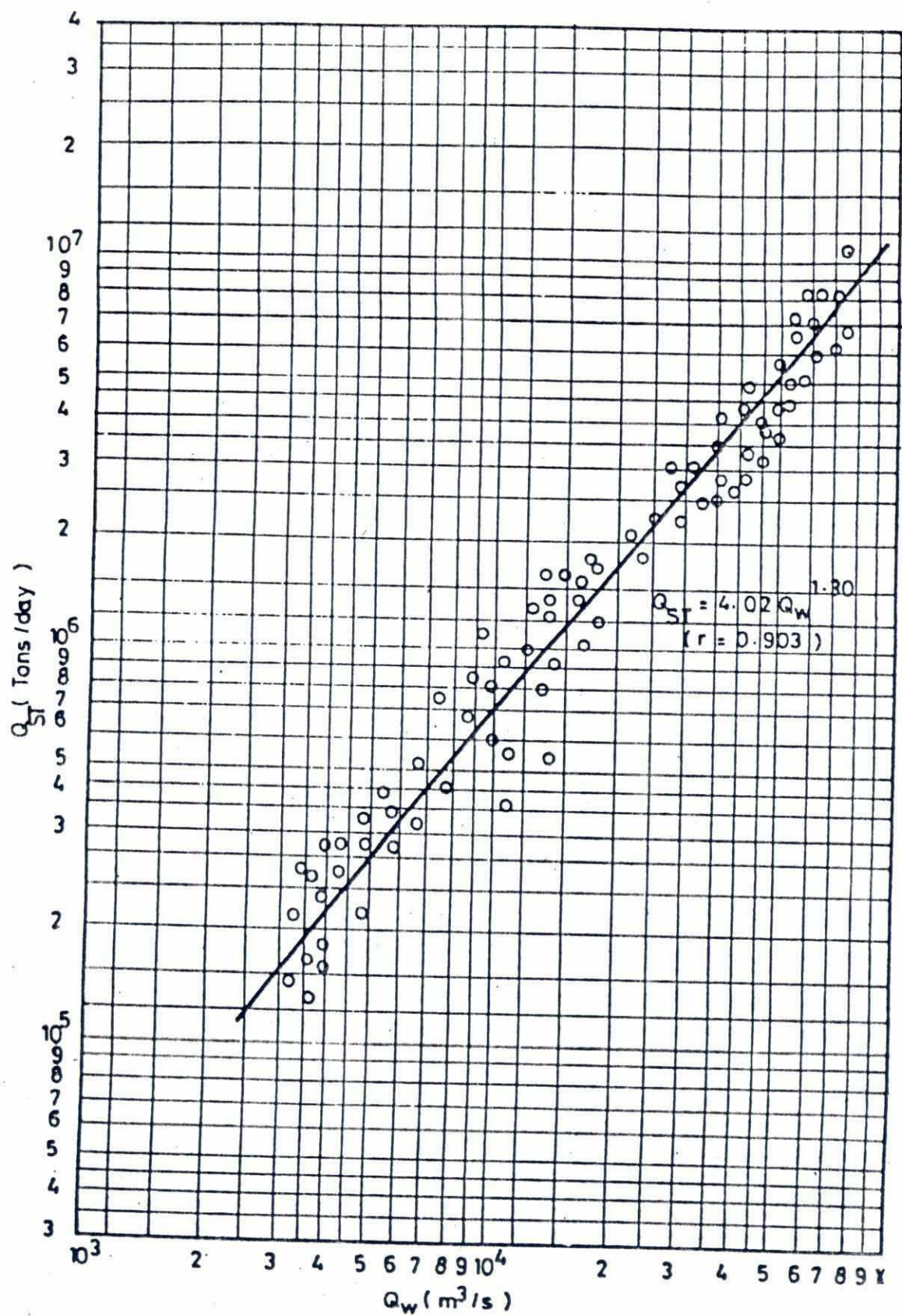


FIG. 2 WATER SEDIMENT DISCHARGE RELATIONSHIP FOR THE RIVER JAMUNA AT BAHADURABAD ( 1982 - 88 )



data of 1967-'69<sup>17</sup>. This is shown in Table 1.c. It is seen from this table that Jamuna carries a suspended sediment flow which varies from 421 to 675 million tons, the average being 541 million tons. On the other hand, the Ganges on average carries some 520 million tons of suspended sediments annually. Alam and Hossain<sup>3</sup> analysed the data for 1969, '70 and '72 for the Jamuna and found that their estimates closely approximate the estimates made by BWDB. This is shown in table 1.b. A percent wise distribution of catchment area, annual average water flow and annual average sediment flow for the Ganges-Jamuna-Meghna is shown in Table 3 for comparison.

**TABLE - 3**

*Percent wise distribution of catchment area, annual average flow and annual sediment flow of the Ganges Jamuna - Meghna*

| River and Station         | Catchment area | Annual average flow | Annual average sediment flow |
|---------------------------|----------------|---------------------|------------------------------|
| Ganges at Hardinge bridge | 59             | 31                  | 44                           |
| Jamuna at Bahadurabad     | 37             | 53                  | 55                           |
| Meghna at Bhairabazar     | 4              | 16                  | 1                            |

Another interesting trend may be seen if we examine the estimated annual sediment flows of the Ganges and Jamuna over the years as shown in the tables. The sediment flow in the Jamuna is gradually increasing whereas the sediment flow in the Ganges is gradually decreasing at a very slow rate. This may be due to changes taking place in the basins of these rivers from where sediments are delivered into these river system. For example, a number of watershed management program or barrage storage works are taking place in the basin of the Ganges, which is reducing sediment supply whereas the Jamuna basin is facing deforestation or other works which contribute more sediment supply into this river.

Regarding the question how much of the transported sediments reach the Bay of Bengal, it can be said that this aspect has not been studied in great details. However, it is important to

remember that the sediment transport figures obtained either from measurement or from computation are a certain percent of the soil material that is removed from the basin each year. Moreover, majority of the sediments are deposited before reaching the estuary due to reduction of velocity and turbulence in the flow as well as tidal effect. The sedimentation and erosion sub-committee of the federal Inter departmental Task Force<sup>18</sup> has estimated that only about 5% of the products of erosion from the potomac River Basin reach the river mouth. This gives an indication of the quantum of sediment reaching the Bay of Bengal through the Ganges and Jamuna.

### Conclusions :

The maximum daily total sediment flow in the Ganges at Hardinge bridge during flood season may be taken to be of the order of 10 million tons and that in the Jamuna at Bahadurabad as 12 million tons. The combined maximum daily suspended sediment flow in the Ganges and Jamuna as reported by Coleman which is about 13 tons a day may be accepted.

The total annual sediment flow for the Ganges at Hardinge bridge varies over a wide range over years. The sediment flow in the Ganges show a slightly decreasing trend and an average annual flow of 480 million tons may be accepted. For the Jamuna, the sediment flow is found to be increasing with year. An average value over the last thirty years may be of the order of 650 million tons annually. The combined annual sediment flow through the Ganges and Jamuna may be taken to be 1.1 to 1.2 billion tons on average which closely approximates Colman's figure. The peak sediment flow however, may be as high as 1.5 billion tons annually, but in no case the sediment load in the Ganges and Jamuna is 2.4 billion tons, as reported by Holeman.

The amount of sediments that reach the Bay of Bengal is not precisely known as yet and a subject of further investigation.

Correlations between sediment flow and water flow for both the Ganges and Jamuna are developed for ready use in case of practical design problems.



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## 4 Planform Changes

## Paper 4.1

GALAY, V.J., (1980), River channel shifting on large rivers in Bangladesh, *Proc. Intern. Symposium on River Sedimentation, Beijing, China*, 543-562pp.



RIVER CHANNEL SHIFTING  
ON  
LARGE RIVERS IN BANGLADESH  
BY

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ABSTRACT

The Ganges, Jamuna (Brahmaputra) and Meghna Rivers are three of the largest rivers in the world, with flood flows in the order of 2.5 million, 3.0 million and 5.5 million cubic feet per second, respectively. Detailed river-bank erosion studies have been carried out in connection with bridge crossings, power-line crossings and the location of flood control embankments. These studies show that three distinct channel shifting processes occur: gradual migration of the major flow channel, rapid shifting of the major channel during the development of cutoffs, and sudden creation of new flow channels on the flood-plain outside the bankfull channel. These shifting processes are discussed in detail. Rates of gradual migration of the river-bank are presented for various locations along these large rivers. In some cases river-bank migration has been as much as 5,500 feet in one year.

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RIVER CHANNEL SHIFTING  
ON  
LARGE RIVERS IN BANGLADESH

INTRODUCTION

Several of the largest rivers in the world, the Ganges, the Brahmaputra, and the Meghna, flow through one of the smallest countries in the world, Bangladesh, (See Figure 1 for location). The Brahmaputra, which changes its name to the Jamuna after its bifurcation with the old Brahmaputra, experiences flood flows as high as 3 million cubic feet per second (cfs). The Ganges River floods have been slightly lower, 2.5 million cfs. This river changes its name to the Padma at the confluence with the Jamuna River and the Padma joins the Meghna which experiences combined flood flows up to 5.5 million cfs.

During flood, major flow channels within bankfull channels shift and erode river-banks (see Figure 2 for nomenclature). With the possible construction of major engineering works across or adjacent to these rivers it becomes imperative that the shifting of the major flow channels and the erosion rates of the river-banks be fully appreciated.

The Hardinge Bridge was the earliest engineering structure to be constructed across the Ganges River (Gales, Ref. 1 and Inglis, Ref. 2). The bridge site was studied in detail and guide banks were found necessary to contain the down-river shift of the river's meander pattern. More



recently, the East Pakistan Master Plan (Ref. 3) has shown preliminary plans for a large barrage as well as flood control embankments on the Brahmaputra (Jamuna). On this same river extensive studies have been carried out for an electric power line across the river (East-west Interconnector), (Ref. 4,5 and 6). The tentative crossing location for the power line has been chosen near the confluence of the Jamuna-Ganges. Also, a major feasibility study (Ref. 7) has recently been completed for a bridge to be located across the Jamuna at Sirajganj. Another major project, the Chandpur Irrigation Project, Ref. 3, 8, and 9) is almost complete and is located adjacent to the Meghna River. In all of these projects the future location of the river banks is of importance, especially where bridge abutments or river training works are to be constructed.

Several of these projects have involved studies of the rate of bankline erosion, but in some cases these studies have been over short time periods and did not indicate substantial erosion. More recent studies by Northwest Hydraulic Consultants Ltd. (Ref. 6 and 9) using successive air photographs, ERTS satellite photos and maps have revealed that erosion of river banks is closely linked to the shifting of the main flow channel. The various modes of main channel shifting, along with rates of river bank erosion will be discussed in more detail.

#### TYPES OF SHIFTING OF THE MAJOR FLOW CHANNEL

From study of maps and airphotos of the Ganges, Jamuna and Meghna rivers, three distinct channel shifting processes have become apparent:

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1. A gradual downstream migration of the major flow channel.
  2. Rapid shifting of the major flow channel such as occurs during the formation of a cutoff within the bankfull channel, and
  3. Sudden creation of new channels that are outside of the bankfull channel and on the local floodplain. This process is more common to braided rivers such as the Jamuna (Brahmaputra).

The bankfull channel and the major flow channel are defined graphically in Figure 2.

#### Gradual Migration

Whenever rivers flow in single defined channels, the shifting of the main flow channel is referred to here as "gradual migration". The main flow channel shifts by eroding its banks as opposed to developing totally new channels where none existed previously.

The down-river movement of a meander bend occurs by progressive erosion of the concave bank and deposition in the form of point bars along the convex bank of river bend. This is graphically indicated by the migration of the Ganges River in the vicinity of the Hardinge Bridge (see Figure 3). Over a period of 42 years, the river has moved approximately 1-3/4 miles eastward which corresponds to a migration rate of about 200 feet per year. The existence of clay within the riverbank at Sara has kept the migration to a rather low value as compared to shifting at other locations. Training works in the form of guide banks have been constructed to prevent migration of the river at the bridge.



Further downstream, at the Ganges-Jamuna confluence the migration of the main flow channel has been somewhat erratic. From 1952 to 1973, the Ganges did not change its location significantly; Figure 4, an ERTS satellite photo, shows the channels in 1973. Over the next two flood seasons, 1973 and 1974 (the floods take place in July and August), the Ganges migrated approximately 4,000 feet northward (shown in Figure 5) and during the 1975 flood it migrated a further 5,300 feet to the north (shown in Figure 6.) In comparing the locations of the Ganges in the various figures, the old meander loop (point A) is visible in all the photos and maps. The reason for this rapid northward migration is not clear, especially since the flood flow of 1975 was smaller than that of 1974, but it is apparent that the migration is highly erratic. Since 1974, the channel has not shifted significantly.

In studying bankline erosion it is, therefore, important to study the river over long time periods. When shifts can vary from 0 to 5300 feet in one year it appears that there is no steady rate of shifting. Major shifting would probably take place in short time periods as suggested in Figure 7.

Proceeding to the Meghna River, the gradual migration of the Meghna eastward has been carrying on for years (see Figs. 8 and 9). This migration has endangered the Town of Chandpur and has also resulted in erosion of several miles of embankment of the Chandpur Irrigation Project. The maximum migration rate south of Chandpur has been 1400 feet/year (see Figure 9), but this has now been reduced by bank protection at Chandpur.

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From field measurements and airphotos, the rate of gradual migration of the Ganges, Jamuna and Meghna varies from 200 to 5500 feet per flood season. In designing engineering works along these rivers the possibility of large amounts of migration must be allowed for.

#### Rapid Shifting of Major Flow Channel

Rapid shifting of a major flow channel takes place when the main channel switches from one side of an island or bar to the other or when a cutoff is formed. This shifting is difficult to predict. Also, once a main channel shifts, the process of gradual migration of the channel carries on within the new channel. Rapid shifting usually takes place within the bankfull channel, from one main bank to the other. This shift can take place in one flood season.

An example of rapid shifting of the main flow channel is the development of a cutoff on the Meghna River, to the east of Mear Char (Figure 8). As the main flow channel slowly migrated eastward near Chandpur, more and more flow entered the secondary channel on the east side of Mear Char. From 1963 to 1973, the left bank near Hajimara was migrating slowly at approximately 300 feet per year (see Figure 9). However, during the 1974 flood, this secondary channel became the main flow channel; the left bank eroded approximately 2000 feet and the original main channel slowly silted up (see Figure 8).

The development of cutoff channels makes it difficult to predict amounts of bank erosion; a bank that has not moved for years can suddenly undergo rapid erosion when the main flow channel shifts rapidly.



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### Creation of New Channels Outside Active Flood Channel

As well as gradual bank migration and the rapid shifting of the major flow channel, the Jamuna (Brahmaputra) River experiences major changes when new channels are created outside the bankfull channel. Figure 10 illustrates the braided nature of this river. Several small channels near the river are also in evidence; these channels are either remnants of the main channel or newly created flow channels which could become large channels during the coming flood. Consider the small channel A-A during the coming flood (this river floods every year). This channel could well take more flow than it has in the past; the main flow channels may develop bars due to the heavy sediment load being introduced into the Brahmaputra, and after several floods this channel could be the main flow channel. This would mean a shift in the bank line of approximately one mile to the east. This shifting process is similar to that of cutoff development except for the fact that the newly created flow channels are usually further away from the main flow channel.

The prediction of the future location of a river-bank for a braided river is, therefore, very difficult.

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## CONCLUSIONS

The construction of engineering works along or across any of the large rivers of Bangladesh requires an in-depth evaluation of the types of channel shifting that can take place.

Various conclusions arrived at are as follows:

1. The shifting of major flow channels on the Jamuna River takes place in three ways.
  - a. gradual migration
  - b. rapid shifting
  - c. development of channels on the floodplain
2. The shifting of major flow channels on the Ganges, Padma and Meghna is generally by the first two of the above processes: by gradual migration and rapid shifting.
3. The prediction of gradual migration rates is very difficult; the rates vary widely, from 200 feet/year to as much as 5,500 feet/year.
4. The development of new channels on the floodplain along the Jamuna River can result in major channel shifts.
5. The possibility of rapid shifting or cutoff development can only be assessed from study of successive air photos.



ACKNOWLEDGEMENTS

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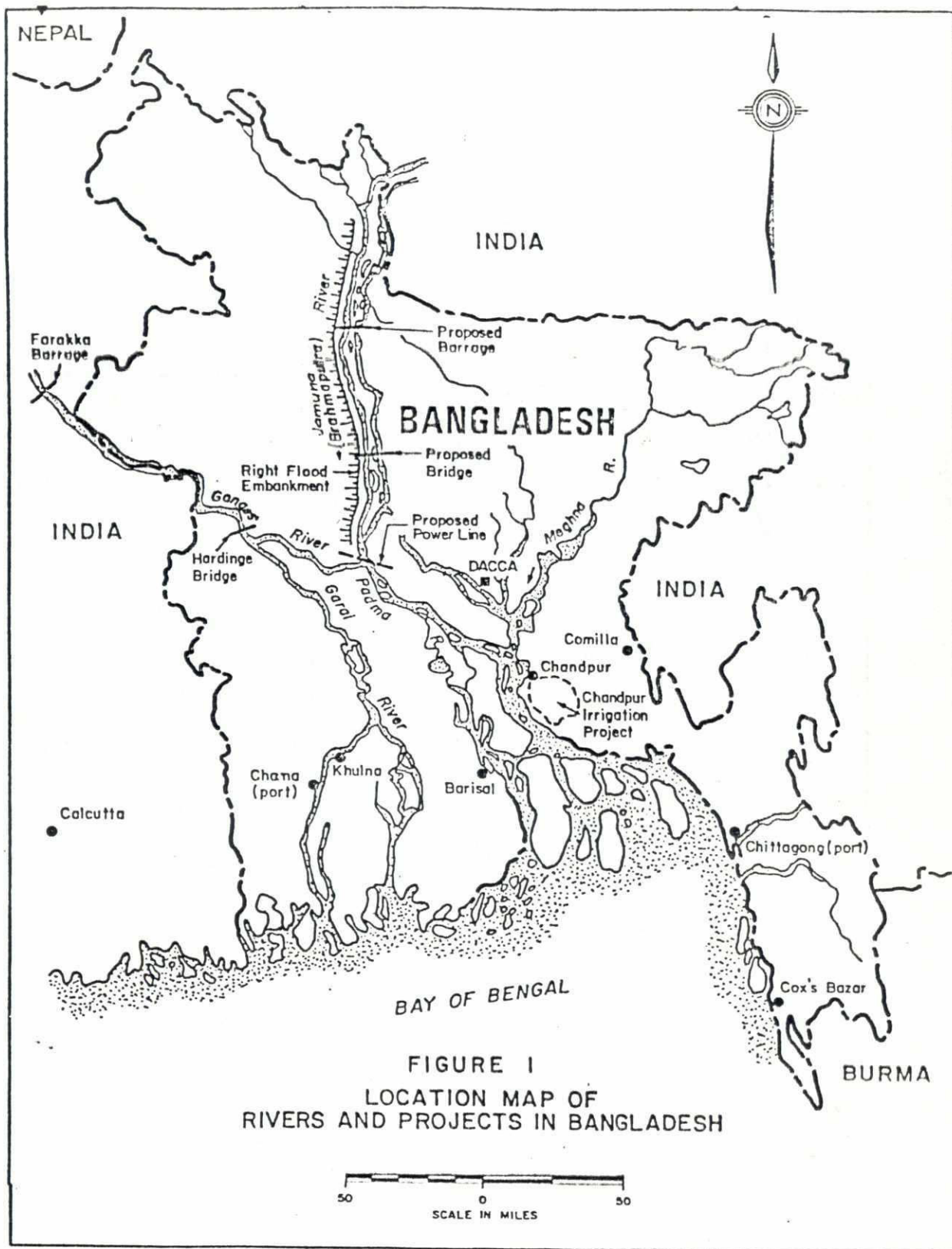


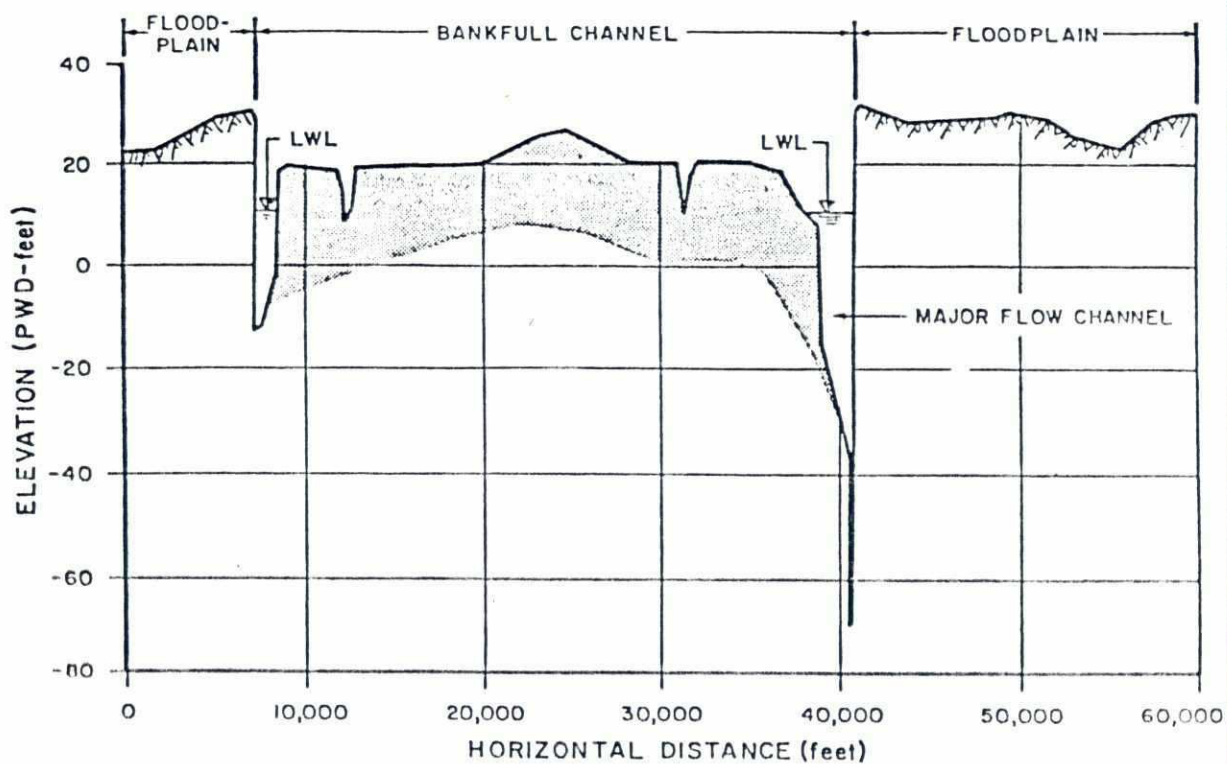
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NOTE:  
CROSS SECTION VIEWED LOOKING DOWNSTREAM

SOURCE:  
ACRES (1970)

FIGURE 2  
CROSS SECTION NOMENCLATURE  
JAMUNA RIVER AT ARICHA (1967)



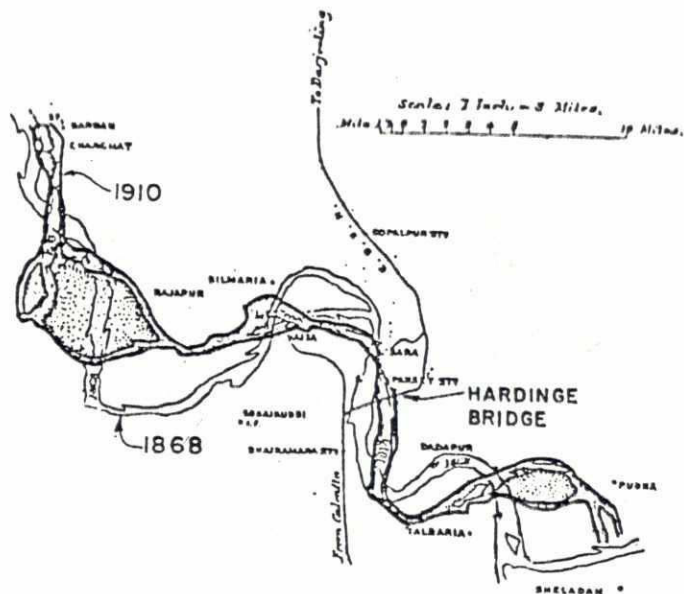


FIGURE 3  
MIGRATION OF THE GANGES RIVER  
AT HARDINGE BRIDGE (1868-1910)

286



FIGURE 4  
GANGES - JAMUNA CONFLUENCE  
( ERTS Photo - Jan. 16, 1973)



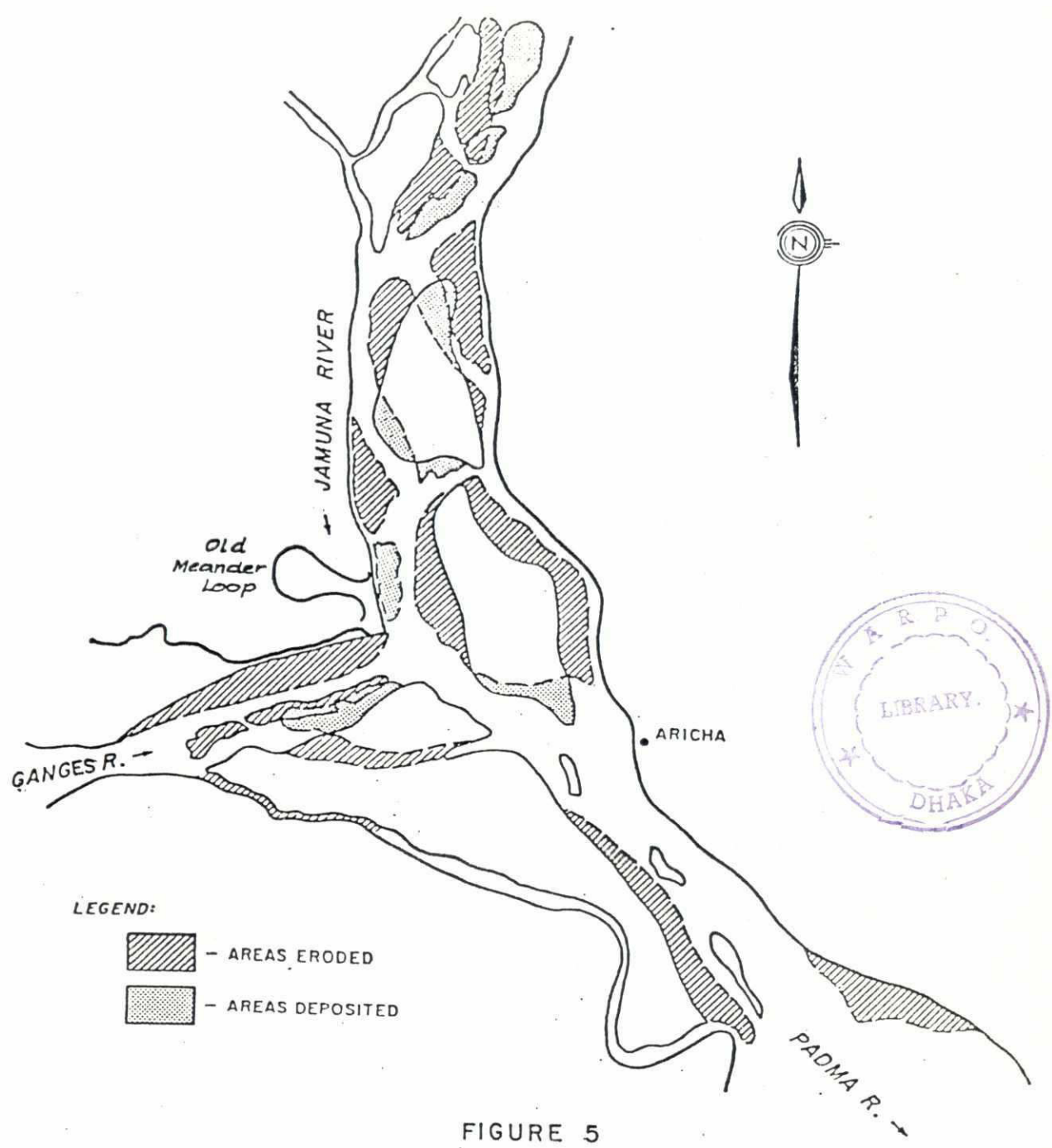


FIGURE 5  
MAJOR CHANGES AT  
GANGES-JAMUNA CONFLUENCE  
DUE TO 1974 FLOOD

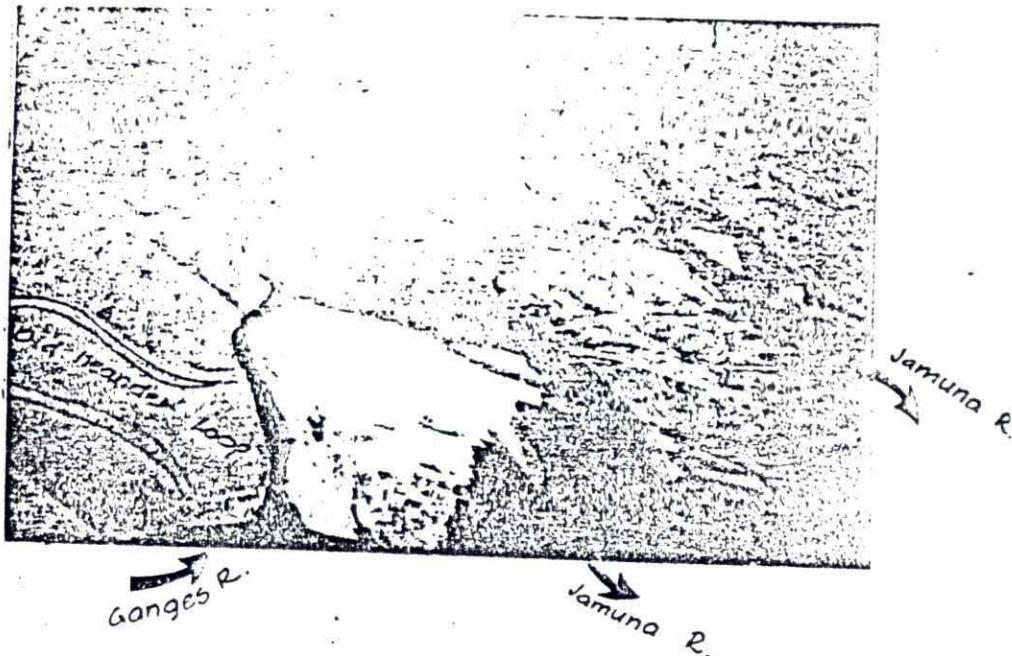


FIGURE 6  
GANGES - JAMUNA CONFLUENCE  
April 16, 1976

THE GANGES RIVER HAS MIGRATED NORTHWARD AND IS  
THE OLD MEANDER LOOP. THE JAMUNA MAJOR CHANNEL HAS  
ALSO SHIFTED AWAY FROM THE WEST BANK (Photo - GALAY, 1976)



CUMULATIVE RIVER-BANK MIGRATION

NOTE : TYPICAL MIGRATION OF GANGES  
AND MEGHNA RIVERS, BANGLADESH

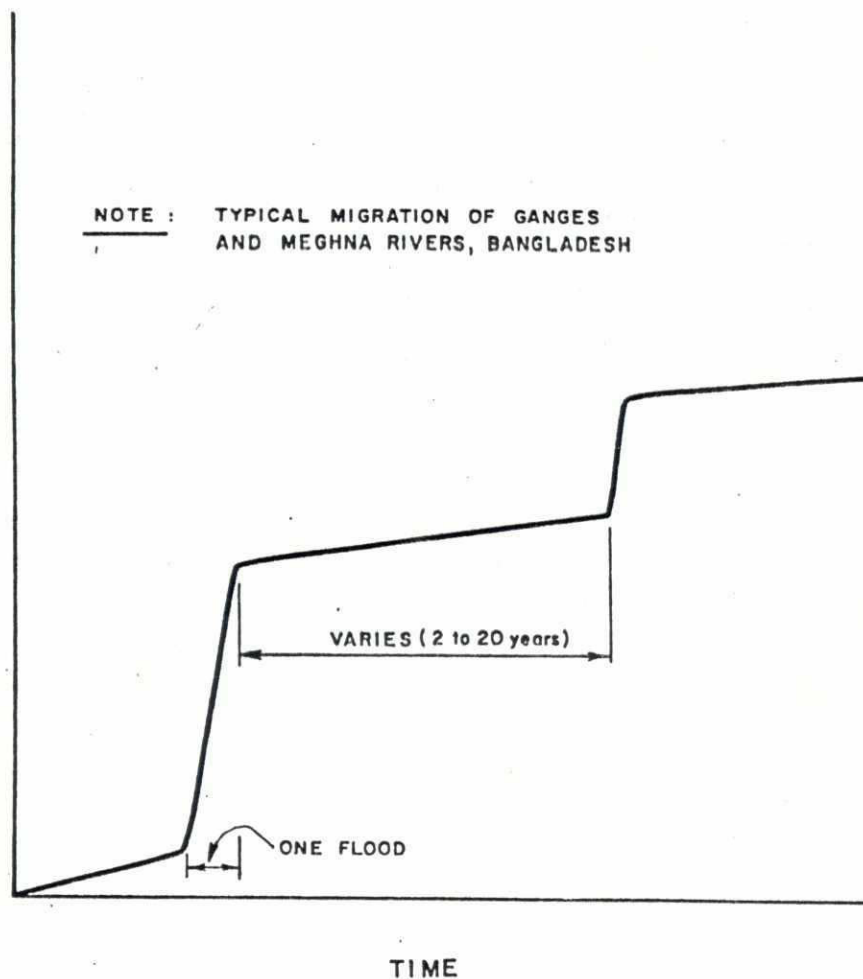


FIGURE 7  
ERRATIC MIGRATION OF  
MAIN FLOW CHANNEL WITH TIME

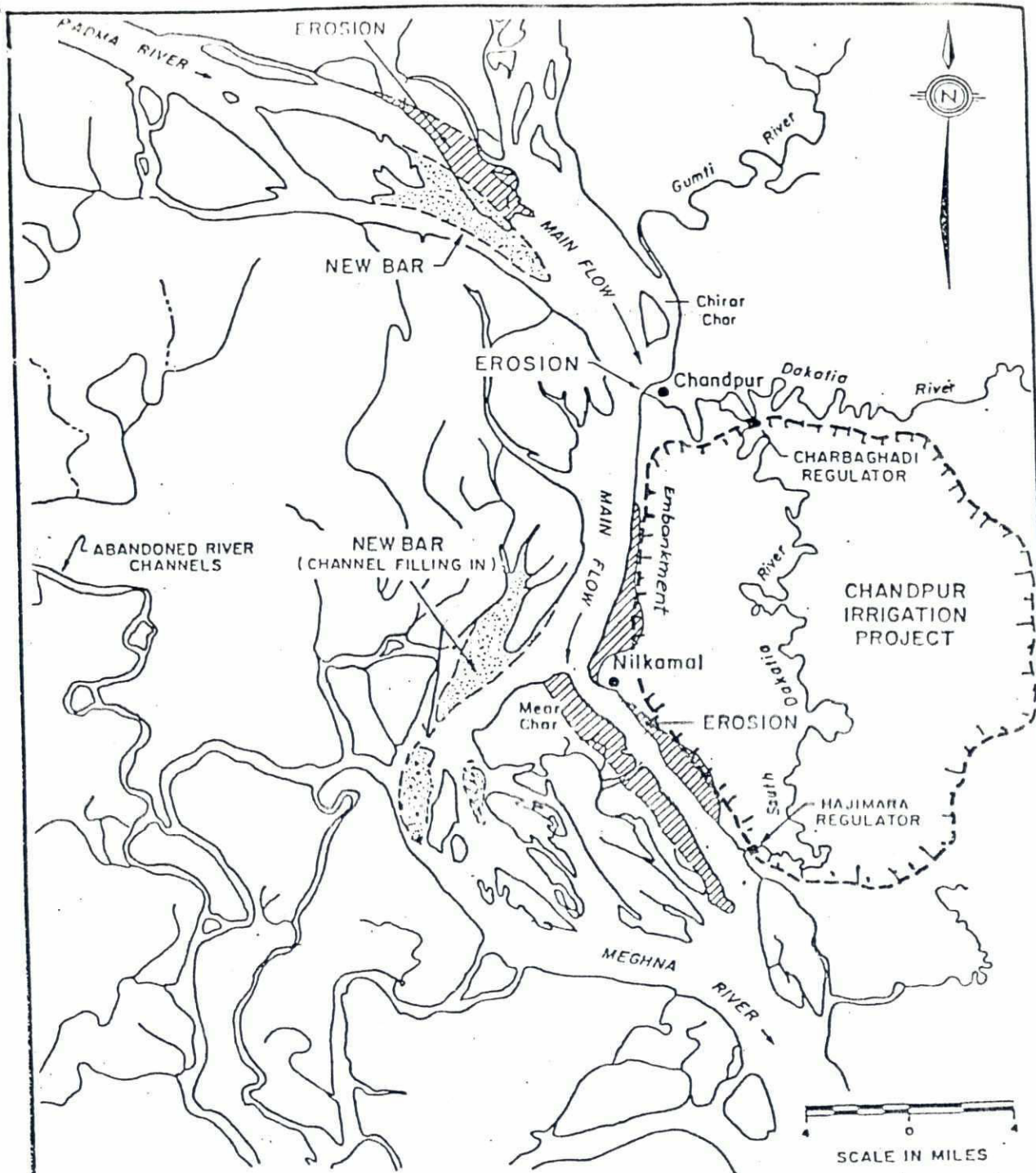


FIGURE 8  
MAJOR CHANGES IN MEGHNA RIVER  
DUE TO 1974 FLOOD



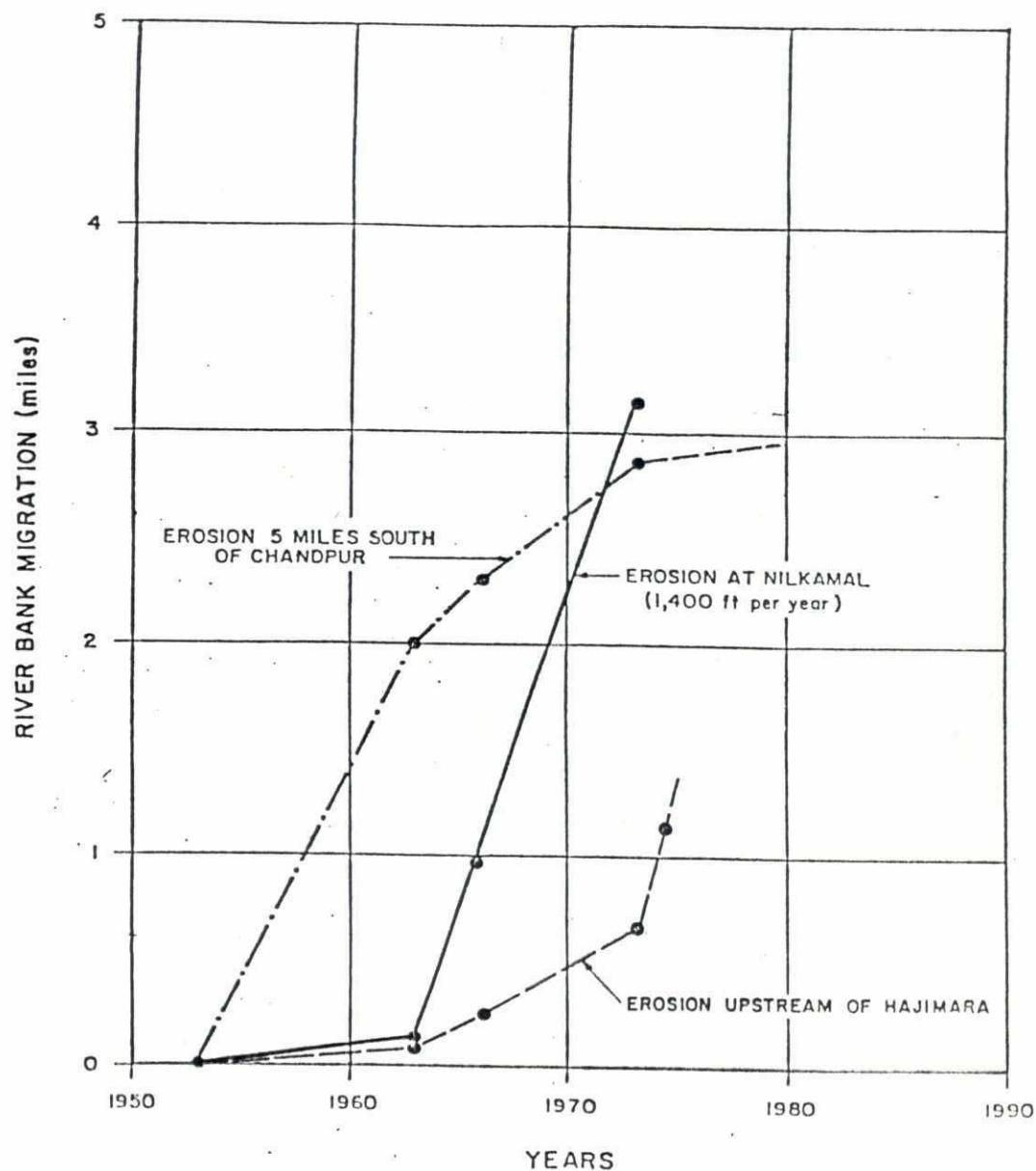


FIGURE 9  
RATE OF EROSION OF THE LEFT BANK  
MEGHNA RIVER NEAR CHANDPUR

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FIGURE 10  
JAMUNA (BRAHMAPUTRA) RIVER  
UPSTREAM FROM CONFLUENCE WITH GANGES RIVER  
(ERTS Photo - Dec. 29, 1972)

C14-20



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## Paper 4.2



BRISTOW, C.S., (1987), Brahmaputra River: channel migration and deposition, *The Society of Economic Paleontologists and Mineralogists*, 63-74pp.



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# BRAHMAPUTRA RIVER: CHANNEL MIGRATION AND DEPOSITION

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**ABSTRACT:** This study follows the pioneering work of Coleman (1969) and presents the results of a study of a 200-km-long reach of the Brahmaputra River in Bangladesh. Channel pattern and migration has been monitored using LANDSAT imagery and historic maps. Channel pattern in the Brahmaputra is varied. The river is mostly braided but some reaches are anastomosed or meandering. Channel movement is dominated by lateral migration with some minor channel switching and one major avulsion in the last 200 years. There is a hierarchy of channels in the Brahmaputra. First-order channels encompass the whole river and may comprise several second-order channels which, in turn, have third-order channels within them. The channels divide and rejoin around bars which scale with the bankfull width and depth of each channel. Bar types include lateral (point) bars, diagonal bars, medial bars and tributary bars. Bars sometimes coalesce to form semi-permanent bar assemblages. Channel cross sections observed by echo-sounding show a high degree of channel asymmetry and extremely complex cross sections which can be related to bar development. The pattern of deposition is divided into four styles: new mid-channel bars (13 percent), channel abandonment (15 percent), lateral accretion to bank (19 percent) and addition to bars (53 percent). The latter is the most important and is subdivided into three elements of deposition: upstream accretion (14 percent) downstream accretion (29 percent) and flank accretion (57 percent).

## INTRODUCTION AND PREVIOUS WORK

Current models for sand-bed braided river deposits depend heavily upon studies of relatively small rivers (Smith, 1970; Williams and Rust, 1969; Collinson, 1970; Cant and Walker, 1978; Blodgett and Stanley, 1980), yet many ancient fluvial sand bodies were deposited by braided rivers of probably much larger magnitude (Campbell, 1976; Conaghan and Jones, 1975; Kirk, 1983). The earliest study of the Brahmaputra was made by Rennell (1781) who first described the growth and formation of bars around a point of nucleation. A more detailed study was made by Coleman (1969) which revealed very large bedforms with rapid rates of migration and high rates of channel movement. His studies of bedform patterns and migration revealed a recurring sequence of events during single flood cycles. The most significant modifications occur during falling stage when sediment is deposited within the channels, accompanied by rapid changes in local flow directions and migration of the thalweg. Coleman also used aerial photographs and maps to show braid bar migration in response to channel movements and large-scale river migration (from geomorphic mapping). Coleman studied paleocurrents from crossbedding measurements on exposed bars and described some of the internal structures. Other work on the Brahmaputra includes engineering studies (Latif, 1969; J.I.C.A., 1976) and a study of suspended sediment transport and basin aggradation (Goswami, 1983).

This study provides additional information on rates of channel migration and deposition and their patterns to gain a more comprehensive understanding of the deposits of a large sand-bed braided river. Channel movements have been recorded from historical maps and LANDSAT images. These reveal large and smaller scale patterns of channel migration and deposition. Field work has included some studies of deposits at low flow stage and the collection of 400 km of echo-sounder traces over a 2-month period during and immediately following the major 1984 flood. This paper presents the conclusions of this first phase, viz. investigation of the patterns of channel morphology and migration. Future reports will deal in more detail with bedform distribution, flow dynamics and floodplain sedimentation.

## RIVER HYDROLOGY

The Brahmaputra River is 2840 km long and drains an area of some 380,000 km<sup>2</sup> from Tibet to the Bay of Bengal. The section of the river described in this paper is a 200-km-long reach in Bangladesh immediately upstream from the confluence with the River Ganges (Fig. 1). This is similar to the area studied by Coleman (1969). The study reach is a multichannel system with a braided appearance in plan view. The maximum discharge recorded in the river is 76,500 m<sup>3</sup>/sec in 1970, and the maximum discharge recorded during this study was 75,700 m<sup>3</sup>/sec (18/9/84) (data from Bangladesh Water Development Board gauging station at Bahadradab, Fig. 2). This discharge is in excess of the predicted 10-year flood and gave rise to widespread flooding and overbank deposition. The minimum discharge in 1984 was 3,950 m<sup>3</sup>/sec (20/2/84). The large variation in discharge results from the seasonal monsoon rains.

During the present field study, the maximum measured surface velocity in the Brahmaputra was 3 ms<sup>-1</sup>. Mean flow depth was 5 m and the maximum recorded depth was 35 m. The width of the river varies from 3 km to 18 km but the average width is about 10 km. Width/depth ratios for individual channels vary from 50:1 to 500:1. The gradient of the Brahmaputra in Bangladesh is 0.000077 decreasing to 0.00005 near the confluence with the Ganges.

## GRAIN SIZE AND SEDIMENT TRANSPORT

The Brahmaputra River has a high suspended load of up to 4,544 ppm (Latif, 1969). The suspended load is dominantly silt with some very fine sand (Goswami, 1983). Total suspended load discharge has been estimated at 725 million tons per annum (Latif, 1969), which is close to the maximum 696 millions tons reported by Coleman (1969) and somewhat more than his calculated mean annual suspended sediment discharge of 478 million tons. It has not been possible to measure accurately total sediment transport in the Brahmaputra River, but some estimates have been made based on measurements of suspended sediment discharge. Goswami (1983) used a series of different formulas to calculate bedload and total load discharge. Goswami





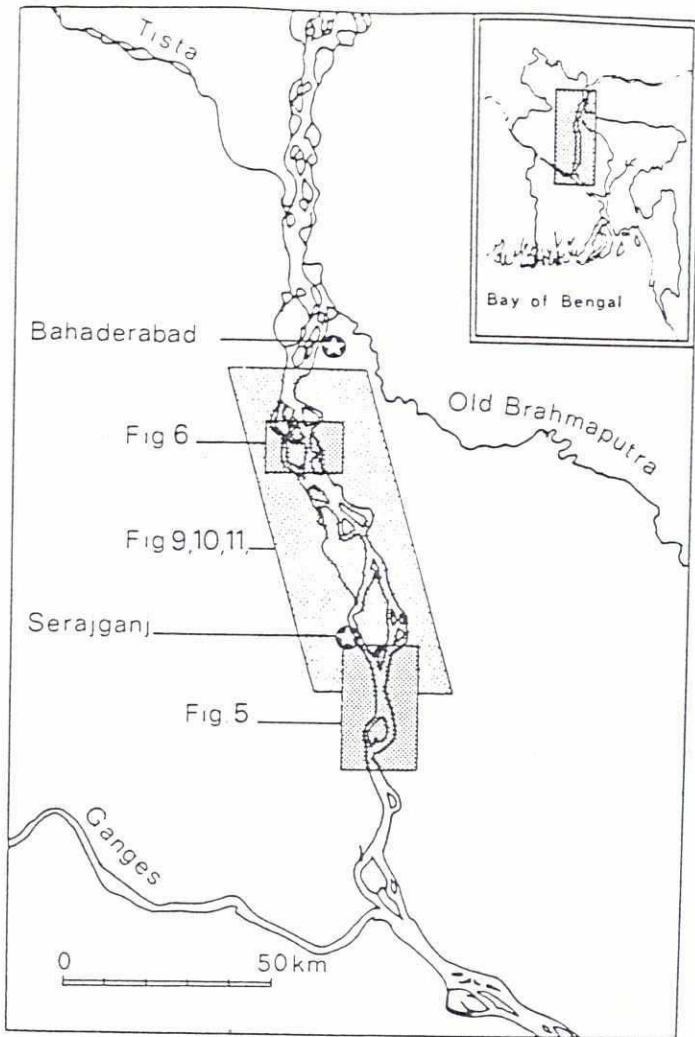


FIG. 1.—Location of the study area within Bangladesh. Shaded areas indicate locations of Figures 5, 6, 9, 10 and 11. Old Brahmaputra River is the former course of the Brahmaputra before the avulsion which occurred around 1787.

preferred the estimate of 15 percent bedload based on the formula of Laursen (1958). This gives a total annual sediment discharge of 4 million tons, two orders of magnitude less than the estimates of Coleman (1969) and Latif (1969). The difference is due to an anomalous measured suspended load of 3,520,631 tons (Goswami, 1983). Coleman (1969) estimated bed load discharge at 25 percent but later (in Cur-ray and Moore, 1971) suggested that the percentage may be as high as 50 percent.

#### *Bed load grain size.*—

Bed load grain size changes spatially and temporarily in response to changes in sediment supply and discharge. Measurements of bed load grain size in Assam (Goswami, 1983) do not show a progressive downstream decrease in grain size (Fig. 3). This is due to the large number of tributaries contributing sediment of varying quantity and calibre along the Brahmaputra. Temporal changes in bed load grain size have been observed in Bangladesh (J.I.C.A., 1976) by sampling bed load each month at two localities (Fig. 3).

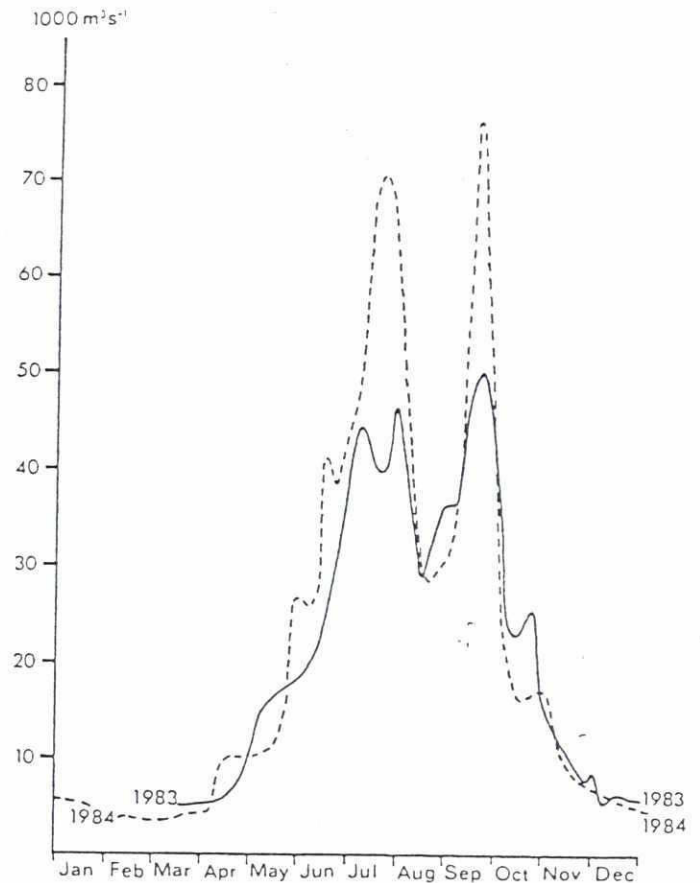


FIG. 2.—Brahmaputra River hydrograph for 1983 and 1984. Discharge measurements made by Bangladesh Water Development Board at Bahaderabad. Peak discharge of 75,700 m<sup>3</sup>/sec on September 19th, 1984 was in excess of the predicted 10-year flood and caused extensive over-bank flooding.

Bed load is finest during low flow, increases during peak flow and again during falling stage. Coarsening of bed load during falling stage may be a winnowing phenomenon. Alternatively, the coarsening of grain size may come from coarser sediment supplied by one or a group of tributaries late in the monsoon season. More observations of bed load supply and transport are needed to resolve the alternatives.

#### THE HIERARCHY OF CHANNELS WITHIN THE BRAHMAPUTRA RIVER

A hierarchy of channels exists in the Brahmaputra River (Fig. 4). The largest first-order channel is the entire river, comprising several second-order channels, which in turn have smaller channels within them (Figs. 5, 6). All the channels are active at bankfull stage but some of the third-order channels become abandoned at low stage. Each channel appears to have a pattern that is formed in response to its slope, discharge and sediment supply. Each channel has slightly different characteristics and as a result each channel behaves differently.

The hierarchy of channels in the Brahmaputra is very different from the one described in the Donjek River (Williams and Rust, 1969) in both scale and process (Fig. 4). In the channel hierarchy of Williams and Rust (1969), each

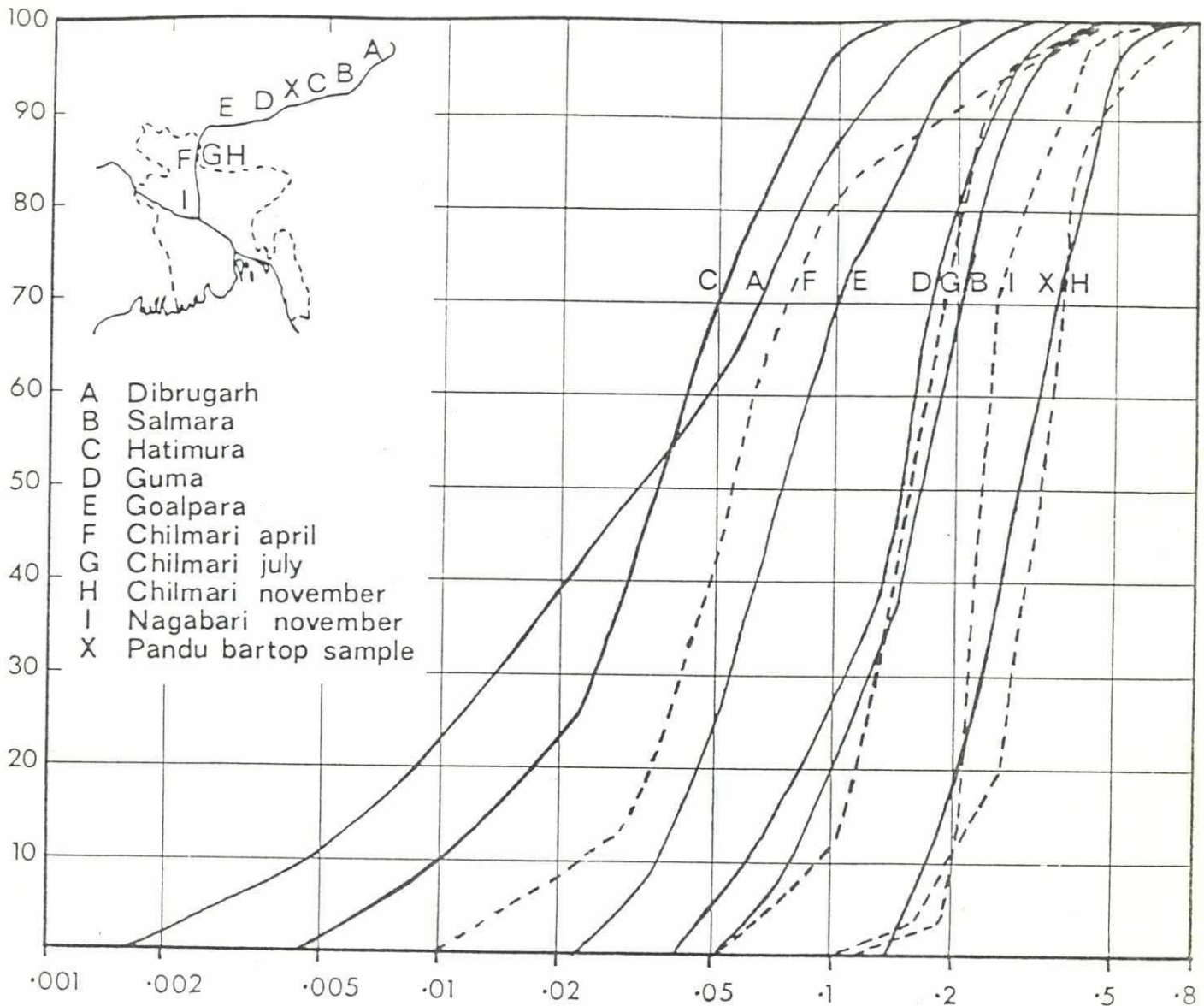


FIG. 3.—Bed load grain size distribution in the Brahmaputra (A-E after Goswami, 1983) shows no downstream decrease in grain size. Samples F-H show seasonal changes in bed load grain size (after J.I.C.A., 1976), which is finest during low flow in April, coarsens during peak flow in July and coarsens again during falling stage in November.

channel scaled with individual bars within the river and its form is dependent on the level of flow around the bar. Thus, the river changes from a single channel at high stage to a multiple channel at low stage. In the Brahmaputra the second-order channels maintain their characteristics and pattern at all stages. Some of the third-order channels may be produced during falling stage, but others appear to maintain their identity at bankfull stage.

#### *First-order channels.—*

The first-order channel includes the entire river (Fig. 4), the channel margin being defined by the outermost river banks. The channel changes its pattern along its length from braided in the north to a more sinuous course in the south. This may be in response to the decrease in gradient. There

are also several narrow straight reaches along the river called node points. Coleman (1969) suggested that the position of these was controlled by more cohesive banks at abandoned channel localities. Historic maps show that node points are not fixed but are transient on a scale of tens of years. One node point identified by Coleman is located just south of Sirajganj. Historic maps (Figs. 7, 8) show that the river has previously been wider here and is not constricted by more cohesive banks but by deposition downstream from a confluence.

#### *Second-order channels.—*

Second-order channels show a variety of patterns. In the area north of Sirajganj (Fig. 7) a large megaform or bar assemblage has developed from a number of medial bars



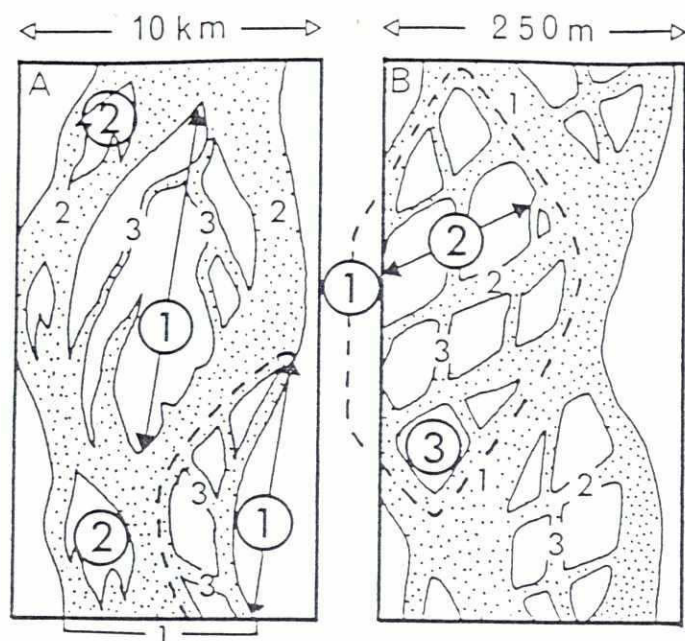


FIG. 4.—Channel hierarchies in the Brahmaputra River (A) and the Donjek River (B) (after Williams and Rust, 1969). Numbers in circles refer to bars, other numbers refer to channels. In the Brahmaputra River the first-order channel comprises the whole river, which includes several second-order channels. Bars scale with the channels in which they occur. Third-order channels modify higher order bars but still have bars within them which are not possible to illustrate at this scale.

since 1954. The western and easternmost second-order channels now have braided and meandering courses, respectively. These channels have maintained fairly constant patterns over the last 10 years (Fig. 8). They appear to be in equilibrium with the local discharge slope and sediment supply. They also migrate independently within the channel belt and could be considered as an anastomosing reach within the braided system. The different behavior of the two channels is probably due to different discharges, assuming other factors to be constant, the braided channel having a greater discharge than the meandering channel.

#### Third-order channels.—

Third-order channels have been observed with meandering and braided courses. Some occur on the top of first- or second-order bars and are produced in response to a hydraulic gradient across bar tops flowing from the main channel into the chute channel. Some third-order channels have been observed by echo-sounding at high stage, however. These may be produced by multiple flow cells in channels with a very high width/depth ratio. There is insufficient data to distinguish the form of these channels or to say if channels exposed at low stage are similar to those observed at high stage.

#### MACROFORMS OR BARS

The type of bars and their distribution within the Brahmaputra have been identified using LANDSAT images, hydrographic charts, echo-sounder surveys, maps and field

examination. A limited number of bar types (macroforms using the terminology of Jackson, 1975; Church and Jones, 1982; and Bridge, 1985) can be recognized. These include lateral bars (including point bars), medial (braid) bars, diagonal bars, scroll bars and tributary (chute) bars. The bars scale with the bankfull channel dimensions. Bar height is between one-half and just less than total bankfull depth. Bar length from crossover to crossover is between 3 and 5 times channel width. Individual bars vary in size because of the hierarchy of different channel sizes. Bars have a time span which is greater than or equal to the passing of a single flood event, which in Bangladesh can be assumed to equal 1 year.

At low flow stage, water level within the Brahmaputra falls as much as 8 m and large areas of bar top are exposed. These emergent areas range in length from a few tens of meters to 15 km. The size and shape is the result of complex sequences of erosion and deposition. Great care has to be taken in distinguishing partially exposed areas of bars in the field from modified mesoforms (dunes or megaripples) (cf. Church and Jones, 1982; Bridge, 1985). Hydrographic charts which show the full three-dimensional form of bars are the most useful data source in this respect.

Lateral (point) bars (Fig. 5D, E; Fig. 6C, D) occur on the inside of channel bends and in straight reaches. They are attached to the bank at their upstream ends and generally have subsidiary scroll bars and chute channels (Fig. 5E). Lateral bars are commonly modified by third-order channels which flow over the bar from first- or second-order channels into the chute channel, at which point third-order tributary bars have been observed. One sinuous chute channel observed at low flow stage had point bars which scale with bankfull width of the chute channel. Medial (braid) bars (Fig. 5C; Fig. 6A, B) are detached forms which are deposited in areas of flow divergence. With further flow divergence and channel widening, a more complex medial bar may be formed with scroll bars attached to one or both sides and with third-order channels dissecting it. Tributary bars (Fig. 5A, B; Fig. 6E, F) occur at junctions of channels of the same or different order. Figure 5A shows a tributary bar at the junction of two second-order channels; Figure 5B shows a tributary bar deposited where a third-order chute channel enters a second-order channel at the downstream margin of a medial bar which may be evolving into a diagonal bar. Diagonal bars (Fig. 6G) are attached to the bank at both ends and occur at crossovers where flow diverges as the main current moves from one bank to another.

#### Bar assemblages or megaforms.—

In several places bars become coalesced to form bar assemblages (the sedimentation zones of Church and Jones, 1982, or flood plain segments of Romashin in Church and Jones). The length of bar assemblages from crossover to crossover is around 30 km. This does not scale with first-order channel width (3 km at node points). Bar assemblages are associated with Channel anastomosis in the Brahmaputra (Fig. 7). Church and Jones interpreted bar assemblages as the product of locally increased sediment deposition. Relatively rapid deposition is also associated with channel anastomosis in other rivers (Smith and Smith, 1980).



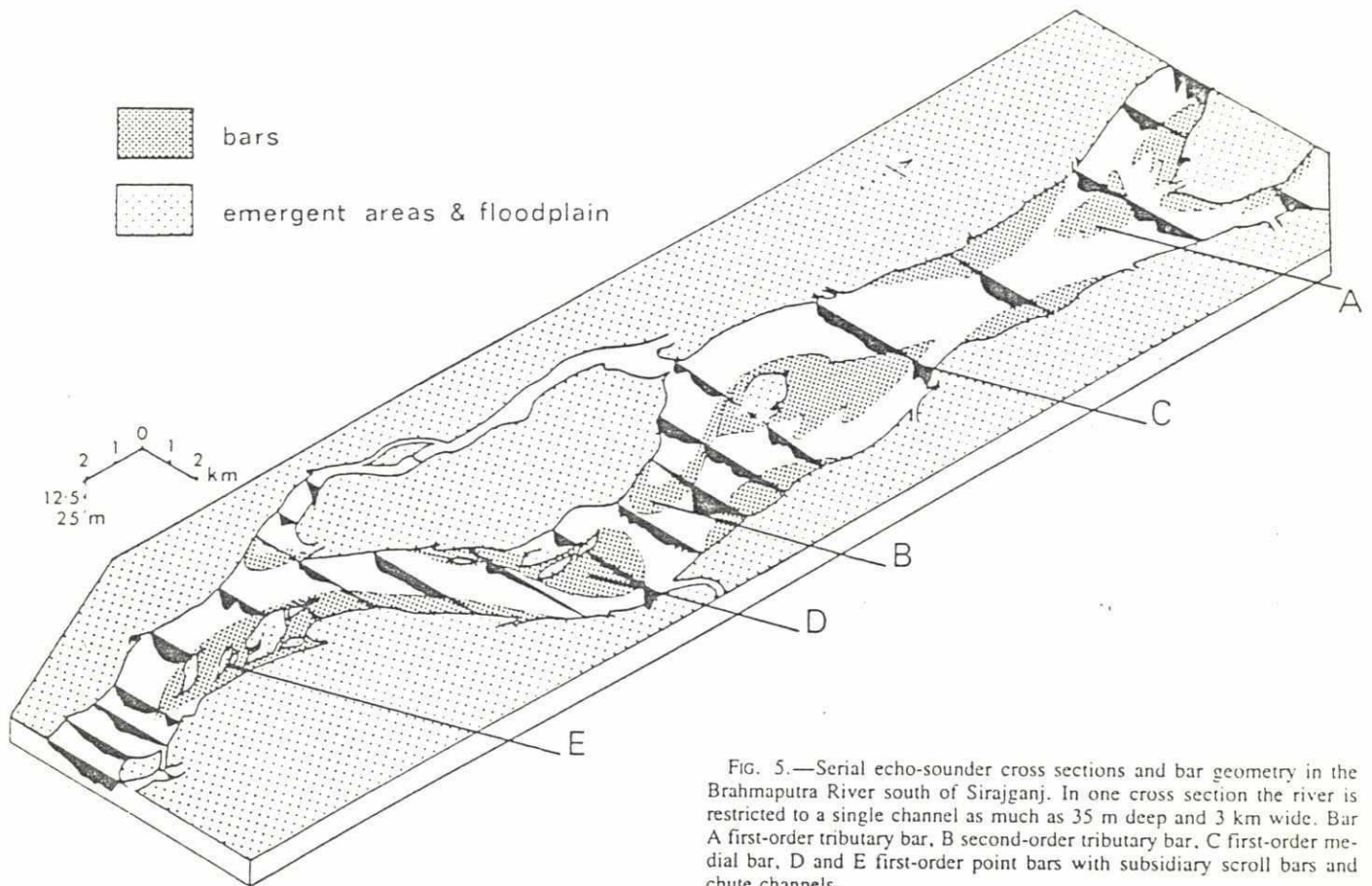


FIG. 5.—Serial echo-sounder cross sections and bar geometry in the Brahmaputra River south of Sirajganj. In one cross section the river is restricted to a single channel as much as 35 m deep and 3 km wide. Bar A first-order tributary bar, B second-order tributary bar, C first-order medial bar, D and E first-order point bars with subsidiary scroll bars and chute channels.

The time span of these megaforms is on the order of tens of years (Fig. 7); several of these megaforms are shown in Figure 9.

#### CHANNEL MOVEMENT

Channel displacement in the Brahmaputra River occurs at a variety of magnitudes. First-order channel migration and avulsion (switching of the channel belt) occurs on a larger time scale than second- and third-order channel movements. Coleman (1969) described two distinct types of channel movement: gradual and continuous transverse movement (lateral migration), and sudden shifts (channel switching within the channel belt). The different methods of channel movement produce different patterns of deposition. Lateral migration produces lateral accretion, whereas switching produces channel enlargement in one area and channel fill in another. Both processes have been observed at all levels of the channel hierarchy, and the deposits of the Brahmaputra contain examples of lateral accretion and cut-and-fill at different scales.

#### *First-order channel movements.—*

The movements of the first-order channels have been recorded from historical maps. Rennell (1781) shows the Brahmaputra River running along a course now occupied by the Old Brahmaputra River (Fig. 1). Around 1787 the

Brahmaputra avulsed into its present course, capturing the Tista River. There is some dispute in the literature as to the nature and trigger for the avulsion (Morgan and McIntire, 1959). Coleman (1969) concluded that the change in course took place gradually, due to increased flood discharge, faulting, or a combination of both. Latif (1969) refers only to a devastating flood in 1787. The avulsion resulted in a maximum channel displacement of 100 km.

The avulsion occurred gradually over 50 years. The new course of the Brahmaputra was initially sinuous, as shown in Wilcox's 1833 survey (Fig. 8). By 1867 the river had become braided. This change in pattern was probably due to increased discharge as less water diverted along the Old Brahmaputra. On Rennell's (1781) map the Old Brahmaputra was braided; since the avulsion, this river has assumed a meandering course in response to decreased discharge. The Brahmaputra avulsed into an unnamed tributary of the Konaie River, not the Jenni as reported by Coleman (1969). Maps from 1830 to the present (Fig. 8) show that the river has migrated back and forth within a channel belt approximately 15 km wide. Coleman recognized a net westward migration of as much as 10 km over this 150-year period, a rate of approximately 70 m per annum. This net migration is most apparent in the upstream reaches. There has been less first-order migration downstream and the confluence with the Ganges has remained almost static.

Coleman concluded that net migration westward was due



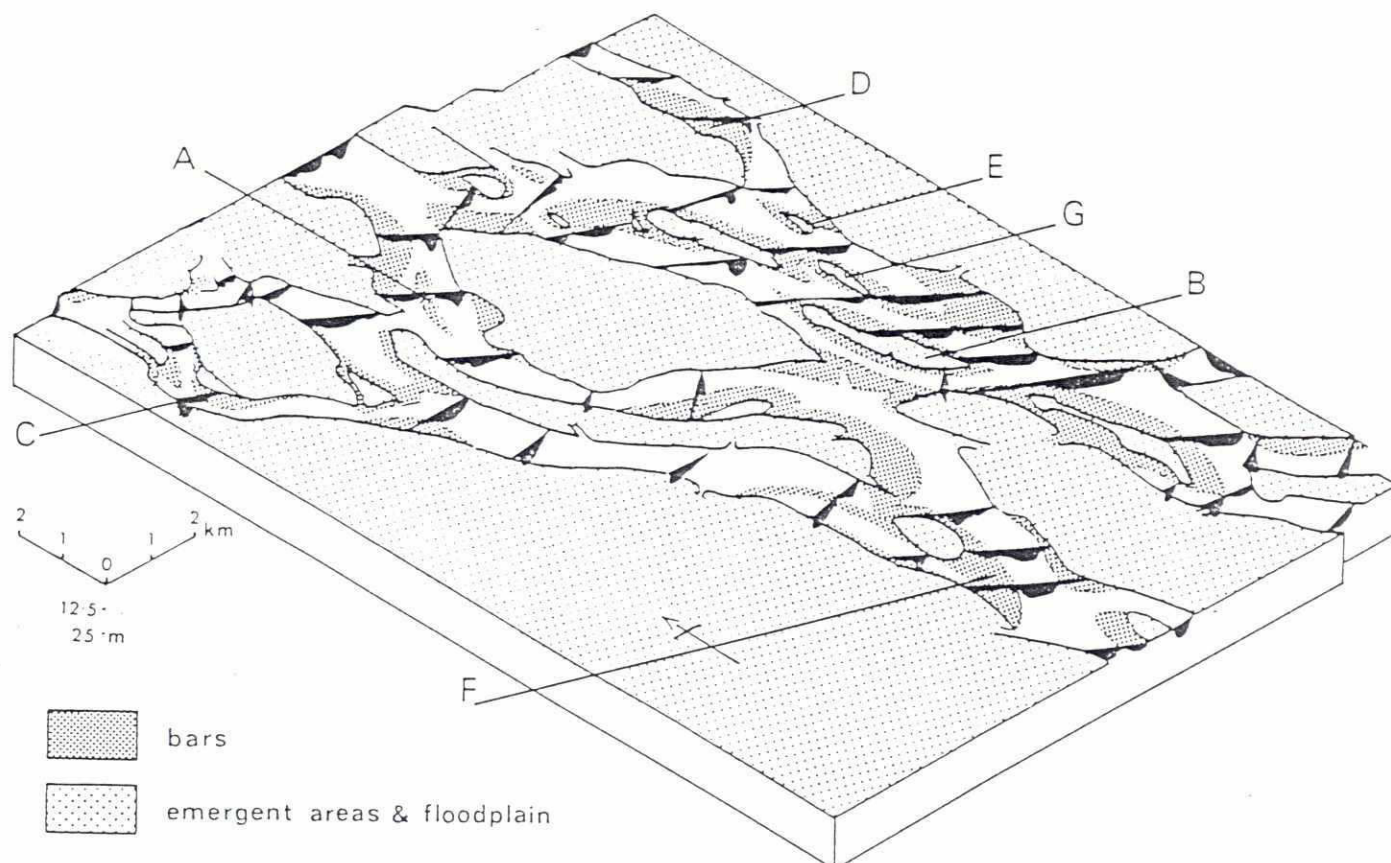


FIG. 6.—Multichannel reach of the Brahmaputra River between Shureakhandi and Maderganj with echo-sounder cross sections and bar geometry. Bars A and B are elongate second-order medial bars. C and D are second-order point bars on the sides of medial bars with strongly asymmetric channel cross sections. E and F are second-order tributary bars. G is a diagonal bar with a series of small third-order channels superimposed.

to tectonic tilting, whereas different rates of migration along the river were substrate controlled. Westward migration is against the regional tectonic slope, but the river course is probably influenced by intrabasinal faults (Morgan and McIntire, 1959). The changing rate of migration may also be due to changes in the pattern of the river downstream and the reduced gradient near the confluence with the Ganges.

#### *Second-order channel movements.—*

Studies of LANDSAT images from 1972–1978 show second-order channel switching and lateral migration (Fig. 9). Second-order channel migration is comparatively rapid; migration of 1 km per annum is not uncommon. This rate of movement exceeds the average figure of 76 m per annum for first-order channel migration and suggests that second-order channels will totally rework any first-order accretion surfaces. Most of the movement of second-order channels is by lateral migration, which has a major effect on the internal sand body geometry.

When a second-order channel erodes one bank, there is generally an equal amount of deposition on the opposite bank (Figs. 9–11). This is contrary to Coleman's (1969, p. 159) observation that "in a braided channel . . . this type of cut and fill does not occur." The difference lies in Coleman's use of bankline studies which were restricted to ob-

servations of the first-order migration without considering the second- and third-order channels, which are fundamental component parts of a multichannel river. Using the channel hierarchy concept, it is easier to explain channel migration and resulting patterns of deposition (see below). It is hoped that the same concept can be used in interpreting ancient sand bodies deposited by multichannel rivers. The movements of the first-order channel control the regional distribution of sand and sand body shape. Second-order channel migration controls the internal stratigraphy.

#### *Third-order channel movements.—*

Third-order channels are too small to be clearly defined on LANDSAT images which have a resolution of 80 m on the ground. Studies are restricted to observations at low flow and echo-sounder traces. Coleman (1969) described a third-order channel moving within a second-order channel and changing its behavior in response to changes in discharge. During the rising stage, movement is gradual lateral migration of as much as 1 km. During peak flood, there is little movement of the third-order channels, but during falling stage the channels switch in a sudden and irregular fashion. Coleman attributed this to deposition of material during falling stage producing changes in local flow direction. Thus, in a period of bed aggradation, channel switching is greatest, many new third-order channels are formed



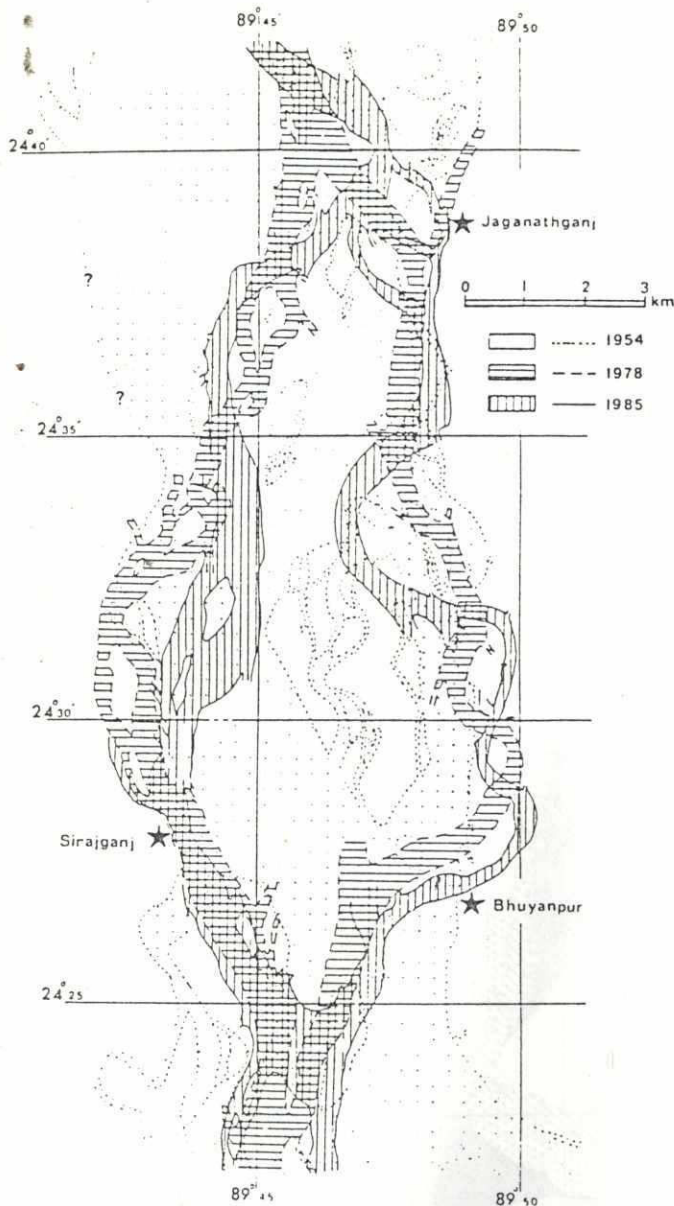


FIG. 7.—An anastomosed reach of the Brahmaputra which has developed following the coalescence of medial bars into a bar assemblage. The division of discharge has produced two contrasting channel patterns in the anastomosed reach. The western channel is braided, the eastern channel is meandering. The bar assemblage megaform has remained stable over the past 15 years, while the second-order channels have migrated independently along either side.

and function for a short time and are then abandoned. The rapid and contrasting changes in movement of third-order channels will produce complex internal erosion surfaces and sequences of deposition (see below). The third-order channels, which are low stage features draining off higher order bars, do not migrate very much due to the infrequent discharge and high cohesion of the substrate on bar tops. Drainage channels may be enlarged during floods and then become preferential sites for higher order channel avulsions, or abandoned and filled with mud plugs at the top of the sand body.

#### PATTERNS OF CHANNEL MIGRATION AND DEPOSITION

Maps showing channel migration and deposition (Figs. 9, 10, 11) have been compiled from LANDSAT imagery with resolution of 80 m on the ground: this is sufficient to monitor migration of channels and bars in a large river such as the Brahmaputra. The images used were all selected for the dry season periods (1972–1978) because at this time of year, there is very little cloud cover and low water level exposes the bars. During flood stage, the high sediment load in the water unfortunately prevents any use of LANDSAT imagery for bathymetric or bedform observations.

The images reveal many changes in channel position (Fig. 9). Areas of deposition have been analysed to show temporal variations (Fig. 10). The style of deposition has been interpreted from these successive images (Fig. 11). The areas of deposition shown in Figures 10 and 11 are areas of net deposition: some of these areas were subsequently eroded so that the gross pattern of deposition is likely to be more complex than that illustrated. The erosion of some bars explains the apparent misnomer of some deposits classified as addition to bars (Fig. 11) when there is no pre-existing bar indicated due to its subsequent erosion.

The styles of deposition recorded here refer only to the deposits resulting from migration of the first- and second-order channels. With LANDSAT imagery, it is not possible to record the movements of third-order channels and bars. Thus, the geometry of sand bodies inferred from the styles of deposition refer only to the large-scale first- and second-order channel deposits. The third-order elements may show a different architecture within this framework. Smaller scale "deposition" patterns from third-order channels and bedforms have been measured from echo-sounder cross sections, aerial photographs, and field observations.

#### Temporal changes in deposition.—

Figure 10 shows the net pattern of deposition from six annual floods on the Brahmaputra River. The amount of erosion, sediment transport and deposition varies from year to year. These are probably controlled by discharge. Very high discharges, or prolonged high discharges, produce greater erosion, channel migration, sediment transport and deposition. The period December 1972 to January 1976 includes the major 1973 flood when overbank flooding was maintained for 2 months. This period shows the greatest amount of change.

#### STYLE OF DEPOSITION

The areas of deposition identified in Figure 10 have been subdivided into four categories: addition to bars, lateral accretion to the bank, new mid-channel bars and channel abandonment (Fig. 11). The relative proportions of these four styles of deposition are shown in Figure 11. From this compilation it is apparent that the most important component of deposition in the Brahmaputra River is addition to pre-existing bars. This category accounts for 53 percent of the area of deposition, whereas lateral accretion to the bank, new mid-channel bars and channel abandonment account for 19, 13 and 15 percent, respectively.



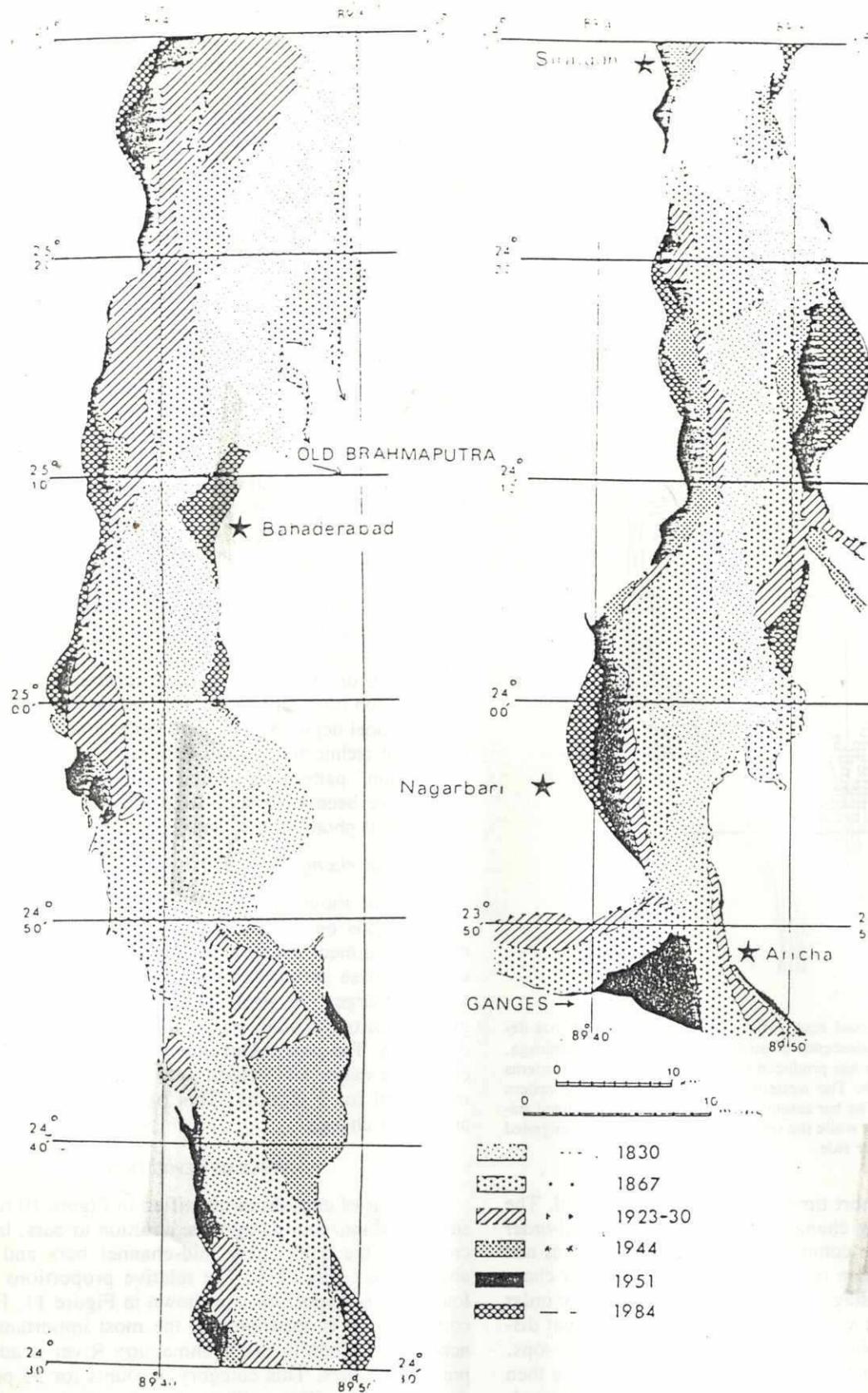


FIG. 8.—Bank line changes over the past 150 years. The river has changed from a sinuous course in 1830 to a braided course by 1867. The channels have migrated back and forth within a 15-km-wide belt which shows a minor amount of net migration to the west.

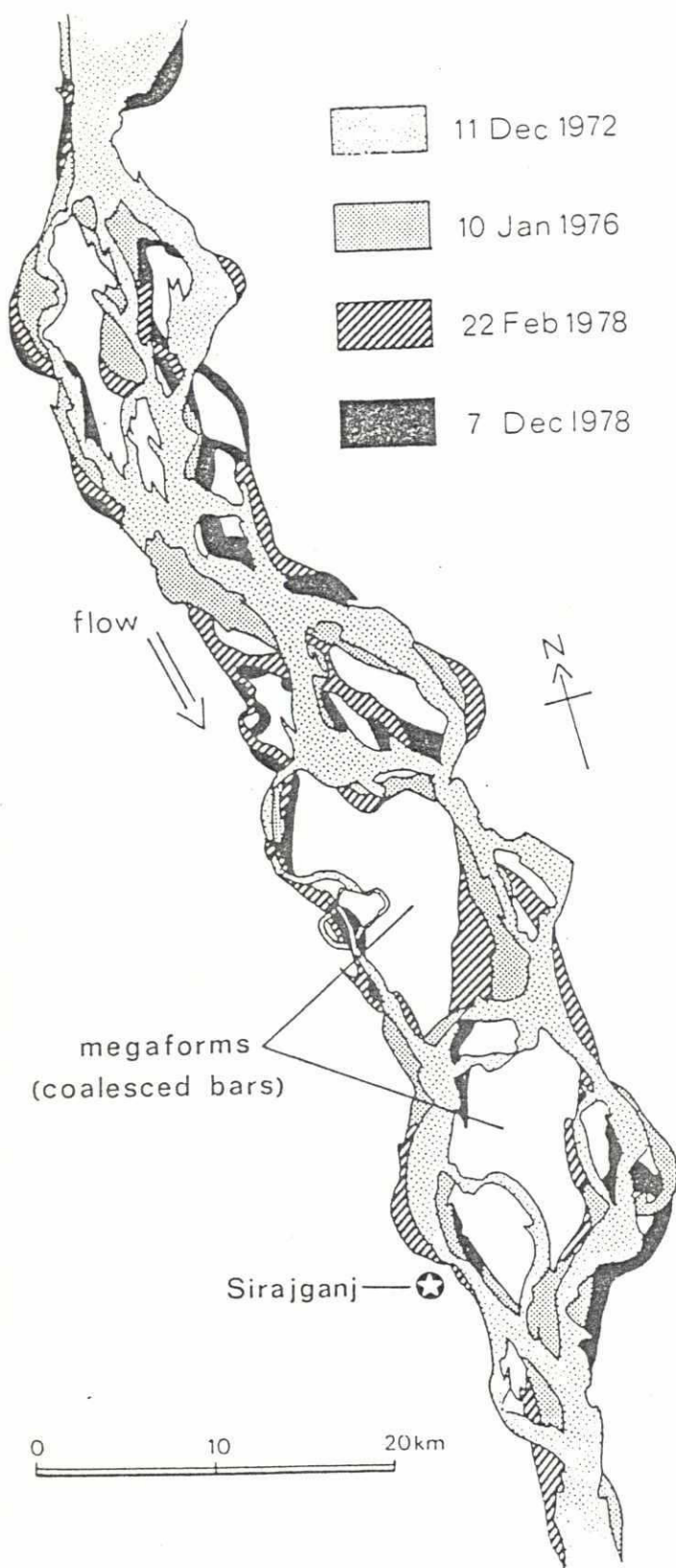


FIG. 9.—Second-order channel locations recorded from LANDSAT images, 1972 to 1978. Most channel movement occurs by lateral migration at rates of as much as 1 km per annum. The megaforms have remained stable over this period.

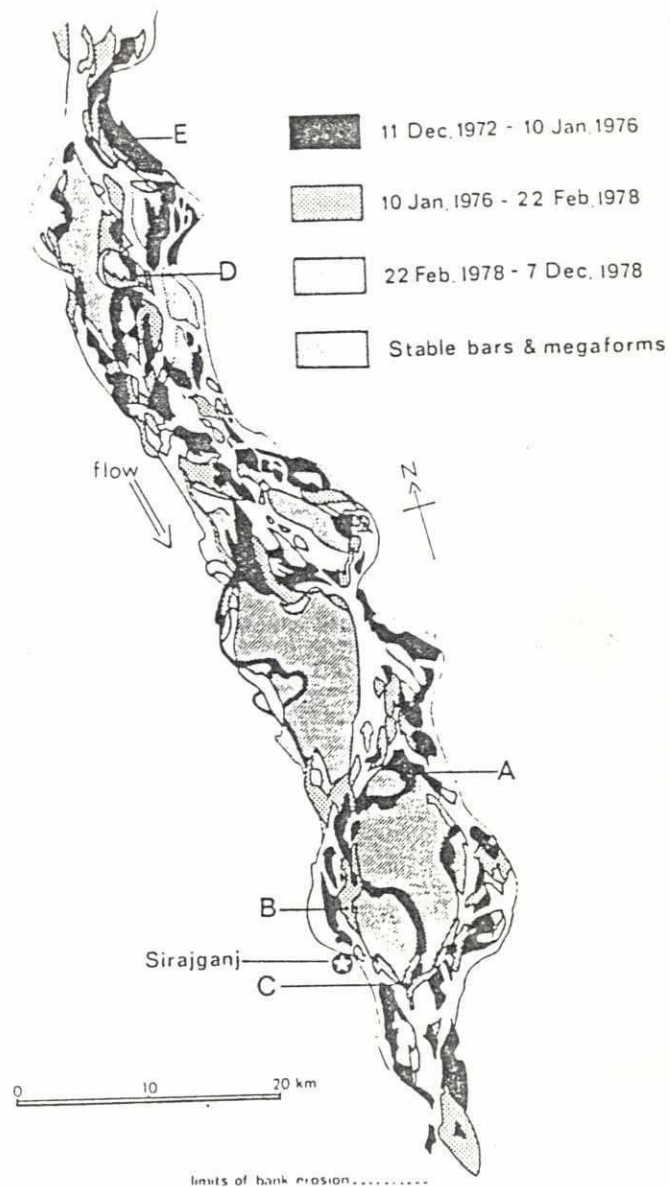


FIG. 10.—Areas of deposition from the map of successive channel locations; most deposition occurs on the margins of the megaforms. A, upstream accretion; B, Flank accretion; C, Downstream accretion; D, shows the development of a complex medial bar from an initial new mid-channel bar, and E is a large point bar accreted on the left bank. Blank areas within the limits of bank erosion are areas characterized by erosion or nondeposition.

#### Addition to bars.—

The addition to bars can be subdivided into three elements of deposition: upstream accretion, flank accretion and downstream accretion. All three elements combine different proportions of vertical and lateral accretion as the bar is enlarged. Upstream accretion (14 percent) may be accomplished by megaripples and scroll bars wrapping around the upstream margin of a bar in a manner similar to that described for the formation of sandflats in the South Saskatchewan River (Cant and Walker, 1978). Alternatively, deposition may occur in a zone of comparatively slack water



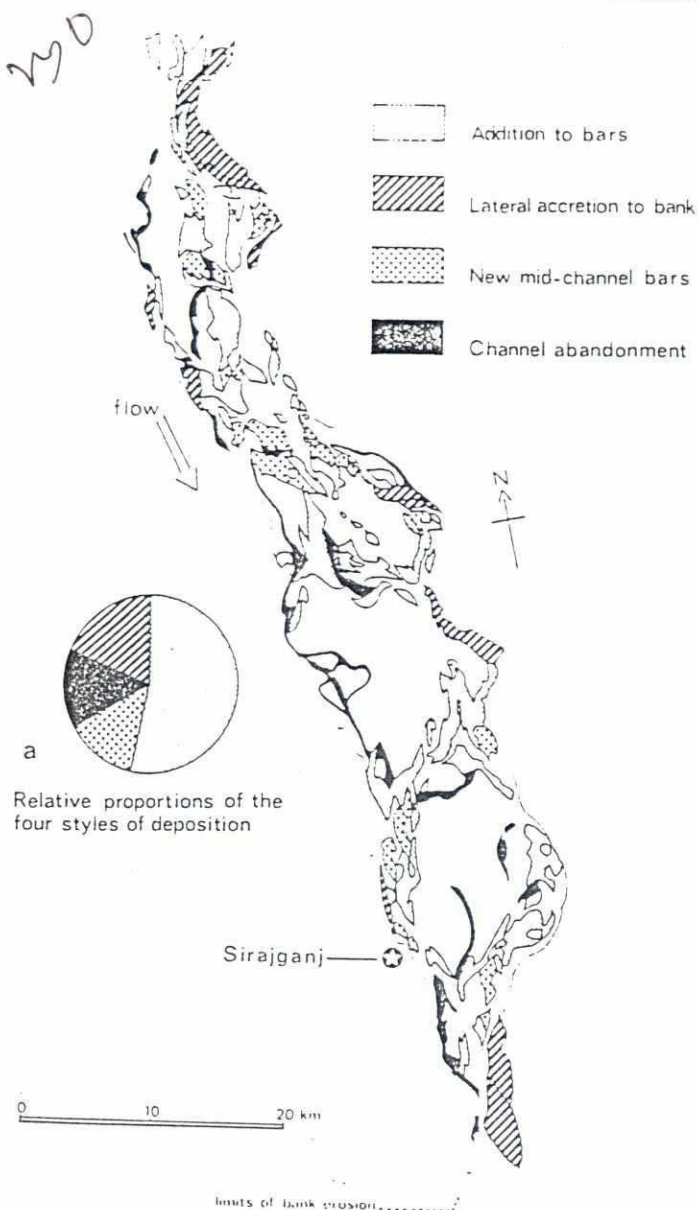


FIG. 11.—Styles of deposition, 1972 to 1978. The areas of deposition identified in Figure 10 are classified according to the area in which they were deposited. This figure represents gross deposition; subsequent erosion in many areas complicates the pattern of net deposition.

upstream from a bar where the current diverges. There is not necessarily a scour pool upstream of every bar as described by Coleman (1969).

Flank accretion is the most important element in medial bar growth (57 percent). Flank accretion is produced by megaripples and scroll bars migrating around and up the flanks of convex bar margins in a manner similar to that described from point bars in meandering rivers. Allen (1968) suggested that braid bars could form as double point bars and Bridge (1985) proposed a similar model. Recognizing this lateral accretion in the deposits is very difficult due to the extreme width/depth ratios and very low angle of lateral accretion surfaces in the Brahmaputra.

The downstream accretion element (29 percent) includes

a considerable amount of deposition from suspension. During falling stage, large flow separation zones exist downstream from bars. In these areas current velocity is reduced and sometimes moves upstream. Deposits are dominated by climbing ripple lamination and mud drapes up to 2 m thick. Megaripples and scroll bars also migrate around the downstream margin of medial bars.

These three elements can be distinguished on the maps of deposition (Figs. 10, 11). The dominance of lateral accretion produces ridge-and-swale topography on bars (Fig. 12) which records successive downstream and lateral growth of the bar. Superimposed on the ridges and swales are natural levees. Thus, the bar is highest at its margins and lowest in the middle. Similar accretion topography has been observed on bars in the low sinuosity Calamus River (N. D. Smith, and J. S. Bridge pers. commun.).

#### *Lateral accretion to bank.—*

Lateral accretion to the bank comprises 19 percent of the deposition, most of which is in the form of large point bars as much as 10 km long and 3 km wide (Fig. 8E). In some localities the lateral accretion may be onto a bar margin which has been incorporated into the floodplain following channel abandonment. The largest areas of lateral accretion are all on the east bank of the river (Fig. 8), which correlates well with Coleman's observations that the Brah-

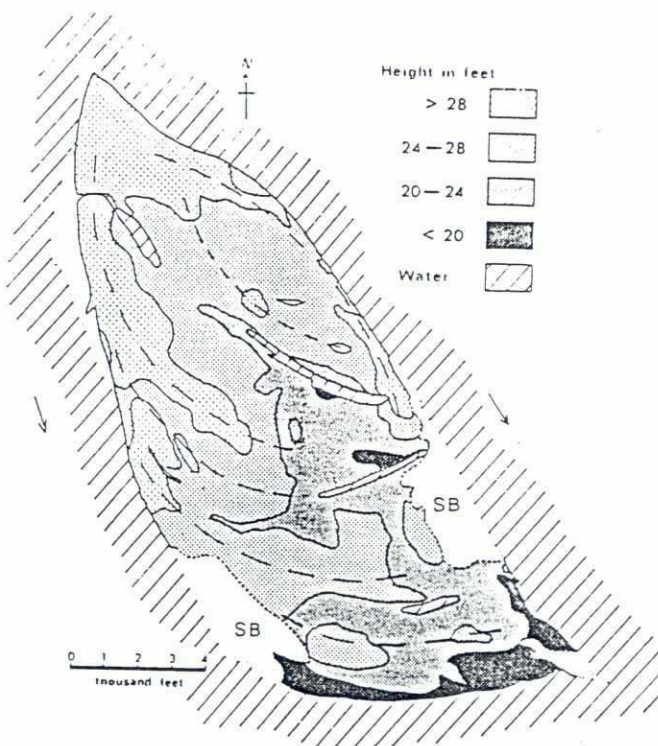


FIG. 12.—Bar topography (compiled from maps with 1-foot contours) shows ridge and swale topography from lateral and downstream accretion. Lateral accretion is accomplished by scroll bars (SB) wrapping around the downstream margin of the bar. The highest areas of the bar are along the upstream and lateral margins which have been raised as natural levees. The lowest part of the bar is along the center and downstream margins, where flood waters are ponded and sediment is deposited from suspension.



maputra River is migrating westwards. It is interesting to note that Coleman interpreted all the recent deposits of the Brahmaputra as those of a meandering river with very broad low-curvature meander loops. This interpretation is supported by the 1830 map, but the large component of lateral accretion observed in bar growth (57 percent) produces a topography similar to ridges and swales on point bars (Fig. 12). This presents problems for the interpretation of recent and ancient deposits of a large braided river based on the conventional association of lateral accretion with meandering rivers.

#### *New mid-channel bars.—*

New mid-channel bars form the smallest component of the styles of deposition 13 percent. This is due to the small size of new bars. New mid-channel bars generally have an elongate diamond or petaloid form. If the channel widens, they may enlarge into a more complex medial bar.

#### *Channel abandonment.—*

The channel abandonment comprises 15 percent of the deposition recorded, which is considerably less than expected from previous models of braided river sedimentation (Bridge, 1985). Furthermore, the channel fill deposits generally preserve the minor second- and third-order channels (Fig. 11). Coleman's observations suggest that the smaller third-order channels are more susceptible to avulsion than the larger second-order channels. Using these abandoned channel-fill sequences for paleodischarge measurements should be treated with caution, because the abandoned channels may not be representative of the true magnitude of the fluvial system.

### DISCUSSION

Recent sedimentation in the Brahmaputra River is predicted to produce a sand body of a sheet-like nature approximately 40 m thick and 20 km wide. The dimensions of the sand body and total sediment distribution are controlled by first-order channel movements. The most important of these are the 1787 avulsion and the subsequent westward migration. This movement occurs on a time scale of  $10^2$ – $10^3$  years and is probably controlled by tectonic movements within the Bengal basin (Morgan and McIntire, 1959).

The internal stratigraphy of this sand body is controlled by second-order channel movements. This is dominated by lateral migration at rates of as much as 1 km per annum. Due to the extremely high width/depth ratios of channels in the Brahmaputra (as much as 500:1), the resulting lateral accretion surfaces will be very difficult to recognize in the geologic record. More asymmetric sinuous channels (Fig. 6C), however, have width depth ratios of 45:1. Continual reworking within the channel belt and occasional channel switching will produce complex internal erosion and deposition surfaces in a multistory sand body. During falling stage, large amounts of suspended sediment are deposited within the channel, particularly on the downstream margin of medial and lateral bars. These silt drapes may preserve some record of second-order bar and channel form. Low

stage reworking and the entrenching or abandonment of third-order channels will further complicate the internal geometry of the sand body.

### CONCLUSIONS

The Brahmaputra River is a multichannel river. Channels of many different sizes (from hundreds of meters to kilometers wide) and different patterns (braided, meandering and anastomosed) occur in Bangladesh.

The presence of three different channel patterns within the Brahmaputra River shows that there is a continuum between braided, meandering and anastomosed rivers. The main factor controlling channel pattern in the Brahmaputra is channel discharge. Other factors, sediment supply, grain size and slope, are locally similar. The change in channel pattern from braided in the north to a more sinuous course in the south is probably caused by the downstream reduction in valley gradient from 0.000077 to 0.00005. There is also a possible downstream reduction in peak discharge due to diversion of flood waters along the Old Brahmaputra and Dharleswari Rivers and into the Atrai Gur flood basin. These changes in slope and discharge may affect the rate of channel migration which also decreases downstream.

The channels are organized in an hierarchy. The first-order channel comprises the entire river. This has an average width of 10 km and maximum depth of 45 m. The first-order channel has made one avulsion in the last 200 years. Since then, it has migrated within a channel belt as much as 20 km wide. First-order channel migration is an average 70 m per annum. Second-order channels have maximum widths of 5 km and depths to as much as 40 m. They migrate at rates of as much as 1 km per annum. Most channel movement occurs by lateral migration, although some channel switching has been observed. Third-order channels occur within second-order channels. They have widths of hundreds of meters and migrate in response to local hydraulic gradients within second-order channels. Third-order channels dissect and modify higher order bars at falling stage. Some may also be active at high stage when a high degree of switching has been observed.

Five types of bar are recognized in the Brahmaputra: lateral bars, medial bars, tributary bars, diagonal bars and scroll bars. Of these the first three are dominant. All of these bars scale with the bankfull width and depth of the channel. Bar length from crossover to crossover is approximately 3 to 4 times channel width, and bar height is greater than one-half channel depth. There is a wide range in bar size due to the variety of channel magnitudes. Thus, there are first-, second- and third-order bars which scale with first-, second- and third-order channels, respectively. Bars are stable over several floods and are modified by succeeding depositional and erosional events which produce complex bar forms.

The most important component of deposition on bars is lateral accretion (57 percent), followed by downstream accretion (29 percent) and upstream accretion (14 percent). Together, these form 53 percent of all deposition within the river. Other sites of deposition are lateral accretion to the bank 19 percent, channel abandonment 15 percent and new mid-channel bars 13 percent.



In some localities bars coalesce to form bar assemblages which are as much as 20 km long. At these points, the river is anastomosed and each second-order channel migrates independently within the channel belt. These megaforms are stable over tens of years.

Recent deposits of the Brahmaputra form a 20-km-wide multistory and multilateral sand body which is predicted to have a complex internal geometry. This will be marked by a hierarchy of internal erosion and deposition surfaces, scaling with the hierarchy of channels. Due to the high width/depth ratios, lateral accretion surfaces will be extremely low-angle. Second-order bounding surfaces are expected to dominate the internal stratigraphy due to their high rates of migration and deposition within the channel belt.

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### Paper 4.3

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Bank erosion and channel processes in the Jamuna River,  
Bangladesh, *Proc. International Symposium on the Impact of  
River Bank Erosion, Dhaka, Bangladesh.*



BANK EROSION AND CHANNEL PROCESSES IN THE  
JAMUNA RIVER, BANGLADESH

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**SUMMARY:** For an overall approach to flood protection in Bangladesh, insight is required into the processes that cause bank erosion along the main rivers. Within the framework of the Jamuna Bridge Appraisal Study, Phase I and Phase II, studies into bank erosion and channel processes were and are being carried out in the Jamuna River in Bangladesh. Some results of a detailed analysis of cross-sectional soundings are presented, together with an outline of and some preliminary results of a continuing analysis of a sequence of LANDSAT-images of the river. It is indicated how increased understanding of these processes can help in optimizing flood protection measures.

1. Introduction

In general it can be stated that in Bangladesh bank erosion along the main rivers is usually associated with deposition of material on the opposite bank. Actually most rivers do not increase their width, but their channels meander in their flood plains. Consequently land that is lost on one side, is gained on the other side. This is probably the main reason for the approach of retiring embankments that has been followed in Bangladesh in the past. The bank erosion is threatening an embankment along one of the main rivers at a particular spot and preferably before the embankment fails a second embankment has already been constructed that reduces the consequence of the failure in reducing the flooded land. Further erosion resulted in further retirement. Especially along the Jamuna River, where the river has gradually shifted to the West in the last centuries (Coleman, 1969), substantial retirement has been required.

In the long run such an approach may not be acceptable any longer from a social and also from an economic point of view. From a social point of view while the loss of land very often leads to impoverishment of the small farmers and from an economic point of view when farmlands under command of an irrigation project are being lost.

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Furthermore, it seems that in view of expensive training works to protect towns like Sirajganj and the future Jamuna Bridge continued bank erosion may not be acceptable any longer.

Whether or not anti-erosion measures on a large scale along the main rivers will be feasible, can only be evaluated when the costs of such measures can be assessed in combination with sufficient insight into where and when these measures have to be applied. This calls for a good insight into bank erosion processes and how they are linked to the channel processes in the rivers itself. Such insight would also help in the drawing up of an improvement plan that will cover say 20 years. In such a plan it should be indicated what works will have to be executed, and what lay-out of the protection works is required. It is obvious that such an overall plan should not only be limited to flood protection, but also drainage aspects should be considered and, last but not least, construction methods that can be applied in Bangladesh. In addition to existing bank protection methods (see e.g. Alam and Faruque, 1986) new methods may have to be developed taking into account the possibilities and constraints applicable to Bangladesh.

It is against this background that the contents of the present paper should be appreciated. It provides some results of continuing studies into bank erosion and channel processes in the Jamuna River in Bangladesh. These studies are being done within the framework of the Jamuna Bridge Appraisal Study, Phase II. The project stems from the required integration of the Western and the Eastern regions of Bangladesh; this integration has been set by the Government of Bangladesh as one of its highest priorities. The project consists of the construction of approximately 5 km long Multi-purpose Bridge across the Jamuna River, which bisects the country from North to South.

The Jamuna Multi-purpose Bridge Authority (JMBA) has been created by the Government of Bangladesh to further the studies and construction of the Project and to ensure the necessary funds for its implementation. The World Bank as Executing Agency for the United Nations Development Programme, has appointed Rendel Palmer & Tritton, NEDECO and Bangladesh Consultants Ltd. to undertake Phase II of an Appraisal Study into the proposed crossing. This study is being undertaken in 1987 and 1988. The Consultants will complete the engineering and geotechnical study, commenced in Phase I of the study, and will prepare the final design of the approaches and the river training works together with designs of two river bridge alternatives.

In order to study whether or not the Jamuna River is stable in its present position and to be able to evaluate risks of possible substantial future bank erosion upstream of the proposed bridge, an analysis was carried out of available data, including a detailed analysis of available satellite images.

The results obtained during this study are of relevance, too, for bank protection works along Bangladesh's main rivers, because they aim at increased understanding of the relevant processes governing bank erosion along this large braiding sand-bed river. As such it expands on the recent increase in understanding of bank erosion in meandering rivers and the development of methods for prediction.

In the present paper two studies are considered in some detail, viz.:

- an analysis of soundings by the Bangladesh Water Development Board (BWDB), to study horizontal displacements over the last two decades;
- an analysis of satellite-images of the Jamuna River over the period 1973-1987 to study in more detail the relationship between bank erosion and channel processes.



The first study is almost finished and results and preliminary conclusions are given in Chapter 2. The second study is still in progress and only the approach and some preliminary results are presented in Chapter 3.

It should be emphasized here that, in addition to the studies described in this paper, within the framework of the Jamuna Bridge Appraisal Study a number of other studies are being carried out to improve the understanding of processes in a braiding river like the Jamuna River. The following studies should be mentioned in this respect:

- (i) an extensive survey, aiming at collecting field data on bed forms and on scour at confluences, in outer bends and at protrusions (see Klaassen and Vermeer (1988b) and Klaassen, Vermeer and Uddin (1988));
- (ii) a movable-bed scale model in which channel patterns and the effect of river training works for the bridge are being studied (Klaassen, 1987).

## 2. Analysis of cross-sectional data

### 2.1 Introduction

The Jamuna River in Bangladesh is the lowest reach of the Brahmaputra River upstream of its confluence with the Ganges River (see Figure 1). The Jamuna River is a large, braiding sand-bed river. The average discharge during floods is about 50,000 m<sup>3</sup>/s, the maximum width during floods about 17 km. The bed material is quite uniform and D<sub>50</sub> varies from 0.25 mm near the Indian border to 0.16 mm at the confluence with Ganges River. The valley slope in Bangladesh decreases gradually from 0.10 to 0.06 m/km. More details on the river characteristics can be found in e.g. Coleman (1969) and, more recently, in RPT/NEDECO/BCL (1987), notably appendix C.

About two centuries ago the Brahmaputra River shifted from its original course (presently named the Old Brahmaputra River, see Figure 1) to its present course, now known as the Jamuna River. The reason for this avulsion is not discussed here as it is outside the scope of this contribution. Readers interested in this phenomenon are referred to Morgan & McIntyre (1967) and Coleman (1969). Since its avulsion the Jamuna River has 'increased its total width and has gradually shifted in Western direction, in some reaches up to 10 km (see Coleman (1969) or IECO (1969)). An important issue was whether or not the river has reached an equilibrium condition in the centuries that have passed since the avulsion. This was especially important because the risk of out-flanking of the bridge had to be evaluated as part of the feasibility study.

During this evaluation a detailed analysis was made of all the available data on the Jamuna River. Initially only the river width and the location of left and right banks and in particular its changes over time were considered. Figure 3 is an extension of the work reported upon in IECO (1969) on these changes in time.

In the present context especially the cross-sectional characteristics of the Jamuna River are of importance. They were studied on the basis of soundings which have been made by the Bangladesh Water Development Board (BWDB) since 1966/67. The analysis carried out was mainly aiming at the identification of any consistent changes in channel location. Apart from horizontal displacement also vertical changes (degradation or aggradation) were studied. Results on vertical changes are not given here, but reference is made to Klaassen & Vermeer (1988a), a paper which considers also planform changes.

The structure of this Chapter is as follows. The available basic data (i.e. BWDB soundings) are described in Section 2.2. The processing of these data is described in Section 2.3, while the horizontal changes of the Jamuna River are discussed in Section 2.4.



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The present analysis expands on recent work by Ullah (1987), who compared soundings of 1965/66 with more recent ones from 1983/83. By including soundings from other years, here a better insight is obtained into the fluctuations over the years in relation to any trends in bed levels and channel locations.

In this paper no attention is given to the possibility of another major avulsion. Figure 3 provides some extended cross sections over the Jamuna River. It seems that due to geological processes a depression has occurred on the Eastern side of the river. Whether or not this may induce an avulsion can only be evaluated after a more detailed study, including the overall topography of Bangladesh, flood levels, etc.

## 2.2 Available data

Cross-sectional soundings of the Jamuna River have been made since 1965/66. Initially these measurements were done every year, but they were interrupted in 1970 due to the War of Independence. In 1975/76 the soundings were taken up again, although not every year the Jamuna River was included in the country-wide morphological measurements. In the period between 1965/66 and 1986/87 for most of the cross-sections in total some ten soundings have become available for analysis.

The soundings were carried out during the low flow season. Both the submerged areas and the dry islands (locally called "chars") in between the braids were sounded. The bed levels are related to the local Public Works Datum (PWD). The horizontal distance was measured from monuments. Sometimes the monuments got lost either by bank erosion or by other causes. Consequently for a number of cross sections it is not possible to make a quantitative comparison of channel location in the horizontal direction over the full period.

Initially the distance between the cross sections was 4 miles, but gradually more cross-sections have been introduced over the years. Figure 2 provides an overview of the sounded cross-sections.

In the present study twelve cross-sections were selected for detailed analysis considering:

- i) conveyance;
- ii) width (of channels and chars);
- iii) depth (average and maximum);
- iv) average cross-section level.

Cross-sections J5, J5-1, J6, J6-1, J8 were selected in view of their location close to the proposed bridge site (J6-1), the others to investigate possible changes in channel characteristics upstream and downstream of the bridge site. Figure 5 provides the results of soundings of two cross-sections (notably J13-1 and J6-1) for a number of years. Other cross-sections may have a total width of more than 15 km.

## 2.3 Processing of cross-sectional data

The cross-sectional data contain important information on channel characteristics and horizontal stability. To isolate this information, the cross-sectional data have to be processed further. A computer program was developed for this purpose. This program uses the sounded data as input, and subsequently the following processing is being done per cross-section:

- for three discharge levels (44,000 m<sup>3</sup>/s approximately corresponding to bankful stage, 12,000 m<sup>3</sup>/s and 4,000 m<sup>3</sup>/s representing average low flow conditions in the Jamuna River) stages are plotted in the sounded cross-sections, and channels and sand bars (chars) are differentiated;
- the total wet area for each of the three discharge levels is computed;



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- going from the left (Eastern) bank to the right (Western) bank monument, the cumulative conveyance is computed as a function of the distance to the left bank monument; in a second step this relationship is made dimensionless by dividing it by the total conveyance for that particular cross-section and stage (for an example of the obtained results, see Figure 6);

It should be stressed here that the underlying assumption for the lines in Figure 6 is that the low flow cross sections are representative for bankfull stages also. The scarce information available on changes of the cross section through the year seems to support this assumption.

## 2.4 Horizontal stability

Horizontal stability is considered on the basis of the relative conveyances computed according to the method outlined in the previous section. The relative conveyances for a sequence of years were plotted for all cross sections considered. Some examples are presented in Figure 7, notably the cross-sections J6 and J5-1. Any significant horizontal shift in river location would show up in the figure as a consistent shift in the relative conveyance in either direction. Such a trend is not detectable. The relative conveyance does show a substantial scatter, and this is due to the phenomenon that gradually minor channels become more important and vice versa. This gradual changes (see also Figure 5) is different from what has been stated by Coleman (1969). Coleman's data relate to thalweg locations and these may vary considerably in one year, while the actual changes in depth of different channels in one particular cross-section may be fairly small. Other analyzed cross-sections show a similar picture as Figure 7. Therefore, it seems that over the last two decades there is no significant shifting of the Jamuna River in either direction.

What can be observed from an inspection from Figure 7 and from similar pictures for different cross sections, is that periods of erosion are followed by periods of accretion. This is in line with the observations in the movable bed scale model (Klaassen, 1987): the location of bank erosion is determined by the channel pattern of the river and this channel pattern changes over the years (see Chapter 3). So of the outer banks erosion along a braided river is not a continuous process as it is along meandering rivers.

## 3. Bank erosion and channel processes

### 3.1 Introduction

For the planning of bank protection works a good insight in the relationship between bank erosion and channel processes is required. In fully meandering rivers the location of bank erosion can fairly well be predicted at this moment, and using a sequence of maps or satellite images also the celerity of bank erosion can be deduced. Recently this has resulted in the development of numerical models for the prediction of meander changes (e.g. Johannessen and Parker, 1985). Although the reliability of the predictions with this type of models is still fairly limited, they represent a major step forward as far as the planning of bank protection works along meandering rivers is concerned.

For braiding rivers the insight in the controlling processes has not yet reached such a level that predictive models can be used. Within the framework of the Jamuna Bridge Appraisal Study, Phase II, a fairly extensive analysis of a sequence of satellite images is foreseen, aiming at improving the insight into channel processes and their importance for bank erosion processes. Two items are being studied, notably:

- the celerity of bank erosion in relation to channel characteristics (conform the work by Hickin and Nanson (1984) for meandering rivers),



the process which causes that after some years, the function of major branches is taken over by other channels. The approach followed is discussed in this paper and some preliminary results are presented.

### 3.2 Approach

The study discussed here is based on the comparison of satellite images of the Jamuna River for successive years, the determination of the bank erosion velocity that can be observed for different reaches and channels, and finally the interpretation of the results considering the characteristics of the involved channels. The various steps taken in the analysis are discussed below. The first step in this approach, notably the comparison of LANDSAT-images for different years was done earlier for the Jamuna River (Roque, 1983). The other steps are new.

For the analysis a number of LANDSAT-images were selected. An overview of the images used is provided in Table 1. It should be emphasized here that apart from the "normal" criteria for the selection of an image (quality, cloud cover), here an additional criterion was used. Only images were selected in which on the date of the image being obtained, the stage at Sirajganj was between 6.00 and 7.00 m + P.W.D. (P.W.D. stands for Public Works Datum, corresponding to an agreed standard level in Bangladesh). This was done to truly isolate the changes in channel location due to bank erosion and not to mix it up with changes in channel width for different stages. In total 14 images were selected and CCT's (Computer Compatible Tapes) were purchased. Most of the CCT's relate to the lower reach of the Jamuna River (148(138)/43, downstream of Bahadurabad), some relate to the more upstream reach (148(138)/42) (see also Table 1). Figure 8 provides a non-geometrically corrected print of the 1973 images of the lowest reach.

The processing of the CCT's with digital information was done by the National Aerospace Laboratory in The Netherlands by means of the RESEDA processing system. The tapes were read and data were subsequently geometrically corrected and re-sampled. In doing so the original pixel size of 80 m was reduced to about 50 m. This was done to increase the accuracy of the analysis. The geometrical correction was done on the basis of some ten points which could be identified on the image and from which the co-ordinates could be obtained from topographical maps. The accuracy of this correction was evaluated and the inaccuracy is not greater than the size of the pixels after resampling.

The next step in the analysis is classification. For the present study four classes were distinguished, i.e. water, bare land, sand and vegetation. The classification was done as the basis of band 5 and band 7. Figure 9 provides the classification diagram applied for the image for the year 1977. This classification diagram is different from the diagrams for other years, but the shape of the diagram is similar. Figure 10 provides a part of a classified image. The following colour code is used in Figure 9:

- blue : water
- grey : bare land
- orange: sand
- green : vegetation

In earlier studies (e.g. Roque, 1983) the class "bare land" was not identified separately and it may be argued that this class can be mixed up with e.g. wet sand. It was found, however, that deleting this class resulted in substantial loss of information on landscape features.

Changes over the years in water and land distributions are obtained by comparing the classification for different years. Figure 11 provides the results

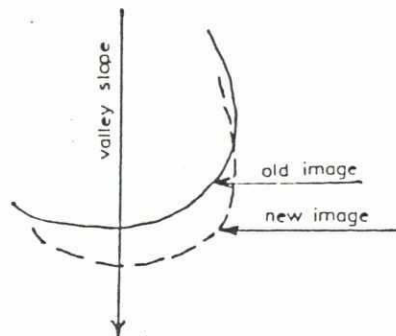


of comparing the classified pattern of the year 1986 with the classification for the year 1987. In total 16 possibilities are present, based as the four identified classes. The used code is indicated in Table 2.

### 3.3 Interpretation of results

At the present stage no definitive conclusions as to the relationship between channel process and bank erosion can be given on the basis of the comparison of LANDSAT-images as described here. This is due to the fact that the obtained results are still being interpreted. However, some observations can already be given:

- (i) It seems that after a major flood (like the one which occurred in 1984) the channel pattern has straightened, the major channels having a smaller deviation from the direction of the valley slope.
- (ii) Channels which have a substantial deviation from the direction of the valley slope seem to move in downstream direction as indicated below:



A possible explanation of this phenomenon is that the movement of such channels is not primarily caused by bank erosion, but that they are "pushed away" by extending chars. This would imply that sediment transport over the chars is more important than earlier envisaged.

- (iii) Additional bank erosion along the main channels seems to occur after the peak of the flood has passed. This may be comparable to what is happening in meandering rivers.

A more detailed analysis is still in progress, during which it is being attempted to correlate the bank erosion to channel processes in a quantitative way. Also the straightening process is being studied in more detail.

### 4. Conclusions

In the present paper some preliminary results are presented of investigations related to bank erosion and channel processes in the Jamuna River, Bangladesh. Although at present no final conclusions can be formulated and also yet no predictive deterministic methods have been developed, it seems that the detailed analysis described here will help in obtaining more insight in the complex processes occurring in large braided rivers. Also parallel studies (see Klaassen (1987), Klaassen & Vermeer (1988 a and b) and Klaassen, Vermeer & Uddin (1988)) help to increase this understanding. It is felt that based on this increased insight, tools can be developed that can be used for the drawing up of an overall programme for bank protection works along the Jamuna River.

### 5. Acknowledgements

The present studies are being carried out within the framework of the Jamuna Bridge Appraisal Study. They are being funded by the United Nations Develop-

ment Programme (UNDP) through the World Bank. The Jamuna Multi-purpose Bridge Authority (JMBA) is furthering the studies for and the construction of the bridge. The present study is being carried out by Rendel, Palmer and Tritton, NEDECO and Bangladesh Consultants Ltd. The soundings of the Jamuna cross-sections has been carried out by the Bangladesh Water Development Board. Mr. Kamal collected the BWDB soundings and analyzed monument data. The analysis of the LANDSAT-images is being done by the National Aerospace Laboratory, The Netherlands. Mr. Z. de Jong of the State University of Utrecht has assisted in this analysis and is doing the geomorphological interpretation of the results. Publication of this Paper has been kindly permitted by both the World Bank and the Jamuna Multi-purpose Bridge Authority.

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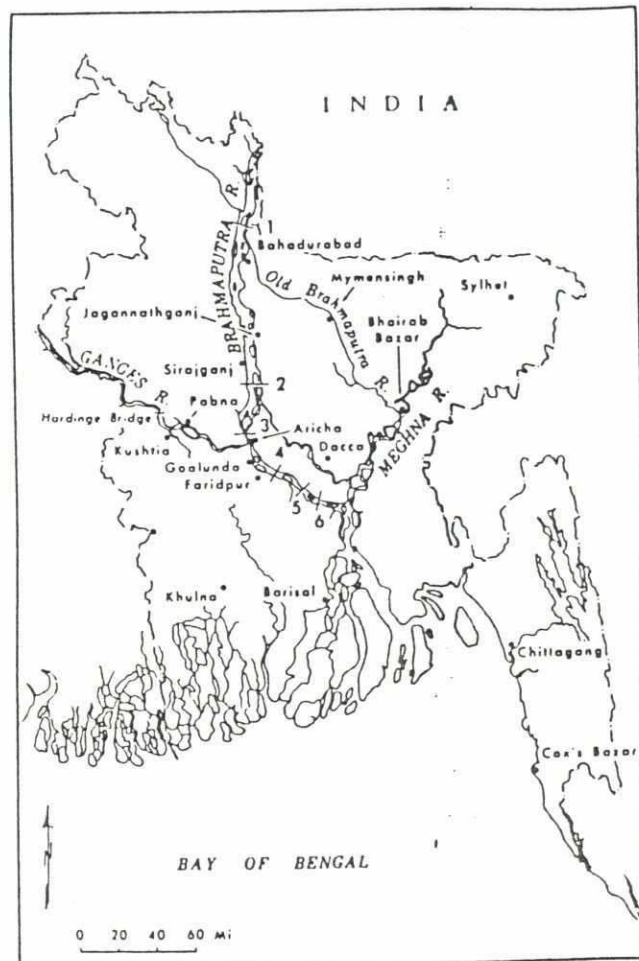


Figure 1 Index map of Bangladesh with major rivers (Coleman, 1969)



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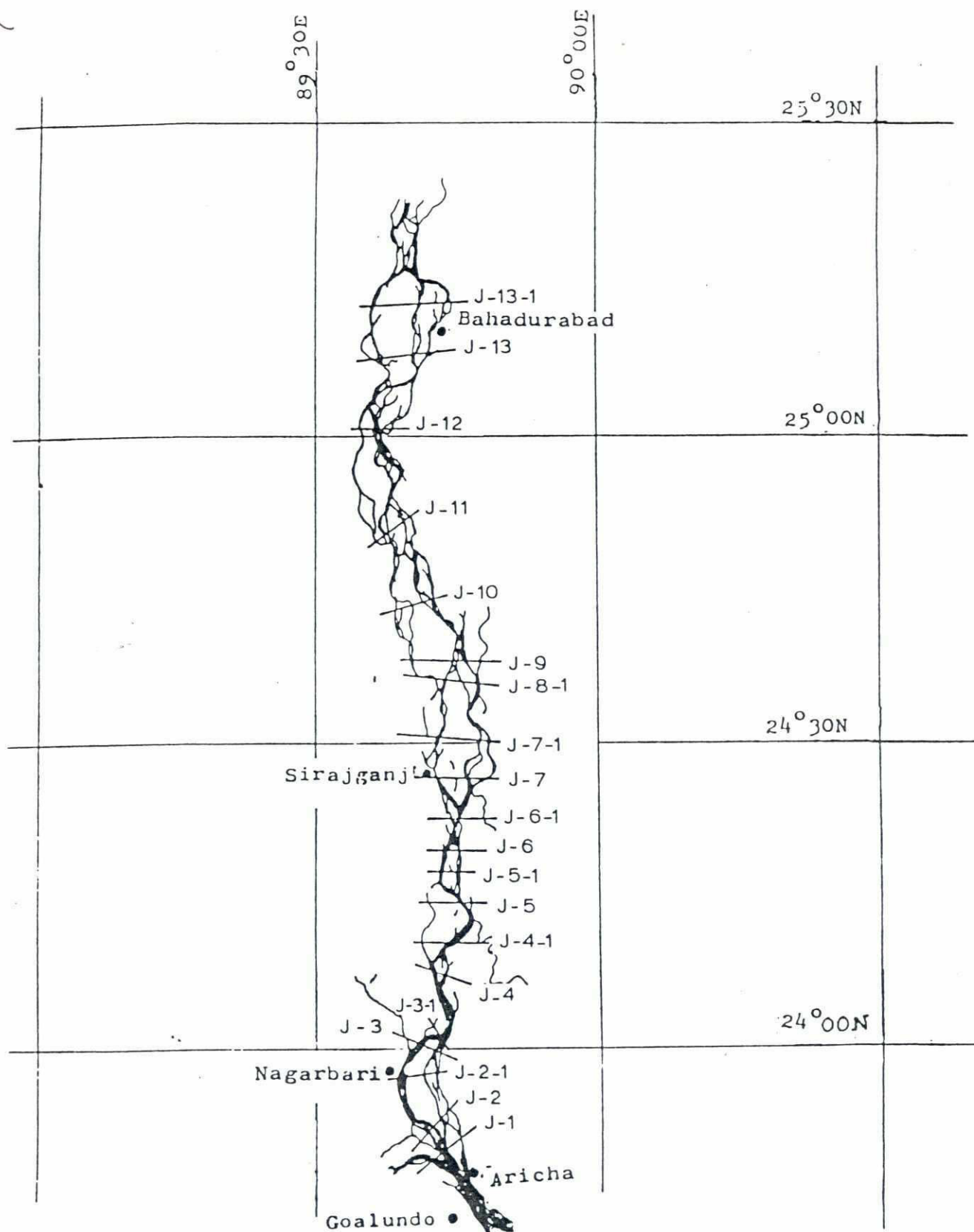


Figure 2 Cross sections measured by BWDB in the period 1966-1987



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Figure 4 Cross-sections perpendicular to Jammu River (data provided by MPD)

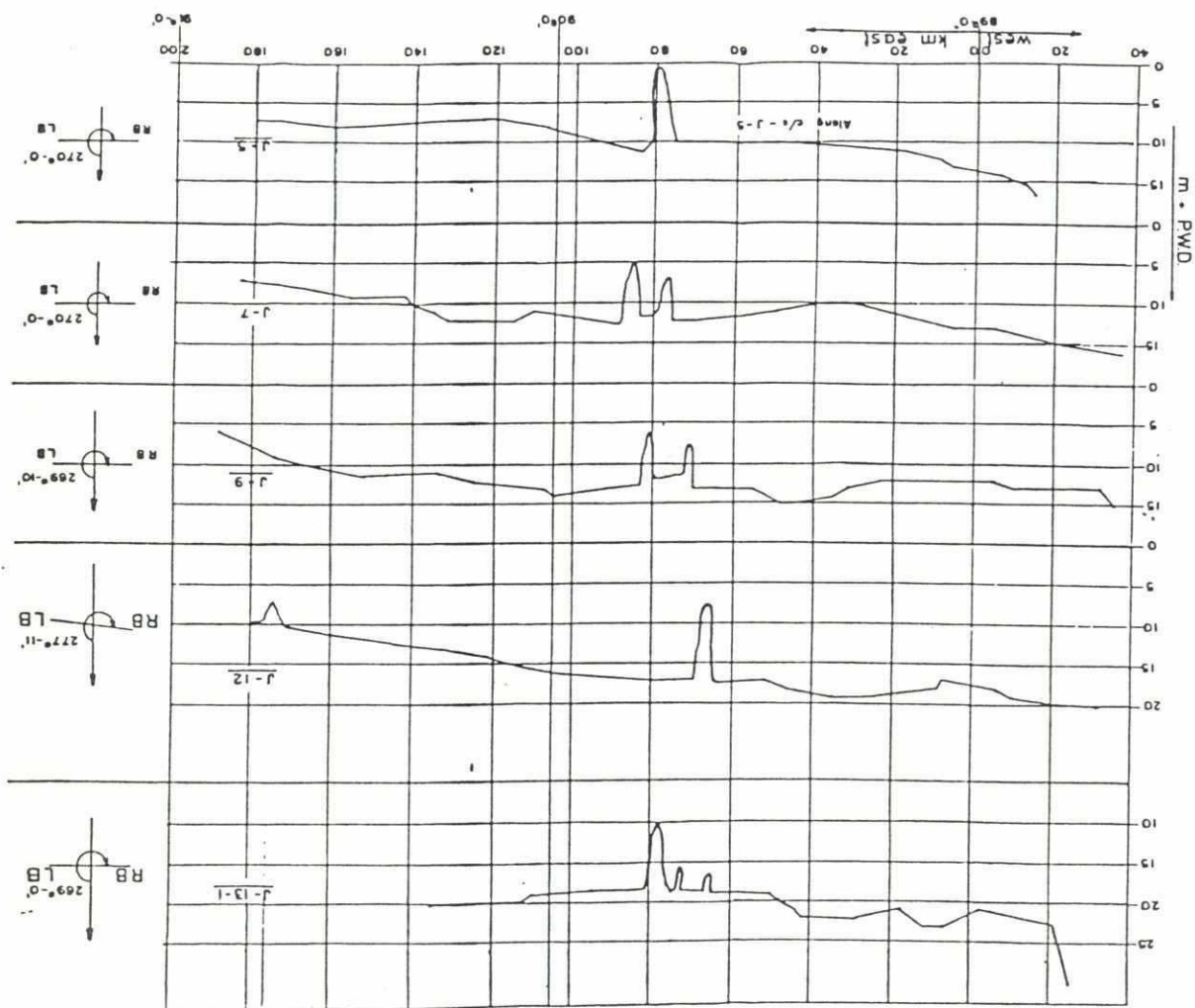
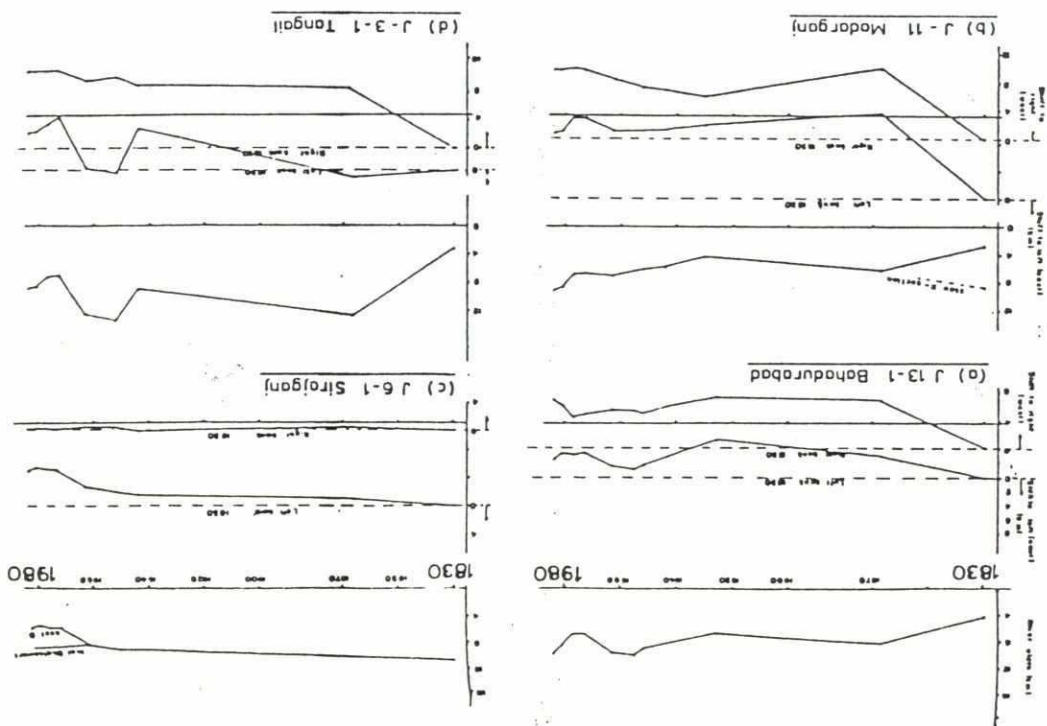
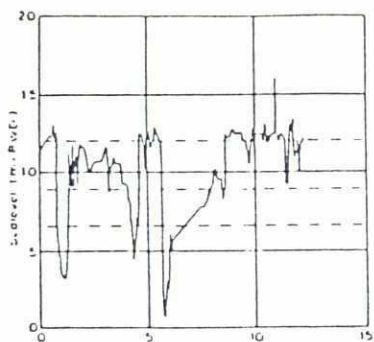
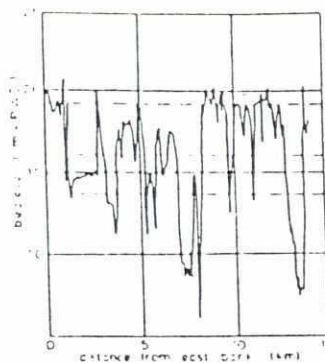


Figure 3 Changes of total width and of channel location over the period 1830 - 1985.

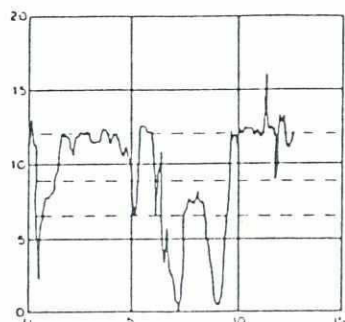




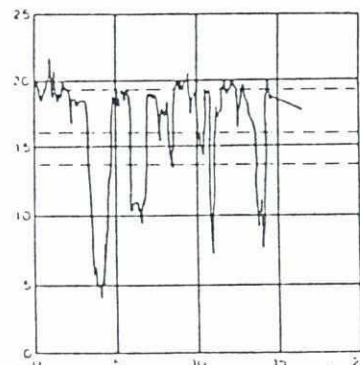
J6-1  
57/68



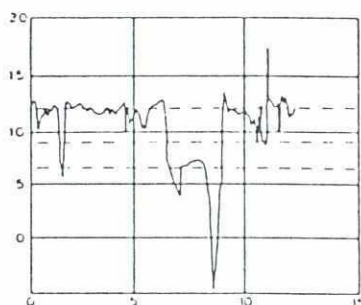
J13-1  
66/67



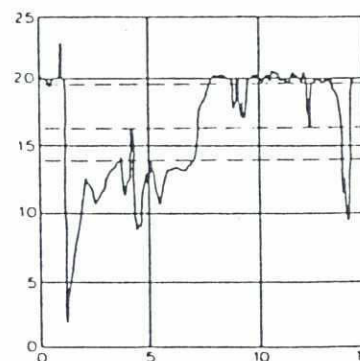
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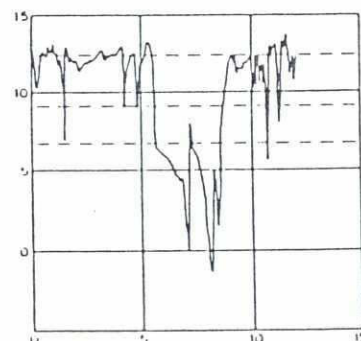
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69/70



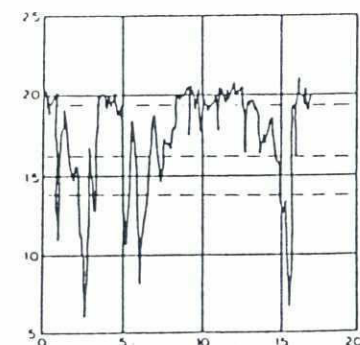
J6-1  
76/77



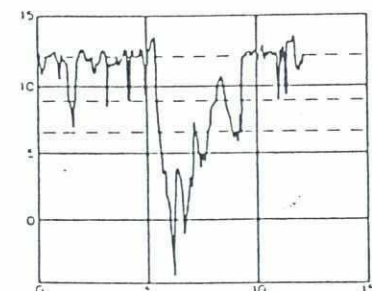
J13-1  
76/77



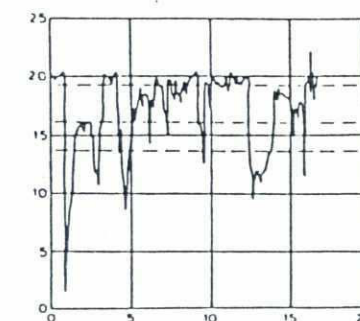
J6-1  
80/81



J13-1  
81/82



J6-1  
86/87



J13-1  
85/86

Figure 5 Two-cross sections of the Jamuna River and their variation in time



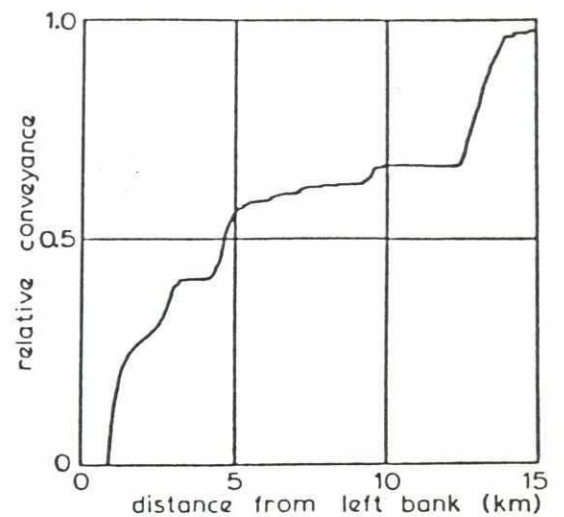
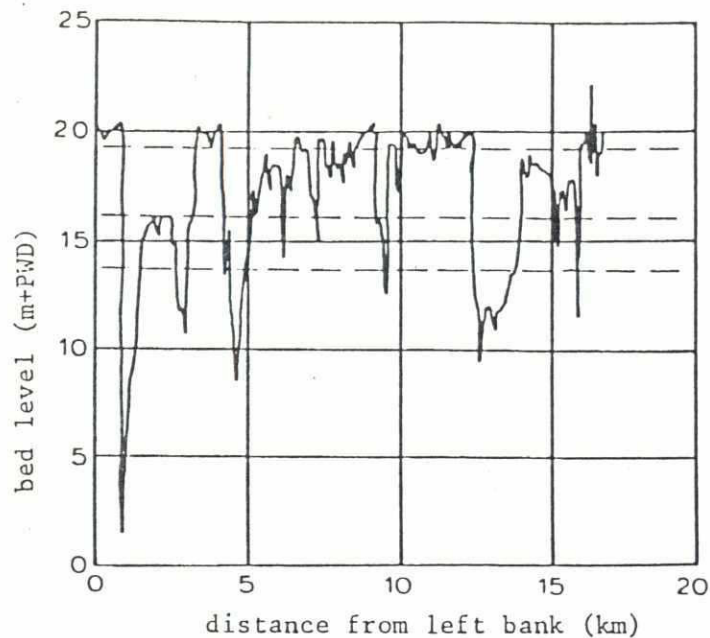


Figure 6 Cross section J13-1 in the year 1985/1986 and the relative conveyance as function of distance from the left bank.

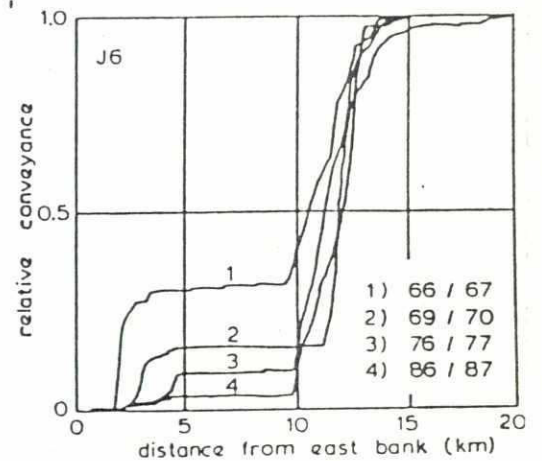
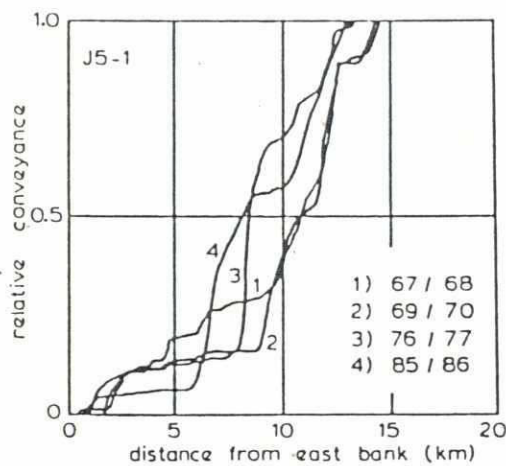


Figure 7 Two examples of the variation of the relative conveyance in time

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Image 148(138)/42 148(138)/43

| Year | Date    | Date    |
|------|---------|---------|
| 1976 | 4 Mar.  | -       |
| 1977 | 9 Feb.  | 9 Feb.  |
| 1978 | 22 Feb. | -       |
| 1984 | 23 Feb. | 23 Feb. |
| 1986 | 20 Feb. | 20 Feb. |
| 1987 | 7 Feb.  | 7 Feb.  |

Table 1 LANDSAT-images used in for present studies

| new image<br>old image | water                                                                               | sand                                                                                | bare soil                                                                           | vegetation                                                                            |
|------------------------|-------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------|
| water                  |  |  |  |  |
| sand                   |  |  |  |  |
| bare soil              |  |  |  |  |
| vegetation             |  |  |  |  |

Table 2 Classification scheme used for changes in land classes between two images



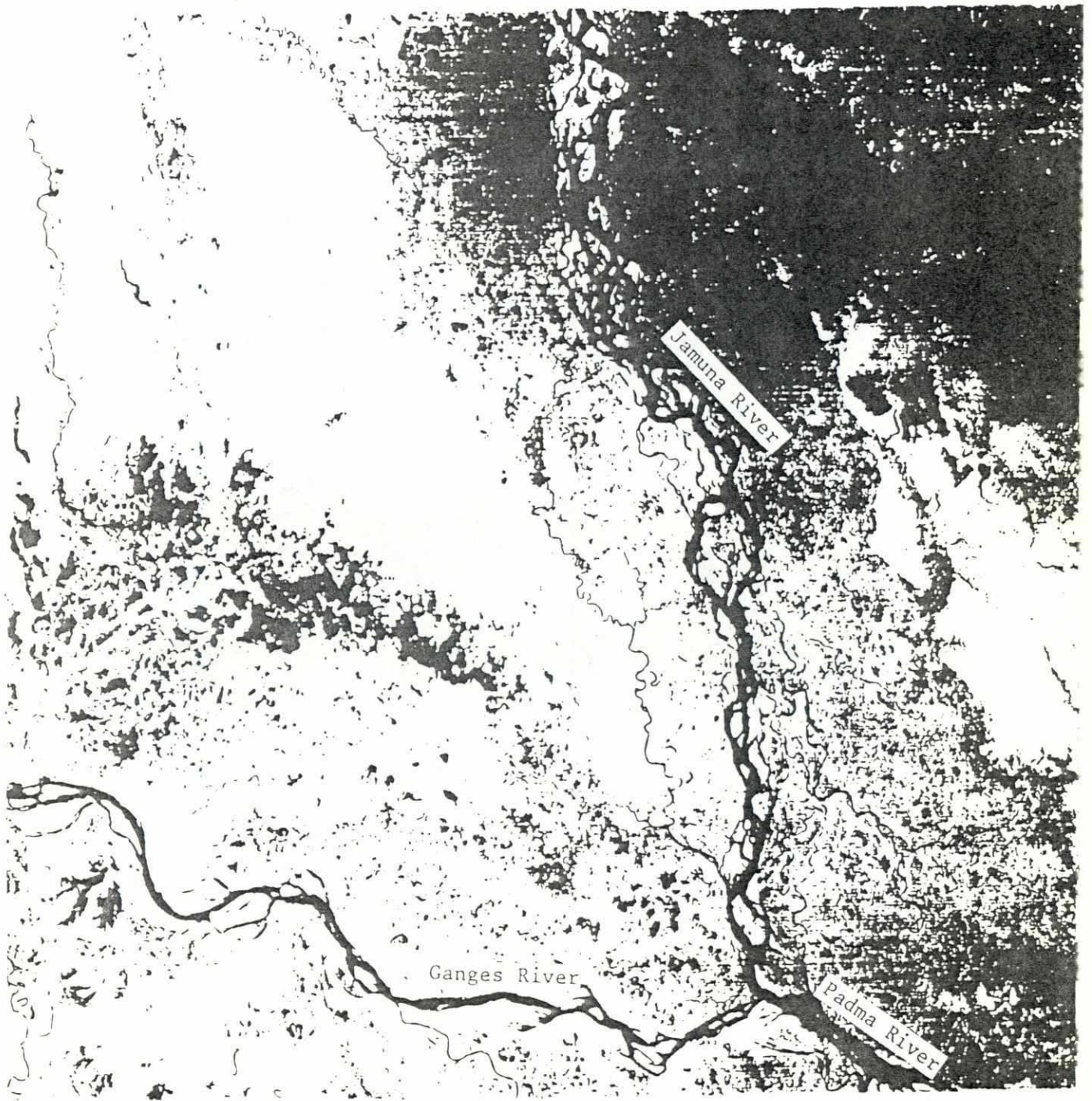


Figure 8 LANDSAT-image lower reach of the Jamuna River, data 27-2-1973  
(scale  $\sim 1:1,000,000$ )

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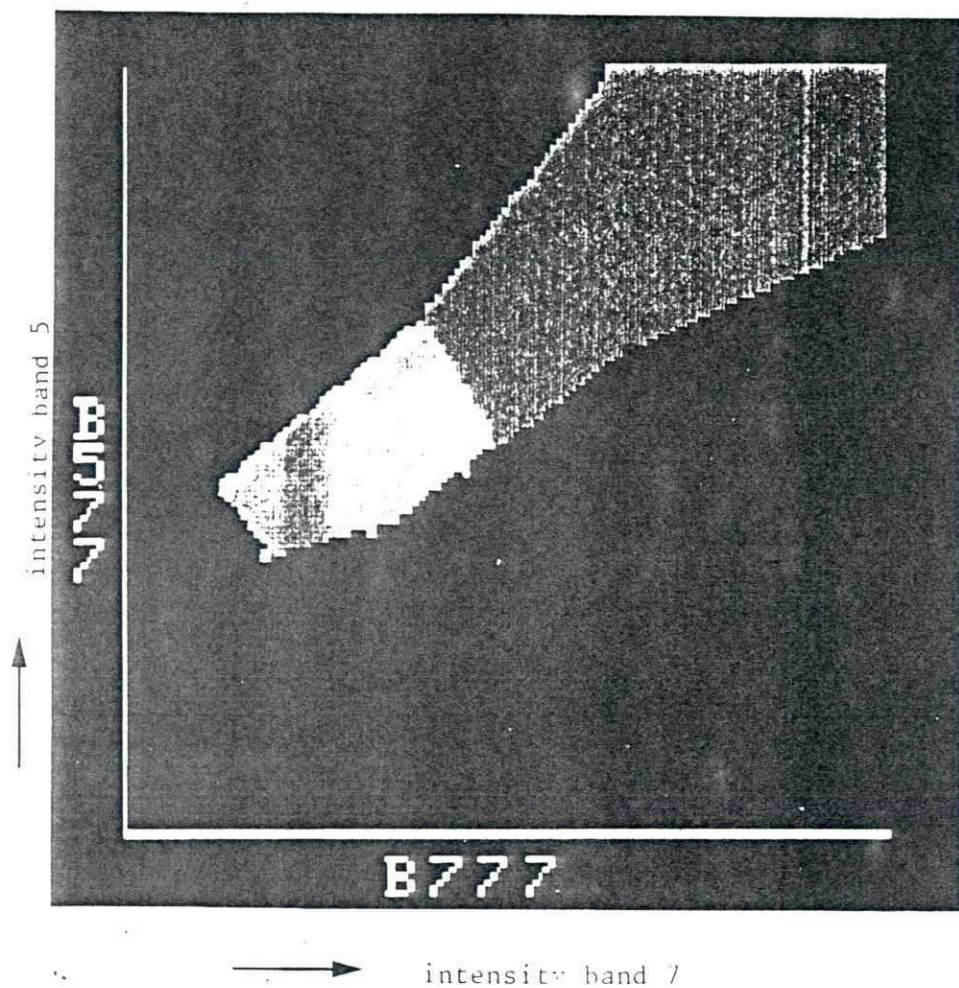


Figure 9 Classification diagram used for the present study



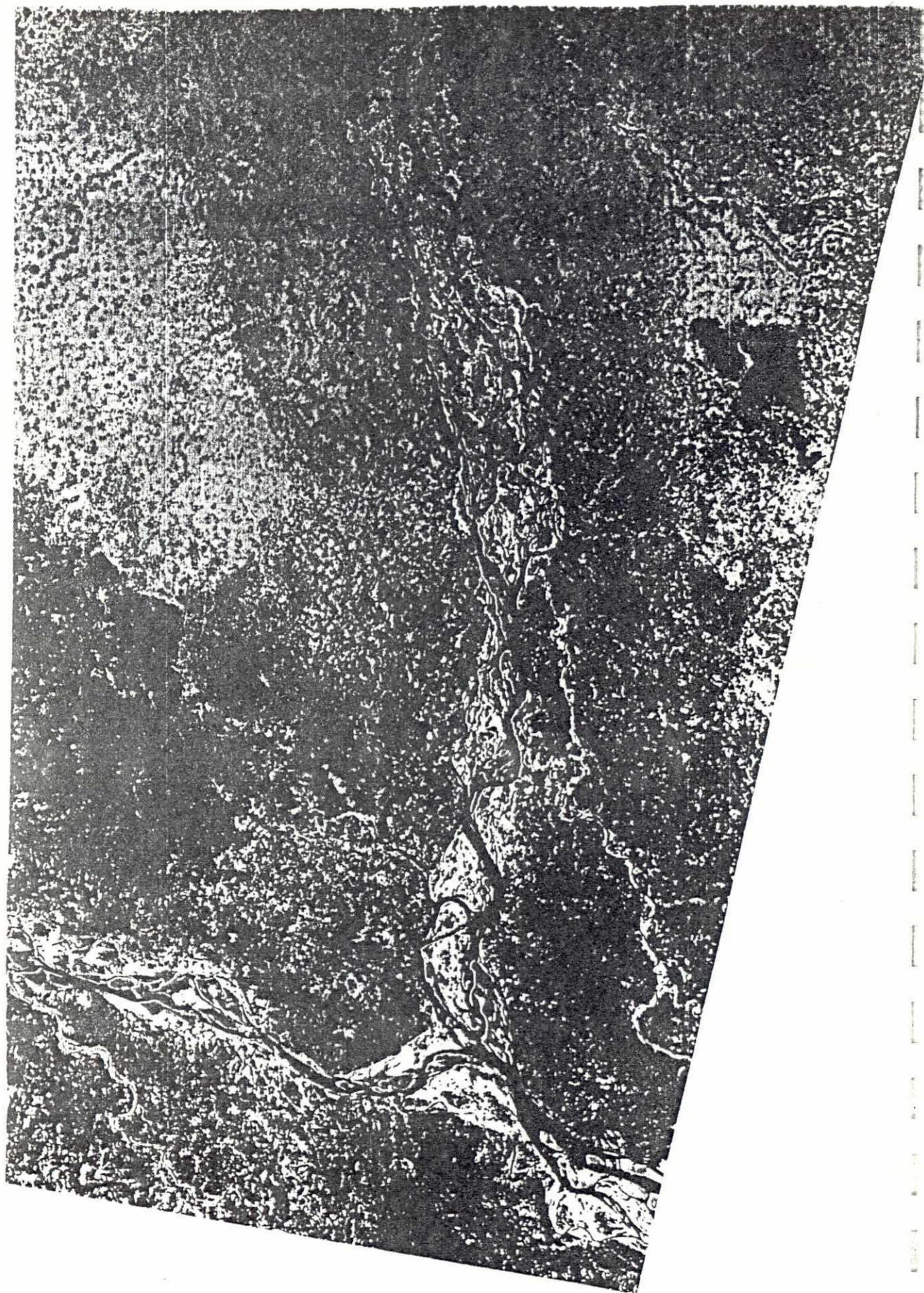


Figure 10 Classified image Jamuna River 1987 (approximate scale 1:725,000)



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Figure 11 Changes in land/water classification Imuna River  
between 1986 and 1987 (scale 1:275,000)



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## Paper 4.4

KLAASSEN, G.J. & B.H.J. van ZANTEN (1989), On cutoff ratios of curved channels, *Proc. XXIIIrd IAHR Congress, Ottawa, Canada, B-121 - 130pp.*

ON CUTOFF RATIOS OF CURVED CHANNELS

by

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Summary

An analytical method is presented that can be used to derive a criterion for the cutting off of curved alluvial channels. The method is based on the (un)balance between the incoming sediment transport and the transport capacity of the channel potentially to be cut off. The method can only be used for the initial stage of the cutting off process of meandering rivers and it is outlined how this method can be extended to later stages via numerical computations. The implications of the derived criterion for the effect of extreme floods and for braided rivers is indicated.

1. Introduction

The length of a meandering river increases regularly due to bank erosion along the outer banks. This would cause a meandering river to increase its length indefinitely, if there would not be a mechanism that counteracted this tendency. This mechanism is the cutting off of meanders during floods. When the length of a particular bend exceeds a certain value the bend will be cut off by the scouring of a cutoff channel in the floodplain through the neck of the meander. An oxbow lake will remain, and this lake will silt up gradually. A typical example is shown on Plate 1, depicting a part of the Tana River in Kenya where still many former bends and oxbow lakes can be identified. In this paper it is attempted to derive analytically a criterion to predict when such

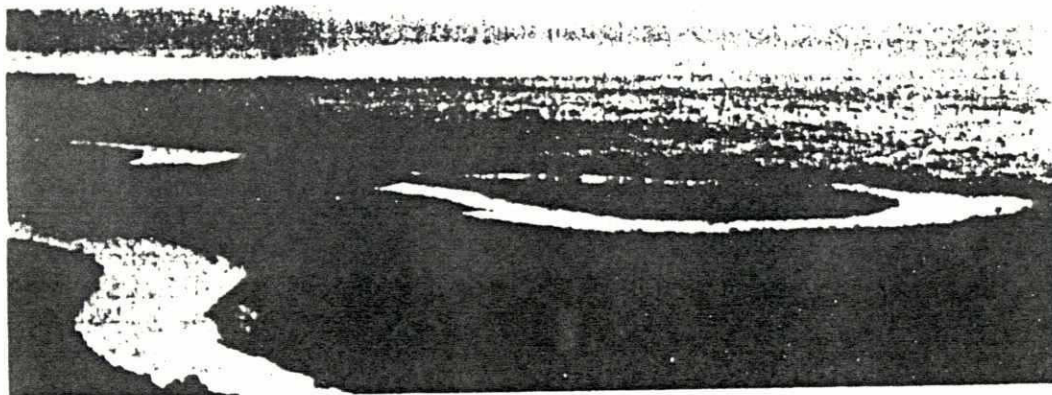


Plate 1 Overview of a reach of the Tana River (Kenya) with cutoff river bend still noticeable as oxbow lake



Figure 1 Definition of cutoff-ratio (Joglekar, 1971)

a cutoff will occur. It is obvious that such a criterion is needed as an essential element in meander modelling (see Johannesson and Parker (1985) and Crosato (1987)).

In the relevant literature (see e.g. Joglekar (1971)) a parameter is introduced that characterizes the extent of meandering before a cutoff occurs. This parameter is called the cutoff ratio, and it is defined as the ratio of the length of the bend to that of the cord (thus, in Figure 1 the ratio  $ACB/AB$ ).

It is stated in Joglekar (1971) that "this ratio varies according to the characteristics of the river at the site, such as the magnitude of the discharge, height of the flood rise, surface fall, bed material and its suitability for the growth of protective grass and bushes". Some numerical values for the cutoff-ratio are given in the same publication and they vary between 1.7 for Punjabi rivers in India and Pakistan and 8 to 10 for the Mississippi River.

The present analysis may help to get a better understanding of the parameters that influence the cutting-off process and that determine the value at which cutting off takes place. In Chapter 2 a theoretical analysis is given, initially for a simplified and later for a more complete case. Also time, as important parameter, is dealt with. In Chapter 3 the results obtained with the present method are discussed and the implications as to the importance of extreme floods and a possible application of the analysis to braided rivers are indicated. Required research to (experimentally) verify the derived criterion is outlined in Chapter 4, whereas some conclusions are listed in Chapter 5.

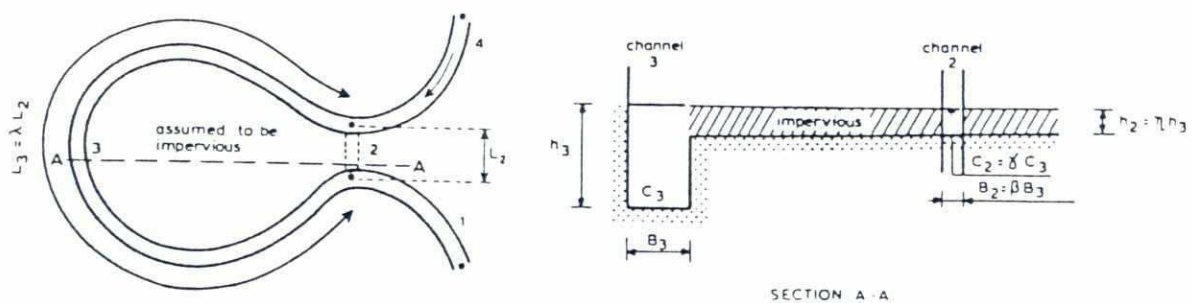


Figure 2 Definition sketch

## 2. Theoretical considerations

### 2.1 General

Consider the schematized river bend in Figure 2. During floods the river will exceed its bankfull conditions and part of the total discharge will follow a shorter route over the neck of the meander. When the bend is not too long, the difference in water level over this short cut will be small. The velocity over the flood plain will remain below the critical value for erosion. With increasing length of the meander bend the critical value may be exceeded. Whether or not this will result in a cutoff channel to develop, depends not only on whether the critical value is exceeded, but also on the quantities of sediment transported over the flood plain, as erosion is caused by a gradient in sediment transport and not by the sediment transport itself. It is therefore appropriate to consider the sediment balance of the channel potentially to be cut off.

This is done in the following sections, where in Section 2.2 first an analysis is given for the very simple case in which it is assumed that the upstream water level is not affected by the discharge flowing over the flood plain. This analysis does not hold for the conditions where a substantial discharge is flowing via the short cut. This case is analyzed in Section 2.3. Both cases considered are dealing with the initial erosion. For a cutoff channel to take over the conveying function of the channel that is being cut off, it has to be scoured to sizeable dimensions. In Section 2.4 some considerations as to the establishment of a representative time scale for this process are given.

### 2.2 Simplified case

Consider the simple network of channels that is shown in Figure 1. Channel 3 is due to be cut off by channel 2. It is assumed that the channels 1, 3 and 4 have the same width, the same slope, the same water depth during flood and the same roughness. Although the river is assumed to flow above bankfull conditions, it is also assumed that there is no water flowing perpendicular to the channels 1, 3 and 4 apart from channel 2. Channel 2 is assumed to be still a channel of minor dimensions, such that the flow through this channel does not influence the water level at the bifurcation. This implies that

$$\beta = B_2/B_3 \ll 1$$

Furthermore the cutoff-ratio  $\lambda$  is defined via:

$$\lambda = L_3/L_2$$

The same ratio exists between the slope of channel 2 and the slope of channel 3, because the drop in water level is the same over the two channels. Thus:

$$i_2 = \lambda i_3$$

Finally the parameter  $\eta$  is introduced as the ratio between the initial water depths in the channels 3 and 2, respectively, in such a way that:

$$h_2 = \eta h_3$$

It is assumed that the lengths of the channels 2 and 3 are sufficiently long to have normal depth at their upstream bifurcation. Expressed in the parameters introduced above, the discharge distribution over the channels 2 and 3 can be obtained from the equation for the normal depth. The following expression is found:



$$\frac{Q_2}{Q_3} = B (\eta)^{3/2} (\lambda)^{1/2}$$

The sediment entering channel 2 depends on the geometry of the bifurcation. For this simple case it is assumed that the sediment distribution is the same as the distribution of the discharge (an assumption that is only fair if most of the sediment is carried as suspended load), thus for the sediment entering channel 2 the following expression is found:

$$S_2 \text{ incoming} = B (\eta)^{3/2} (\lambda)^{1/2} S_3$$

The sediment transport capacity of channel 2 can be estimated with an appropriate sediment transport formula. It is assumed that the formula of Engelund-Hansen (1967) is applicable. The sediment transport capacity of channel 2 can then be expressed as a function of the sediment transport in channel 3 and the following expression is obtained:

$$S_2 \text{ transport capacity} = B (\eta)^{5/2} (\lambda)^{5/2} S_3$$

For erosion to take place in channel 2, the incoming sediment transport should be less than the transport capacity of this channel. This leads to the following criterion to be fulfilled for a cutoff to develop:

$$S_2 \text{ incoming} < S_2 \text{ transport capacity}$$

which can be re-written using the expressions derived above as

$$\eta (\lambda)^2 > 1$$

This criterion is graphically depicted in Figure 3. Of course, in the present context only the conditions for  $\lambda > 1$  are relevant.

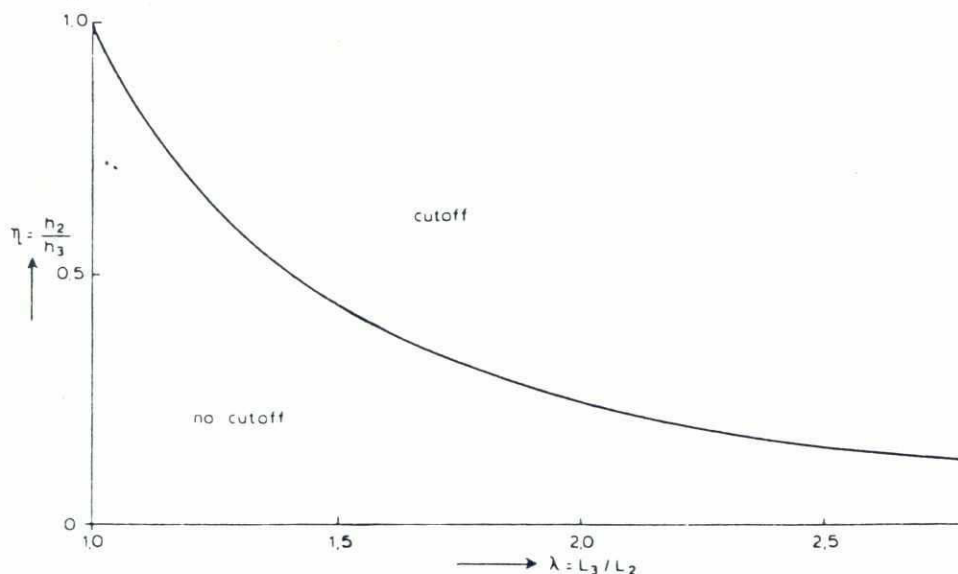


Figure 3 Criterion for the development of a cutoff for simplified case ( $B \ll 1$ ,  $n = 5$ ,  $a = 1$ ,  $\gamma = 1$ ; for explanation of these parameters reference is made to Section 2.3)

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Considering Figure 3 and the derivation underlying it, the following remarks can be made:

- (1) The larger the value of  $\lambda$ , the more probable the development of a cutoff, which is of course what is to be expected.
- (2) From the graph it follows that the larger the flood (which is reflected in the value of  $\eta$ ), the more bends will be cut off. This is in line with observations that a meandering river has a straighter course after an extreme flood.
- (3) The criterion is rather sensitive for the relation between the sediment transport  $s(m^2/s)$  and the velocity  $u$ . Often (see e.g. Jansen (1979) or de Vries (1987)) this relation is approximated by

$$s = m u^n$$

where  $m$  and  $n$  are coefficient. In more general terms the above criterion can therefore be expressed as:

$$\eta(\lambda)^{(n-1)/(n-3)} > 1$$

The value of  $n$  varies between say 3 and 10. The smaller the value of  $n$ , the easier cutoffs occur.

- (4) The above criterion only holds for the beginning of the development of the cutoff channel. Gradually the value of  $\eta$  will increase during the lowering of the bed level of the cutoff channel, which will accelerate the cutoff process. This may be counteracted by e.g. the reduction of the water level at the bifurcation (see Section 2.3).

### 2.3 More complete case

In the above derivation the main simplifications were:

- $\beta$  is small, so the water level at the bifurcation is not affected by the discharge via channel 2,
- hydraulic roughness is the same in channels 2 and 3,
- the sediment distribution at the bifurcation is conform the water distribution.

In this Section a criterion for the more general case will be derived. For this the following assumptions are made:

$$C_2 = \gamma C_3$$

$$\frac{s_{2 \text{ incoming}}}{s_{3 \text{ incoming}}} = \frac{1}{\sigma} \frac{Q_2}{Q_3}$$

The value of  $\sigma$  depends on the geometry of the bifurcation (see e.g. Bulle (1926) and Habermaas (1935)), on the dominant mode of the sediment transport (bed load or suspended load) and probably also on the value of  $\eta$ . Furthermore the assumption that  $\beta$  is small is dropped, allowing for a drop in water level  $\Delta h$  at the bifurcation. It is still assumed, however, that  $\Delta h$  is small compared to the water depth in channel 2. Again assuming that in both channels normal depth is present at the bifurcation, it is possible to arrive at a criterion similar to the criterion derived in Section 2.2. This criterion reads for  $n=5$  as:



for  $\beta \ll 1$  :

$$\eta \sigma \gamma (\lambda)^2 > 1$$

for any  $\beta$ :

$$\gamma \lambda^2 \eta \left(1 - \frac{2}{3} \beta \gamma \lambda^{\frac{1}{2}} \eta^{\frac{1}{2}}\right) \left(1 + \beta \gamma \lambda^{\frac{1}{2}} \eta^{\frac{1}{2}} - \frac{10}{3} \beta \gamma \lambda^{\frac{1}{2}} \eta^{3/2}\right) >$$

$$\frac{(1 + \beta \gamma \lambda^{\frac{1}{2}} \eta^{\frac{1}{2}})^2}{\beta \gamma \lambda^{\frac{1}{2}} \eta^{3/2} + \sigma (1 + \beta \gamma \lambda^{\frac{1}{2}} \eta^{\frac{1}{2}} - \beta \gamma \lambda^{\frac{1}{2}} \eta^{3/3})}$$

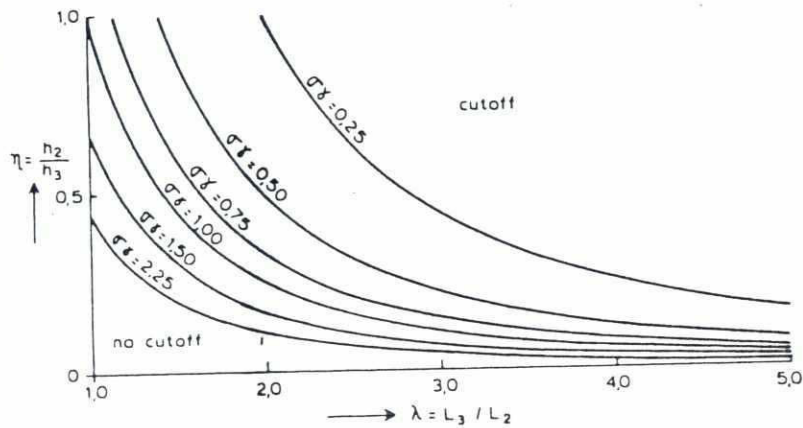


Figure 4 Influence of sediment offtake ( $\sigma$ ) and relative roughness on cutoff criterion ( $n=5$ ,  $\beta \ll 1$ )

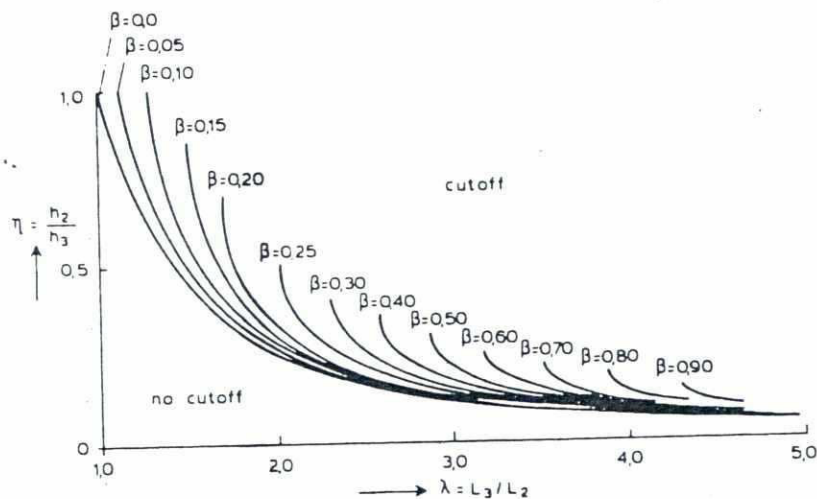


Figure 5 Influence of relative width of cutoff channel ( $\sigma=1$ ,  $\gamma=1$ ,  $n=5$ )

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The latter criterion cannot be solved analytically, so a computer program was made to solve  $\eta$  as a function of  $\lambda$ ,  $\gamma$  and  $\beta$ . There are so many variables involved that it is not possible to make one figure depicting the obtained results. Therefore, Figures 4 and 5 were prepared. Figure 4 shows the influence of  $\sigma$  and  $\gamma$ , but assuming that  $\beta$  is small, while Figure 5 addresses the effect of  $\beta$ , but assuming that  $\sigma=1$  and  $\gamma=1$ .

The following observations can be made upon inspection of Figures 4 and 5:

- (1) The larger the value of  $\sigma$ , the easier a cutoff will take place. This implies that in rivers where bed load is dominant over suspended load the sinuosity will be less, provided that all other factors are the same. The influence of  $\gamma$  is similar, but it may be expected that the variation in  $\gamma$  is less than the variation in  $\sigma$ .
- (2) The effect of  $\beta$  is substantial. In general it can be stated that an increase of the width of the cutoff channel results in more stable conditions. This is due to the reduction of the water level at the bifurcation. This reduction in water level results in a less water per unit width entering the channel to be scoured. It may be assumed that initially the conveying width of the scouring channel has the same order as the width of channel 3. In later stages a decrease in  $\beta$  may occur because the channels scoured in the neck of a meander are relatively narrow and deep, and widening of the scoured channel will only occur in later stages.

#### 2.4 Development of the cutoff in time

The considerations in the previous sections deal with initial stages of the cutoff. Whether or not a cutoff will take place during a certain flood does, however, not only depend on the factors discussed above, but also on the duration of the flood. If the flood considered is a flashy one, then there is probably not sufficient time for the cutoff to develop. Then it may happen that scour in the neck of the meander does occur and is noticeable after the recession of the flood, but the actual cutoff may occur only after some more floods. In principle it is possible to derive a typical time scale for the cutoff process via dimensional analysis. Such a typical time scale can be found by dividing the volume to be scoured by the difference between the sediment transport capacity and the incoming sediment transport in channel 2. The following typical time scale  $T_c$  (cutoff) is obtained in this way:

$$T_c = \frac{\beta * B_3 * L_2 * (h_3 - h_2)}{(S_2 \text{ transp. cap.} - S_2 \text{ incoming})}$$

This time scale should then be compared with the duration of the flood. The following remarks should be made:

- (1) The above derivation of a time scale for the cutoff process is a very crude one. It would be preferable to try to derive such a time scale either via analytical models (see Klaassen (1988) for such an approach for other morphological phenomena) or via numerical computations.
- (2) From the considerations in the previous sections it is clear that scouring of channel 2 may result in an acceleration of the erosion process via an increase in  $\eta$ . This would imply that the above time scale is a conservative estimate and the actual cutoff process may proceed more quickly. Also this can only be evaluated via more detailed research.
- (3) During the scouring process the channel will adapt not only its bed level, but also its width. For both processes probably different time scales will apply.



### 3. Discussion and implications

In the preceding Chapter an analytical method is presented that allows the determination of a cutoff criterion. It is obvious that this criterion depends on the conditions in a particular river reach. Especially the distribution of the sediment discharge at the bifurcation is an important parameter which will depend on the local conditions and on the river characteristics, notably on the dominant mode of transport. Also the flood level, in particular in relation to the floodplain level and the average depth in the minor bed, appears to be a dominant parameter. The implication is that the criterion cannot be determined without considering the actual conditions in the field. For the time being such a criterion can therefore only be derived for a particular river reach considering the river characteristics present. It seems appropriate to concentrate initially on the relationship between  $\eta$  and  $\lambda$  by considering the dimensions of curved channels that have been cut off in the past, and only later to try to get a more basic understanding via the analysis given in Chapter 2.

The present model predicts erosion first to occur at the upstream reach of the cutoff channel. This is contrary to what is frequently observed during the cutting off of meanders: the cutoff is initiated by gully formation at the downstream side of the cutoff channel. This probably calls for the inclusion of the beginning of motion in the analysis. At the downstream side of the cutoff channel, velocities are highest owing to the draw-down which may occur.

Assuming that the present method appears to provide a fair description of the cutoff process, it should be included in meander models as described by Johannesson and Parker (1985) and Crosato (1987). An interesting application will be the prediction of the need for protection works along meandering rivers. Also, the method could be used for the dimensioning of a lead channel through a meander neck.

An important application of the present analysis is the application of it to future conditions in rivers, which are regulated by reservoirs. Owing to the construction of these reservoirs the medium floods will be reduced; consequently the periods when cutoffs that occur will be present less frequently. As a consequence it may be expected the cutoffs will occur for larger values of  $\lambda$  only. The result may be that the width of the meander belt will increase. Because extreme floods are hardly affected by reservoirs, this influence may be limited, but this has to be studied for each river separately. A problem is that, at present, no prediction methods are available for changes in  $\eta$  that will occur too as a result of the change in frequency distribution of the discharges.

Some braided rivers have lower values of  $\lambda$  for which cutoffs occur. An analysis of satellite data from the Jamuna (= Brahmaputra) River in Bangladesh has shown that cutoffs occur for  $\lambda$ -values not in excess of 1.5. This is substantially lower than the values for other rivers as cited by Joglekar. It is tempting to explain this phenomenon by the specific characteristics of a braided river: the reduction of water level at the bifurcation, due to the development of the cutoff channel, may at least partly be compensated by a redistribution of the total discharge over the various braids, counteracting the stabilizing tendencies shown in Figure 5. It is felt, however, that the whole analysis presented here may be less appropriate for low values of  $\lambda$ .

#### 4. Required additional research

The analysis presented here is preliminary only and additional research is required to develop it into a methodology that can be applied successfully in river engineering. At present the following additional studies can be identified:

(1) Further theoretical studies.

In particular these can comprise the time-dependent process of cutting off. Accepting the schematization used, it is possible to carry out numerical computations with a one-dimensional morphological model. This will allow to get a better understanding of the time scale of the process. Also it can be attempted to develop an analytical time-dependent model using methodologies described by de Vries (1987). In addition, more research is required in e.g. the change in  $\delta$  and the different time scales of the deepening and the widening process. Also it may be required to include the initiation of motion in the analysis.

(2) Comparison with field data.

The present method has not yet been tested against field data. It is advisable to identify appropriate data sets to test the predicted trends.

(3) Extension to braided rivers.

If the model presented here proves to be acceptable, an attempt should be made to extend the approach to braided rivers. Some preliminary ideas are already given at the end of Chapter 3.

#### 5. Conclusions and recommendations

A method has been developed that can be used for the prediction of the occurrence of a cutoff of a curved alluvial channel during a certain flood. Although the method allows to identify the relevant parameters, it can not be used for a particular river without considering the actual field conditions. More research is needed to develop the method in a reliable tool for river engineering purposes like the protection against erosion by meandering channels. In future the inclusion of the method in meander models seems appropriate. For application of the method to braided rivers more research is required.

#### Acknowledgements

The present research work was initiated by and data were used from the Jamuna Bridge Appraisal Study. This study is being funded by the United Nations Development Programm (UNDP) through the World Bank. The Jamuna Multi-purpose Bridge Authority (JMBA) is furthering the studies for and the construction of the bridge. The Jamuna Bridge Appraisal Study is carried out by Rendel, Palmer and Tritton (U.K.), Nedeco and Banlandesh Consultants Ltd. The first author is the senior river morphologist in this study.

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## Paper 4.5

KLAASSEN, G.J. AND G. MASSELINK, (1992),  
Planform changes of a braided river with fine sand as bed  
and bank material, *Proc. 5th Intern. Symp. River Sediment-  
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## Planform Changes of a Braided River with Fine Sand as Bed and Bank Material

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**ABSTRACT:** As a first step towards the development of a predictive model for channel changes and related bank erosion, morphological processes were studied in a braided river with fine sand as bank and bed material. This was done using mainly (geometrically corrected) satellite images, the use of which was justified because of the large scale of the river studied: the Jamuna River in Bangladesh with a total width of up to 17 km. The study concentrated on bank erosion, channel shifts, and processes at bifurcations and confluences. It was found that the bank erosion depends on the relative curvature of the curved channels, and that the rate is an order of magnitude larger than predicted by Hickin & Nanson (1984). Several types of channel shifts were identified, and it was observed that cutoffs occur already at very low cutoff ratio's. The main conclusions from the study are that on the one hand much more studies are needed to improve the understanding of these different processes, and on the other hand that there is a clear limit to the period over which predictions can be made due to the observed chaotic behaviour of the channel changes.

### 1 INTRODUCTION

Understanding of future planform changes of rivers is essential for the siting of infrastructure (like embankments, supply canals, roads, etc.) and of towns and villages along rivers. Nowadays the understanding of planform changes of meandering rivers is fair: meandering via bank erosion can be predicted to some extent (Hickin & Nanson, 1984), including the occurrence of cutoffs (Klaassen & van Zanten, 1989), and predictive models for the development of meanders (Parker & Andrews, 1981; Crosato, 1990). No predictive models are available for braided river systems, although the need for predictions of future behaviour is even more acute, because of the apparently erratic behaviour of such river systems (Burger et al, 1988).

This paper reports on a study that was carried out to improve the understanding of the processes in braided river systems, with the ultimate aim to develop a deterministic method for predicting the future changes in channel planform and the corresponding bank erosion. Connected to this is the question whether there is a limitation as to the period over which bank erosion along a braided river can be predicted in

advance. The present study was initiated in relation to the design of a bridge across the braided Jamuna River in Bangladesh. The Jamuna River is the lowest reach of the Brahmaputra River in Bangladesh (see Fig.1). Its main characteristics are a water level slope that gradually decreases from 0.10 to 0.06 m/km, the bed material is quite uniform and  $D_{50}$  varies from 0.25 mm near the Indian border to 0.16 mm at the confluence with the Ganges River, the average annual flood is about 60,000 m<sup>3</sup>/s, and during low flow between 4,000 and 12,000 m<sup>3</sup>/s. The Jamuna River is a large braided sand bed river, the number of braids (during low flows) varies between 2 to 3, and the total width of the braided channel pattern varies between 5 and 17 km. Flood conditions usually prevail from May through October (see e.g. Coleman (1969)) while the low flow season lasts from December through March. For more details on the Jamuna River reference is made to Coleman (1969), Bristow (1985), Klaassen & Vermeer (1988a and 1988b) and RPT et al (1987, 1990a and 1990b).

Within the Jamuna Bridge Appraisal Study, an extensive analysis was made of rates of bank erosion along and channel processes in the Jamuna River in Bangladesh, using satellite images and cross-sections.

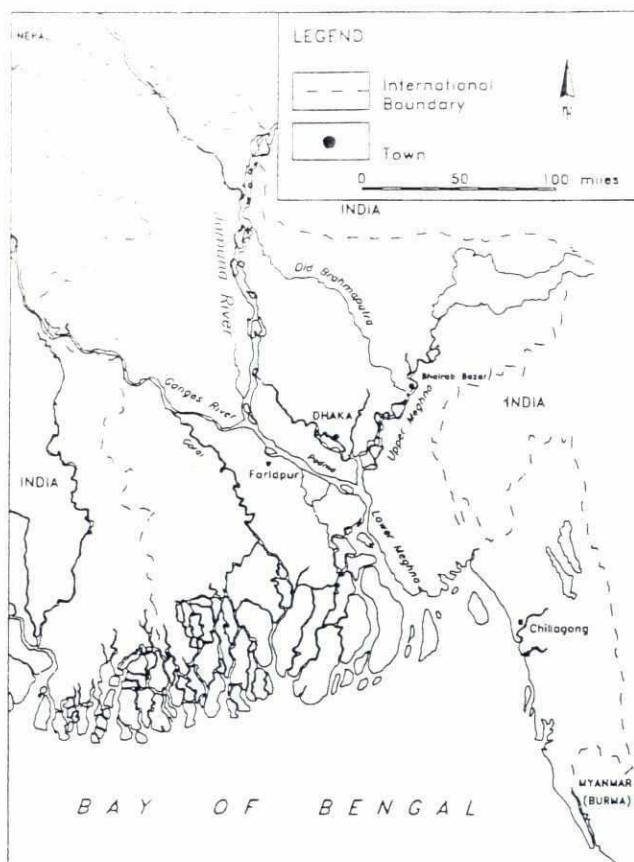


Fig. 1 Map of Bangladesh with Jamuna River

This paper summarizes the results of this study. For more details reference is made to Masselink (1989).

Several studies on the Jamuna River have already been carried out, leading to an increased understanding of the processes that take place in such large, braided sand-bed rivers (Coleman (1969), Bristow (1985), Klaassen & Vermeer (1988a,b), Klaassen et al (1988)). These studies have revealed processes that take place on a scale which can not be compared to the results of studies on "normal" braided rivers (see e.g. Rust, 1972). In particular the use of satellite images in the present study does enhance the understanding of the processes in the Jamuna River substantially. Because of the large scale of the river involved the accuracy of the LANDSAT MSS images, in combination with the rapid and substantial changes, makes the use of the satellite images very fruitful. For the time being, SPOT images are not a real alternative as (i) they cover a much shorter period and (ii) the detailed insight they provide, is not really needed in this stage.

## 2 APPROACH

### 2.1 Data used

Yearly bank erosion rates were studied on the basis of comparisons of geometrically and otherwise corrected satellite images. Here LANDSAT MSS images of in total 6 years were used, with time intervals of 1, 2 and 6 years. The data that were used in this study comprised cross-sectional data and planform data derived from LANDSAT images.

### 2.2 Cross-sectional data

The cross-sectional characteristics of the Jamuna River were studied on the basis of soundings made by the Bangladesh Water Development Board (BWDB) since 1966. The locations of the studied cross-sections are indicated in Fig. 2.

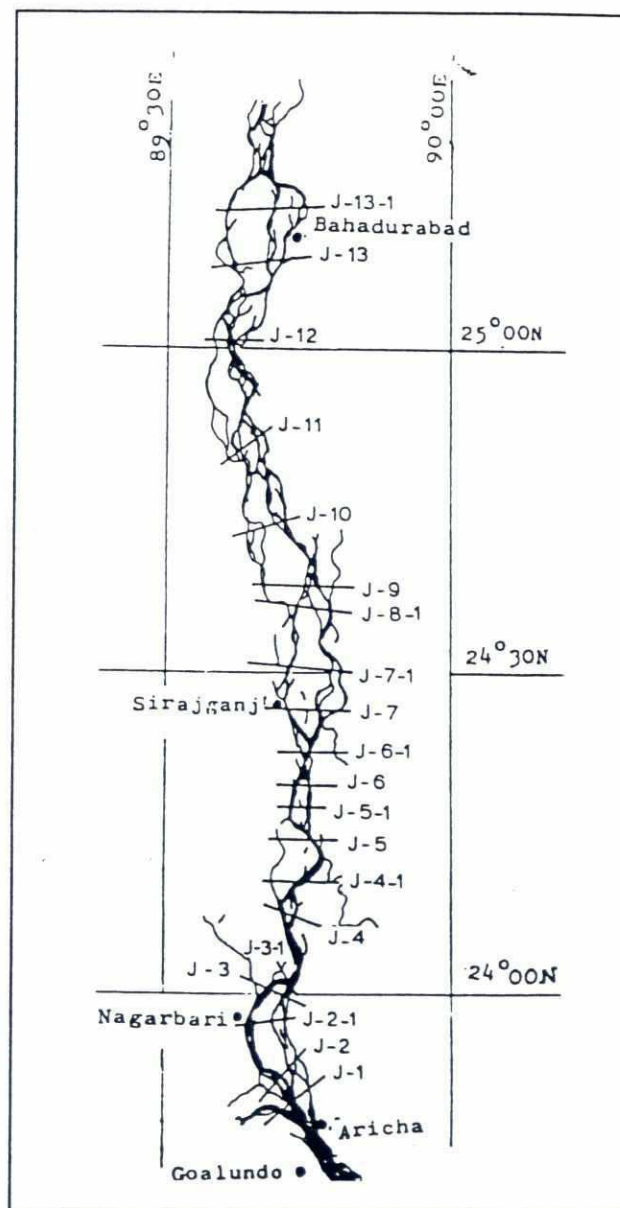


Fig. 2 Location of BWDB cross-sections

An extensive analysis of all cross-sections has been carried by Klaassen & Vermeer (1988a). They



concluded that over the last decades the Jamuna channel bed was stable both laterally and vertically. Hence, the Jamuna River is on the average not aggrading, and can be supposed to be in equilibrium.

### 2.3 Selected satellite images

For the LANDSAT MSS images selected see Table 1.

| Image Scene number <sup>1)</sup> | South 138-43 | North 138-42 |
|----------------------------------|--------------|--------------|
| Year                             | Date         | Date         |
| 1976                             | 4 Mar        | -            |
| 1977                             | 9 Feb        | 9 Feb        |
| 1978                             | 22 Feb       | -            |
| 1984                             | 23 Feb       | 23 Feb       |
| 1986                             | 20 Feb       | 20 Feb       |
| 1987                             | 7 Feb        | 7 Feb        |

<sup>1)</sup> LANDSAT 4/5 numbering

Table 1 LANDSAT images used for the present study

The research was primarily based on the images of the Southern part of the Jamuna, because these images form a more or less continuous series, whereas the Northern images did not. Only images were selected for dates for which the stage at Sirajganj (see Fig.2) was between 6.00 and 7.00 m+PWD (PWD refers to the agreed reference level in Bangladesh). Hence, all images show the Jamuna River during low flow conditions. The images were geometrically corrected (de Jong, 1988), hence mutual comparisons between different years could be made.

### 2.4 Classification

In a next step the images were classified according to the type of soil and the vegetation cover. For this an algorithm was applied, using two spectral bands. For the procedure applied see Klaassen et al (1988) and de Jong (1988). The following classification and corresponding colour coding was adopted:

- water : blue;
- bare land : grey ;
- sand : orange;
- vegetation : green.

The classified images are called statical composites. An example of a series of these statical composites is given in Fig. 3. Because it concerns a black and white copy, here the water corresponds to the darkest colour. Changes over the years in water and sand distributions are obtained by comparing statical composites. An "overlay" (of the digital data) of two statical composites is made, and based on the four identified classes, there are 16 possible combinations,

yielding 16 new classes. These can subsequently be interpreted and analyzed. The new picture obtained (being the comparison of two successive years) is called a dynamical composite.

### 2.5 Processing, analysis and interpretation

The statical and the dynamical composites were used, together with the BWDB cross-sections, for the study of processes active in this braided river system. The following topics were investigated:

- (1) bank erosion rates of curved channels,
- (2) cutoffs,
- (3) processes at bifurcations,
- (4) processes at confluences.

The rationale for selecting these topics is that it is felt that these processes in combination are responsible for the changes in channel patterns. The results of the various analyses are presented in the subsequent Chapters 3 through 6.

The different processes were studied via measurements in the statical and dynamical composites. The accuracy of the LANDSAT-image after processing is approximately 200 meters at a significance level of 97%. De Jong (1988) presents a complete discussion on the selection, processing, classification and accuracy of the images.

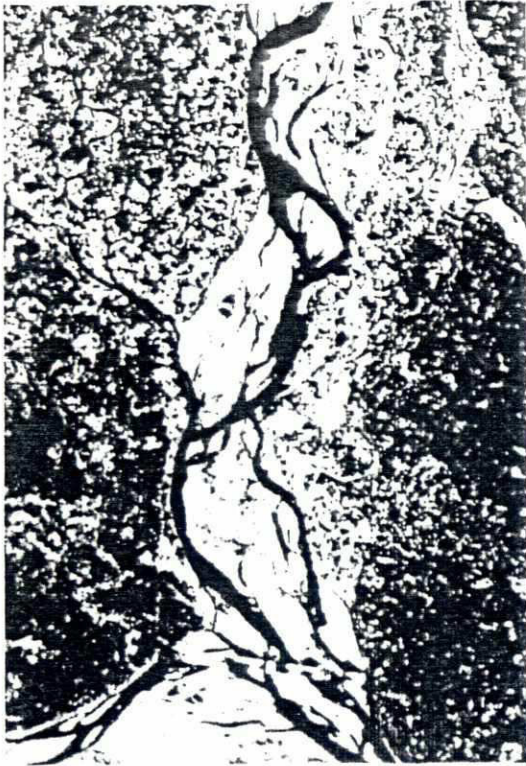
## 3 BANK EROSION RATES

### 3.1 Bank erosion rates

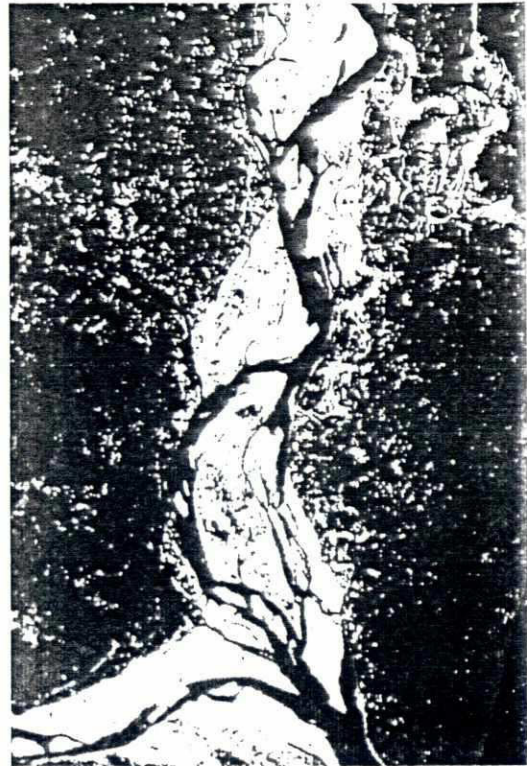
Using the dynamical composites, the bank erosion rates along curved channels could be studied. Not all curved channels were included in the analysis: channels that had significantly narrowed or widened in one or two years were excluded, as for these cases it was difficult to distinguish between bank line changes due to an increase or decrease of discharge and 'true' bank erosion. The bank erosion does obviously vary along the bend: here only the maximum bank erosion rates were studied.

Fig. 4 provides a summary of the observations for the four dynamical composites of the south image, via a plot of the observed erosion rates (divided in classes) versus the number of observations in each class. Yearly erosion rates  $E$  apparently vary between 0 and 1,000 m. Fig. 5 provides the same information, but here the average number of occurrences for the four different periods is presented. It appears that the bank erosion rate along the curved channels of the Jamuna River in most of the cases is between 0 and 500 m/year with larger values up to 1,000 m/year under exceptional conditions. This is conform the findings of Coleman (1969).

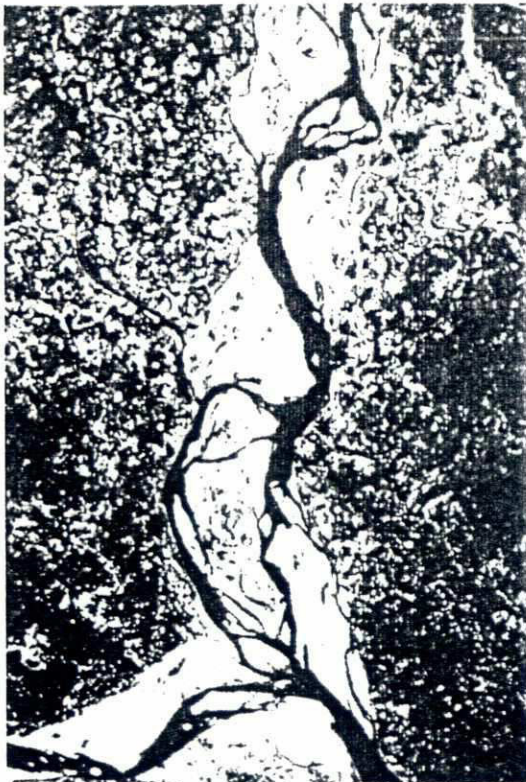




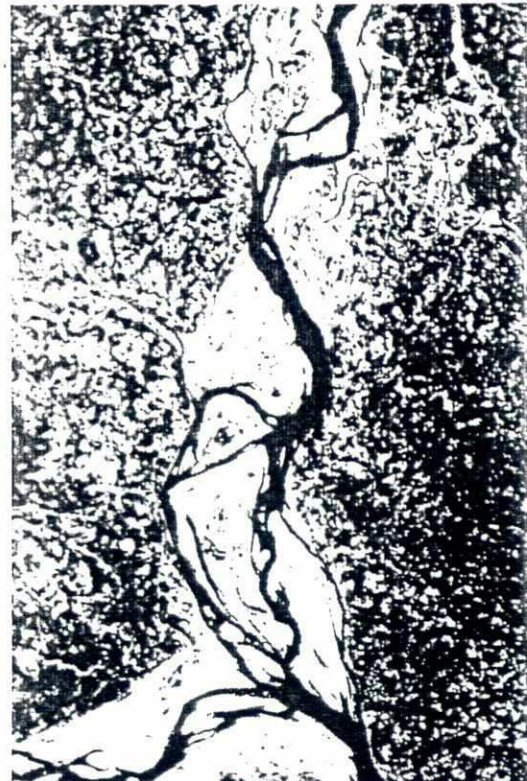
(a) 1978



(b) 1984



(c) 1986



(d) 1987

Fig. 3 Classified LANDSAT images of the Southern part of the Jamuna River for four years (approximate scale 1:500,000)



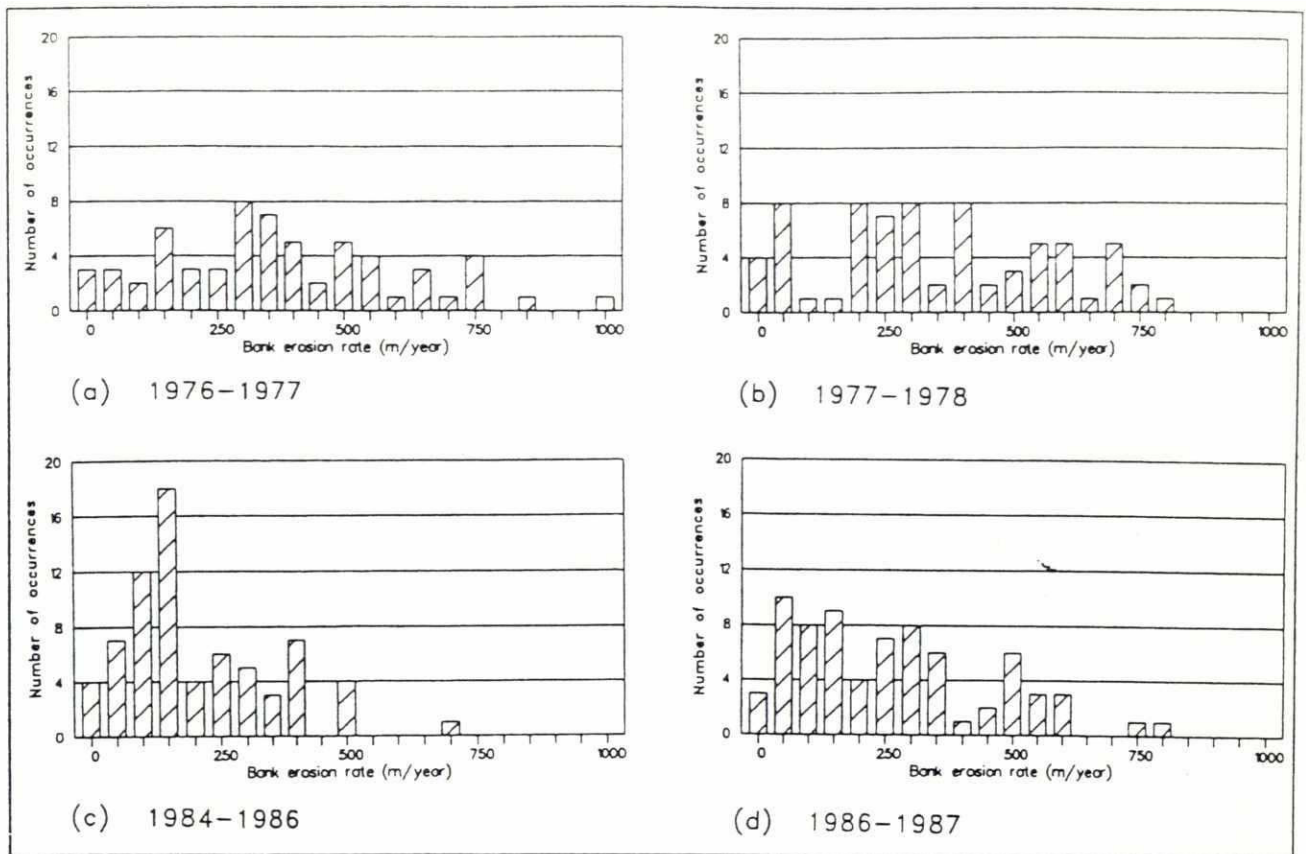


Fig. 4 Observed bank erosion rates along curved channels of the Jamuna River

Also some other aspects of the bank erosion were studied using the satellite images. The direction of the bank erosion was studied by plotting in Fig.6 the direction of the maximum erosion relative to the valley slope. The direction of the largest erosion is on the average approximately perpendicular to the valley slope, while for smaller erosion rates it frequently deviates substantially from the valley slope.

Also the influence of vegetation was studied. It was

found that chars with minor or absent vegetation do not erode faster than vegetated floodplains along the

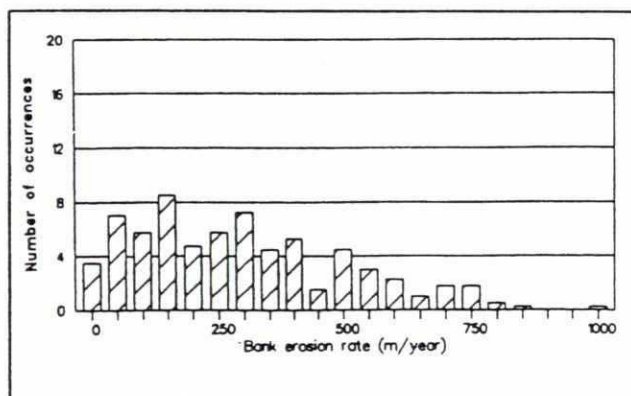


Fig.5 Average bank erosion rates along curved channels of the Jamuna River for the four periods considered in Fig. 4

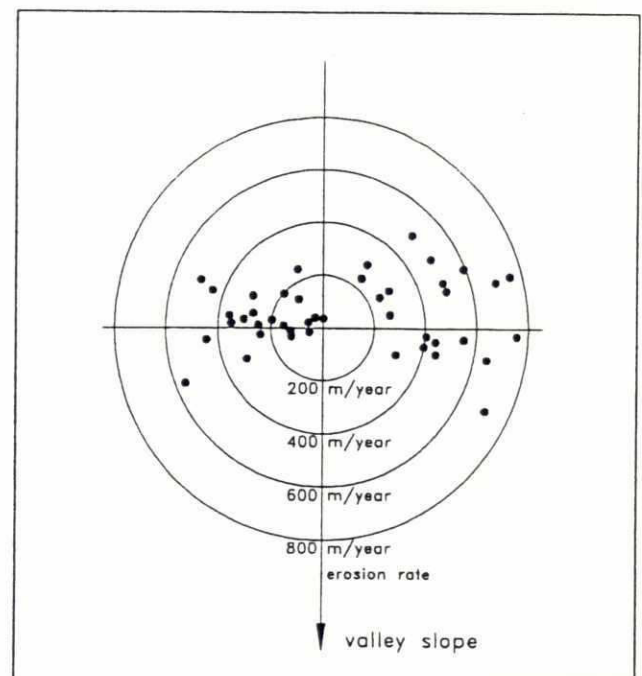


Fig. 6 Direction of bank erosion along curved channels of the Jamuna River

edge of the channel pattern. Hence it can be concluded that for the deep channels of the Jamuna River (see Klaassen & Vermeer, 1988b) the influence of vegetation only effective in the upper layers is negligible.

Furthermore a distinction was made between the following bank erosion mechanisms (see the below Fig. 7): rotation (A), extension (B) and translation (C).

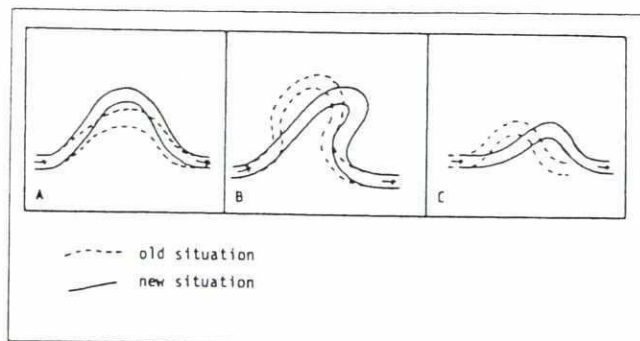


Fig. 7 Possible bank erosion mechanisms studied here

It was found that both rotation and extension do occur; translation is absent along the Jamuna braids. Translation does usually occur for cohesive banks. The chars and flood plain deposits of the Jamuna River exhibit hardly any cohesion, so this may be an explanation.

Finally it is remarked that apparently the yearly flood hydrograph may play a role in the magnitude of the yearly erosion rate (see the difference between e.g. 1976-1977 and 1984-1986). This has not been explored in more detail yet.

### 3.2 Analysis

The observed bank erosion rates were analysed within a frame-work similar to previous work by Nanson & Hickin (1985) for meandering rivers, by establishing the relation between the bank erosion rates and the relative curvature of the channels. Here a complicating factor is the fact that not one channel is considered, but that several parallel braids are present. In a meandering river the bankfull width is fairly constant (and fairly good related to the 'dominant' discharge). In the present case the width of a channel will differ depending on whether the channel is in regime, scouring or vanishing. This makes scaling with the width  $W$  as parameter (as done by Nanson & Hickin) more risky. Nevertheless this has been attempted hereafter.

According to Nanson & Hickin the yearly bank erosion  $E$  (in m) can be expressed as:

$$E = f(W, R/W, C, B) \quad (1)$$

where  $E$  = yearly bank erosion (m),  $W$  = channel width (m),  $R$  = radius of curvature of the curved channel (m),  $C$  = Chezy coefficient ( $m^{1/2}/s$ ), that represent the channel roughness, and  $B$  = overall bank resistance coefficient. For the time being it was assumed that the parameters  $C$  and  $B$  do not vary along the Jamuna River, hence for this river the equation would reduce to :

$$E = f(W, R/W) \quad (2)$$

This expression was tested for the Jamuna bank erosion data. The determination of the applied radius of curvature was done in the following way. Usually the values of relative curvatures ( $R/W$ ) for the two years were averaged. If, however, one of the values for  $R/W$  was smaller than 5.0, and the other was greater than 5.0, then the value of the bend with the smallest relative curvature was used. The reasoning behind this choice is that it was considered that this bend was most active in the eroding process.

Initially a distinction was made as to the channel width: four classes were identified (see Fig. 8). The results, related to bends selected according to the criteria given in Section 3.1, are presented in Fig. 8. It can be observed that the largest channels tend to correspond to the largest erosion rates. Hence a similarity collapse was attempted, by plotting  $E/W$  versus  $R/W$ . The result is given in Fig. 9.

Upon inspection of Fig. 8 and Fig. 9 the following observations can be made:

- (1) There is definitely a negative correlation between the relative bend curvature ( $R/W$ ) and the erosion rate ( $E/W$ ). Low relative curvatures lead to relatively fast erosion rates, and vice versa. There is no tendency for very sharp bends ( $R/W < 2.5$ ) to have smaller erosion rates, as was observed for meandering channels by Nanson & Hickin (1984). The value of  $W$  used here corresponds to the low flow width. As on the average the bankfull width is about 3 times larger (Klaassen & Vermeer, 1988a), a possible peak would have occurred here for the low flow data for  $R/W$ -values of approximately 7.5.
- (2) The largest bends ( $W > 1000$  m) plot low in Fig. 9, indicating that relative erosion rates by large channels is relatively small. However, only 7 large channel bends were studied. From Fig. 8 it appears that there is no significant difference in relative erosion rate between the smallest ( $W < 500$  m) and the intermediate ( $500 < W < 1000$  m) channels.
- (3) Both Fig. 8 and Fig. 9 show a substantial scatter. Apart from the assumptions made during the



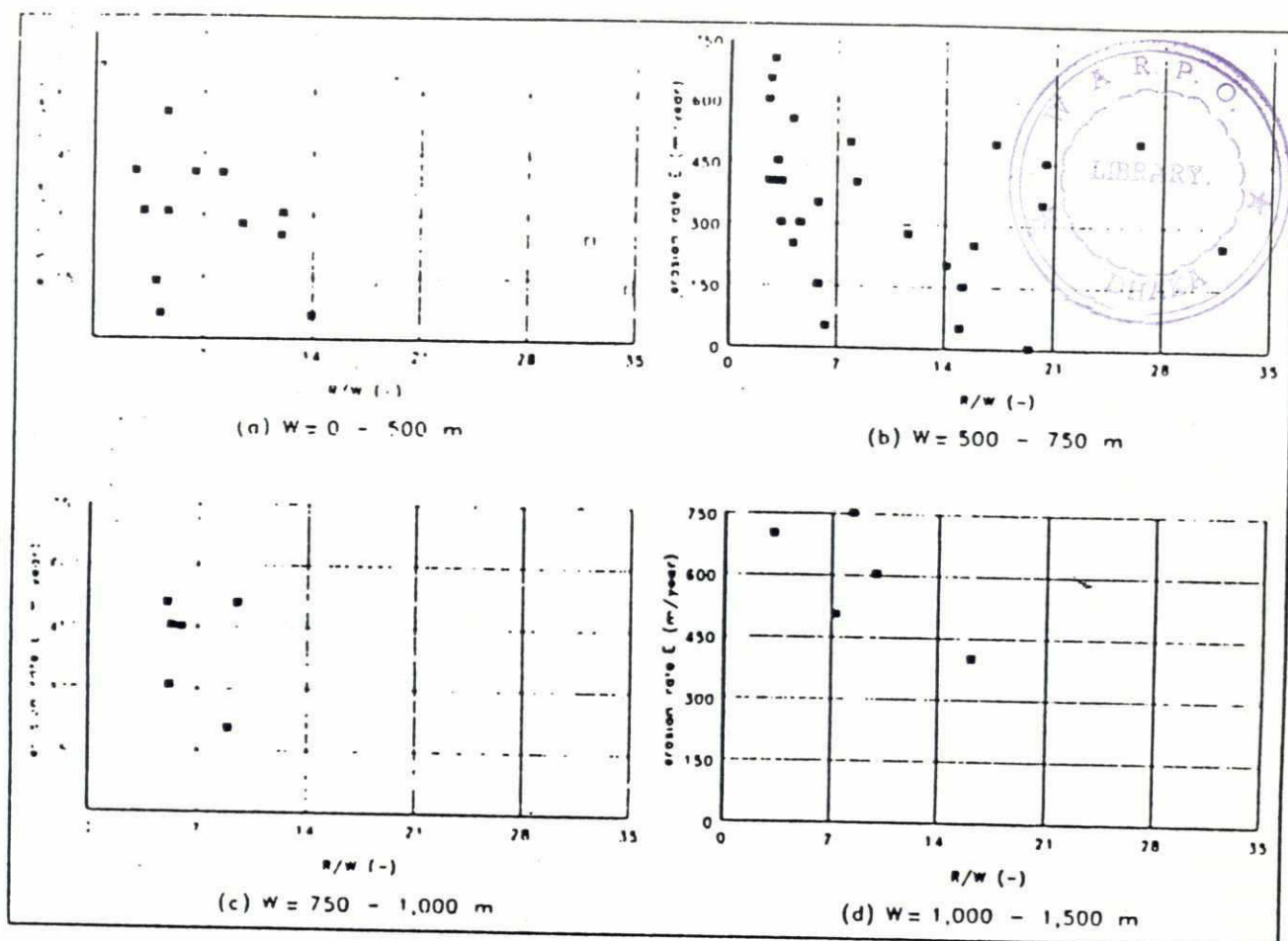


Fig. 8 Dimensionless erosion rate  $E/W$  versus  $R/W$  for the channels classified according to width

derivation of Eq. (2), possible explanations for this scatter may be:

- measuring accuracy (see before);
- use of low flow widths;
- use of one value of the radius of curvature, where in reality  $R$  may vary considerable along the bend;
- the inclusion of less active bends possibly being rather shallow

A comparison was made between the presently observed erosion rates and the rates predicted by Iylickin & Nanson (1984). Assuming a bankfull discharge of a channel of  $20,000 \text{ m}^3/\text{s}$  and using the other river characteristics described in Chapter 1, the estimate of several tens of meter of erosion a year. It is clear that this estimate is an order of magnitude smaller than what was actually observed. A possible

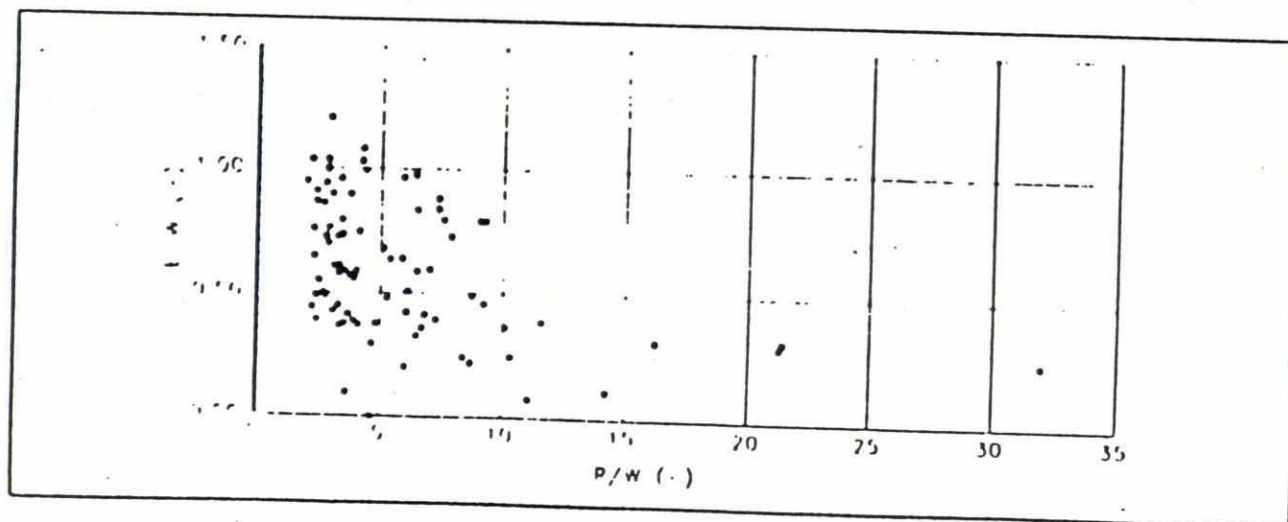


Fig. 9 Dimensionless erosion rate  $E/W$  versus  $R/W$  for all curved channels along the Jamuna River

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explanation could be that in the case of the Jamuna River, the floods extend over a period of some three months. It may be appropriate to include in any prediction method also the duration (and magnitude?) of the floods.

#### 4 CHANNEL SHIFTS

Using the dynamical composites, also channel shifts were studied. The channel pattern of the Jamuna River changes continuously: large channels being abandoned, and new channels developing in a few years only, are common features. Coleman (1969) refers to this process as to 'sudden shifts'. A channel shift is accomplished by the development of a completely new channel, or, more common, a pre-existing channel takes over the conveyance function of another channel. The new channel may flow through the original channel deposits or through the floodplain. Three types of channel shifts were identified and these are discussed hereafter:

##### (1) Bar induced shifting

Sand bar induced channel shifting is not often identified in the literature, but is a rather important process in the Jamuna River. Large sand bars migrating in a channel or developing in specific channel reaches block the entrance of small channels. Consequently these channels will receive less discharge and are "abandoned" subsequently. Sand bars can also redistribute the flow so that one channel receives more discharge than another. A complete channel shift can be the result (see Fig. 10).

##### (2) Development of a cutoff

The development of a cutoff occurs frequently. Of the 23 cases of channel shifts studied, 11 relate to cutoffs. To get some insight into this phenomena, it was approached in two ways, notably (i) considering the relative curvature  $R/W$ , and (ii) considering the cutoff ratio (Klaassen & van Zanten, 1989).

##### Re (i) Relative curvature

The relative curvature of a bend is assumed to be a primary control on the processes which determine channel shifts of this type. From a study of channel shifts, it appeared that the average relative curvature ( $R/W$ ) of the pre-shift bends was 3.6 (standard deviation 1.8). Bends with  $R/W$ -values  $< 3.6$  are apparently 'unstable' and result in a channel shift within a period of two years. This may be explained by the shear stress distribution in the bend.

##### Re (ii) Cutoff ratio

Klaassen & van Zanten (1989) have shown the importance of the cutoff ratio  $\lambda$ , being the ratio between the length of the channel along the curved reach and the direct line, along which the cutoff channel is developing. For a number of cutoffs in the Jamuna River the value of  $\lambda$  was determined and a cumulative frequency distribution was prepared. The

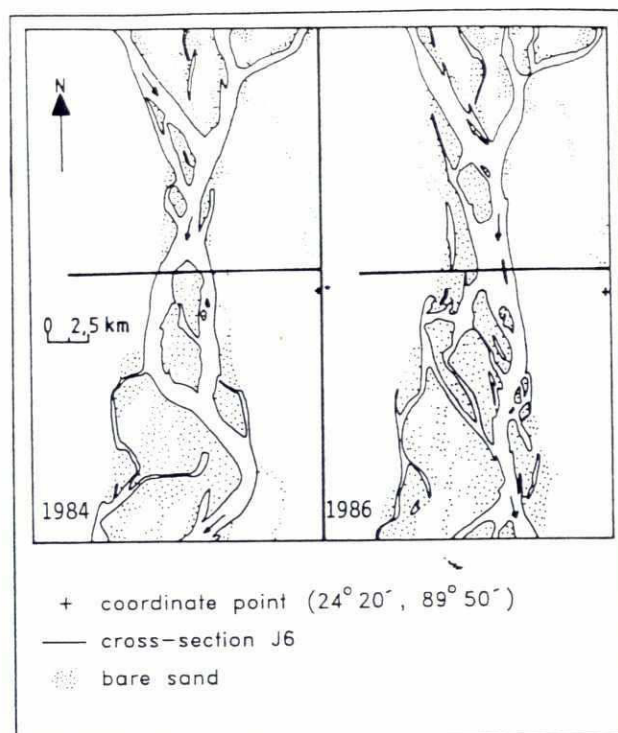


Fig. 10 Bar-induced channel shift

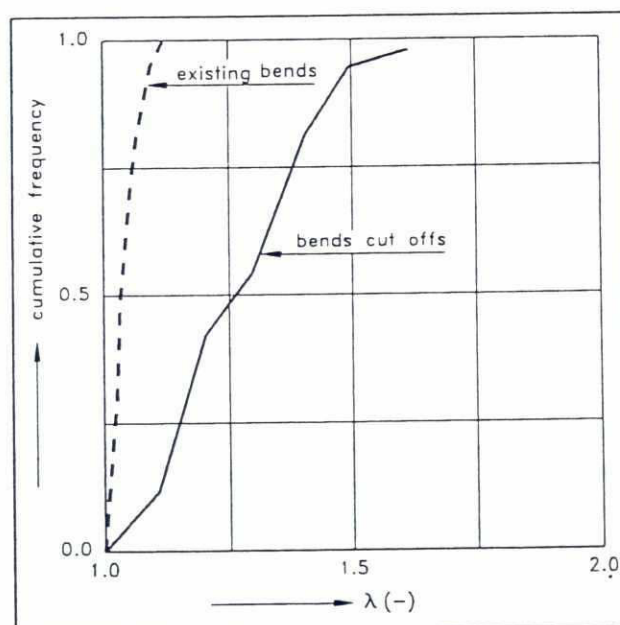


Fig. 11 Cumulative frequency distribution of cutoffs compared with existing bends

results are presented in Fig. 11. It appears that the values of  $\lambda$  vary between 1.0 and 1.7. These are very low values for the cutoff ratio, demonstrating that in this type of braided river with fine bed material channel shifting via cutoffs occurs relatively quickly. For comparison: in meandering rivers values of  $\lambda$  in the range of 5 to 30 have been observed (Joglekar, 1970; Klaassen & van Zanten, 1989). Furthermore it is observed that the 'critical' value of  $\lambda$  varies



between a wide range, hence the usefulness of Figure 11 for predictive purposes is very limited. For the time being a 50% value of 1.25 can be used.

In Fig. 11 also the frequency distribution of potential cutoff ratio's of existing bends is added, and it is observed that larger (potential) values of  $\lambda$  apparently result almost immediately into a cutoff.

### (3) Outerbend channels in bends

Also the formation of what is called here an outer bend overflow channel occurs frequently (9 times out of the 23 cases studied). Also this may be related to the shear stress distribution in the bend (Masselink, 1989). Often such an outer bend overflow channel develops where previously an old channel has been present. See Fig. 12 for an example of the development of an outer bend channel, where a fairly rapid shift took place. The new channel started as an outerbend overflow channel and was located in the floodplain. Within two years (1979-1981) the overflow channel deepened from 7 to 17 metres, and the original channel had been filled in completely. The original channel is not always filled in completely, but may still convey a certain amount of discharge.

The previously discussed types of channel shifts were rather simple in that a clear cause and effect relationship could be determined: e.g. a sand bar blocks a channel, and consequently the channel is abandoned. Many channel shifts occur without such an obvious reason. Usually sand bar formation enhances a channel shift, but it is not the direct cause. It is proposed that changes in flow and sediment distribution cause local bed aggradation in one channel. Consequently the flow efficiency of this channel is reduced through a decrease in channel slope. Sand bar formation may occur. Another channel may take over the function of the aggrading channel because of its higher efficiency. All depends on the complex interaction between channel morphology, discharge and sediment load. This relationship is too complex to be studied on the basis of LANDSAT images alone. Complex channel shifts are a fairly important process in the Jamuna River, the reasons why such shifts take place however, are not fully understood.

## 5 PROCESSES AT BIFURCATIONS

When a major channel continues in two smaller channels, a bifurcation is present. A bifurcation is a very important feature in a braided river system, as the development in time of the two bifurcating channels is determined by the flow and sediment distribution at the bifurcation. Here a description is given of the processes that are active at a bifurcation. The two channels arising from the bifurcation are

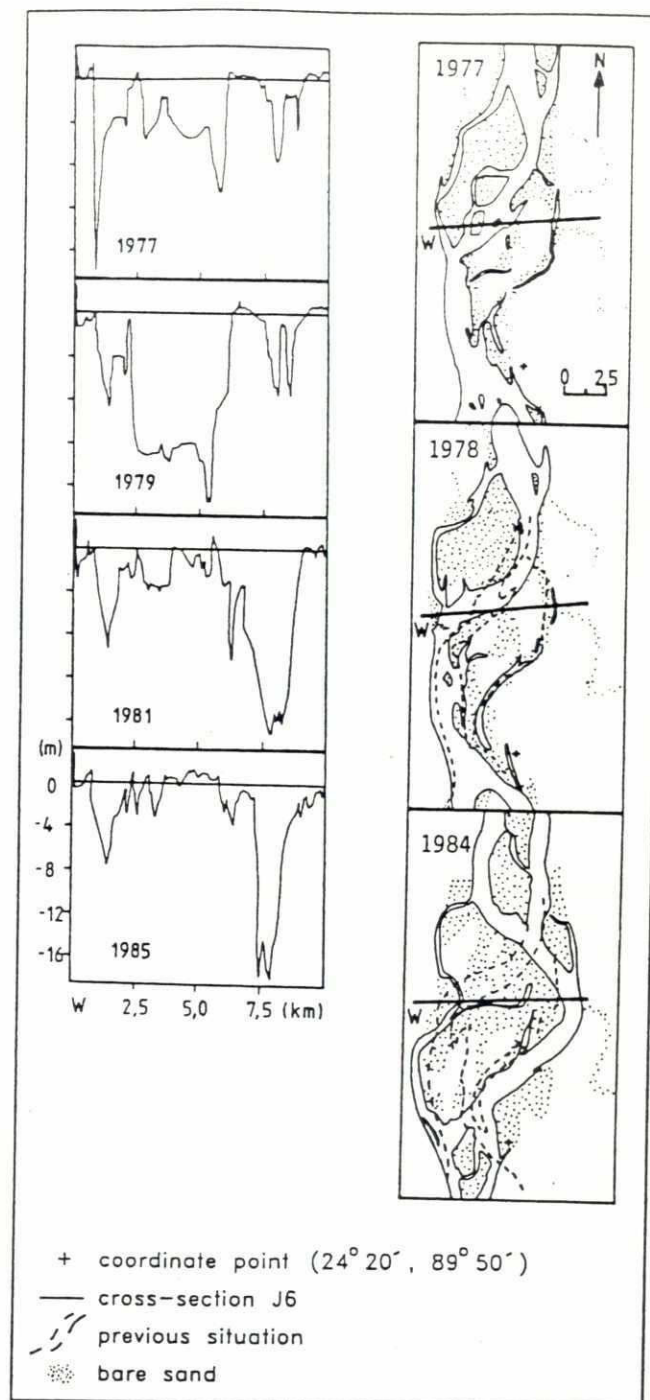


Fig.12 Development of an outer bend channel into an already existing old channel

separated from each other by a char; the surface of this char is above bankful discharge level.

The development of the bifurcation depends on the character of the planform; it may be a symmetric or an a-symmetric one. A symmetric bifurcation is characterized by the flow direction of the upstream channel being different from the two downstream channels. A symmetric bifurcation is characterized by upstream accretion. An analysis of several large bifurcations revealed an average propagation speed (in upstream direction) of approximately 900 m/year.



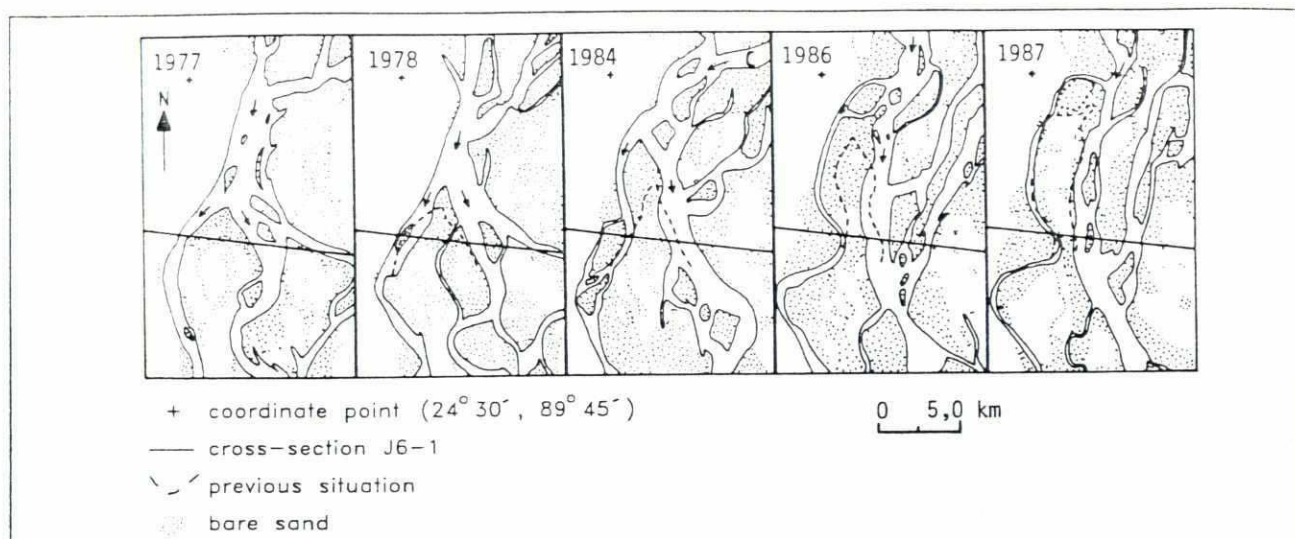


Fig. 13 Development of a bifurcation during the period 1977-1987

(in upstream direction) of approximately 900 m/year. It appears that the propagation rate scales with the size of the channels.

An a-symmetric bifurcation is characterized by one of the downstream channels having approximately the same direction as the upstream channel. The other downstream channel is usually much smaller in size, and because of blocking of this smaller channel by chars, it usually disappears in one or two years.

Fig. 13 outlines the development of a bifurcation in the period 1977-1987. The behaviour is characterized by a symmetric and an a-symmetric phase. In the period 1977-1984 a symmetric bifurcation is observed which progrades in an upstream direction at a rate of about 625 m/year. Between 1984 and 1986 a major channel upstream of the bifurcation (marked with a c on the drawing for 1984) is abandoned. The result is an asymmetric bifurcation where the eastern bifurcation channel is transporting most of the discharge. A small char is formed, blocking the entrance of the western channel. In 1987 the small char is attached to the major char and the bifurcation point has disappeared.

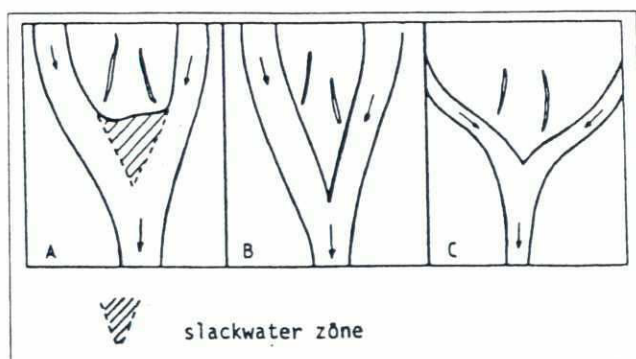


Fig. 14 Different types of confluences

## 6 PROCESSES AT CONFLUENCES

The development of a confluence appears to depend on its shape. Different shapes of confluences are schematically drawn in Figure 14. They are different in the presence of a slackwater zone (A versus B and C) and in the angle between the two confluent channels (B versus C). A slackwater zone is prone to quick deposition and usually vanishes in one year. The channels that drain the char surface have an important role. The char surface slopes in downstream direction so most of these channels discharge near the confluence. If they discharge in the slackwater region all the sediment load is depositing there, giving a significant contribution to the downstream accretion. If the confluence is a smooth one, all the sediment will be discharged in the main channel and be transported downstream. In this case these channels may even lead to erosion of the confluence as most of their sediment load is extracted from the downstream part of the bar.

Bristow (1985) mentions downstream accretion is an important mechanism for the growth of medial chars (major bars in main channels). In the present study this was not confirmed for the major confluences studied here. In general if the confluence margin is smooth, and the two confluent channels do not change significantly, confluences are relatively stable river sections. This corresponds to the conditions in the Yellow River, where some of the nodal points are formed by channel confluences (Chien, 1961). If on the other hand one of the channels becomes dominant, the confluence moves in the direction of the minor channel by means of erosion at the major channel side of the confluence, and sedimentation at the minor channel side. On the average, confluence points do not move significantly upstream or downstream.



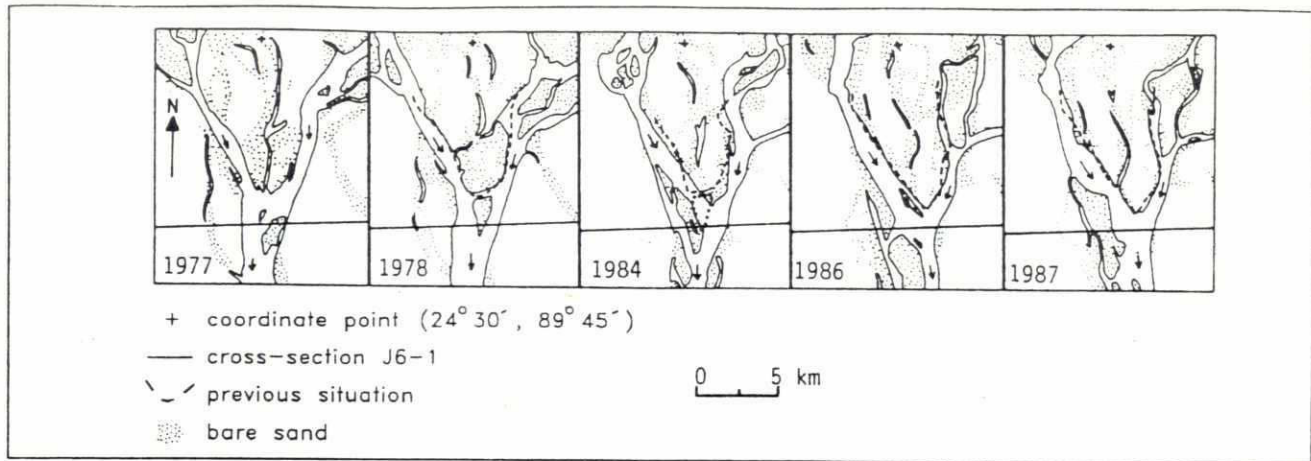


Fig. 15 Development of a confluence in the period 1977-1987

As an example here the historical development of a confluence near Sirajganj over the period 1977-1987 is shown Fig. 15. This confluence had a fairly blunt shape in 1977. In 1978 a bar had been formed by sedimentation in the slackwater region, smoothing the confluence. Over the period 1978-1984 the western channel became more important, and the confluence migrated to the east. In the period 1984-1987 the situation did not change significantly, and over the whole period the position of the confluence is remarkable stable. Old maps, however, show that the confluence was not present at this particular location in the 19th century. Hence, this so-called nodal point (Coleman, 1969) is probably more determined by the presence of a confluence in the last decades than by other possible causes, like larger resistance to erosion due to deposits of a former river (Coleman, 1969).

## 7 TOWARDS A PREDICTIVE MODEL?

The ultimate objective of this study was to investigate whether the development of a predictive model for braided rivers with fine bed and bank material is feasible. With such a deterministic model one should be able to predict future channel development and related bank erosion with reasonable accuracy and preferably on a time scale of at least several years.

From the results of the study reported here it can be concluded that presently it is not possible to develop such a deterministic model, for at least two reasons:

- (1) the limited understanding of the prevailing processes, and
- (2) the observed chaotic behaviour of the system.

These two reasons are discussed hereafter.

### Re (1) Limited understanding

Even for the limiting assumptions made here (e.g. in the analysis of the bank erosion rates that the channel should not 'deteriorate' too much over the period of

comparison), the resulting relationships exhibit a large scatter. This is a.o. caused by not including all relevant features in the analysis. Some examples of this are:

- (1) The analysis was generally based on the LANDSAT images from which only two-dimensional features can be studied. The BWDB cross-sections (taken only every 4 km) were only of limited help. Especially channel shifts are strongly influenced by the surface morphology. Relatively low parts of the floodplain are preferred locations for new channels, and generally such locations cannot be determined from LANDSAT images.
- (2) The studied images form a discontinuous series (1976 (only partly usable, because of cloud cover), 1977, 1978, 1984, 1986, 1987). Consequently the development of a bend or a secondary channel could only be "followed" for two successive years. After the 'gap' that then followed, usually the bend or secondary channel did not exist anymore.
- (3) The shape of the discharge hydrograph was not yet taken into account in the analysis of e.g. the bank erosion rates. The same holds for channel shifts: Klaassen & van Zanten (1989) have shown theoretically that the magnitude of the flood and its duration are important parameters in the actual occurrence of a cutoff.

### Re (2) Chaotic behaviour

Morphological processes in braided rivers like the Jamuna River ('choked with sediment', because of the fineness of the bed material and the large sediment loads during floods), appear to be characterized by chaotic behaviour, and this makes it difficult if not impossible to model the morphological processes in a deterministic way. A.o. sand bar induced channel shifting is producing this chaotic behaviour, but also the discharge hydrograph being different for different years adds to the chaotic behaviour.



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The results of the present study imply that for the time being predictions can be made, but only for the conditions one or two years ahead. The only locations for which predictions can be made for a longer period ahead are the nodal points (major confluences), which appeared to be quite stable over decades.

## 8 CONCLUSIONS AND RECOMMENDATIONS

Based on the study reported here, the following conclusions can be drawn:

- (1) The study of the satellite images, in combination with cross-sections has provided an improved understanding of changes in channel pattern of and the related bank erosion along large braided rivers with fine sand as bank and bed material.
- (2) This improved understanding relates especially to the relation between the erosion and the relative curvature ( $R/W$ ), the negligible importance of vegetation on bank stability, the occurrence of channel shifts and the importance of sand bars, the propagation in upstream direction of bifurcation when the two channels are of almost equal importance, and the stability of major confluences.
- (3) Nevertheless, the understanding of these processes is still very limited, and does not allow for the development of a deterministic model for the development of the channel pattern and the prediction of where bank erosion will take place and how large the bank erosion will. Only near confluences prediction over some years may be possible, because usually these reaches are fairly stable.
- (4) The fact that a deterministic model cannot be developed, is not only due to the as yet too limited insight in the processes in these rivers, but probably also to chaotic behaviour.

Based on these conclusions, the following recommendations are made:

- (1) to study the morphological processes in more detail by:
  - including also information over channel dimensions, in particular the relative importance of eroding channels,
  - studying a more continuous series of satellite images,
  - to include other aspects in the analysis like the duration of the yearly hydrograph;
- (2) to carry out a special study into the possible chaotic behaviour of the river system, with the aim to identify this chaotic behaviour and to study its implications for the time span over which a reliable prediction of channel changes and bank erosion can be made.

## 9 ACKNOWLEDGEMENTS

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## LIST OF SYMBOLS

B = bank erosion coefficient  
 C = Chezy coefficient ( $m^{1/2}/s$ )  
 E = yearly erosion (m)  
 R = radius of curvature (m)  
 W = channel width (m)  
 $\lambda$  = cutoff ratio (-)

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## Paper 4.6

KLAASSEN, G.J., E. MOSSELMAN and H. BRÜHL  
(1993), On the prediction of planform changes of braided  
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ON THE PREDICTION OF PLANFORM CHANGES  
IN BRAIDED SAND-BED RIVERS

Gerrit J. Klaassen<sup>1</sup>, Erik Mosselman<sup>2</sup> and Hartmut Brühl<sup>3</sup>

ABSTRACT

Braided sand-bed rivers are characterized by rapid planform changes. The prediction of these changes requires a thorough insight into the morphological processes in this type of rivers. We report an ongoing study in which these morphological processes are studied on the basis of a series of satellite images covering almost all years in the period 1973-1992. The ultimate aim of this study is to develop a model for planform changes in braided sand-bed rivers. Possibilities and limitations of such an approach are discussed and the concept of the model is described and exemplified with an application to a reach of the Jamuna (= Lower Brahmaputra) River. From analyses it has become clear that it is not possible to develop a fully deterministic model. Predictions of planform changes can only be done by including the probability of occurrence of the different potential developments. This is the more so because recent work suggests that some of the individual processes might exhibit chaotic behaviour. Hence, the model under development can be visualized as a gradual transition from a mainly deterministic model for the first year to a fully stochastic model for a more extended period.

INTRODUCTION

One of the projects of the Flood Action Plan (FAP) for Bangladesh is the Bank Protection and River Training Pilot Project (FAP 21/22). Within the framework of this project, which has a duration of 5 years, some test structures (groynes and revetments) have to be constructed along the braided Jamuna River (see Figure 1), and the performance of these structures has to be monitored. Lessons have to be learnt from the experiences obtained during the monitoring and maintenance, ultimately leading to guidelines for the design of bank protection structures along the Jamuna River.

An obvious prerequisite for the testing of these structures is that they will be attacked by the river during the 5 years the project will last. This is not a simple condition to fulfill as the attack of the river is very difficult to predict. A braided sand-bed river like the Jamuna (= Lower Brahmaputra) River in Bangladesh is characterized by frequent and substantial changes in planform during individual floods, causing the locations of attack by bank erosion to shift rapidly over the years (Coleman, 1969; Klaassen & Masselink, 1992). For the planning of bank protection works and for the maintenance of structures like river training works for bridges in general and for the test structures of FAP 21/22 in particular, a good insight is required into future planform changes. Hence prediction methods for planform changes of these rivers are urgently needed.

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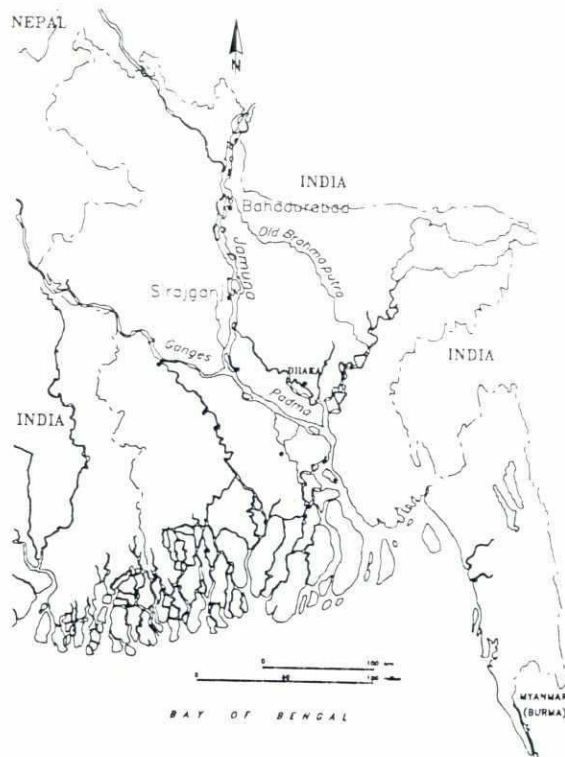


Figure 1. Map of Bangladesh with Jamuna River

The phenomena occurring in a braided river system are much more complex than the ones in meandering rivers. In a meandering river, bank erosion along outer bends and occasional cut-offs are the most prominent features, the channel width often remaining fairly constant along the river and in time. The same processes are active in a braided system, but in addition widening and narrowing are important and cut-offs occur frequently, all processes being very fast in the Jamuna River due to the fine nature of its bed material (almost uniform sand of 0.2 mm).

We report an ongoing project aiming at developing tools for the prediction of planform changes of the Jamuna River. Earlier results are reported by Klaassen & Masselink (1992). The basis for this study is an extensive set of satellite images, obtained during yearly low-flow seasons when there are no clouds and no floodplain inundations, so that the individual channels can be observed clearly. The change-detection images were constructed by combining two consecutive single-year images (see Figure 2). Red, purple and pink represent erosion and pale colours represent accretion. These images allow the identification of bank erosion rates, cut-offs and other morphological phenomena over one flood season. Some examples are shown in Figures 3 and 4. The satellite images are supplemented by soundings and other information on the river system. For more information on the satellite imagery and their use together with other data we refer to Klaassen & Masselink (1992).

The satellite images were used to identify underlying processes and to derive prediction methods for each of them. In a next step we combine these methods in an integrated way to make predictions for the changes in the planform at specific locations along the Jamuna River. The present paper concentrates on this last step and discusses the approach taken, together with its possibilities and limitations. The prediction methods aimed at will form tools for the assessment of morphological changes that occur autonomously as well as in response to, for instance, bank protection works.

For reference we give some characteristics of the Jamuna River. The water level slope decreases gradually from 0.1 to 0.06 m/km. The bed material is fairly uniform and its median grain size ( $D_{50}$ ) varies from 0.25 mm near the Indian border to 0.16 mm near the confluence with the Ganges River. The annual flood is about 60,000 m<sup>3</sup>/s, and during low flow conditions the discharges are between 4,000 and 12,000 m<sup>3</sup>/s. The number of major braids varies between 2 and 3 per cross-section and the total width of the braid belt varies between 5 and 17 km. Flood conditions prevail from June to November, whereas the low-flow season lasts from December to April. For more details on the Jamuna River we refer to the bench-mark paper by Coleman (1969) and to more recent literature (Bristow, 1987; Klaassen & Vermeer, 1988a and 1988b; Klaassen & al, 1988; Klaassen & Masselink, 1992; Thorne, 1993).



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Figure 2. Single-year and change-detection images



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Figure 3. Change-detection image of bend erosion

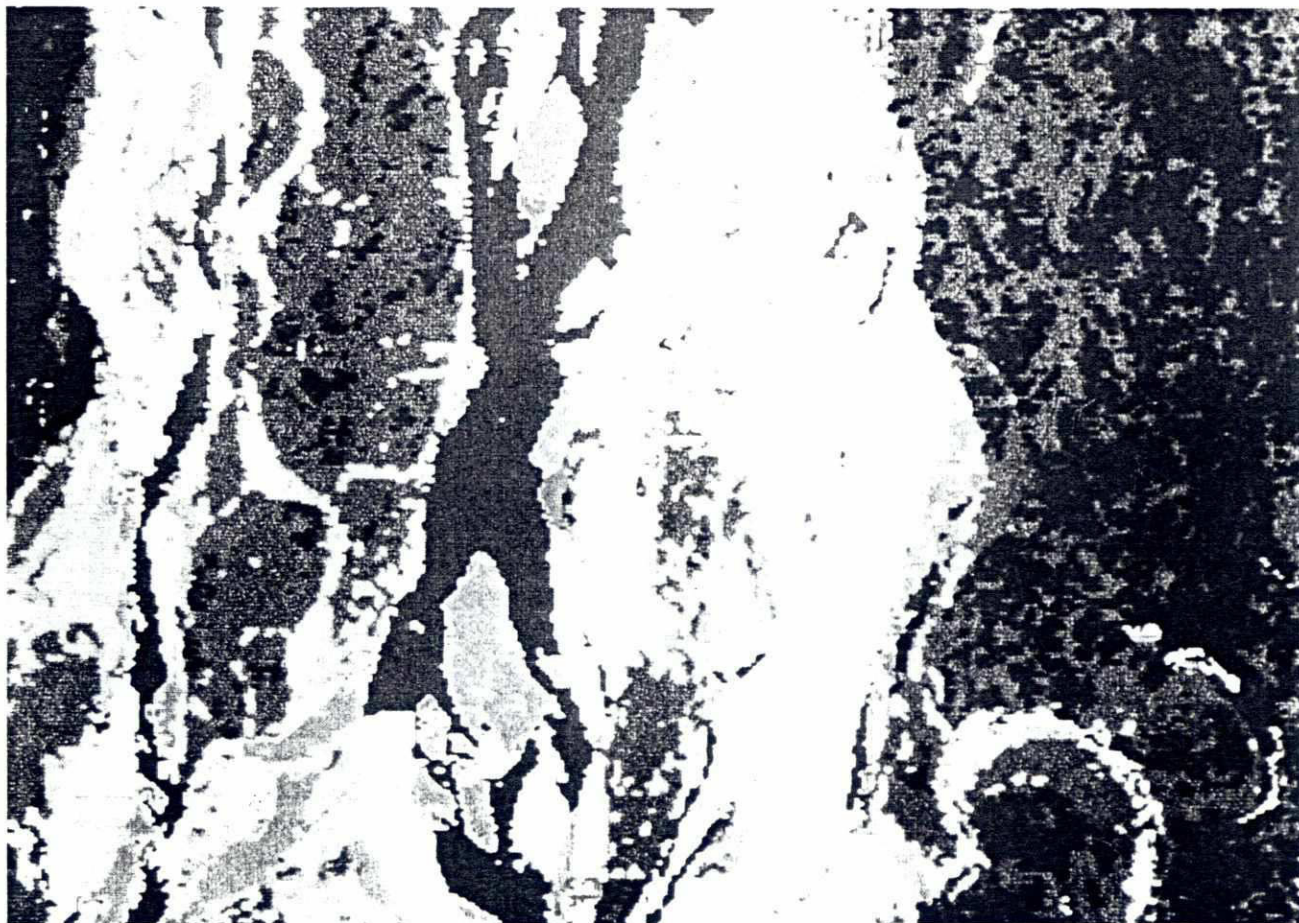


Figure 4. Change-detection image of channel abandonment



The processes in the Jamuna River are so complex, that they sometimes seem almost random. Therefore, it is important to reflect upon the predictability of these processes. The predictions are based on initial conditions, boundary conditions and the formulation of the predictive model. We discuss first how all these factors put limits to the predictability and in the next section how considerations of predictability lead to a choice of the type of model to be used.

The initial conditions are determined from observations, which usually have limited accuracy and do not provide complete information. In particular, the following problems can be mentioned:

- the resolution (pixel size) of the satellite images is on the order of 50 m;
- there are no data on the morphological changes during a flood (only the net end result can be seen on the satellite images taken in the dry season);
- cross-sections are not available for every channel;
- even when a cross-section is available for a certain channel, it is not necessarily representative because of the strong non-uniformity of the channels;
- inundated banks cannot be distinguished from eroded banks on the satellite images (hence, higher stages produce spurious bank erosion on the images).

The boundary conditions include the upstream input of water and sediment, the downstream water levels and the properties of the surrounding terrain. Here, the problems are not only related to shortcomings in observations, such as the limited information on soil inhomogeneities. Problems also arise because the boundary conditions are affected by complex dynamical systems with their own limited predictability, in particular by meteorology and geology.

The meteorological processes determine the discharges. As long-term meteorological predictions are not possible, discharges must be treated as stochastic inputs. The planform changes are very sensitive to the discharges. Magnitude and duration of a flood determine the travel distances of wandering bars during one year, and hence whether or not such a bar arrives at a bifurcation. The arrival at a bifurcation may induce the rapid silting up of one of the bifurcated channels, which would have remained open otherwise. This represents one of the main thresholds in the system. Another influence of meteorology is that, on larger time scales, climate changes may affect the downstream boundary through sea level rise.

Bangladesh lies in one of the most active tectonic zones of the world (Coleman, 1969) and the influence of tectonics on the rivers of Bangladesh is unusually high (Baker, 1986). It is generally recognized that major avulsions are controlled by tectonic movements (e.g. Coleman, 1969), but there are indications that also the channel patterns within the present river course are influenced in this way. This might explain why channels in the river seem to have preferred courses. The influence of tectonics is currently also being studied on the basis of the satellite images.

Predictions covering a long period are also made difficult by the possible occurrence of exceptional events. A major earthquake could completely alter the present-day trends and result in a completely new orientation of the river courses (cf Coleman, 1969).

Theoretically, a feasible predictive model can be derived from fundamental laws for the motion of water and sediment by a formal integration over time and space. This is not a direct integration from a small to a large scale, but a series of integration steps to ever higher levels in a cascade of scales. Due to the non-linearity of the system, each step requires the introduction of empirical closure relationships. This implies that even 'theoretical' models become to a large extent empirical, and hence inaccurate.

Another problem is that the model may be very sensitive to initial conditions, thus producing chaotic behaviour. Phillips (1992a and 1992b) discusses general arguments of why chaotic behaviour is likely to be very common in geomorphic systems. In river morphology this is still a hardly explored subject, but the sensitivity of channel abandonment to discharge as well as the non-periodicity of the meanders in the numerical simulations by Howard & Knutson (1984) and Crosato (1990) do suggest chaos. Furbish (1991) explains explicitly that bend shape variations can be ascribed to the sensitive dependence of meander planforms on initial conditions and entrance conditions.

Considerations on predictability for the period of interest prescribe the type of model to be used. Table 1 gives an overview. We illustrate this by discussing models for planform changes of meandering rivers first. The boundary conditions are the discharges and the properties of the surrounding terrain. The discharge hydrograph can be more or less predictable, for instance in rivers with regular monsoon flood waves, or to a large extent irregular and unpredictable.

Planform changes are very difficult to predict if the period is short. Banks erode by mass failure during discrete events when a critical stability criterion is exceeded. This does not happen uniformly along the whole bank, but each time in small zones only, seemingly at random. This means that the surrounding terrain properties are stochastic boundary conditions and that the only modelling approach possible is a stochastic one.

For intermediate periods and relatively regular discharge hydrographs, a good prediction of river meandering is possible. The modelling can be described as deterministic. However, the sensitivity of the model to initial and boundary conditions and the possible occurrence of cut-offs make that the model should contain stochastic elements as well.

For longer periods, the question whether a certain place at a certain time is attacked is determined by the statistical distribution of water and land over a long period. Hence a statistical model can be applied, in which the spatial distribution of frequencies of water occurrence reflects patterns of soil properties, geology, terrain elevation and local human interference. This approach was used for determining the design conditions for a bridge over the Jamuna River (with an economic life time of 50 years and the river completely changing its planform in 5 to 10 years). The statistical distribution for the period 1973-1992 is shown in Figure 5-a, where colour intensities indicate the number of years in which locations were a part of the deep channels. Figure 5-b represents a superposition of the successive channel planforms from the same period.

Similar considerations hold for the choice of a model for a braided river like the Jamuna River. For intermediate simulation periods (one to three years), however, the greater complexity and the greater importance of thresholds cause the stochastic elements to be more important here than for meandering rivers.

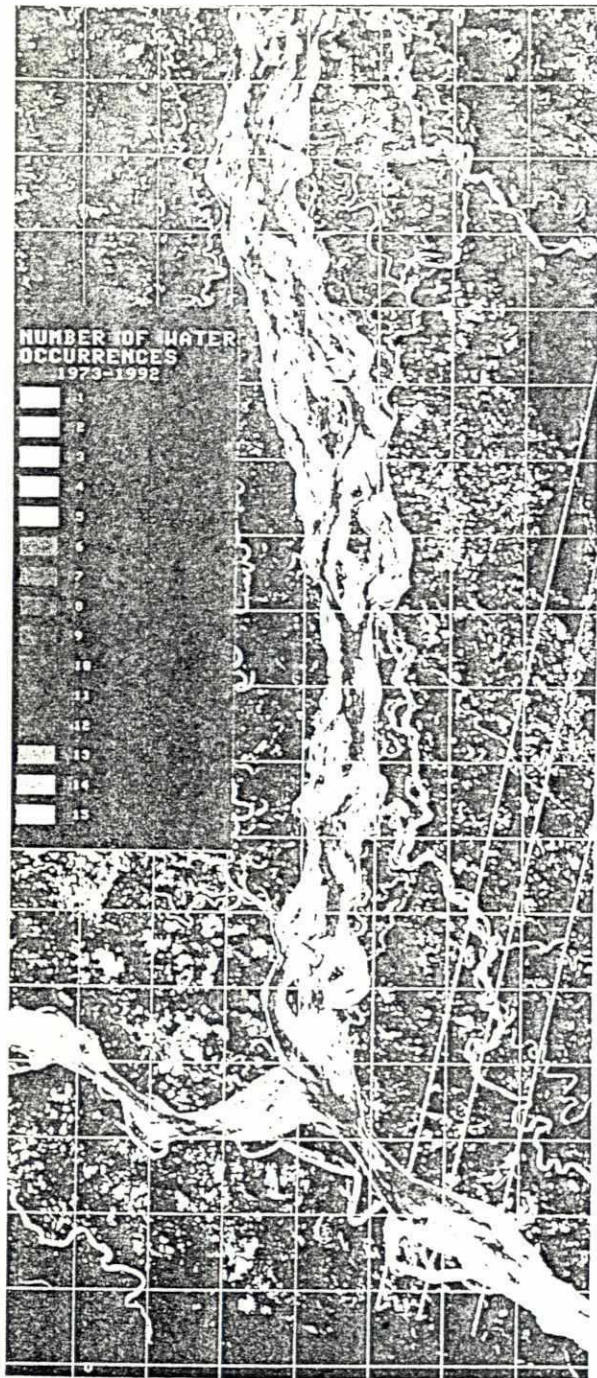
| boundary conditions | model sensitivity to boundary conditions | duration of simulation period relative to characteristic time scale of morphological processes |                                 |               |
|---------------------|------------------------------------------|------------------------------------------------------------------------------------------------|---------------------------------|---------------|
|                     |                                          | short                                                                                          | intermediate                    | long          |
| predictable         | low                                      | deterministic                                                                                  | deterministic                   | deterministic |
|                     | high                                     | deterministic                                                                                  | deterministic / stochastic      | statistical   |
| unpredictable       | low                                      | stochastic                                                                                     | deterministic for time averages | statistical   |
|                     | high                                     | stochastic                                                                                     | deterministic / stochastic      | statistical   |

Table 1. Selection of type of model to be used

## MODELLING CONCEPT

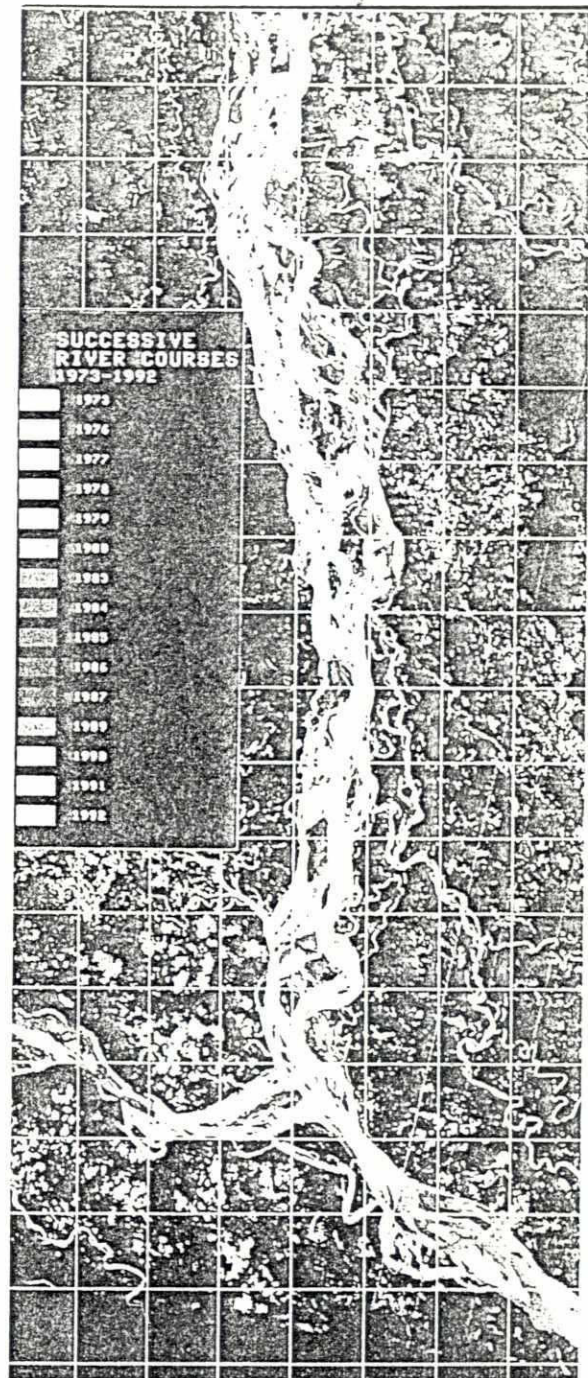
As the derivation of a feasible predictive model for the Jamuna River from fundamental laws would involve so many integration steps that many empirical closure relationships would have to be introduced, we believe that it is better to induce a model directly from the observations. This leads to an approach in which we (1) decompose the overall phenomena into a number of individual processes at a high level of aggregation, (2) investigate the individual processes separately, and (3) integrate the individual processes into a combined predictive tool. The high level of aggregation means that process descriptions represent the complex behaviour of large-scale river elements rather than the elementary behaviour of infinitesimal volumes of water and sediment. The individual processes at a





(a)

Statistical distribution  
of water occurrences



(b)

Successive river courses

Figure 5. Multi-temporal planforms from period 1973-1992





high level of aggregation are channel migration, mid-channel bar formation, behaviour of bifurcated channels (i.e. width adjustment, channel abandonment and channel creation) and the migration of confluences and bifurcations. Some examples are shown in Figure 6.

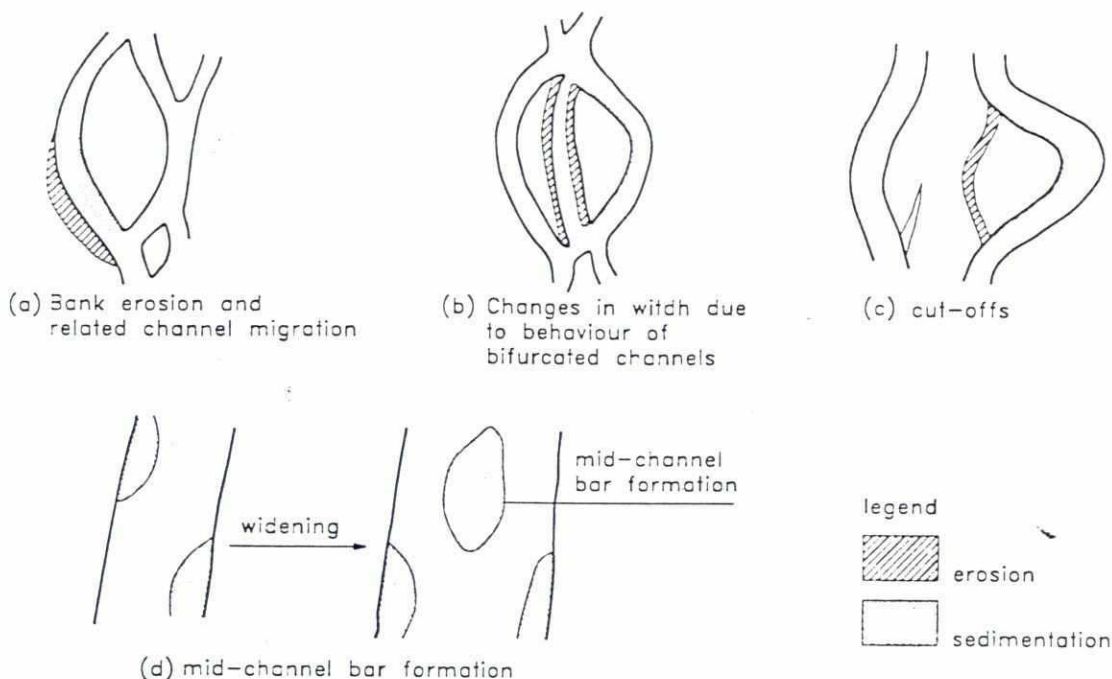


Figure 6. Planform changes in a braided river

We consider two ways of combining submodels for individual processes into a prediction tool, see Figure 7. The first way is already operational and consists of evaluating the combined action of the individual processes by hand according to a formal procedure. We refer to this tool as the 'methodology' and show its application in a later section. The second way consists of combining the submodels in a computer model. The development of the latter just started. We use the general word 'model' for both the methodology and the computer model.

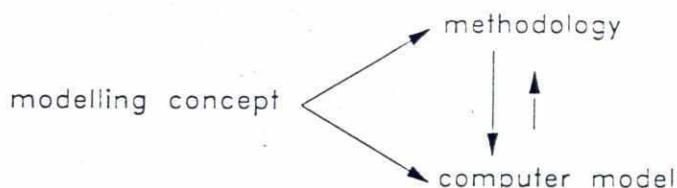


Figure 7. Modelling concept and prediction tools

The methodology and the computer model have stochastic and deterministic elements. The general structure of the computer model will be a Monte Carlo simulation for both stochastic processes (e.g. channel abandonment) and stochastic input data (e.g. discharge hydrograph). The Monte Carlo simulation will not produce a continuous spectrum of possible outcomes, because the system contains certain thresholds between whether events occur or not, for instance the silting up of a channel due to the arrival of a wandering bar at a bifurcation. Hence the Monte Carlo simulation can be represented as a tree of events, similar to fault trees in risk analyses. This is the essence of the procedure in the methodology.

In principle, two types of methods are possible for the description of the processes involved, one related to a bar approach and one related to a channel approach, see Figure 8. In the bar approach, the pattern changes are taken to be determined primarily by changes of a submerged bed topography. In the channel approach, the patterns are schematized as a network of migrating bends, interconnected at bifurcations and confluences that are also migrating. Within this network, new channels can be created and existing channels can be abandoned. We mainly follow a channel approach, but with some elements from the bar approach.



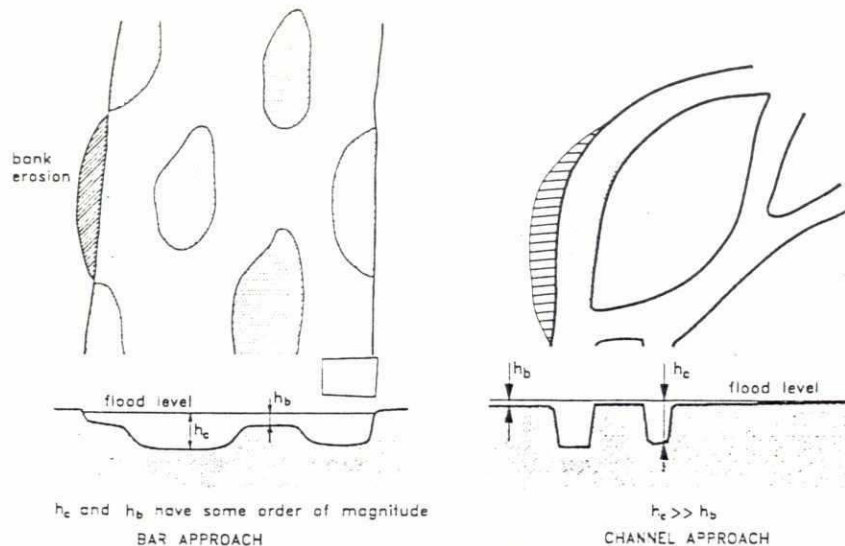


Figure 8. Bar approach and channel approach

## MODEL COMPONENTS

The model components are submodels for the individual processes. Generally, they describe planform changes as a function of geometry and discharge. In addition, the preprocessing and postprocessing of data form an important part of the computer model.

### Channel Migration:

Several mathematical models for river meandering exist. Roughly, they can be subdivided into five categories. Listing them in the order of increasing complexity, these categories are:

- (1) Simple formulas in which bank migration rates are a function of local channel curvature (Hickin & Nanson, 1984);
- (2) Simple kinematic models in which bank migration rates are a function of local and upstream channel curvatures (Ferguson, 1983; Howard, 1983; Howard & Knutson, 1984);
- (3) Meander models based on the equations for water flow and bank migration in curved equiwidth channels (Ikeda et al, 1981);
- (4) Meander models based on the equations for water flow, sediment transport and bank migration in curved equiwidth channels (Johannesson & Parker, 1989; Crosato, 1990);
- (5) Meander models based on the equations for water flow, sediment transport and bank migration in channels with arbitrary geometries (Mosselman, 1992).

Klaassen and Masselink (1992) followed the approach of Category 1 by studying the correlation between bank erosion and local radius of channel curvature. However, a model based on this type of formulas cannot simulate bend migration properly, as was pointed out by Howard & Knutson (1984). We therefore now use the approach of Category 2 by using a linear relation between bank erosion and a weighted average of local and upstream channel curvatures. For that, we re-analyzed the bends in the Jamuna River and revised the relations between bank erosion and curvature. The sophisticated models from Categories 4 and 5 would require too much computation time and are not justified in view of the data available. As Parker (1983) argues that models from Category 2 are equivalent to the model of Ikeda & al (1981) in Category 3, we think that Category 2 indeed serves our purposes the best.

### Mid-channel Bar Formation:

The formation of mid-channel bars is modelled as a function of the width-to-depth ratio of the channel. This is the main bar-approach element in the model. Mid-channel bars are formed in wide and shallow channels and, in their turn, enhance channel widening by further outward deflection of the channels on either side. Struiksma & Klaassen (1988) base a criterion for mid-channel bar formation on the stability of linearized equations for water and sediment motion. A linear prediction of exponential growth, however, does not necessarily imply that the bar will actually reach the water level, because non-linear interactions may prevent growth beyond a finite amplitude. The criterion therefore needs recalibration.

## Behaviour of Bifurcated Channels:

Changes in the relative importance of bifurcated channels are determined basically by the distributions of discharge and sediment transport at the bifurcation. The sediment transport distribution is very difficult to assess, but will strongly depend on the geometry of the bifurcation. Therefore, the changes in the bifurcated channels will be related directly to bifurcation geometry. Channel abandonment and channel creation arise as special cases of the behaviour of bifurcated channels. A new channel appears when a mid-channel bar emerges from the water level or when an avulsion or a cut-off occurs. The latter case can only be simulated with the computer model if the position of a probable new channel has been specified by the user in advance. New channels frequently occur in connection with a drainage channel which was probably formed during the fall of the previous flood, but this is not always the case.

Channel cut-offs are a special type of channel creation, often occurring in combination with simultaneous channel abandonment. Different from what was suggested implicitly by Klaassen & van Zanten (1989) and Klaassen and Masselink (1992), it appears that in the Jamuna River the cut-off ratio (i.e. the ratio between the lengths of the old and the new channel) is less important than the angle between the channel to be abandoned and the channel upstream. Figure 9 shows that the probability of channel abandonment is negligible if this deflection angle is less than 30°, but increases for higher values of the deflection angle. Bifurcation asymmetry is also an important parameter.

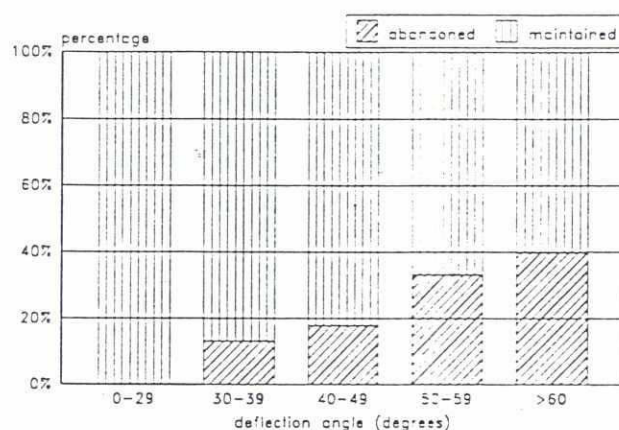


Figure 9. Probability of channel abandonment as a function of deflection angle

## Migration of Confluences and Bifurcations:

For the time being, no separate formulation for the migration of confluences and bifurcations is included. The migration is assumed to result simply from nearby channel migration and the formation and coalescence of mid-channel bars.

## Pre- and Postprocessing:

An important element in the envisaged computer model is the preprocessing and postprocessing of data. The preprocessor must allow that the channel network is derived directly from the digital satellite image. For this, we will use the skeleton algorithm of Hilditch (1969), also used for rivers by Moll and Overmars (1990). The postprocessor must produce a map in which colour intensities indicate the probability that a pixel becomes water. As a matter of course, this will also provide the probabilities of bank erosion, bend cut-off, etc.

## SAMPLE APPLICATION OF METHODOLOGY

To demonstrate the potentials and the limitations of the modelling approach, some results of its application to a reach of the Jamuna River are shown here. The reach is located upstream of the confluence with the Ganges River. It is slightly untypical for the Jamuna River in that the number of braids per cross-section, and hence the braiding intensity, is less compared with the more upstream reaches. This is the very reason why we selected this reach here, because it serves as a clearer example than some of the other reaches where the methodology was applied. Some results of the application of the methodology are shown in Figure 10.



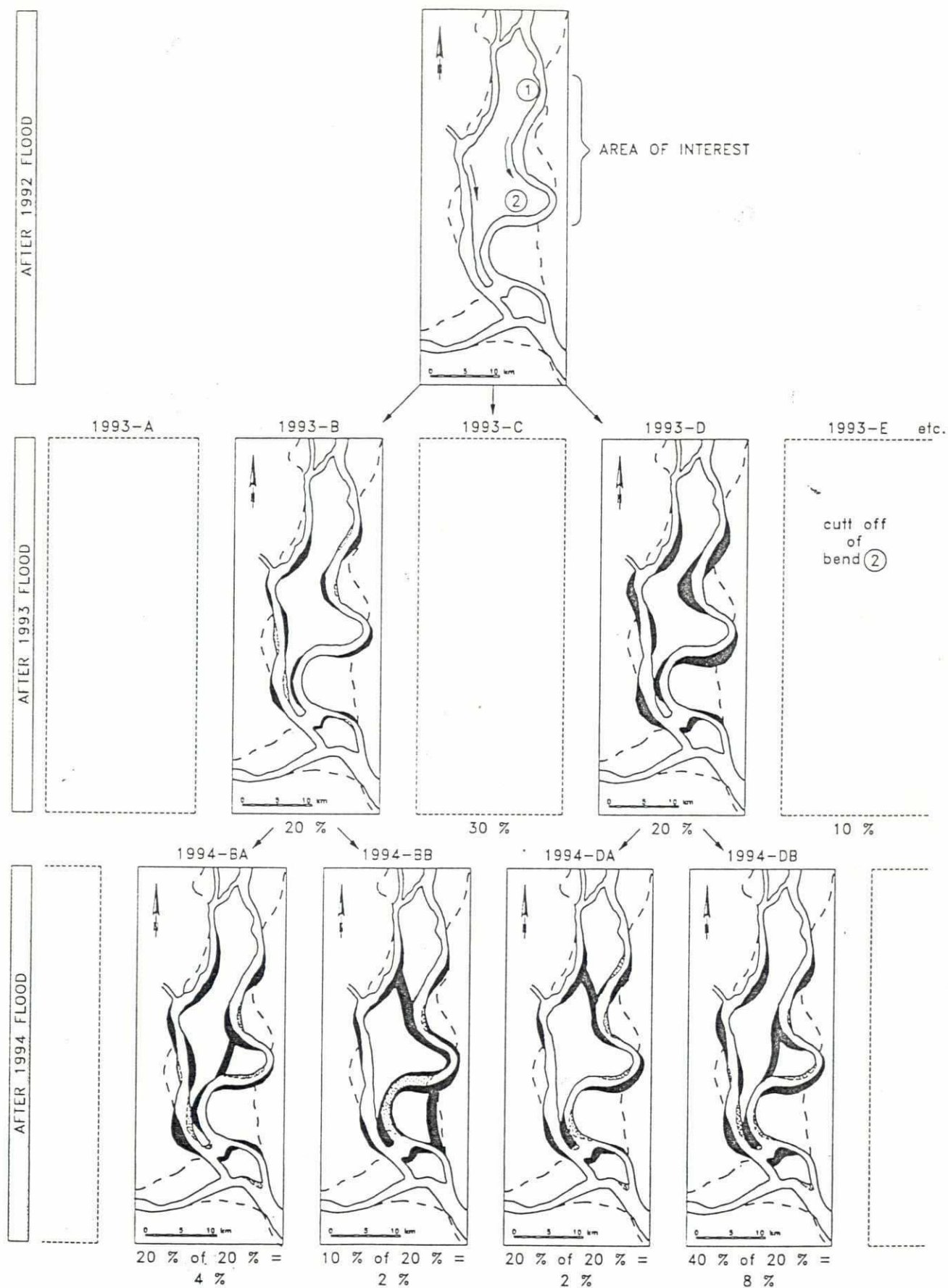


Figure 10. Sample application of methodology

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The area of interest is located along the left bank of the river. Two bends are present, marked as 1 and 2. The question to be answered is: what is the probability that the bank in bend 2 will still erode in the 1995 flood season and afterwards? Figure 10 shows that several developments can take place. During the 1993 flood season, the bank erosion along the various bends may be minor or major, depending on the magnitude and the duration of the flood. Note that the two developments shown correspond to only 40% of the possible developments. The probability that a cut-off of bend 1 occurs is certainly not negligible. Each of the possible planforms in 1993 will develop further in 1994. A cut-off conform the development 1994-DB becomes more and more probable. This will lead to a reduction of the bank erosion along bend 2. Note that also development 1994-BB leads to such a reduction. The overall probability that the bank erosion along bend 2 will stop after 1994 is at least  $2 + 8 = 10\%$ , but many other developments (not shown in the figure) may also result in such a reduction. We estimate that the probability of no further erosion in 1995 is about 50%. Note also that for all developments foreseen in Figure 10, the erosion along bend 1 will continue. From the two potential sites for test structures a location along bend 1 is to be preferred.

The number of possible developments obviously increases with the number of years considered, which makes the analysis complicated and time-consuming. This demonstrates the need for a computer model.

## CONCLUDING REMARKS

In this paper we have outlined a modelling concept for the prediction of planform changes in braided sand-bed rivers. Because of the stochastic approach necessarily taken here, the possible developments can only be expressed in terms of probability. Initially the future developments can be predicted with a fairly high accuracy, but when the ratio of simulation period to typical time scale of the morphological changes becomes larger, the number of possible developments increases and the accuracy of the prediction of the actual development decreases.

The components of the model were mainly derived from the analysis of a 20 year long series of satellite images of the Jamuna River during low flow conditions. Essentially these satellite images allow only a two-dimensional picture, whereas the actual developments are definitely affected by three-dimensional phenomena (like spiral flow, bar heights, etc.). A further refinement would be possible if the three-dimensional bed topography could be used. In future, radar images from satellites can help in providing such a three-dimensional picture by also yielding the extent of the flooding during the flood season when clouds prevent observation with optical and infrared sensors.

One particular aspect of the Jamuna River is that, due to the fine nature of the bed and bank sediments, morphological changes are also substantial during lower stages. More insight in the response of the river to these lower stages is needed to improve the prediction techniques for the various model components.

Furthermore, it should be stressed that until now the methodology could only be tested over the most recent (1992) flood. As the prediction covers a number of years, it is required to obtain experience by comparing predicted and actually observed planform changes in the coming years too. This also raises the point of 'verifying' or 'falsifying' the modelling concept. This can only be done in a statistical sense. Methods for verification are currently being developed.

In long-term perspective, we consider the present development of prediction tools as a first step towards an early warning system for undesired planform changes during a flood. Such a system would be based on numerical predictive models, discharge measurements, soundings and radar remote sensing, linked together through data assimilation. Radar penetrates clouds and can sense bed topography patterns up to tens of meters deep (cf Vogelzang & al, 1992).

## ACKNOWLEDGEMENTS

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The processing of the satellite data was carried out by NLR (National Aerospace Laboratory, The Netherlands). We acknowledge Messrs H.H.S. Noorbergen and W. Verhoef in this respect. Special thanks are due to Mr. E. Koomen and Ms. M. Huisink who did most of the geomorphological interpretation of the satellite images as trainees from the Free University of Amsterdam.

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## Paper 4.7

MOSSELMAN, E., M. HUISINK, E. KOOMEN AND  
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## MORPHOLOGICAL CHANGES IN A LARGE BRAIDED SAND-BED RIVER

by

E. Mosselman<sup>1</sup>, M. Huisink<sup>2</sup>, E. Koomen<sup>2</sup> and A.C. Seymonsbergen<sup>3</sup>

### ABSTRACT

Processes governing morphological changes in the braided Lower Brahmaputra river have been studied. We present results on width adjustment, channel abandonment, bend migration, the influence of tectonics and correlations with the magnitude of the annual flood.

### INTRODUCTION

The Brahmaputra is a large braided sand-bed river that flows through India and Bangladesh. The total width of its braid belt varies between 5 and 17 km and individual channels are up to 2 km wide. The annual flood is about 60,000 m<sup>3</sup>/s. Scour depths can reach 40 m and bank erosion rates can locally be as high as 1000 m/year. The associated rapid planform changes are very complex and include phenomena like bend migration, formation and propagation of bars, channel creation, channel abandonment and migration of confluences and bifurcations. Previous studies of these processes are reported by Coleman (1969), Bristow (1987), Klaassen & Vermeer (1988a and 1988b) and Klaassen & Masselink (1992). General considerations on predictability and model concepts are discussed by Klaassen & al (1993). Here we present the main results from our studies in 1992. For these studies we used daily discharges from Bahadurabad station, cross-sectional data and an extensive set of satellite images from Landsat MSS, Landsat TM and MOS-MESSR sensors of the Lower Brahmaputra in Bangladesh. The images cover almost every year from 1973 to 1992. Noorbergen (1993) describes the processing of these images, which includes the production of multitemporal change-detection images.

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## RELATION BETWEEN BANK EROSION AND DISCHARGE

The discharge through a particular channel within the braid belt can be estimated from the total discharge by assuming that the discharges are distributed over the channels in proportion to the conveyance of each channel. Cross-sectional data, however, were not available for every channel, and even when a cross-section was available, it was not necessarily representative because of the strong non-uniformity of the channels. Indeed, attempts to correlate morphological changes with discharges thus obtained yielded poor results. We then tried to find a correlation between an overall measure of planform change and a parameter characterizing magnitude and duration of the total annual flood. Figure 1 shows the relation between the total area of eroded land along 14 major bends and the fourth moment of the discharge record,  $\alpha_4$ , defined as

$$\alpha_4 = \frac{1}{T} \int_0^T \{Q(t)\}^4 dt$$

in which  $Q(t)$  denotes discharge as a function of time and  $T$  represents one year. Discharges were truncated at the bankfull value of 44,000 m<sup>3</sup>/s. Here a weak trend that the amount of bank erosion increases as discharge increases becomes visible. The trend is less clear for lower moments of the discharge record with an exponent of, for instance, 1 or 2.

Inundated banks cannot be distinguished from eroded banks on the satellite images, so that higher stages produce spurious bank erosion on the images. The arrows in Figure 1 indicate in which direction the points would move if a correction for this spurious erosion could be made. It must be noted, however, that spurious bank erosion is less significant than spurious bank accretion because the eroding banks are usually steep.

## WIDTH ADJUSTMENT

The hydraulic geometry of individual channels changes continuously due to changes in the supply of water and sediment from upstream. This supply depends on the distribution of water and sediment at upstream bifurcations. The distribution of discharges over the bifurcated channels can be calculated easily when the geometry of the channels is known, but the distribution of sediment transport is very difficult to assess. As this distribution depends on the geometry of the bifurcation, we tried to find a correlation between width adjustment and upstream bifurcation geometry. The latter was represented by the deflection angle, i.e. the angle between a bifurcated channel and the channel upstream, and by parameters expressing the degree of bifurcation asymmetry. We could not find a clear correlation. As an example, we show the relation between width adjustment and deflection angle in Figure 2. Width adjustment appears here more or less like a random process.

## CHANNEL ABANDONMENT

Channels are frequently abandoned as a result of chute cut-offs and channel avulsions. We tried to correlate channel abandonment with the key parameters of the model for bend cut-offs by Klaassen & van Zanten (1989) as well as with the bifurcation geometry parameters used in the study on width adjustment. Correlations with the latter group of parameters gave the best results. The relation between the frequency of channel abandonment and the deflection angle is shown in Figure 3. Comparing the clear trend in this figure with the randomness in Figure 2, we conclude that channels are abandoned through shallowing rather than narrowing.



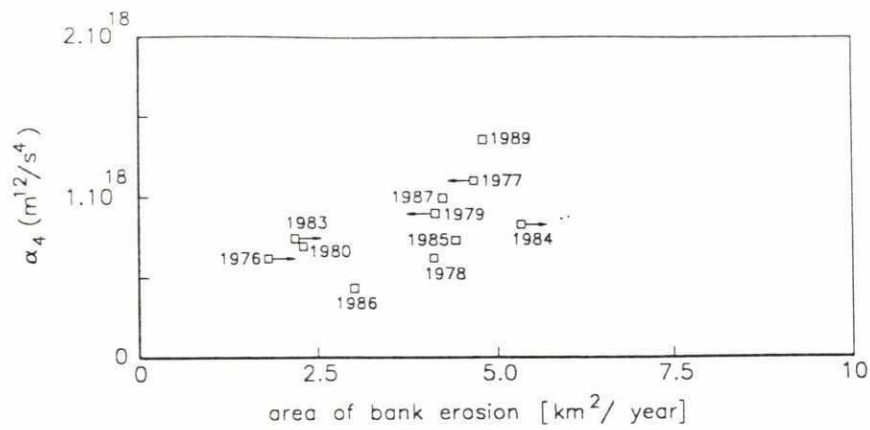


Figure 1: Relation between erosion area and fourth moment of flood hydrograph.

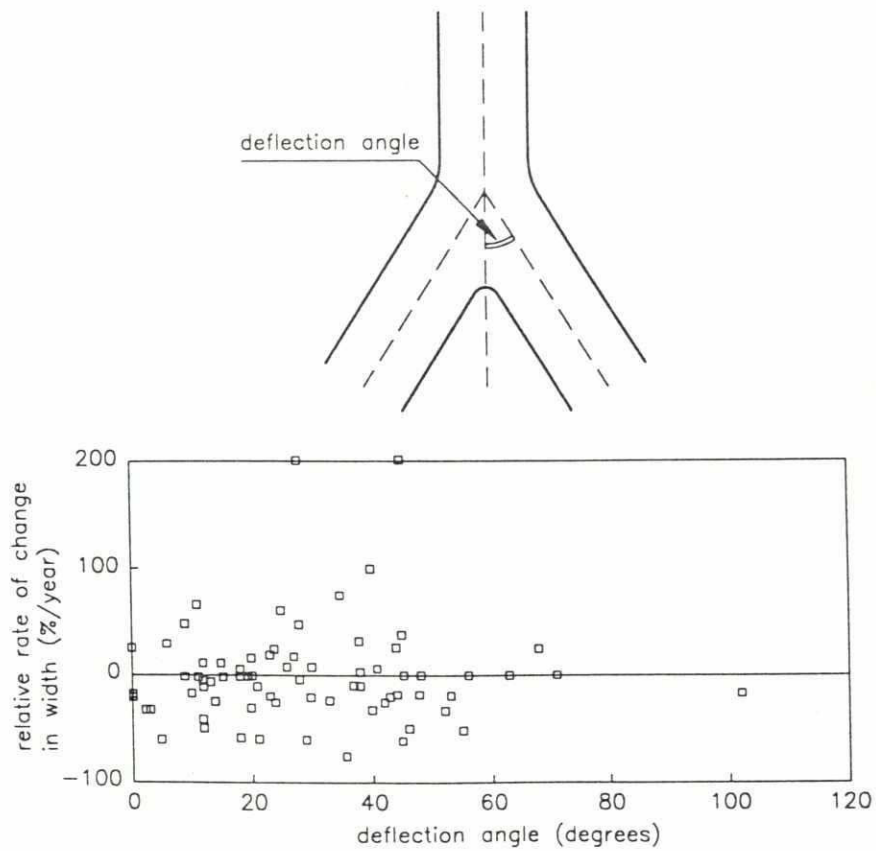


Figure 2: Relation between width adjustment and deflection angle.

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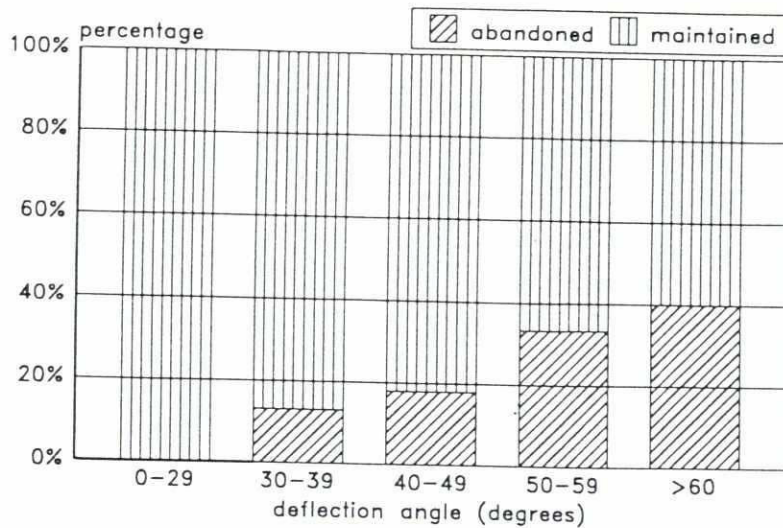


Figure 3: Relation between probability of channel abandonment and deflection angle.

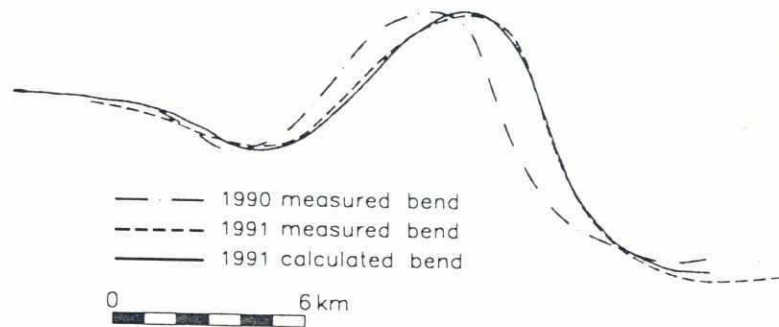


Figure 4: Comparison between observed and simulated channel migration for a large bend around latitude  $24^{\circ} 0'$ .

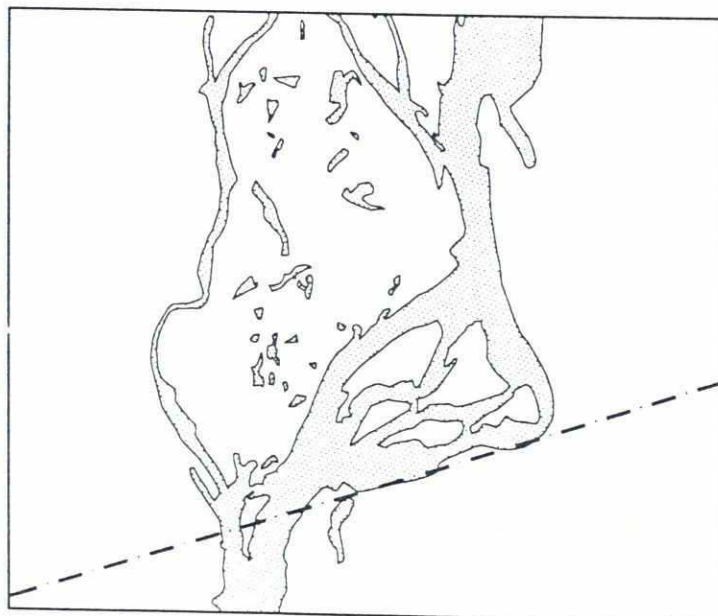


Figure 5: Inhibition of channel migration at a fault by tectonic block activity.



## BEND MIGRATION

Klaassen & Masselink (1992) studied bend migration by correlating bank erosion rates with local channel curvatures. We attempted to improve this by simulating the migration of a few bends with a simple kinematic meander model, in which bend migration is a function of local and upstream channel curvatures (cf Howard & Knutson, 1984) and also a function of the absolute value of the sine of the angle between channel centre-line and direction of maximum valley slope. The latter dependence was assumed to represent the downvalley migration due to cross-channel flow at high stages in analogy to the downstream propagation of a dredged trench. Figure 4 shows the results for the migration of a large bend around latitude  $24^{\circ} 0'$ , close to the confluence with the Ganges river. The characteristic length scale in the convolutional relation between migration and upstream channel curvatures was 10 km. Computed planform changes appeared to be more sensitive to errors in the position of straight reaches than to errors in curved reaches.

## INFLUENCE OF TECTONICS

The Lower Brahmaputra river lies in one of the most active tectonic zones of the world. The tectonic activity is related to the collision between the Indian and Eurasian plates (Molnar & Tapponier, 1977), the subsidence in this zone being the direct counterpart of uplift in the Himalayan belt. The activity has fractured the terrain into various geological compartments or blocks, separated by vertical fault planes. We identified these faults by an analysis of lineaments on the satellite images. The major faults are presented in Figure 6. Details are reported by Hartmann & al (1993).

There are indications that the channel pattern within the Lower Brahmaputra is influenced by differential uplift or subsidence and tilting of the geological compartments. Firstly, the channels are sometimes remarkably straight in directions parallel to the faults. Secondly, changes in planform are sometimes difficult to explain from an autonomous motion of water and sediment. An example is shown in Figure 5, where the downstream translation of a bend at latitude  $24^{\circ} 10'$  seems to be inhibited between 1986 and 1987 by an apparent rectilinear obstacle complying with one of the faults we identified in both the braid belt and the surrounding terrain. Field evidence of vertical terrain motions is needed to confirm these findings.

## CONCLUDING REMARKS

We studied the planform changes in the braided Lower Brahmaputra river by using satellite images, discharge records and cross-sectional data. Some conclusions can be drawn, but the phenomena are so convoluted that it is difficult to extract information on individual underlying mechanisms from observations only. It is necessary to bring descriptions of the underlying mechanisms together in a model to see whether their combined behaviour can reproduce the observed morphological changes. We started the development of a probabilistic predictive model. Its general concepts are described by Klaassen & al (1993).

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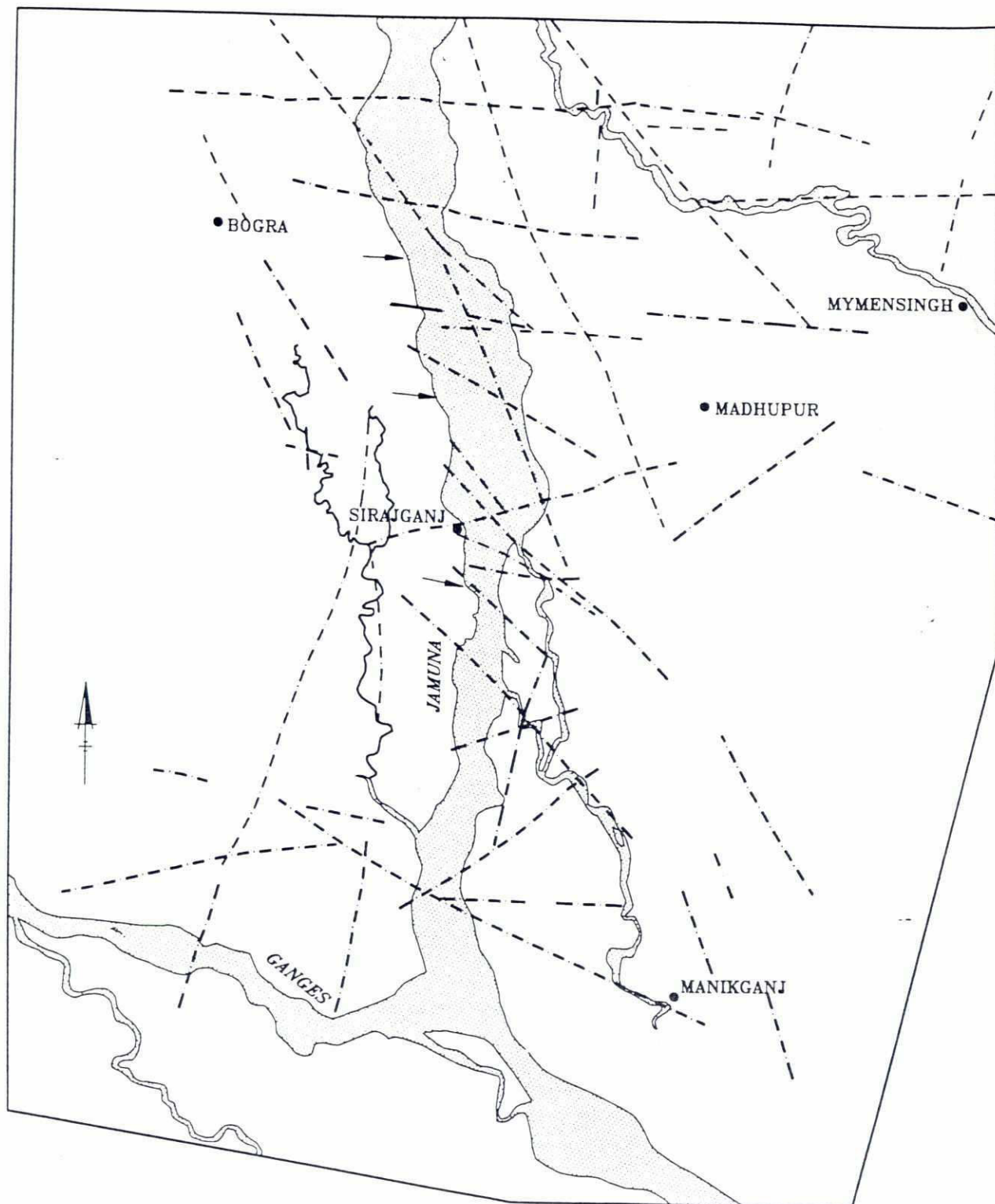


Figure 6: Major faults in the Bengal Basin as inferred from an analysis of lineaments.



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processes and phenomena, Edinburgh, U.K.*



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## GEOMORPHIC STUDY OF BANKLINE MOVEMENT OF THE BRAHMAPUTRA RIVER IN BANGLADESH

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### INTRODUCTION

#### The Brahmaputra River

Three great rivers drain through Bangladesh and have been responsible for building the great alluvial fan which forms the landscape (Figure 1). The Ganges and Brahmaputra are much the larger, both ranking in the top ten rivers in the world. The Meghna is much smaller and is the only one of the three to rise within Bangladesh's own borders.

The Brahmaputra rises on the northern flanks of the Himalayas and flows almost due west through China as the Tsangpo, before turning south and entering India. It then flows southwest through Assam as the Brahmaputra before turning almost due south as it enters Bangladesh. Until the late eighteenth century the curve of the river continued, to produce a southeasterly course that took the Brahmaputra River to the east of the Madipur Tract and into the Sylhet Basin. Here the Brahmaputra joined the much smaller Meghna River and the combined flow entered the Bay of Bengal along the present course of the Meghna Estuary.

This drainage pattern is clear on Major Rennel's map of around 1780, but by the time of the Wilcox map of 1830 the situation had changed drastically. Sometime in the intervening period, the Brahmaputra broke through the higher ground along its southern flank to shift sixty kilometres and adopt a new course to the west of the Madipur Tract. In Bangladesh the reach of river so created is called the Jamuna; the name Brahmaputra being retained by the former course, which has dwindled to become little more than a high flow distributary. There is evidence that this change did not occur catastrophically, but took place gradually over a period of perhaps thirty years.

As a result of this shift the Brahmaputra formed a new confluence with the Ganges, creating a very large river called the Padma. For a few years the Padma followed the old course of the Ganges, along the course of the present Arial Khan, but soon the great increase in the volume of flow led to a last major avulsive shift when the Padma broke through a neck of higher terrace land separating it from the Meghna. For the first time the three large rivers met near Chandpur, the situation which still prevails today.

When considering the present hydrology and geomorphology of this great river it is important to bear in mind its dynamic nature over both the geological time-scale and the recent past.

#### Characteristics of the Brahmaputra River

##### Hydrology and Geomorphology

With an overall length of nearly 3,000km and a drainage area of around 580,000km<sup>2</sup>, the Brahmaputra is one of the world's greatest rivers. The average discharge of around 20,000



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cumecs ranks the river as the fourth largest freshwater flow known to humankind. The annual hydrograph of the river is characterised by high flows during the summer and very low flows during the winter. Summer high flows are driven by a combination of snowmelt runoff from the Himalayas together with local monsoon rainfall particularly in the Assam valley and within Bangladesh itself. Annual flood peaks are of the order of 60,000 cumecs.

The basin is bounded by the highest and most tectonically active range of mountains in the world, the Himalayas. The Himalayas are being uplifted by compression of the Earth's crust due to collision between the Indian and Asian tectonic plates and the young alpine mountains so produced are naturally subject to severe erosion. As a result, the Brahmaputra carries a very heavy sediment load of sand and silt, estimated to be of the order of 500 million tonnes per year on average. This ranks the Brahmaputra third in the world in terms of sediment load and first in terms of sediment concentration. The combination of large discharges of water and sediment gives the Brahmaputra a braided pattern of multiple, shifting anabranches separated by braid bars, called locally 'chars'. Braided rivers are characterised by unstable banklines and rapid rates of lateral movement, and the planform of the Brahmaputra changes annually in response to each year's sequence of flows.

The geomorphology of braided rivers is much less well understood than that of single-thread, meandering rivers, although some general morphological principles are believed to govern the relationship of channel form with fluvial and sedimentary processes. For instance, hallmark work published by Leopold and Wolman in 1957 established the basic mechanics of braiding. This begins by the deposition of a medial bar which deflects the flow to each side of it to produce erosional embayments in each bank. Flow diverges as it approaches a medial bar and then converges towards the tail of the bar. As the embayments grow through bank retreat, the increasing width creates space for the medial bar to grow, while the sediment input from bank erosion supplies the sediment necessary to fuel continued bar building and so produce clusters of bars that eventually merge to form a larger and more permanent island clusters. At the downstream end of an island cluster, flow converges, scouring the bed to produce a deeper, narrower channel. In time a mature braided pattern develops, with longer, wider reaches containing substantial islands and multiple anabranches, separated by shorter, narrower sections where the flow converges to form a node or crossing.

The bankfull capacity of the braided channel of the Brahmaputra is around 55,000 cumecs, so that summer flooding of low lying wash lands is an almost annual event along this river, as it is for most of Bangladesh's rivers. As a result, in an average year, between 20% and 30% of Bangladesh is flooded. Not only is flooding on this scale not a problem to the population, but it is a vital part of their agriculture and food production. Annual summer floods fertilize the land through silt deposition, recharge soil moisture and shallow aquifers needed for winter (dry season) irrigation, and support fisheries that supply much needed protein for a balanced diet. However, much severe floods occur occasionally, with almost entirely negative impacts.

In 1987 and 1988 consecutive floods on the Brahmaputra and other large rivers inundated 40% and 60% of Bangladesh, respectively. In 1988 the Brahmaputra peaked at over 90,000 cumecs, a flow with a return period close to 100 years. Although these floods are not killers on the scale of the coastal floods associated with cyclones like that of 1991, such deep, widespread flooding displaces millions of people, destroys standing crops, and washes away vital bridges and infrastructure that are irreplaceable without major injections of foreign aid. These major floods also cause serious bank erosion that threatens flood embankments, wipes out villages and bazaars, and permanently destroys peoples' farms and dwellings to deprive them of both their homes and their livelihoods.

Despite reports in the popular press to the contrary, there is actually no quantitative evidence that either the magnitude or frequency of severe flooding has worsened in recent years, although the damage inflicted by such floods on the population is certainly increasing. This is the case for three reasons: first, because of Bangladesh's growing population (now over 110 million in a nation the size of Wales); second, because of development of flood plain

England and



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areas through more intensive agriculture, the construction of new buildings, roads, drainage schemes and bridges; and third, because in the case of the Brahmaputra, widening and possible migration of the river channel are leading to more widespread flooding due to breaching of the Brahmaputra Right Embankment (BRE). Each of these dimensions to the impact of floods is reviewed briefly in turn.

### People and Water Resources

In 1981 the last census for which data are available placed the population of Bangladesh at 89.6 million. Early estimates based on the last census (1991) suggest that this figure has risen to 110 to 115 million, a growth rate of 2.5% per annum. The World Bank estimates that the population will rise to 177 million by 2010. Coupled with this rise in population is a migratory trend from the rural areas to the cities. In 1981 only 15% of the population was urban. The World Bank estimates that this will rise to 30% by 2005.

In a small country like Bangladesh a large and rapidly growing population puts enormous pressure on physical resources, food production and employment. Hence, the impacts of severe floods which disrupt production, destroy physical resources and prevent employment are magnified. The lack of an effective welfare state means that people whose land, property or jobs are either temporarily or permanently denied them by floods or flood related destruction become destitute and homeless. As the population rises, so the numbers of people at risk increases as hence the human impacts of the floods become ever more serious.

Bangladesh has benefitted for many years from substantial foreign investment through loans and donations of cash. This has led to the construction and equipping of not only factories and mills, but also schools, colleges, and hospitals and the building of infrastructure in the form of roads, railways, embankments, culverts and bridges. The Bangladeshi economy has been growing substantially as a result, with a very creditable 4% per annum being maintained for some years. Hence, there is much greater potential for damage by severe flooding because of disruption of the economy and destruction of buildings, plant and infrastructure. Not only did the flood years of 1987 and 1988 produce distinct reductions in the rate of economic growth, but also the damage caused by the floods set back development programmes by three to five years.

The heart of the Bangladeshi economy remains agriculture and yet even today very limited use is made of the nation's land and water resources. For example, less than 5% of the annual volume of water flowing through Bangladesh each year (around 795 billion cubic metres) is actually diverted for any municipal, industrial or agricultural use. The amount actively used is within the measurement margin of error. With an efficient system of flood management, drainage control and irrigation, agricultural production in each of the summer, autumn and winter season crops could be greatly increased, allowing Bangladesh again to become a net exporter of food. Also, improved flood control would boost industrial growth and exports that earn valuable foreign income. However, growth is at present also hampered by the nation's difficulties with internal transport and communications. Much of this difficulty also centres on the Brahmaputra River.

### Jamuna Bridge Project

A major impediment to development of the Bangladeshi economy has been the relatively poor internal transport within the nation. In this respect the Brahmaputra River forms a major barrier, because there are no bridges across the river and because its shifting banklines and braid bars make both the establishment of permanent ferry terminals (called locally ferry ghats) and navigation by cross river ferries very difficult. During high summer monsoon floods road and rail embankments, culverts, bridges and ferry ghats are inundated and/or washed away. During the winter dry season, water levels drop so low that chars block all but a few of the anabranches and road and railway ferries are forced to take circuitous routes across the river that add hours to journey times. In the driest winters most ferry routes become completely impassable and all cross-river traffic must use just one or two reliable routes.



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In the 1980s a major initiative was undertaken to select a suitable site for and road and rail bridge across the Brahmaputra/Jamuna in Bangladesh. After a three year study of the river and its geomorphology, a site just south of Sirajganj (Figure 1) was identified as being a morphological node in the braided pattern and therefore being the most desirable site on the basis of its relatively narrow width and bankline stability. A preliminary design for the bridge and river guide bunds to train the river away from the bridge abutments was put forward in 1989 by the Jamuna River Bridge Authority (JMBA, 1989).

Land acquisition and site preparation for the Jamuna Bridge have now begun and international funding for this multi-million dollar project is being assembled. However, morphological channel developments since completion of the JMBA Report in 1989 raise serious questions concerning the stability and hence the continued suitability of the crossing site. Channel changes in the Brahmaputra/Jamuna and the issues they raise for the detailed design of the bridge and its training works must be kept under continuous review in order to ensure the success of this major piece of civil engineering.

### Flood Embankments

The right bank flood plain of the Brahmaputra/Jamuna River is protected from inundation for a length of over 200km between the River Teesta in the north and the Hurasagar in the south by the Brahmaputra Right Embankment (BRE). This is an earthen flood control embankment built in the 1960s. The BRE has never been over-topped, not even by the 1988 flood, which had a return period of around 100 years. Also, it has been very stable geotechnically and there have been no known significant failures of the embankment due to mass instability alone. The BRE has, however, become a permanent refuge for increasing numbers landless people whose land, villages, work places and bazaars have been consumed by bank erosion along the right bank. People dwelling on the BRE cut into its steep sides to create level ground for houses. This seriously weakens the embankment and compromises its effectiveness as a flood barrier, but it is impossible to move the people from their homes as they have no where else to go and unless continuing land loss along the right bank can be halted, the number of people living on the BRE will increase.

The BRE is unprotected by any hard covering or armour layer to prevent erosion by fast flowing water. Hence, it is vital that a wide berm of flood plain land exists between the BRE and the swift currents of deep, near-bank anabranches of the braided Brahmaputra River. When constructed in the 1960s the BRE was built well back from the bank edge and lateral spurs, called cross-bars, were built across the berm at right angles to the flow to prevent significant currents in water standing on the flood plain. But during the last thirty erosion of the right bank of the Brahmaputra has reduced or eliminated the berm and cross-bars at many points along the BRE. Where a current attacks the embankment it fails almost immediately, opening a breach and leaving the area behind the breach open to flooding. Failure occurs either by direct erosion of the soil making up the embankment, or, more commonly, by a sliding failure of the river bank and the embankment together due to deep scouring of the channel bed adjacent to the bank. Breaches allow flooding not only during high flood years, but also during more frequent, lesser events that would normally be comfortably contained by the BRE.

When the BRE is breached a new embankment is built at some distance behind the missing section. This 'retired' embankment is seldom built to the high specification of the original BRE and major problems arise due to the need for land acquisition and the fact that land owners who are on the river-ward side of the retired embankment lose flood protection. As a result further development of their property is blighted and they face financial ruin as its value as real estate diminishes greatly. For these and other reasons retirement of the BRE is seen as undesirable.

Following serious breaching of the BRE in 1987 and 1988 and extensive flooding through the breaches in those and subsequent years, the question arose as to whether it was



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cost effective to continue to try to maintain the embankment along its current alignment, or whether it might be better to start again with a new embankment some distance back from the bankline and create a substantial berm between it and the major right bank anabranches. Hence, a study was proposed to examine all the issues and put forward a plan of action. When the Terms of Reference for the study were determined it immediately became obvious that any attempt to maintain the current alignment would involve very substantial bank protection works that would effectively harden the right bank of the river against erosion along a 200km length of channel. Erosion prevention on this scale constitutes a form of river training and so the BRE Study became the Brahmaputra River Training Study (BRTS). Within months of its inception the Flood Action Plan (FAP) came into being and the BRTS project was subsumed as a component, FAP-1.

## River Training Works

The aim of river training works is to satisfy the needs of users of the river and flood plain by enhancing the stability, flood capacity and navigability of the channel without reducing its ability to convey the wide range of water discharges and associated sediment loads supplied from upstream.

In the case of rivers with single-thread meandering channels river training is an advanced engineering-science. Over many years a great deal of experience has been gained in training large alluvial rivers such as the Rhine and Danube in Europe and the Missouri and Mississippi in North America. Also, advances in the understanding of flow hydraulics and sediment transport in channels with loose boundaries have supplied a theoretical basis for modern river training. Developments in both physical and numerical modelling allow quite complex situations to be simulated over relatively long reaches of river, especially in the case of computer-based models, so that field observation and past experience can be combined with model investigations when planning a new layout for training works. The morphological study of meandering rivers has been the subject of intense activity by fluvial geomorphologists for over 100 years and the excellent qualitative understanding of meander morphology and mechanics that has resulted can be applied as a useful adjunct to the findings of quantitative studies in hydraulic and sedimentation engineering. As a result, when construction of training works begins, there can be a high degree of confidence that the outcome will be successful.

Conversely, there is a dearth of experience in training braided rivers like the Brahmaputra. Some experience has been gained in China on great rivers such as the Yellow River, but river regime, environmental, sociological and political systems are sufficiently different that it cannot be safely assumed that such experience is appropriate to training the Brahmaputra in the 1990s and beyond. The processes of flow and sediment movement in braided rivers are poorly understood and no theory exists for loose boundary flow in a channel with multiple anabranches. Physical models of braided rivers of the size of the Brahmaputra are impractical owing to scaling problems except for small, local models of specific parts of the system, while computer models are unable to reproduce the complex three dimensional flow patterns of a braided system because the algorithms needed to describe such flows do not yet exist. In any case the data requirements to specify the boundary conditions for such models would be so immense preclude their use in practical engineering studies. For all these reasons, training of a braided river presents many challenging problems that cannot be resolved quantitatively using existing approaches in hydraulic and sedimentation engineering. Hence, in the case of training braided rivers, much greater reliance must be placed on morphological studies that use qualitative and empirical descriptions of process-form interactions in and between anabranches.





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## FLOOD ACTION PLAN 1- THE BRTS STUDY

### Brief Overview of FAP-1

The difficulties inherent in conceiving and designing training works on braided rivers were recognised in the Terms of Reference for BRTS/FAP-1 and were reflected in the successful proposal prepared by Sir William Halcrow and Partners. This proposal emphasises an inter-disciplinary approach that combines the results of engineering investigations, physical modelling and computer simulations with a substantial input based on morphological study, to produce a scheme for training works designed to protect the BRE and hence secure an improvement of its performance as a flood embankment. It also sets out the need to design any training works in a way that maintains the channel capacity needed to convey the 100 year flood and allows the development of reliable, permanent locations for ferry ghats with reasonable river crossing routes and times.

The BRTS envisaged that unless it was decided to retire the BRE substantially then it would be necessary to stabilise the right bank on two different timescales: first, through short-term measures to protect the BRE at locations where bank erosion poses an immediate threat and; second through longer-term measures designed to train the river in such a way that the BRE is protected along its full length from the Teesta confluence to the mouth of the Hurasagar. It was recognised that the completion of all the necessary training structures would take decades rather than years and that consequently a Master Plan giving a strategy for phasing their construction would be essential. Since the bank protection works can only be built sequentially over a period of years, a rationale would be required to determine where to start and in what order to protect various sites. The Master Plan would also have to address the issue of predicting the response of the river to training, which could only be attempted on the basis of a sound geomorphological analysis that covered the history of channel development, current morphological status and likely direction of future channel planform evolution with and without training works.

### Morphological Studies in FAP-1

The morphological studies were undertaken using a modular approach which divided the task into a number of clearly defined and almost independent work units. Twenty work units were established, involving:

- i) numerical treatment of archive data on discharge records, sediment rating curves and stage-discharge curves for the principal gaging station at Bahadurabad;
- ii) field observation of present day bank erosion processes and mechanisms and their distribution along the study reach;
- iii) analyses of historical maps, aerial photographs, cross-sections and satellite images to establish ancient, recent and present patterns of channel change at local, reach and regional scales; and
- iv) the synthesis of the results of these geomorphic, historical and contemporary study units to draw conclusions on the past development and present status of the river, and make predictions of possible future trends and directions of change under different scenarios of training and management.

Due to its limited length, the remainder of this paper concentrates solely on the bankline dynamics of the river, as they directly affect the right bank flood embankment. Full accounts



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of the results and significance of the entire morphological study may be found in the BRTS Final Report (Halcrow, 1992) and the Draft Master Plan (Halcrow, 1993) recently published by the BRTS.

## Historical Bank Movements and Erosion Rates

A primary aim was to produce reliable, documented records of rates and distributions of bank erosion along the right bank in the study reach. This is essential because although it is accepted that very high rates of bank erosion are experienced along the Brahmaputra, in the past there has been much reporting of severe bank erosion and very rapid rates of retreat without a sound factual basis and much of the evidence used to document rapid retreat is either based on hearsay or from questionable sources.

The second aim was to develop predictors of bank erosion rate that could be used to gauge the vulnerability of the BRE to breaching in the near future. Such predictors are essential at two timescales: first in the immediate and near future so that the risks of breaching at various locations where the BRE is presently being threatened by bank erosion can be properly assessed and priority locations for immediate bank protection can be correctly identified; and second, in the medium term future to allow a broader view of the vulnerability of the BRE and assess how far the embankment would have to be retired if no training works were constructed in order to make it secure for the next few decades.

Although there is a great deal of historical information available on bankline movement, interpretation is limited because little of it uses a common reference frame that can be used with confidence in making quantitative measurements. For example, the many of the earlier studies use maps prepared by Rennells, Wilcox and the Survey of Bangladesh in computing bankline movement rates over long periods. However, the JMBA Feasibility Report rejects all the mapping data on the grounds of unreliability in reference frame and instead relies heavily on more recent cross-sectional data from BWDB permanent ranges. Yet, field and archive investigations undertaken in the BRTS have revealed that the positioning of BWDB monuments marking cross-sections to be variable in time and uncertain in accuracy. Hence, even this data source is questionable.

Unrectified satellite images are difficult to use to derive data on bankline movement rates because consecutive images have different levels of distortion and likely errors introduced by these differences are of the same order as the bankline changes that it is desired to quantify.

In the BRTS it was decided that no data sources would be excluded initially, but that interpretation should take place in the light of the likely error margins on the results. For example, although the maps produced in the 18th and 19th centuries cannot match the accuracy of the satellite images of the 1970's and 1980's, their much greater antiquity makes them valuable nonetheless because it gives an indication of bankline changes over centuries rather than decades.

Early examination of historical bankline movement concentrated on three sources believed to be reliable: the Survey of Bangladesh map of 1953-56, the LANDSAT imagery of 1973 and the rectified SPOT images of 1989. Preliminary results and bank erosion rates based on tracings of banklines from maps and images were reported in the Second Interim Report of the BRTS, but it was clear by the end of 1991 that the sources being used needed better preparation if acceptable results were to be obtained. Preparation would involve adopting a common reference frame, digitising the banklines for the years of interest and entering the data in a Geographical Information System so that maps to a common scale and datum could be produced and accurate, detailed data on bankline movement over different periods could be generated. These major tasks were performed under the FAP-19 GIS programme conducted



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by ISPAN (Irrigation Support Project for Asia and the Neareast) and the results made available to BRTS starting in September 1992.

The FAP-19 base maps and materials included: rectified and co-registered images for the years 1973, 1976, 1978, 1980, 1984, 1987, 1990 and 1992, all referenced to the transverse Mercator projection of Bangladesh; data-bases containing the bankline Mercator coordinates at half kilometre intervals for these years; bankline maps based on the Survey of Bangladesh maps of both the early to mid-1950s and 1914; and a bankline map based on the Wilcox map of 1830. FAP-19 also produced derived maps showing bankline changes over various periods, rates of widening, rates of centreline shifting and areas of erosion and accretion for different time intervals.

In the BRTS analysis of the FAP-19 data a considerable number of graphs, plots and tables were produced and examined. Consideration is limited here to those which gave the greatest insights into bank retreat distributions and which formed the basis for bank erosion rate predictors.

#### Spatial patterns of bank erosion

Bank retreat was considered spatially by plotting retreat rates against Mercator Y-coordinate at 0.5km intervals for the various periods between satellite images. Figure 2 shows typical results, with the 1987-90 and 1990-92 periods superimposed. The most striking feature of the 1987-90 plot is the series of peaks and hollows in the line. These represent rather evenly spaced zones of severe bank retreat (downward spikes) interspersed with less severely eroding, stable or even accreting banks. The average spacing of erosional spikes is around 8km and it was found that this corresponds to the spacing of embayments in the right bank cut by either an aggressive meander bend, or flow deflection around a rapidly growing braid bar in the right bank anabranch channel. Further examination of the plot for 1987-90 shows that the pattern of fairly evenly spaced erosive spikes has some large gaps in it. For example, there are no significant spikes between metres 750,000 and 770,000. This reach corresponds to a 'nodal' reach between two island clusters, where bank erosion would be expected to be less severe from first principles. Hence, the spatial distribution of severe erosion does tie in somewhat to the morphological pattern of the braided channel form of braided/meandering anabranches flowing around major island clusters which are separated by relatively stable nodal reaches.

#### Development of short-term bank erosion rate predictors

Table 1 contains data used to develop a simple medium-term erosion predictor for the right bank, designed to allow an estimate to be made of the likely position of the right bank in about twenty years if no bank protection or river training works are constructed. The derived data listed in Table 1 were obtained by dividing the study reach into twenty one sub-reaches, each 10km long and containing twenty sampling points at half kilometre intervals. The distributions of erosion rates for the period 1973-1992 was analysed statistically to find the mean rate and standard deviation for that sub-reach. Ten kilometre sub-reaches were used in this exercise because when using a substantial time-scale for erosion predictions, a fairly large spatial scale should also be used for comparability. Since ten kilometres is larger than the characteristic size of an embayment and since twenty years is much longer than the lifespan of an embayment, it may be assumed that within each ten kilometre sub-reach over a nineteen year period the distribution of erosion is random. In that case a normal distribution can be used to describe the probability of a particular erosion rate being equalled or exceeded.

The results indicate that 19 year average erosion rates are much lower than those usually quoted in the literature and in reports concerning the Brahmaputra. In fact the highest 19 year erosion rate *for a ten kilometre sub-reach* is just less than 160m/yr, which although serious would not usually be considered exceptional when dealing with the Brahmaputra. To examine this further a cumulative frequency plot of 19 year mean erosion rates was plotted, not for the sub-reaches but for all available sampling points. Sites showing net accretion were



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excluded since this part of the morphological study is concerned with bank retreat. The results are plotted in Figure 3. The median erosion rate is around 80m/yr and the inter-quartile range of 40 to 170m/yr is probably much smaller than most researchers would expect. Local erosion rates greater than 400m/yr are exceptional and occur at less than 5% of the sample sites.

Plots of the erosion rates for shorter periods are shown in Figure 4. This plot shows that while erosion rates do vary over shorter periods, they tend to follow a rather similar probability function. Hence, the short-term (2 to 4 year) median erosion rate varies only between about 50 and 110 m/yr. The range is only as large as this is because the period of record includes both some very large floods events and exceptionally low flow years. The most rapid erosion occurs in the 1987-90 period, which includes the major floods of 1987 and 1988, and the lowest erosion occurs in the 1990-1992 period, because flows were well below normal and probably also because of bank relaxation following the exceptionally heavy retreat in the preceding period.

These findings illustrate the importance of taking into account the time and space scales when evaluating erosion rates. Very high rates are observed over periods of year or a few years, but they are not sustained for many years at the same location and they do not occur at all locations simultaneously. Similarly, it is known that some locations have enjoyed periods when erosion rates have been slow, or when the bank has actually advanced through accretion, but the duration of such quiet periods is also limited. To investigate the probability distributions for the duration of higher than average and lower than average periods of erosion, the time distributions of retreat at individual sampling locations were plotted and examined. Catastrophic erosion was defined as a rate higher than 350m/yr, very rapid erosion as between 200 and 350m/yr, and rapid erosion as being between 150 and 200m/yr. A cumulative plot of the frequency and duration of each class of higher than average bank erosion is shown in Figure 5.

The results show that there is an inverse relationship between the severity of erosion rate and the length of time over which that erosion rate is sustained. In 80% of observed cases the duration of a period of catastrophic erosion is between two and four years. The median duration is three years. There is less than a five percent chance that catastrophic erosion will persist at a site for more than six years. The distribution of periods of very rapid erosion is very similar to that of catastrophic erosion. The median duration is again three years, but very rapid erosion can be sustained for longer periods than catastrophic erosion. Hence, there is a ten percent chance of very rapid erosion persisting for more than seven years, but only a five percent chance of it continuing beyond eight years. Rapid erosion is much more likely to be sustained for a fairly prolonged period. The median (50%) duration is between six and seven years and there is a ten percent chance of a retreat rate in this category continuing longer than ten to eleven years.

As the higher than average erosion rates dealt with in Figure 5 are almost always the result of embayment formation due to development of a near bank anabranch bend or flow division, these findings concur with the observation that the average lifespan of an anabranch embayment is between around five to eight years.

The probability distributions for higher than average erosion may be used to gauge the danger of breaching of a threatened section of the BRE within the next few years, depending on the width of berm between it and the retreating bank and the current rate of bank retreat. In practice, even given the highest priority treatment, construction cannot begin until one to two years after the decision to proceed, these plots should help to differentiate priority locations where the BRE can be protected by immediate or very prompt action from sites where it is probable that a breach will occur before any works can be successfully completed. At these latter sites embankment retirement is the only viable option. If retirement is undertaken, then the plots can be used to estimate the minimum distance by which the BRE should be retired to give it a reasonable chance of survival over the next few years.



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Figure 6 shows the cumulative frequency vs duration plot for slower than average erosion rates. Again, periods of slow erosion tend to be of limited duration. Slow erosion is classed between 25 and 50m/yr and very slow erosion as less than 25m/yr. Bank accretion is excluded in this analysis since accreted deposits are usually very easily re-eroded and constitute little protection from renewed bank erosion. The 50 percentile for very slow erosion is 5 years and that for slow erosion is 10 years. Hence, if a section of bank has not suffered serious erosion historically for some period, the likelihood of this continuing into the future can be gauged by examining this plot. In the long-term the historical data indicate that erosion is fairly evenly distributed spatial and that promontories tend to be destroyed after a number of years. Hence, the fact that a particular reach of bank has been spared average or above average bank retreat for some time may actually be a sign that severe erosion is more likely to occur in the near future.

#### Development of medium-term bank retreat predictors

While such analyses are useful for short-term predictions at points where the BRE is directly threatened, when making medium to long-term predictions of bankline movement, it is necessary to use typical erosion rates for medium and long time periods.

In predicting the bankline position in 2011, the data in Table 2 were used to indicate the amounts of bank retreat likely to occur with a 50% probability of exceedence (that is the mean erosion rate operating for 19 years) and one with a 5% probability of exceedence (that is an erosion rate equal to the mean plus 2.6 times the standard deviation). The coordinates predicted using this approach were supplied to FAP-19 who produced maps of predicted banklines at 1:250,000 scale (Figure 7).

An implicit assumption in the method described above is that there are no trends towards accelerating or decelerating erosion at a site through time. Exhaustive examination of time trends at many of the sample sites along the river suggested some cyclical behaviour over the 19 year period, probably associated with the life-cycle of anabranch bends and embayments. However, there was no clear evidence that erosion was either accelerating or decelerating over the last 19 years. Comparison of the 1973-1992 and 1953-1973 periods does indicate marked differences in bank erosion rates, particularly for the left bank. However, consideration of the implications of the 40 year record since 1953 is still at an early stage.

### CONCLUSIONS

The Brahmaputra River is a water course of enormous importance to the people of Bangladesh and of world significance geomorphologically. The Brahmaputra Right Embankment provides important flood protection to millions of people living in the flood plain of the Brahmaputra River. While it has never been over-topped and seems quite stable geotechnically, the BRE is highly vulnerable to breaching by fast flowing, deep water in the right bank anabranch. Breaching occurs when an embayment develops in the right bank due to meandering and braiding processes in that anabranch. Embayments can develop very rapidly, with peak erosion rates exceeding 400 m/y and rising to as much as 1km per year in the most extreme cases. However, such severe erosion is rarely sustained at any site for more than a few years and medium term erosion rates are much lower. Some sections of bank have been spared from serious erosion for nearly twenty years. Eroding and non-eroding reaches of bank are spaced somewhat evenly along the bank, with a typical distance between embayments being 8km.

Statistical treatment of high quality data on bankline movement from satellite images for the period 1973-1992 can be used to produce quantitative predictors of likely rates and durations of significant erosion. These can be used to assess the risk of a breach occurring at a particular location and to make recommendations concerning the set-back distance necessary to protect a retired embankment from attack in the near future.



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The results of the morphological analysis indicate that long reaches of the BRE are or will soon be vulnerable to flow attack and breaching and that unless steps are taken to protect the right bank and train the river then serious erosion, land loss and flooding will almost certainly continue into the next century.

### ACKNOWLEDGEMENTS

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Table 1. 1973-1992 rates of right bank retreat  
for 10km sub-reaches

| Mid-Point<br>Y-Coordinate<br>(m) | Mean<br>Erosion<br>Rate<br>(m/yr) | Standard<br>Deviation<br>(m/yr) |
|----------------------------------|-----------------------------------|---------------------------------|
| 842500                           | -74.24                            | 39.84                           |
| 830250                           | -47.28                            | 9.73                            |
| 820250                           | -98.91                            | 101.75                          |
| 810250                           | -21.25                            | 16.54                           |
| 800250                           | -150.12                           | 86.93                           |
| 790250                           | -158.82                           | 97.06                           |
| 780250                           | -66.21                            | 26.98                           |
| 770250                           | -121.20                           | 35.09                           |
| 760250                           | -75.76                            | 31.30                           |
| 750250                           | -102.03                           | 31.12                           |
| 740250                           | -156.12                           | 12.56                           |
| 730250                           | -95.43                            | 24.78                           |
| 720250                           | -130.13                           | 21.36                           |
| 710250                           | 9.58                              | 45.61                           |
| 700250                           | -37.35                            | 14.46                           |
| 690250                           | -56.23                            | 35.06                           |
| 680250                           | -59.89                            | 15.31                           |
| 670250                           | -90.30                            | 44.13                           |
| 660250                           | -4.71                             | 10.63                           |
| 650250                           | -50.08                            | 27.48                           |
| 642000                           | -27.18                            | 34.86                           |

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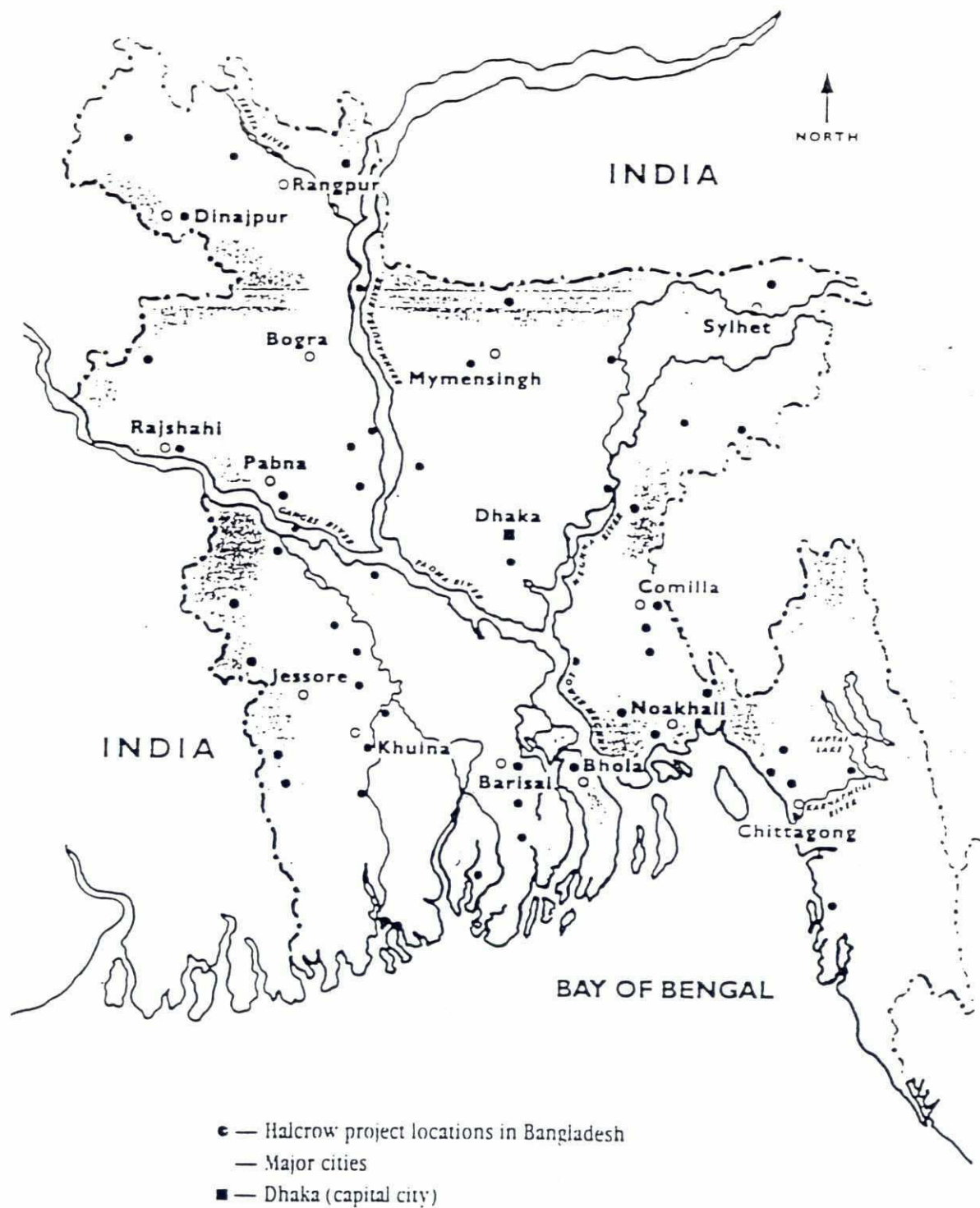


Figure 1 Location map of the Brahmaputra River



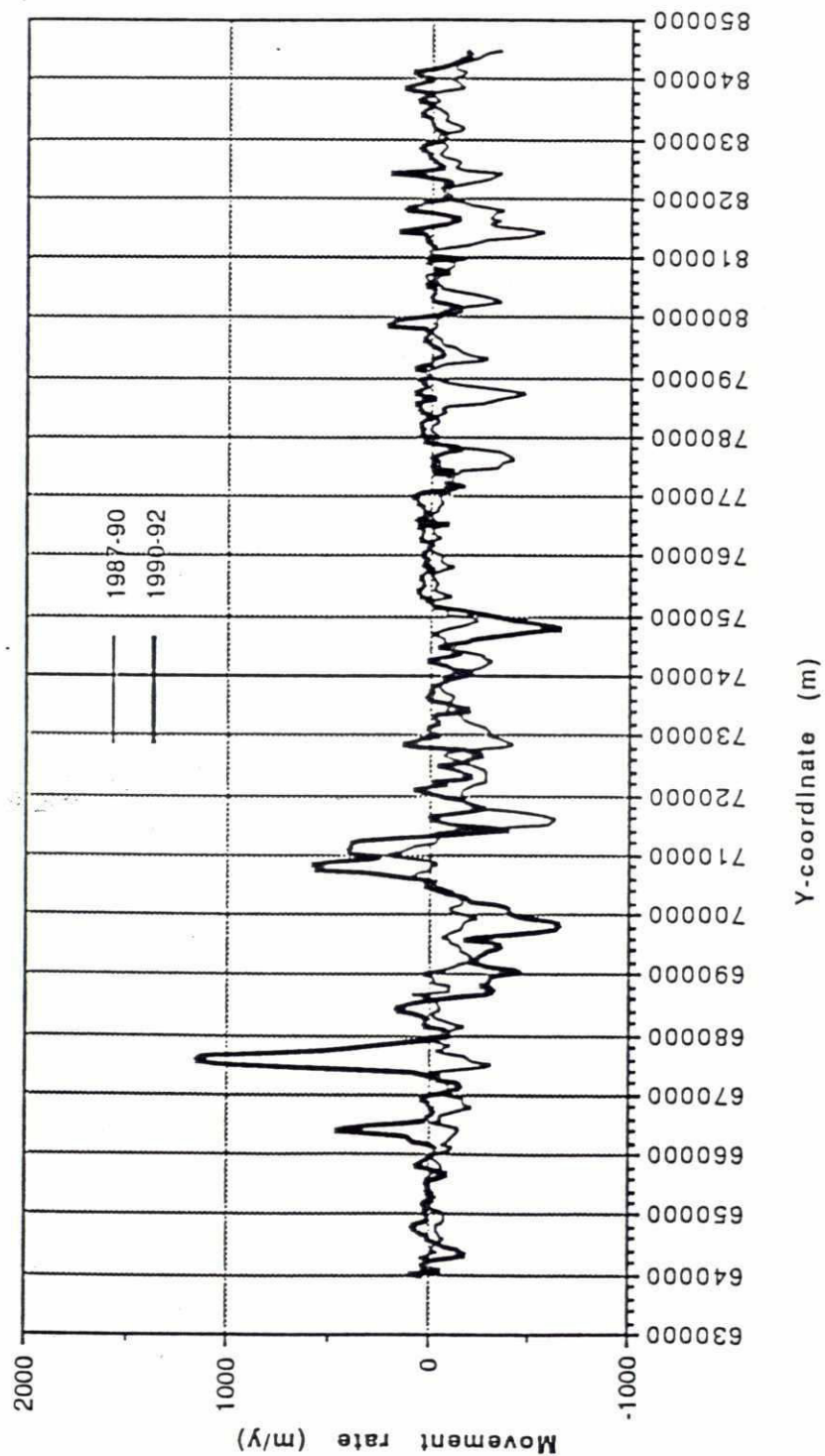


Figure 2. Distribution of bank erosion along the right bank for high discharge (1987-90) and low (1990-92) discharge periods. Notes: negative bank movement indicates erosion; Y-coordinate is a northing based on the Bangladesh Transverse Mercator Projection.

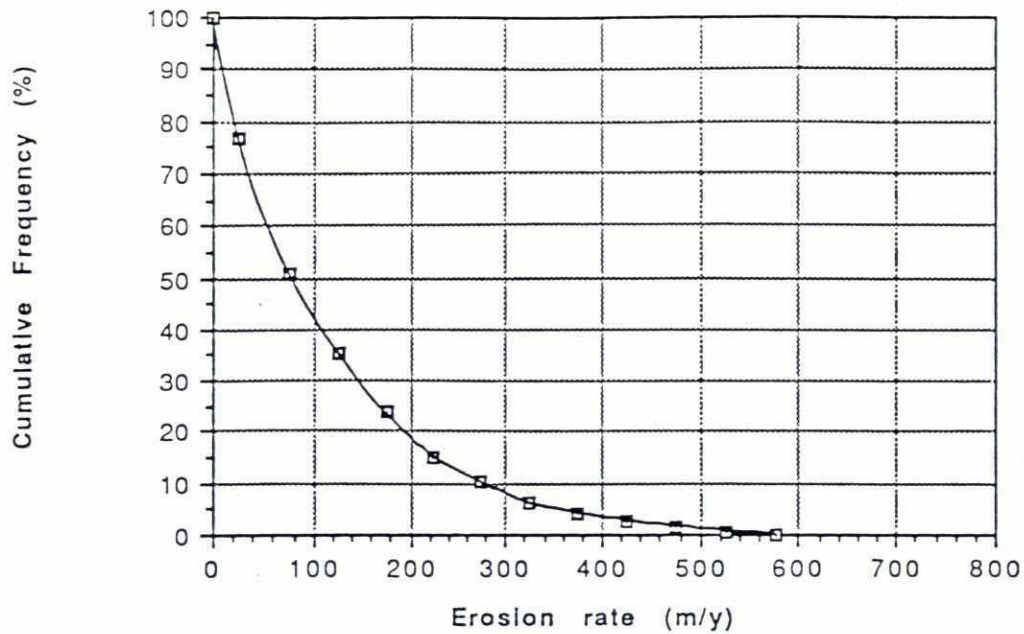


Figure 3. Statistical distribution of right bank erosion rates over the period between 1973 & 1992

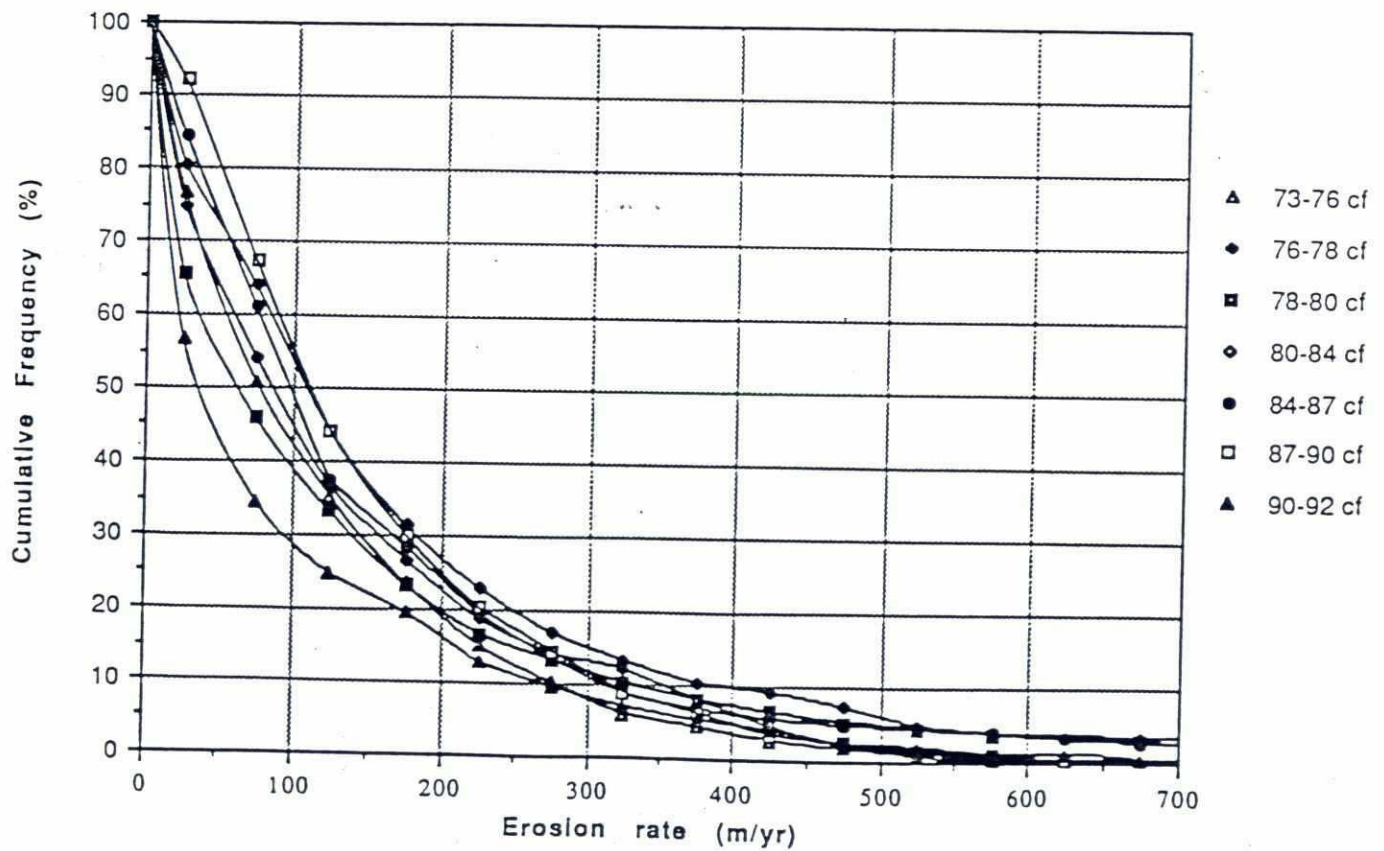


Figure 4. Right Bank Brahmaputra River Bank Erosion Rates for Various Periods



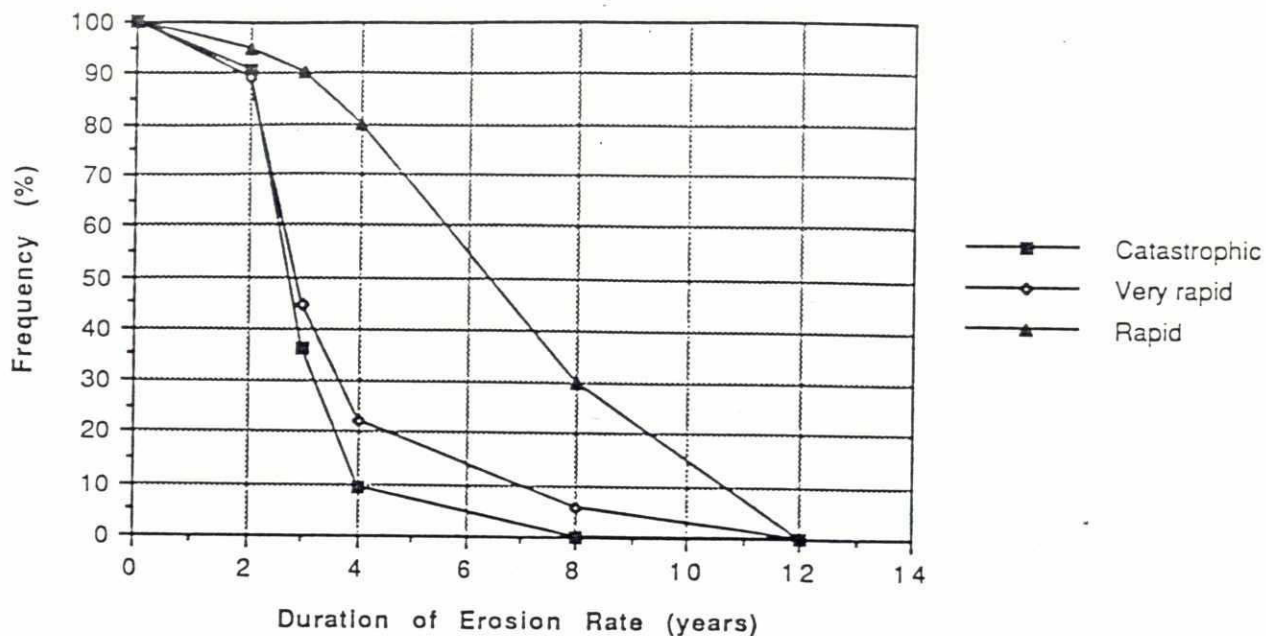


Figure 5. Duration of faster than average bank erosion rates on the right bank

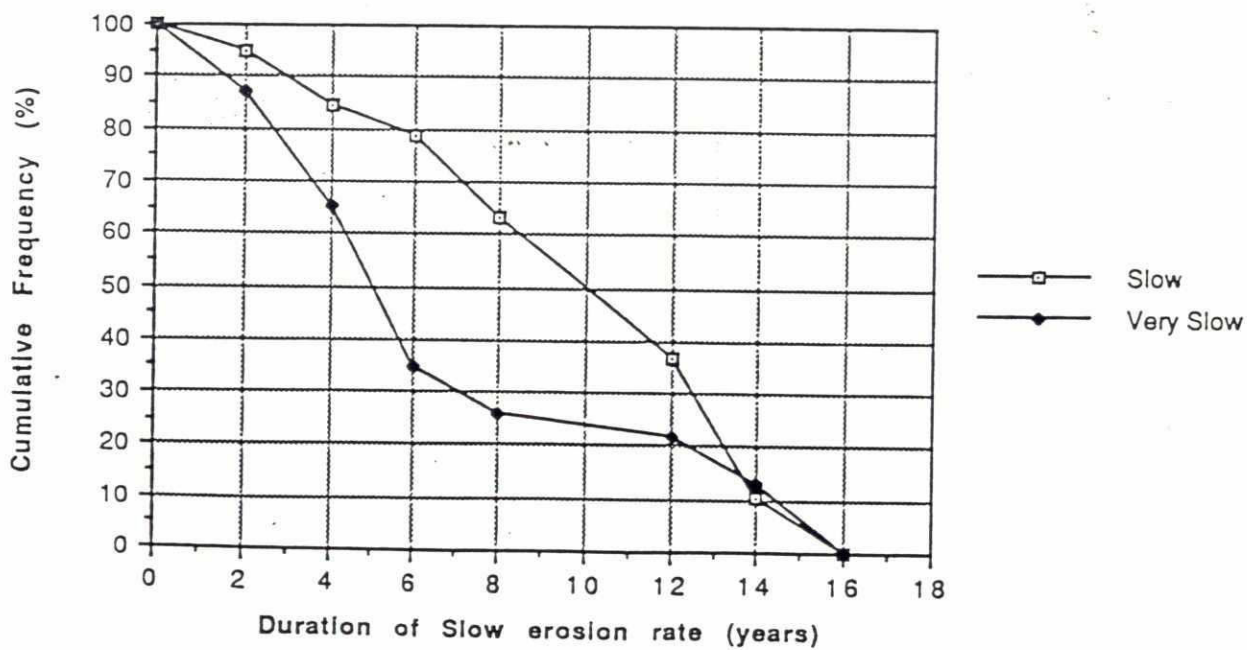


Figure 6. Duration of Periods of Lower than average erosion rates on the right bank

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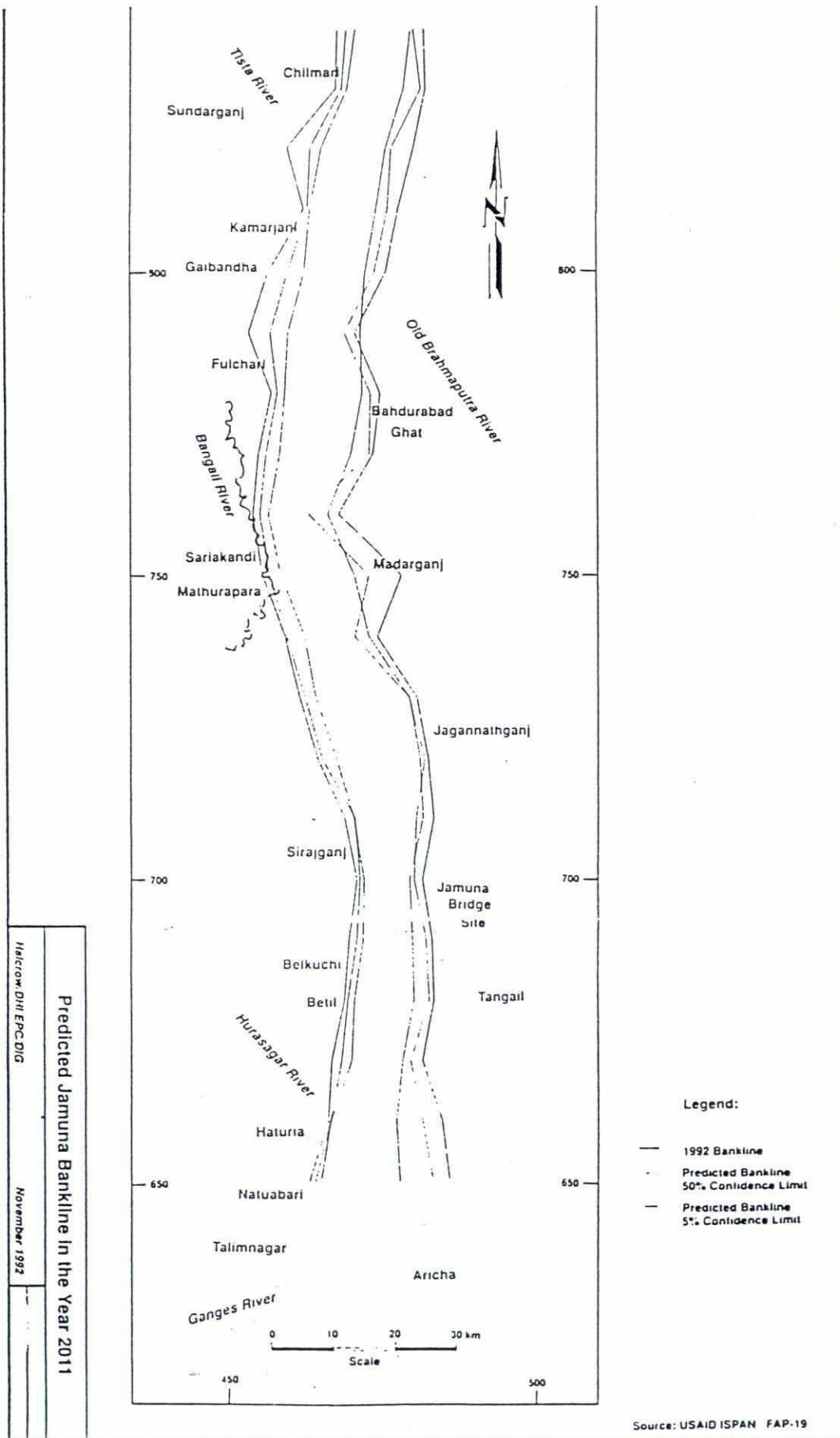


Figure 7 . Predicted position of banklines in about 2010 if no bank protection or river training works are built (Map supplied by FAP-19)



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## 5 Scour

BUDAPEST

1988

INTERNATIONAL CONFERENCE ON FLUVIAL HYDRAULICS  
ON CONSTRICTION SCOUR IN BRAIDED RIVERS

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**SUMMARY:** An analytical method is developed for the estimation of scour in braided rivers due to constriction by the construction of bridge approaches. Both ultimate scour and time dependent scour are considered. The methods are checked with a numerical model.

# 1. Introduction

The Government of Bangladesh has set as one of its highest priorities the integration of the Western and Eastern regions of the country by the construction of a 5 to 6 km long Multi-purpose Bridge across the Jamuna River, which bisects the country from North to South into two. The Jamuna Multi-purpose Bridge Authority (JMBA) has been created by the Government of Bangladesh to further the studies and construction of the Project and to assure the necessary funds for its implementation.

The World Bank as Executing Agency for the United Nations Development Programme has appointed Rendel Palmer & Tritton, NEDECO and Bangladesh Consultants Ltd. to undertake Phase II of an Appraisal Study into the proposed crossing. This study is being undertaken in 1987 and 1988. The Consultants will complete the engineering and geotechnical investigation, commenced in Phase I of the study, and will prepare the final design of the approaches and the river training works together with designs of two bridge alternatives. Within the frame-works of this study constriction scour in braided rivers was studied.

The Jamuna river has a total width in the order of 10 to 15 km. It may be economically attractive to construct a bridge across the Jamuna River that does not span the full width of the river, but rather a bridge with reduced span in combination with earthen bridge approaches. Earthen bridge approaches instead of bridge work, however, have the disadvantage that they virtually block the passage of flood water. Thus the flood waters must be conveyed through the remaining opening spanned by the bridge. This would lead to backing up of the water upstream of the bridge (with possibly adverse effects like extra inundation of adjacent lands, etc.), if it were not compensated by scouring of the river bed. In alluvial rivers, however, the increased discharge through the remaining bridge opening results in constriction scour of the river bed, thus increasing the water transport capacity. This process may be so efficient that it may even yield lower flood levels upstream.

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An understanding of the processes described above is essential for an evaluation of possibilities for constriction. In particular the maximum scour depth and the development of the scour in time are of importance. The maximum scour is affecting the foundation depth of the piers and as such is an important factor for the costs of the bridge. The time-wise development of the scour determines whether this maximum scour is actually reached during the design flood. It is also of major importance for the reduction of the backwater as a consequence of scouring of the river bed underneath the bridge.

Several analytical methods have been proposed to compute the constriction scour (see e.g. Laursen (1963) or Komura (1971)). These methods, however, predict only equilibrium scour and not the time-wise development. Furthermore, the conditions in a braiding rivers have not been addressed specifically. Within the frame-work of the appraisal study mentioned above special attention was given to time-dependent constriction scour in braided rivers. Here a summary of this study is given. For more details reference is made to the Phase I appraisal report (RPT/NEDECO/BCL, 1987).

## 2. Special aspects for braided rivers

What is so special about constriction scour in braided rivers? A braiding river is characterized by channels and bars. From an inspection of the cross-sections measured yearly by Bangladesh Water Development Board (BWDB) it can be concluded that the Jamuna River can be characterized as a flat sand bed in which a number of deeper channels are present (see the Figures 1 and 2). The level of the bars is ultimately determined by deposition processes during floods. It appears that the average level of the bars is about 1 m below average flood level. This implies that the contribution of the flow over the chars (as the bars are called locally) is quite limited. During an earlier feasibility study for a bridge across the Jamuna River (JICA, 1976) the constriction scour was computed based on the average depth of channels and chars. Considering the limited contribution of the chars to the total discharge, it may be possible to constrict the river without causing serious scour as long as the discharge capacity of the deeper channels is not reduced. The considerations given hereafter are based on this reasoning.

There are two more aspects which need attention when dealing with braided rivers, viz.:

- (i) changes in channels pattern,
- (ii) confluence scour.

### Re (i) Changes in channels pattern

Following constriction of the river bed the flow which initially was conveyed by several channels may be carried by a single channel only. The dimensions (average depth, width) of this single channel may be different from the dimensions of the initial channels. Klaassen and Vermeer (1988a) studied the channel characteristics of the Jamuna River and found the following relationship between the average depth (in m) and the discharge (in m<sup>3</sup>/s) for bankfull stapes:

$$\bar{h} = 0.23 Q^{0.32} \quad (1)$$

A change from two channels with equal contribution to the total discharge to one channel carrying the combined (bankfull) discharge would result in a channel with an average depth about  $(2)^{0.32} \approx 1.25$  times larger than the average depth of the two smaller channels. At the proposed crossing site of the Jamuna bridge only one channel is present, so this aspect has not to be taken into account here.

Re (ii) Confluence scour

In the case of one single channel underneath the bridge also confluence scour (see e.g. Ashmore and Parker (1983) or Klaassen and Vermeer (1988b) for confluence scour in the Jamuna River) may have to be taken into account. Available information (RPT/NEDECO/BCL (1987) or Klaassen and Vermeer (1988b)) suggests that in the Jamuna River confluence scour is dominant over outerbend scour.

### 3. Analytical approaches

#### 3.1 General

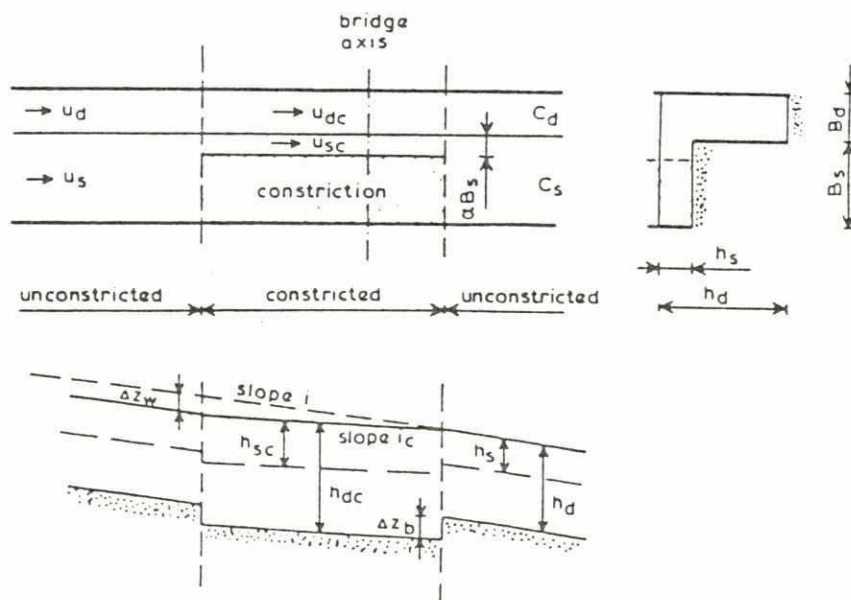
To estimate the scour depth in the constriction during the passage of an extreme flood, essentially two approaches can be used. The first approach is the use of analytical methods based on the governing equations. For the derivation of these analytical methods normally simplifying assumptions have to be made. The second approach consists of the application of numerical models. These models allow for more detailed analysis, e.g. the bed level changes during real flood waves (and not a constant maximum discharge as used in analytical models) can be simulated. The advantage of analytical methods over mathematical models is that they allow for a quick evaluation of proposed solutions.

Analytical methods applicable for the present problems are derived in the present chapter. In Chapter 4 these methods are tested with an exploring computation with a numerical model.

#### 3.2 Equilibrium constriction scour

For the present considerations the braiding river is assumed to consist of one deep channel (representing the summation of the numerous channels present in one cross-section) and a shallow part (representing all chars in a cross-section). Both deep channel and shallow char are supposed to be alluvial. Sediment is transported over both parts and the sediment transport is governed by the local hydraulic conditions. It can be shown that the retarded adaptation of suspended load plays a minor role only for the Jamuna River conditions and it is neglected here. Furthermore it is assumed that:

- the total width of the channels remains the same;
- the deposition downstream of the constriction (caused by the erosion in the constriction) has a negligible effect.





The following notation is used (see above Figure):

- $h$  = water depth (m)
- $i$  = slope (-)
- $B$  = width (m)
- $u$  = water velocity (m/s)
- $\alpha$  = constriction coefficient of shallow chars (-)
- $C$  = Chézy coefficient, defined via  $C = u/(hi)^{0.5}$  ( $m^{1/2}/s$ ) assuming that the bank roughness is negligible.

The index  $d$  refers to the deep channel(s), the index  $s$  to the shallow chars, and the index  $c$  to the constricted reach.

The following assumptions are made:

- $h_s/h_d = \text{constant} = \phi$ ;
- both deep and shallow channel are alluvial, the ratio of changes in  $h_s$  and in  $h_d$  equals also  $\phi$ ;
- $B_s/B_d = \beta$ ;
- $C_s/C_d = \gamma$ ;
- sediment transport  $s$  ( $m^2/s$ ), can be written as:

$$s = a u^b \quad (2)$$

where  $a$  and  $b$  are constants for a particular site.

For the ultimate conditions (that will occur after an infinitely long time) the discharge and the sediment transport through the constricted reach should equal these parameters in the non-constricted reaches. Thus the following expressions are obtained:

$$Q_t = Q_d + Q_s = B_d \cdot h_d \cdot u_d + B_s \cdot h_s \cdot u_s = B_d \cdot h_{dc} \cdot u_{dc} + \alpha B_s \cdot h_{sc} \cdot u_{sc} \quad (3)$$

$$S_t = S_d + S_s = B_d \cdot s_d + B_s \cdot s_s = B_d \cdot s_{dc} + \alpha B_s \cdot s_{sc} \quad (4)$$

where  $Q$  = discharge ( $m^3/s$ ),  $S$  = sediment transport ( $m^3/s$ ), defined via  $S = s \cdot B$ , and the index  $t$  refers to the summation of the contribution of shallow and deep part (so  $S_t = S_s + S_d$ ).

The above Equations together with the Chézy equation and the assumptions made above, results in the following expression for  $h_{dc}/h_d$ :

$$\frac{h_{dc}}{h_d} = \theta_1 \cdot \theta_2^{-1/b} \quad (5)$$

where:

$$\theta_1 = \frac{\beta \gamma \phi^{3/2+1}}{\alpha \beta \gamma \phi^{3/2+1}} \quad (6)$$

$$\theta_2 = \frac{\beta \gamma \phi^{b/2+1}}{\alpha \beta \gamma \phi^{b/2+1}} \quad (7)$$

The constriction scour is computed via:

$$\Delta z_b = \frac{h_{dc}}{h_d} \cdot h_d - h_d = \left( \frac{h_{dc}}{h_d} - 1 \right) h_d \quad (8)$$

In a similar way an expression for the drawdown (no backing up!) can be derived which reads:

$$\Delta z_w = \left( \frac{1}{i} \frac{c}{1} - 1 \right) \cdot L = \left( \frac{1}{\frac{1}{i}} - 1 \right) \cdot i \cdot L \quad (9)$$

where:

$$\frac{1}{i} \frac{c}{1} = \theta_1^{-1} \cdot \theta_2^{+3/b}$$

The above formulae can be used to compute the ultimate constriction scour and upstream drawdown. The following remarks are made:

- (i) The changes in bed level computed with Equation (4) are minor compared to a approach where the average depth and the full width is used averaging of channels and chars alike (JICA, 1976). This is so because the discharge over the chars is small compared to the discharge in the deep channels. Reducing the width occupied by chars does not affect the capacity of the total channel considerably.
- (ii) The above analysis relates to the bed level averaged over the width of the total of the deep channels and of the shallow chars. The variation of the depth over the width may be considerable and this has to be taken into account separately.
- (iii) The present method can also be used to determine the effect of reducing the width of the channels in addition to closing off the width of the chars. In that case the method has to be repeated for  $\beta$ ,  $\phi$  and  $\alpha$  as required for the further constriction envisaged and the obtained value of  $h_{cd}/h_d$  should be multiplied with the value obtained for closing off the chars only.

### 3.3 Development of constricted scour in time

Next the development in time of the constricted scour is considered. It is not possible to solve the complete differential equations describing this phenomena analytically. Here the complete differential equations are simplified to find approximate solutions, allowing to identify the important parameters. This will enable to estimate scour during a design flood wave of limited duration.

In addition to the assumptions made in Section 2.2 it is assumed here that backwater effects are negligible. The local constriction varies in longitudinal direction, which is expressed as  $\alpha = \alpha(x)$ . All parameters in the constriction will vary in longitudinal direction ( $x$ ) and in time ( $t$ ). The governing equations for this time-dependent phenomenon are the continuity equation for the water and for the sediment transport, in addition to the Chezy equation and the sediment transport equation (Equation (2)). The continuity equations for water and sediment read:

$$\frac{\partial}{\partial x} (Q_s + Q_d) = 0 \quad (11)$$

$$- B_d \frac{\partial h_d}{\partial t} - B_s \cdot \frac{\partial h_s}{\partial t} + B_d \frac{\partial s_d}{\partial x} + B_s \frac{\partial s_s}{\partial x} + s_s \frac{\partial B_s}{\partial x} = 0 \quad (12)$$

For given  $B_s = B_s(x)$ , the above set of equations can be used to find solutions for  $h_d(x,t)$ , etc. To do this analytically, it is necessary to linearize these equations. This is done by assuming:

$$u_d = u_{do} + u'_d$$

$$h_d = h_{do} + h'_d$$



$$s_d = s_{do} + s'_d$$

$$u \ll 1$$

where  $u_{do}$ ,  $h_{do}$  and  $s_{do}$  refer to the undisturbed, non-constricted conditions. Introducing the above assumptions and neglecting higher-order terms results in the following equation:

$$\frac{\partial h'_d}{\partial t} + c_{do} \frac{\partial h'_d}{\partial x} + E_o \frac{s_d}{B_s} \frac{\partial B_s}{\partial x} = 0 \quad (13)$$

where:

$$c_{do} = \frac{bs_{do}}{h_{do}} \left( \frac{1 + \beta \phi^{b/2} \gamma^b}{1 + \beta \phi} \right) \quad (14)$$

$$E_o = \frac{(1 + \beta \phi^{b/2} \gamma^b) \beta \phi^{3/2} \gamma^b - (1 + \beta \phi^{3/2} \gamma) \beta \phi^{b/2} \gamma^b}{(1 + \beta \phi)(1 + \beta \phi^{3/2} \gamma)} \quad (15)$$

This simple-wave equation (de Vries, 1987) can be solved provided  $B_s = B_s(x)$  is given.

Assuming that the constriction of the char width varies with  $\tanh x/\lambda$  conform Figure 3, results in the following analytical solution for  $h'_d(x,t)$ :

$$\frac{h'_d}{h'_{d \max}} = \frac{1}{2} \left\{ \tanh \frac{x}{\lambda} - \tanh \left( \frac{x}{\lambda} - \frac{t}{T} \right) \right\} \quad (16)$$

where:

- $h'_d$  = bed level change due to general scour (m)
- $h'_{d \max}$  = maximum general bed level scour (for  $t \rightarrow \infty$  and  $x/\lambda > \sim 3$ ) (m)
- $\lambda$  = characteristic length (m)
- $T$  = characteristic time scale, defined via  $T = h_{do} \lambda / bs_{do}$

The maximum constriction scour is given by

$$h'_{d \max} = (1-\alpha) \cdot \frac{E_o s_d}{c_{do}} = (1-\alpha) \cdot \frac{(1+\beta \phi^{b/2} \gamma^b) \beta \phi^{3/2} \gamma^b - (1+\beta \phi^{3/2} \gamma) \beta \phi^{b/2} \gamma^b}{(1+\beta \phi^{b/2} \gamma^b)(1+\beta \phi^{3/2} \gamma) b} \cdot h_{do} \quad (17)$$

A matter of interest may be how the results of the present time-dependent analysis are related to the solution for the ultimate conditions when a design flood has continued for an indefinite period. More specifically this may be stated as whether Equation (8) and Equation (17) are similar. It can be shown that Equation (8) can be transformed to Equation (17), when it is assumed that:

$$\alpha \beta \gamma \phi^{3/2} \ll 1 \quad \beta \gamma \phi^{3/2} \ll 1$$

$$\alpha \beta \gamma \phi^{b/2} \ll 1 \quad \beta \gamma \phi^{b/2} \ll 1$$

In fact these assumptions were made during the derivation of Equation (17), so it can be stated that Equation (8) is a more general approximation than the other expression. For the conditions in the Jamuna River given in the next chapter Equation (8) and (17) yield approximately the same results.

#### 4. Comparison with numerical model

To check on the analytical models developed here, computations were carried out with a numerical model. The model used was a 1-dimensional morphological model, that does not take into account retarded settling of suspended load (Jansen, 1979; de Vries et al, 1988). Retarded settling is also not included in the analytical models derived above. A schematization was used that resembles to some extent the conditions in the Jamuna River (see Figure 4). The particle size of the size of the sediment was assumed to be 0.2 mm.

Various computations were carried out with the model. Here the results of one computation are given (see Figure 5), being characterized by:

- sediment transport 5 x the sediment transport predicted by the method of Engelund and Hansen (1967),
- a succession of four normal hydrographs and one more extreme (about 1:10) year hydrograph (see Figure 5(a)).

The conditions used in the numerical model can be transferred to coefficients for the analytical approach as follows:

- $B_d = 5,000$  m,  $B_s = 5,000$  m, thus  $\beta = 1$
- constriction of 4,000 m yields  $\alpha = 1,000/5,000 = 0.2$
- $C_d = 70$  m<sup>1/2</sup>/s,  $C_s = 50$  m<sup>1/2</sup>/s, thus  $\gamma = 0.71$
- power of sediment transport formula  $b = 5$
- for normal flood  $h_d = 7.50$  m and  $h_s = 1.50$  m, thus  $\phi = 0.2$
- for extreme flood  $h_d = 9.00$  m and  $h_s = 3.00$  m, thus  $\phi = 0.33$ .

Application of the analytical model derived in Section 3.2 yields:

|               | $\beta$ | $\gamma$ | $\phi$ | $b$ | $\alpha$ | $h_{dc}/h_d$ |
|---------------|---------|----------|--------|-----|----------|--------------|
| normal flood  | 1       | 0.71     | 0.20   | 5   | 0.2      | 1.05         |
| extreme flood | 1       | 0.71     | 0.33   | 5   | 0.2      | 1.095        |

For the scour values of 0.37 and 0.86 m for the normal and the more extreme flood, respectively, are found. These figures correspond nicely with the results of the numerical model for high sediment transport rates ( $5 * EH$ ), when equilibrium is approximately reached, viz. 0.35 and 0.86 m.

Finally, the value of the time-scale  $T = h_{do} \lambda / (b s_{do})$  is determined for the case that the sediment transport is 5 x the transport predicted by Engelund and Hansen (1967). Here for  $\lambda$  a value of 3 km is taken. Extreme flood conditions are considered with  $h_{do} = 9.00$  m. For  $T$  a value of about 5 days is obtained. From Figure 3 it follows that maximum scour is about reached after approximately 20 days. Comparing this value with the hydrographs given in Figure 5(a), it can be concluded that ultimate constriction scour conditions will be reached for the Jamuna River.

Finally, it is of interest to have another look at Figure 5(a). The eroded bed material is deposited downstream of the constricted reach, consequently back-water (initially induced by the constriction), will still be present even when the constriction scour has fully developed, due to deposition downstream.

#### 5. Conclusions

In this paper analytical expression are derived for the ultimate and the time-dependent constriction scour in braided rivers. Comparison with a numerical model shows that the analytical expressions yield a fair estimate of the con-



striction scour depth to be expected. When applied to the future conditions for the bridge across the Jamuna River (Bangladesh) it is shown that the constriction scour develops relatively rapidly. That nevertheless some backwater will occur is due to the deposition of the eroded sediment downstream of the bridge, causing backing up of the flood waters.

#### 6. Acknowledgements

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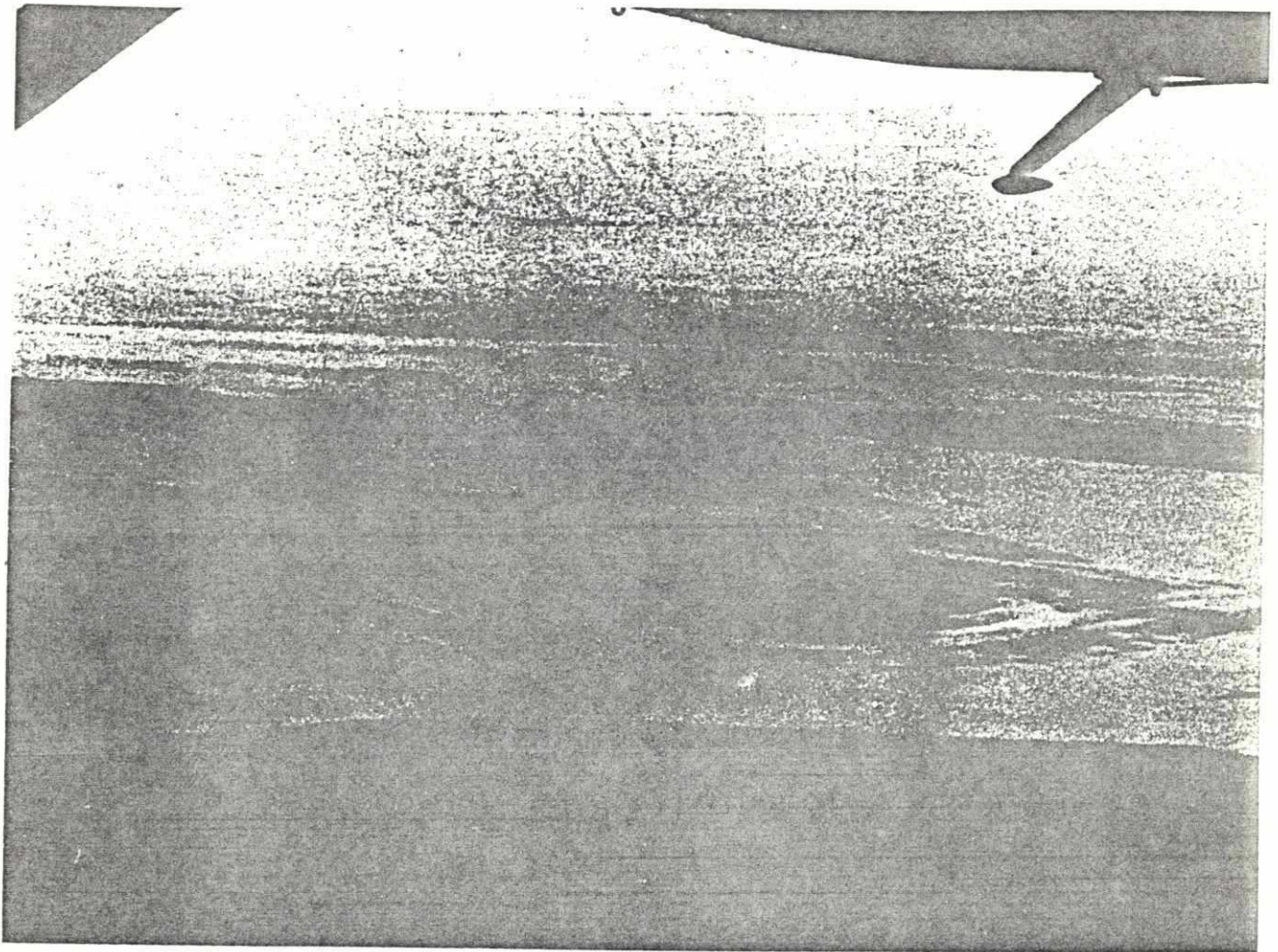


Figure 1 Braiding Jamuna (Brahmaputra) River, Bangladesh during low flow conditions (flow from right to left).

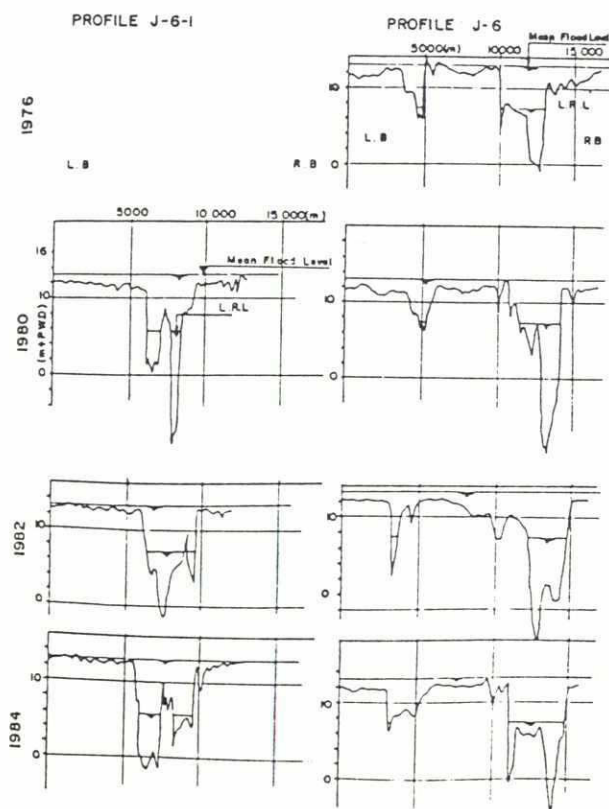


Figure 2 Some cross-sections across the Jamuna River near the proposed crossing site at Sirajganj for various years (measurements by BWDB).



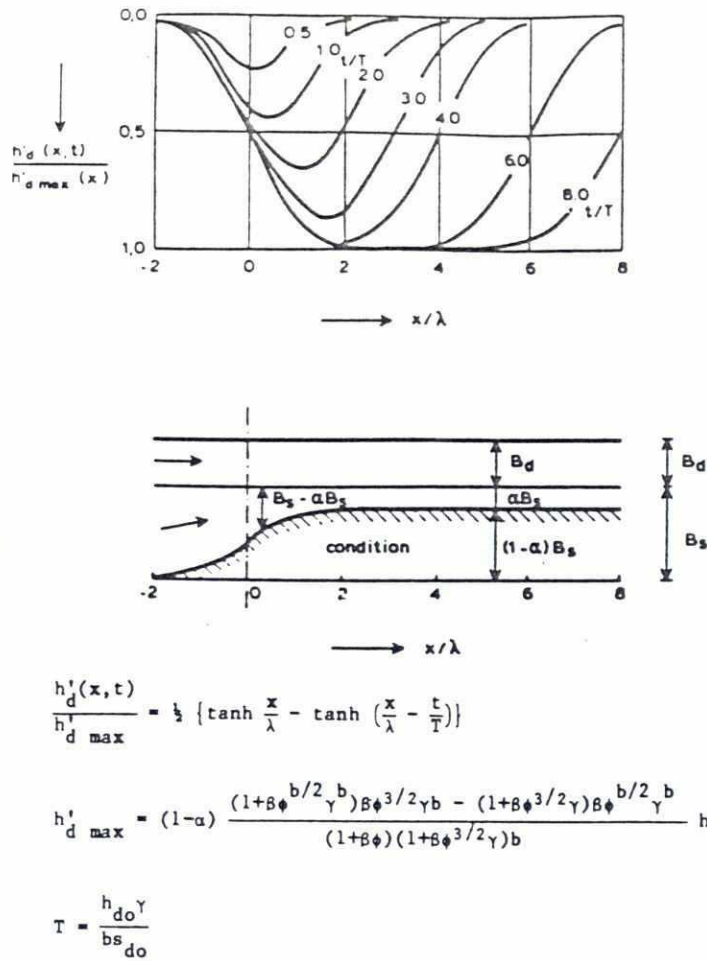


Figure 3 Time-dependent constriction scour in braided rivers.

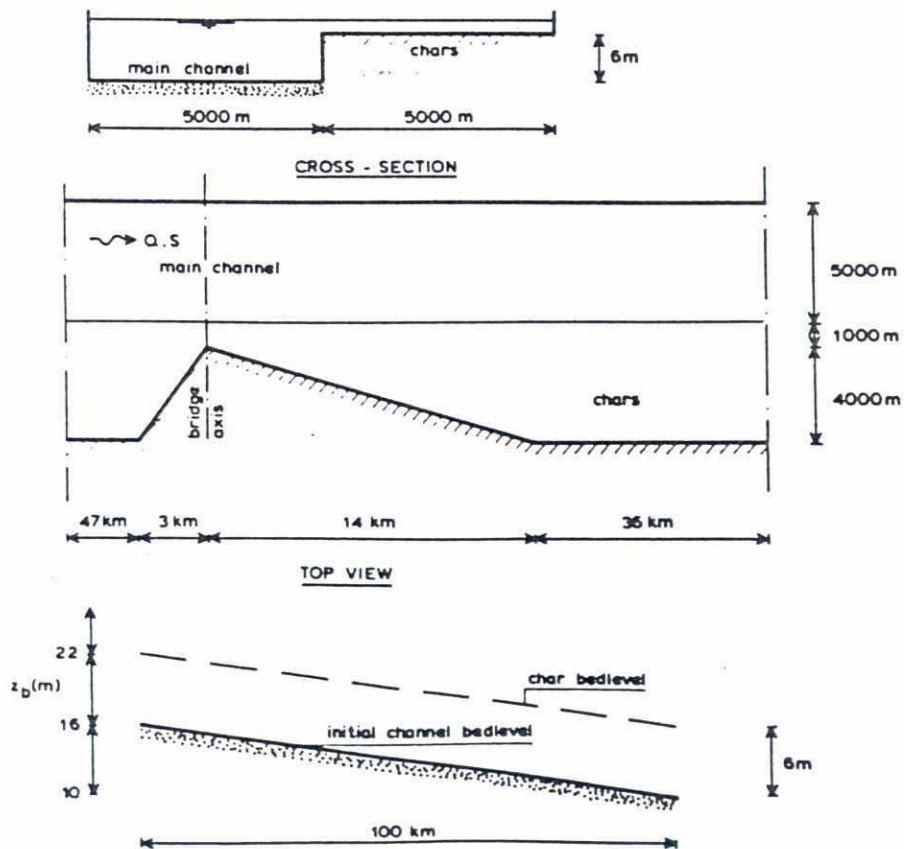
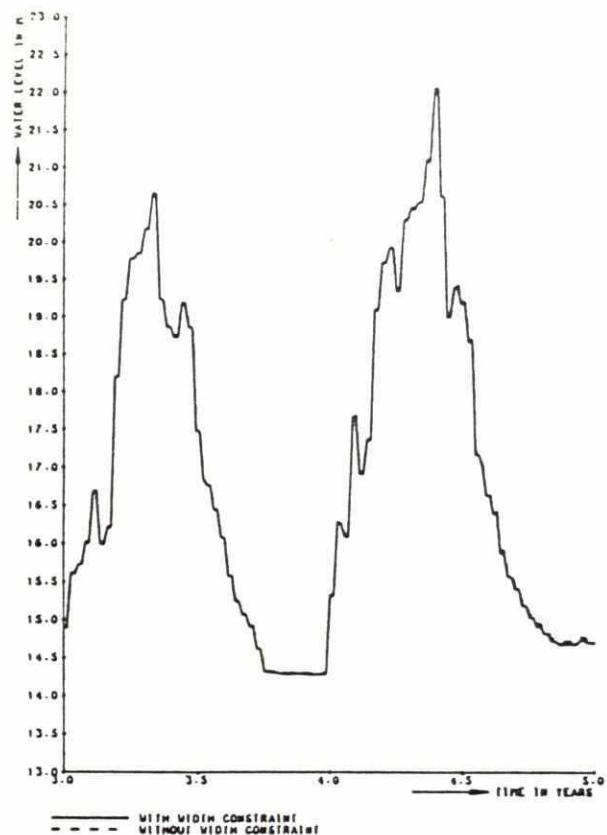
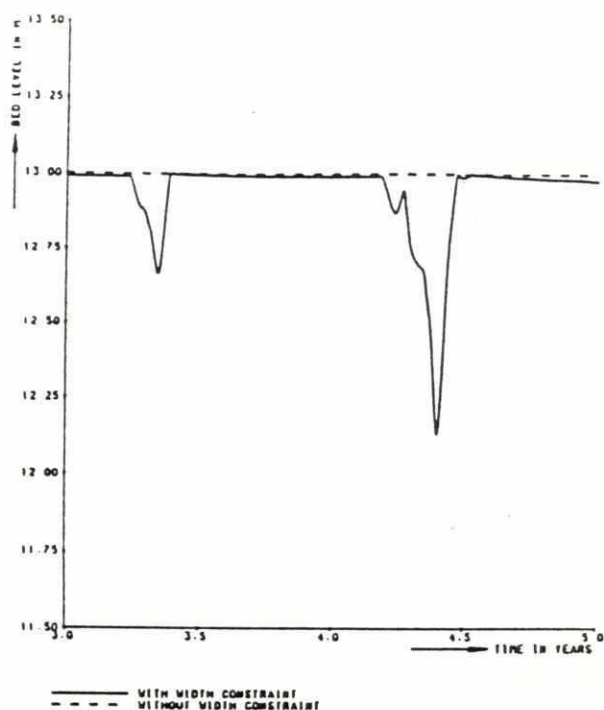


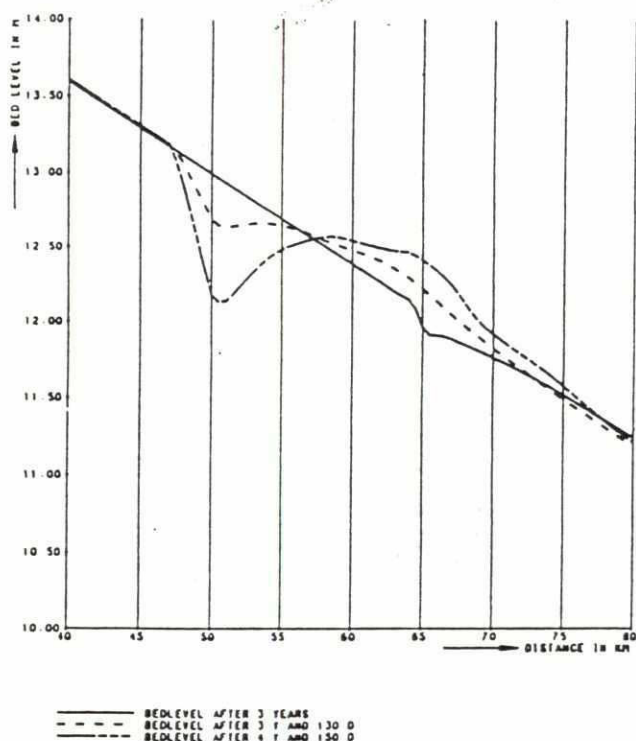
Figure 4 Schematization of modelled river reach.



a. stage versus time.



b. bed level changes as function of time.



c. bed level before and after flood waves.

Figure 5 Some results of simulations with the numerical model.



## Paper 5.2

KLAASSEN, G.J. & K. VERMEER (1988), Confluence scour in large braided rivers with fine bed material, *Proc. Intern. Conf. on Fluvial Hydraulics, Budapest, Hungary*, 395-408pp.

BUDAPEST

1988

## INTERNATIONAL CONFERENCE ON FLUVIAL HYDRAULICS

CONFLUENCE SCOUR IN LARGE BRAIDED RIVERS  
WITH FINE BED MATERIAL

by

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SUMMARY: Results of field measurements in the Jamuna (Brahmaputra) River, Bangladesh, were used to study confluence scour in large braided rivers with fine bed material. A relationship between confluence scour depth and the angle of the confluencing channels is developed, comparable to the relationship established by Ashmore & Parker (1983) for braided streams with coarse bed material. It seems that in sand-bed rivers with considerable suspended load the maximum relative scour depth is less than for gravel streams.

### 1. Introduction

Confluence scour is the scour pattern occurring downstream of confluences of two channels. Such a confluence can be either part of a braiding channel system, or it can be the confluence of two rivers. The scour pattern that develops downstream of such a confluence can be characterized as an elongated reach of relatively low bed levels in the middle of the combined downstream channel.

Relatively little is known about confluence scour. A review of literature in which the phenomenon is mentioned, is given by Ashmore & Parker (1983). Only in the last decade attempts have been made to look more closely at the phenomenon of confluence scour, aiming at identifying the mechanisms responsible for this phenomenon and to arrive at methods to predict the maximum depth and location of the scour. In the relevant literature (notably Mosley, 1976; Ashmore, 1979 and 1982; and Ashmore & Parker, 1983). Two possible explanations were given for the observed phenomenon:

- back-to-back bend scour, explaining the scour as caused by helical flow, generated by the induced curvature of the stream lines,
- increased turbulence due to the bouncing of the two confluencing water "jets", whereby the increased turbulence caused local erosion similar to the scour holes produced by wall jets (Rajaratnam et al, 1981) or, slightly more remote, the scour occurring near the re-attachment point of dunes (de Ruiter, 1982).

In addition, even a third explanation can be considered, in which the produced scour hole is thought to be produced by two confluencing channels with non-equilibrium distributions of water and sand distribution over the width of the

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combined channel downstream (similar to the cause of pools and point bars in river bends, see Struiksma et al, 1985). Exploring computations, with a 2-d mathematical model for bed topography have shown, however, that this phenomenon probably produces scour holes with too small dimensions.

It is not the purpose of the present paper to contribute to the understanding of the cause of confluence scour. It is merely intended to present some results of the analysis of observed confluence scour holes in a large braiding sand-bed river. As such it expands on the studies mentioned above that has exclusively dealt with relatively steep gravel-bed rivers.

The work on coarse bed rivers was mainly initiated by the need to predict maximum scour depth in braided rivers in relation to the required designed depth for pipelines to be buried. The present study was done for the feasibility study for a bridge across the Jamuna River in Bangladesh. This project stems from the required integration of the Western and the Eastern regions of Bangladesh; this integration has been set by the Government of Bangladesh as one of its highest priorities. The project consists of the construction of a 5 to 6 km long Multi-purpose Bridge across the Jamuna River, which bisects the country from North to South. The Jamuna Multi-purpose Bridge Authority (JMBA) has been created by the Government of Bangladesh to further the studies and construction of the Project and to assure the necessary funds for its implementation.

The World Bank as Executing Agency for the United Nations Development Programme has appointed Rendel Palmer & Tritton, NEDECO and Bangladesh Consultants Ltd. to undertake Phase II of an Appraisal Study into the proposed crossing. This study is being undertaken in 1987 and 1988. The Consultants will complete the engineering and geotechnical investigation, commenced in Phase I of the study, and will prepare the final design of the approaches and the river training works together with designs of two river bridge alternatives.

The Jamuna River is the lowest reach of the Brahmaputra River in Bangladesh upstream of its confluence with the Ganges River (see Figure 1), and as such one of the largest braided river systems in the world. The Jamuna River has a slope, typically between 0.1 and 0.06 m/km while the bed material is quite uniform and varies between 0.25 mm near the Indian border and 0.15 mm at the confluence with the Ganges River. More details on the Jamuna River are given in Coleman (1969) and, more recently, in RPT/NEDECO/BCL (1987), Klaassen and Vermeer (1988) and Klaassen, Vermeer and Uddin (1988).

The present paper is structured as follows: Chapter 2 deals with the data which were used in the present study. Also some data on maximum scour depths in the Jamuna River are given, to demonstrate the importance of confluence scour in this river system. In Chapter 3 the confluence scour data are analysed, whereby a similar approach, as used by Ashmore and Parker (see Figure 2), is applied. The results are discussed in Chapter 4, a.o. by comparing the results obtained here for large, sand-bed rivers with Ashmore and Parker (1983)'s results for gravel-bed rivers. Some conclusions are formulated in Chapter 5.

## 2. Used data

In the present study data are used obtained from field measurements in the Jamuna River in Bangladesh. Figure 3 is a satellite image of the lower reach of the Jamuna river in 1973 during low flow conditions. The approximate scale of the Figure is 1:1,000,000. The hydrograph of the Jamuna River is fairly constant (see e.g. Coleman, 1969). From April to July the discharge rises



gradually. The actual flood season is from July through September, during which the discharge varies around a fairly constant value. From October onwards the discharge decreases. So, different from many gravel-bed rivers, long periods are present in which the variation in discharge is relatively small. Because sailing on the Jamuna River is quite possible also during flood conditions, it is possible to obtain soundings of confluence scour patterns which must represent approximate equilibrium conditions.

Two sets of field measurements are used in the present study, viz:

(i) Historical data

Since the late sixties regular soundings have been done by the Bangladesh Inland Water Transport Authority (BIWTA). These measurements are done to identify the best sailing routes on the Jamuna River. The soundings are executed from sailing boats, using echo sounder equipment in combination with a Decca positioning system. Measurements have been done mostly during low flow conditions, but some sounding maps relating to flood conditions are also available. Figure 3 provides parts of some sounding maps of BIWTA, relating to the same river reach downstream of Sirajganj (see Figure 1), but for different years. In this reach the river system is relatively stable, but even here over the years changes do occur. On the maps the confluence scour can easily be identified.

(ii) Special field survey

In the period July - November 1987 a special hydrographic survey was carried out to measure scour depths (and bed forms, see Klaassen, Vermeer and Uddin (1988)) in the Jamuna River, in relation to the feasibility study for the proposed bridge. Scour depths were measured on different sites (see Figure 5) and at different dates. Also these measurements were carried out by BIWTA. In addition to the Decca system, also a Trisponder positioning system (with an accuracy of 1 m) was used. Due to excessive flooding in Bangladesh during the measuring period the measuring program had to be reduced, but still some data on confluence scour were collected. Results are presented on similar sounding maps as partly shown in Figure 4.

During Phase I of the present Jamuna Bridge Appraisal Study an analysis was made of the deepest points in the Jamuna on the basis of the historical BIWTA sounding maps. A distinction was made between:

- outer bend scour,
- confluence scour,
- scour near protrusions (stable points, bank protection works).

The results of this analysis are shown in Figure 6 for both the Jamuna river and the Padma River (Figure 1). The following conclusions can be drawn from this figure:

- deepest scour holes occur near protrusions,
- confluence scour in the Jamuna River is dominant over outer bend scour,
- maximum depth in a confluence is in the order of 20 to 25 m, compared to about 8 m being the average channel depth in the Jamuna River (Klaassen and Vermeer, 1988).

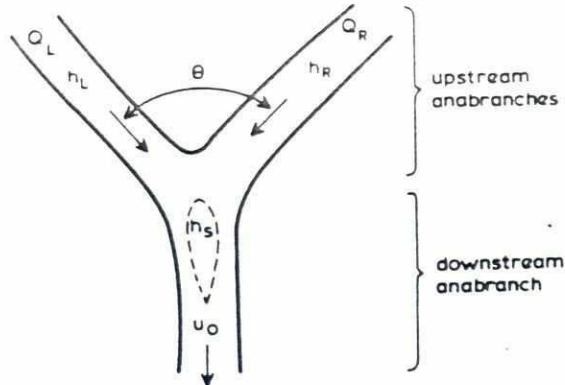
The scatter in the figure is due to:

- (i) measurements done in flood as well as low flow seasons,
- (ii) channel characteristics vary over the years (Klaassen and Vermeer, 1988),
- (iii) both the angle of the confluence and the bend characteristics vary in place and time.



### 3. Analysis

Ashmore and Parker (1983) proposed the following relation for confluence scour:



$$\frac{h_s}{\bar{h}} = f(F_o, \bar{i}_w, \theta, \epsilon) \quad (1)$$

where

- $\frac{h_s}{\bar{h}}$  = maximum (confluence) scour depth (m)
- $\bar{h}$  = average depth of the upstream anabranches, defined via  $\bar{h} = (h_1 + h_2)/2$  (m)
- $F_o$  = densimetric Froude number, defined via  $F_o = u_o / (\Delta g \bar{D}_g)^{0.5}$  (-)
- $\frac{u_o}{D}$  = average anabranch velocity (m/s)
- $\bar{D}$  = average mean grain size of the upstream anabranches (m)
- $\Delta$  = relative density, defined via  $\Delta = (\rho_s - \rho) / \rho$  (-)
- $\rho_s$  = specific density of sediment ( $\text{kg/m}^3$ )
- $\rho$  = specific density of water ( $\text{kg/m}^3$ )
- $\bar{i}_w$  = average anabranch downstream water surface slope
- $\theta$  = angle of incidence of anabranches of confluence ( $^\circ$ )
- $\epsilon$  =  $(Q_L - Q_R) / \frac{1}{2} Q_T$  (-)

$Q_L, Q_R, Q_T$  = discharge in left and right anabranch and total discharge downstream of confluence.

Ashmore and Parker analysed in total 118 points, including Mosley (1981)'s and their own field observations and results from laboratory studies. They found that the influence of  $F_o$  and  $\bar{i}_w$  on  $h_s/\bar{h}$  is very weak. Not discriminating for the values of  $\epsilon$  and thus including all data in the analysis the following relationship between  $h_s/\bar{h}$  and  $\theta$  was obtained (see Figure 7):

$$\frac{h_s}{\bar{h}} = 2.235 + 0.0308 \theta \quad (2)$$

Considering only points with  $\epsilon < 0.75$  (so confluences for which  $Q_1/Q_2 < 2.20$  where the indices 1 and 2 refer to the anabranches with the largest (1) and smallest (2) discharge, respectively), then a relationship steeper than Equation (2) was found. For  $\epsilon > 0.75$  the slope of the regression line for  $h_s/\bar{h}$  versus  $\theta$  was indistinguishable from zero at the 95% significance level.

A similar analysis was carried out for the field data from the Jamuna River. The depths of the upstream anabranches ( $h_R, h_L$ ) were determined by averaging

the observed average depth of at least four cross-sections directly upstream from the confluence. The inclusion of more cross-sections did not yield substantially different average depths. The deepest confluence scour depth in the downstream anabranch,  $h_s$ , was also taken from the sounding maps. The plotted depths are related to the S(andard) L(ower) W(ater), so the figures on the maps were corrected considering the water levels during the period the soundings were done. The angle of incidence was obtained from the sounding maps, too. Some inaccuracy is inherent, because most upstream anabranches are not straight. In the Jamuna River also more complicated confluences are present, where a short distance downstream of the confluence a third anabranch joins the river. Here a more complicated scour pattern occurs and these confluences were discarded from the analysis.

For each confluence values of  $\bar{h} = \frac{1}{2} (h_R + h_L)$ ,  $h_s$  and  $\theta$  were obtained. Computed values of  $h_s/\bar{h}$  are plotted against  $\theta$  in Figure 7. Although there may be an increase in  $h_s/\bar{h}$  for increasing  $\theta$ , this increase is obscured by the larger scatter.

As a second step of the analysis, an attempt was made to discriminate according to the value of  $\epsilon$ . The parameter  $\epsilon$  is a measure for the distribution of the total discharge over the two upstream anabranches. When  $Q_1$  and  $Q_2$  are the larger and the smaller discharge, respectively, then the relationship between  $\epsilon$  and  $Q_1/Q_2$  is as indicated below:

|            |      |      |      |      |      |      |          |
|------------|------|------|------|------|------|------|----------|
| $\epsilon$ | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | 1.50 | 2.0      |
| $Q_1/Q_2$  | 1.00 | 1.25 | 1.67 | 2.20 | 3.00 | 7    | $\infty$ |

To establish the value of  $\epsilon$  the value of  $Q_L$  and  $Q_R$  should be known, but these discharges were not measured simultaneously with the soundings. Here use is made of "at-a-station-relationships" that were derived for the Jamuna River (see Klaassen & Vermeer, 1988). The proposed relationship between the average depth of an individual channel of the Jamuna and the discharge it carries, reads:

$$\bar{h} = 0.56 Q^{0.23} \quad (3)$$

This relation can also be used for estimating the discharge in a channel once the average depth is known. The relation reads for that case:

$$Q = 12.44 (\bar{h})^{4.35} \quad (4)$$

Using Equation (4) an estimate of the discharges in all confluencing anabranches was obtained. Next the value of  $\epsilon$  was computed for each confluence. The result is plotted in Figure 9, where in total six classes for  $\epsilon$  were differentiated. For the results of the special survey in 1987 and the historical data, different symbols have been used. A figure is added to each point, indicating the estimated discharge in  $10^3 \text{ m}^3/\text{s}$  of the anabranch with largest depth.

From inspection of Figure 9 the following conclusions can be drawn:

- (i) For small values of  $\epsilon$  (for  $\epsilon < 0.25$  and to a smaller extent for  $0.25 < \epsilon < 0.50$ ) an increase of the ratio  $h_s/\bar{h}$  with increasing  $\theta$  is found, with a much smaller scatter than present in Figure 7.
- (ii) For larger values of  $\epsilon$ , especially for  $\epsilon > 1.00$ , no relationship between  $h_s/\bar{h}$  and  $\theta$  is found.

In Figure 10 all data for  $\epsilon < 0.50$  (in total 18 points out of 78 data obtained) are presented. A linear regression line was determined through these 18 points which reads:



$$\frac{h_s}{h} = 1.292 + 0.037 \theta \quad (5)$$

The correlation coefficient is about 0.72, implying a significant result. The above relationship is supposed to provide a fair estimate of confluence scour in the Jamuna River.

#### 4. Discussion

The analysis of the available field data from the Jamuna River has resulted in a preliminary design curve for confluence scour in this river. The following remarks can be made:

##### (i) Estimation of $\epsilon$

To estimate  $Q_L$  and  $Q_R$  and thus  $\epsilon$ , use was made of Equation 3 (and 4). As can be observed in Figure 8, the scatter in this relationship is considerable. Consequently, the estimate of  $\epsilon$  has only a limited accuracy. This is also illustrated by the fact that during the further analysis anabranches were present with almost the same average depth, but with quite different width.

##### (ii) Comparison with Ashmore and Parker (1983)

Ashmore & Parker (1983) analyzed confluence scour in gravel-bed rivers. Here data of a sand-bed river with a substantially gentler slope are considered. In the Figures 9 and 10 the relationships obtained for gravel-bed rivers are given also. The present data for the Jamuna River for value of  $\epsilon < 0.50$  are on the average lower than gravel-bed data. For  $\epsilon > 0.75$  no definitive conclusion can be drawn. It should be remarked that from an analysis from cross-sectional data (see RPT/NEDECO/BCL (1987)) it was observed that  $h_{\max}/\bar{h}$  in a cross-section is a

maximum about 3. For values of  $\epsilon$  approaching 2, one would expect that  $h_s/\bar{h}$  is as a maximum about 6. This is substantiated by the provided data.

##### (iii) Influence of suspended load

A possible explanation of the smaller confluence scour depths found for the Jamuna River, in comparison to the gravel-bed rivers studied by Ashmore & Parker (1983), may be the influence of suspended load. Because at present no model is available that explains the occurrence of confluence scour, it is not possible to check whether scour depths are smaller when suspended load is the dominant mode of transport. The effect of suspended load on "local scour", however, is probably a reduction of the scour depth. The data for the Jamuna River (see Figure 9, especially the graphs for  $0.00 < \epsilon < 0.25$  and  $0.25 < \epsilon < 0.50$ ) do suggest that confluence scour is most distinct for small discharges in the anabranches. For larger discharges the value of  $h_s/\bar{h}$  is smaller for the same value  $\theta$ . This is in line with the smaller values of  $h_s/\bar{h}$  found here in comparison with the gravel-bed river data.

#### 5. Conclusions

The Jamuna field data that were processed in the present study provide a fair relationship between the dimensionless scour depth and the angle of incidence of the anabranches. The general character of this relationship is the same as derived by Ashmore & Parker (1983) for gravel-bed rivers, notably an increase in dimensionless scour depth for increasing angle of incidence. Confluence scour depth found here, are smaller than observed in gravel-bed rivers. There are some indications that this is caused by suspended load becoming the dominant mode of transport for higher stages in the Jamuna River.

## 6. Acknowledgements

The present investigations are being carried out within the framework of the Jamuna Bridge Appraisal Study. The study is being funded by the United Nations Development Programme (UNDP) through the World Bank. The Jamuna Multi-purpose Bridge Authority (JMBA) is furthering the studies for and the construction of the bridge. The present study is being carried out by Rendel, Palmer and Tritton, NEDECO and Bangladesh Consultants Ltd. The soundings of the Jamuna River have been done by the Bangladesh Inland Water Transport Authority. The processing of the sounded confluence data was done by Messrs. P.J. de Boer, F. van Zijl and L. Broersma. Publication of this Paper has been kindly permitted by both the World Bank and the Jamuna Multi-purpose Bridge Authority.

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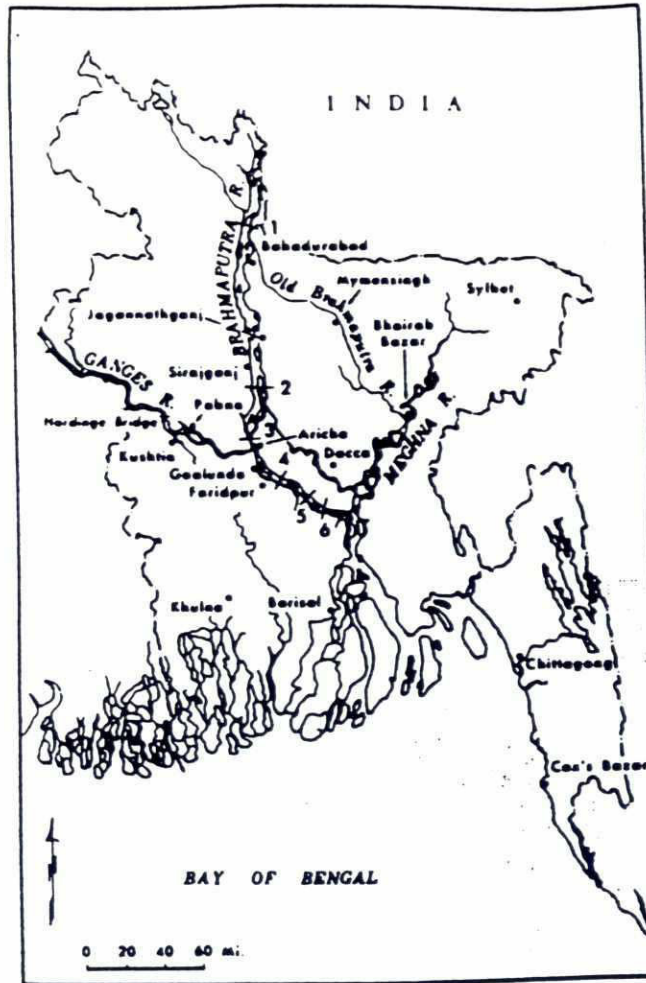


Figure 1 Index map of Bangladesh (Coleman , 1969)

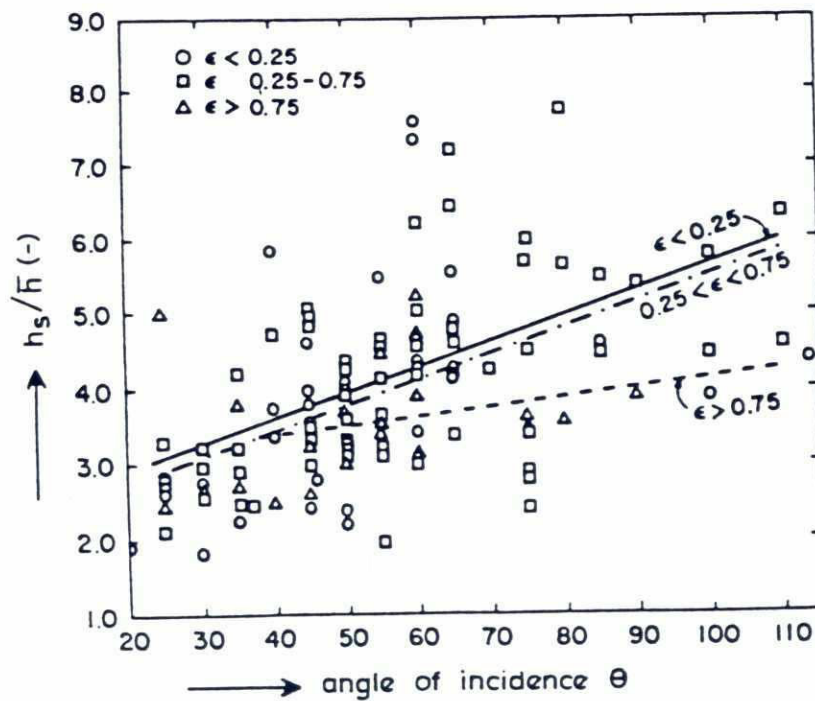


Figure 2 Confluence scour for coarse-bed rivers (Ashmore and Parker, 1983)



Figure 3 Satellite image of lower reach Jamuna River in  
Bangladesh in February 1977  
(approximate scale 1 : 1,000,000)



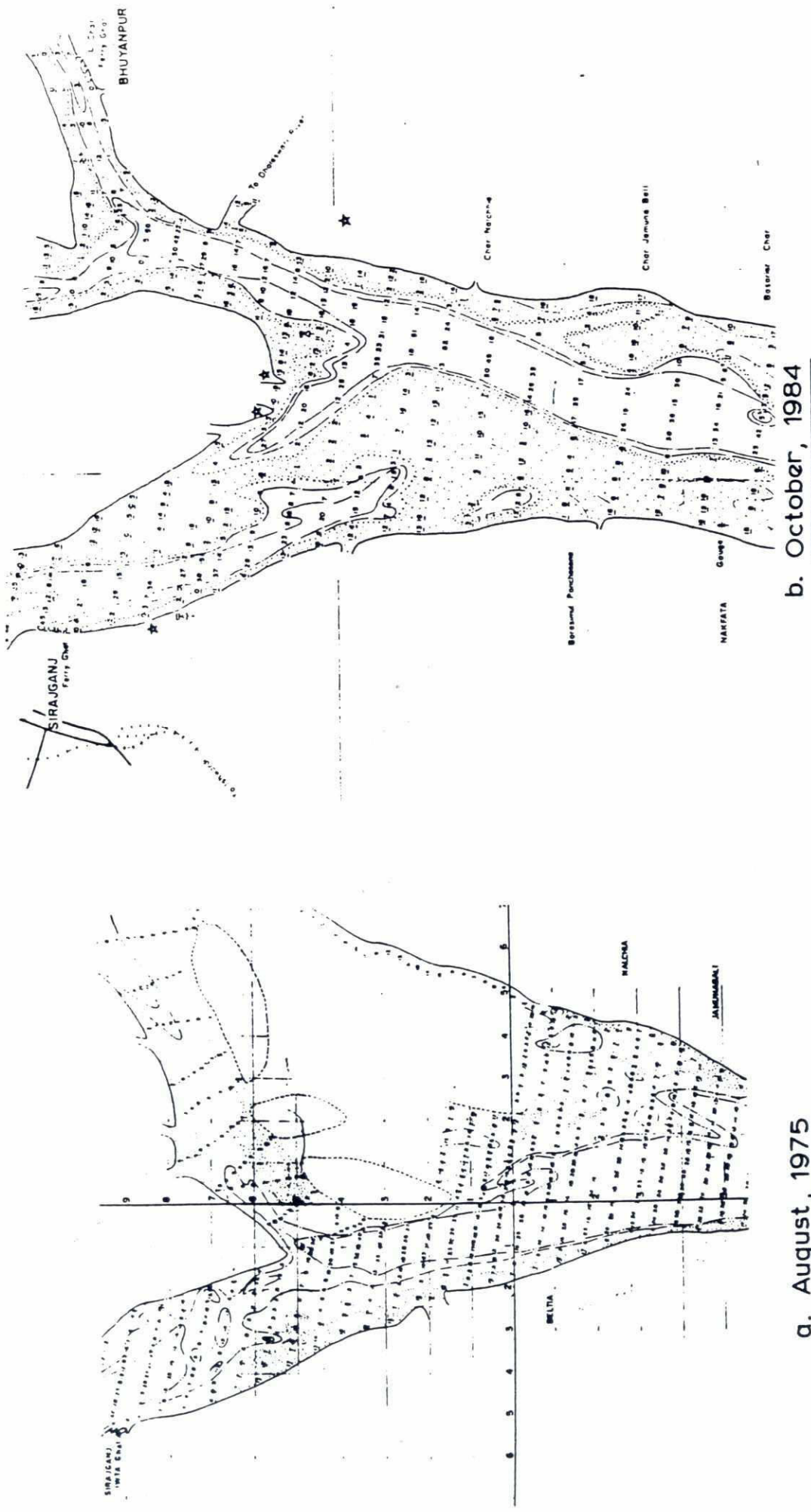


Figure 4 Parts of sounding maps of BIWTA showing confluence scour in Jamuna River (approximate scale 1:100,000).

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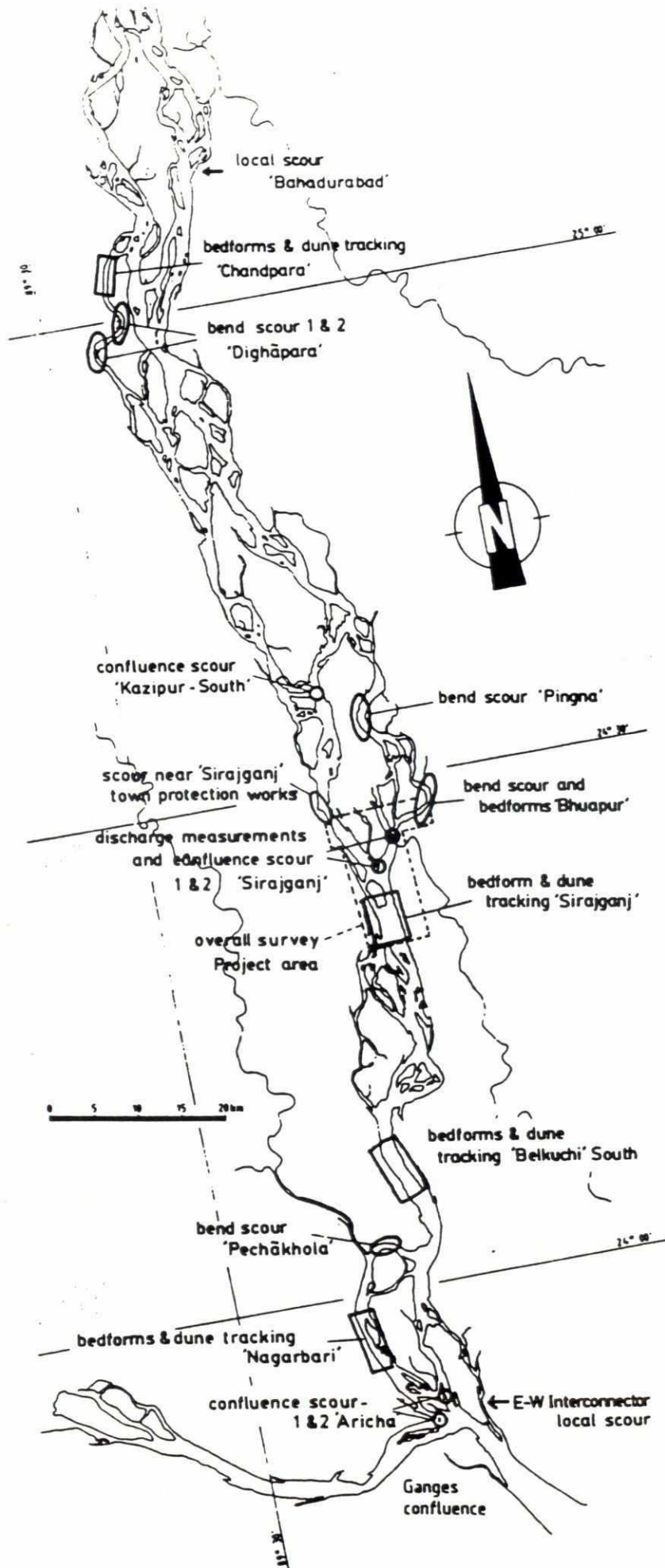


Figure 5 Location of special surveys 1987 on the Jamuna River



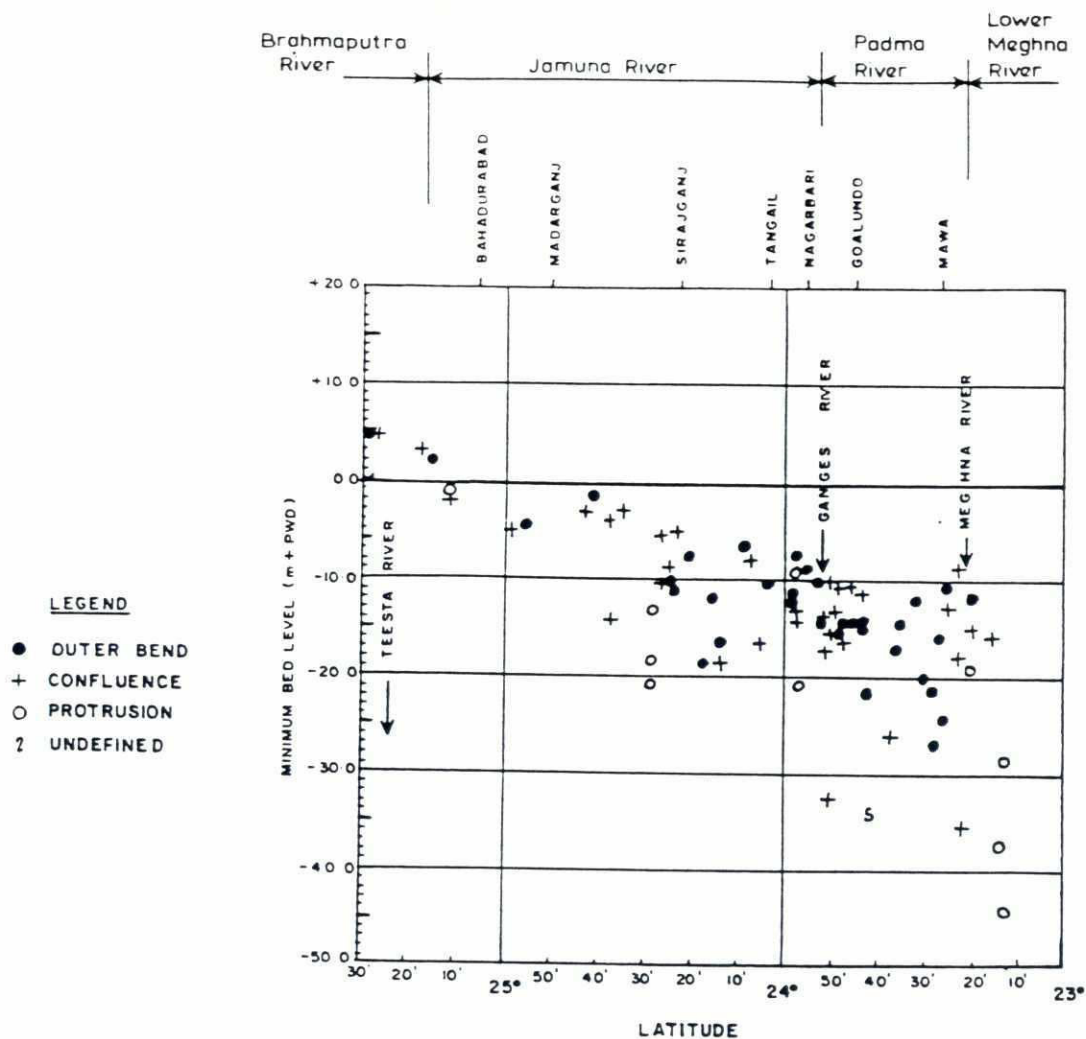
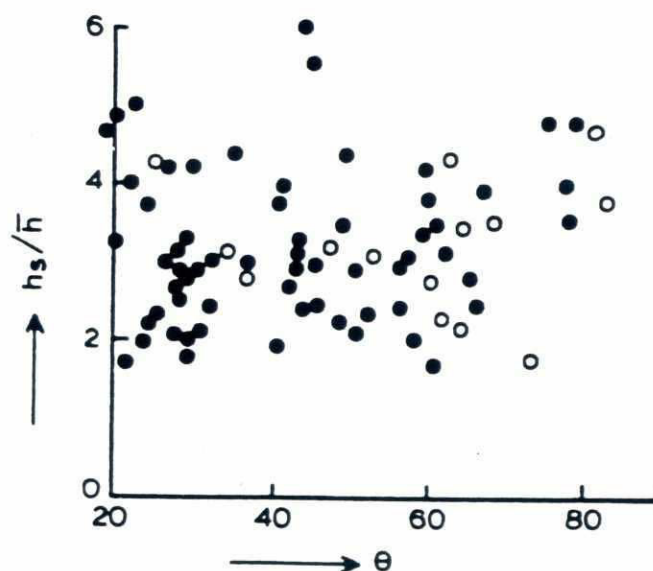


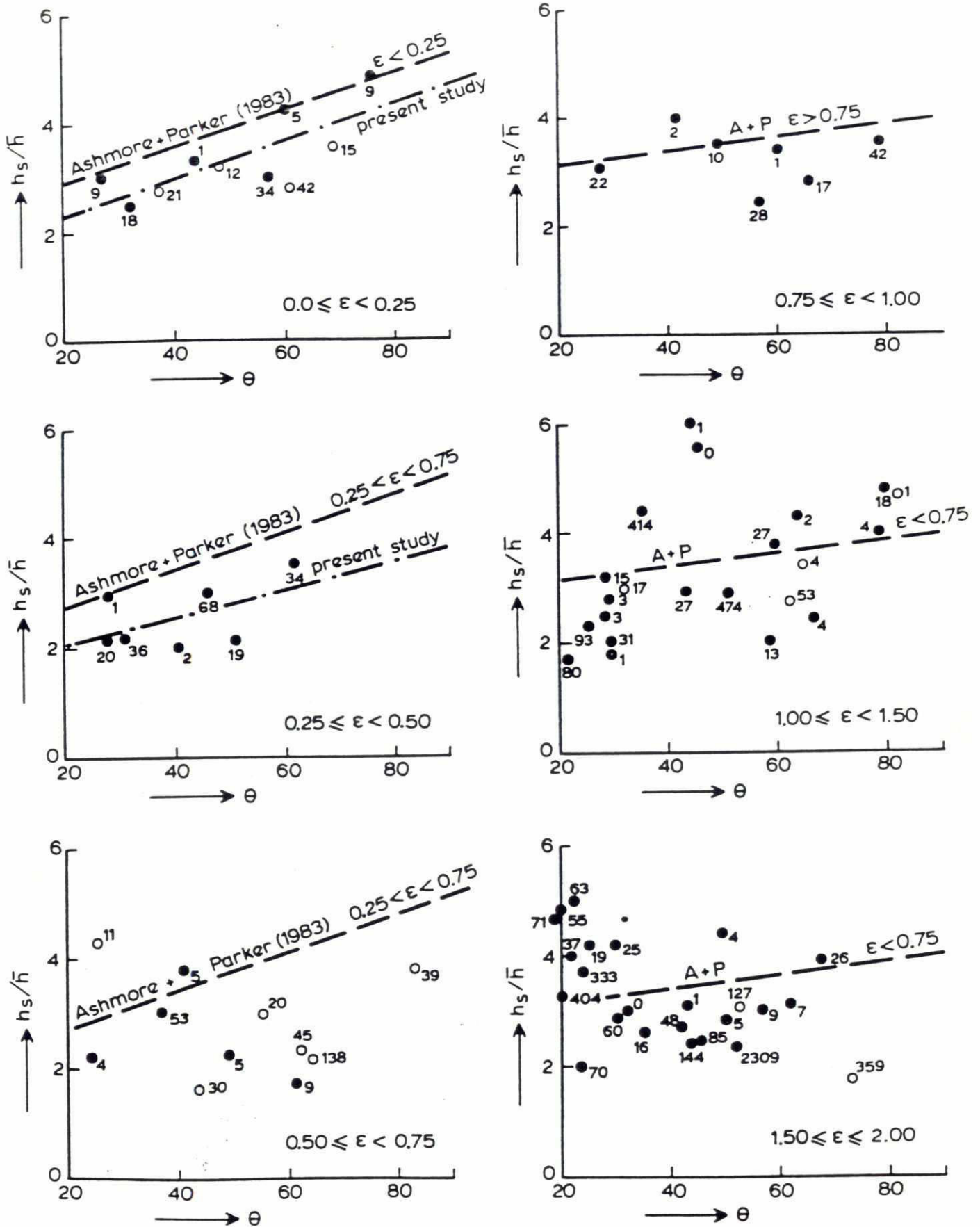
Figure 6 Maximum depths observed in the Jamuna River obtained from BIWTA sounding maps.



LEGEND

- historical data
- special survey '87

Figure 7 Confluence scour Jamuna River, all data.



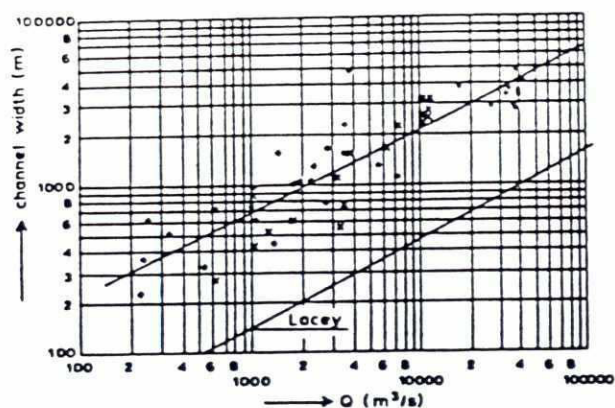
**LEGEND**

- historical data
- special survey '87

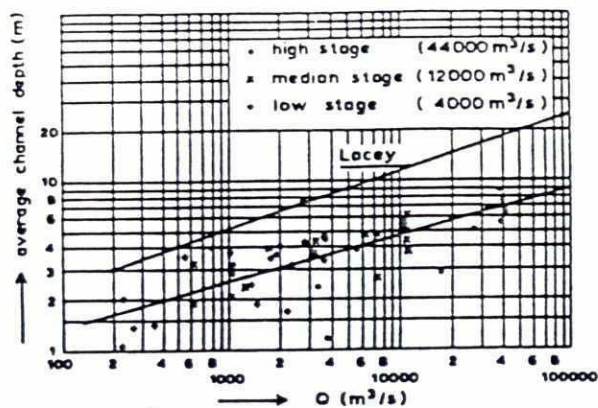
Figure 9 Confluence scour Jamuna River, separated according to ratio of discharge of confluencing channels.



29a



a. width versus discharge



b. average depth versus discharge

(a) width versus discharge

(b) average depth versus discharge

Figure 8 Tentative at a station relationships Jamuna River  
(Klaassen and Vermeer, 1988)

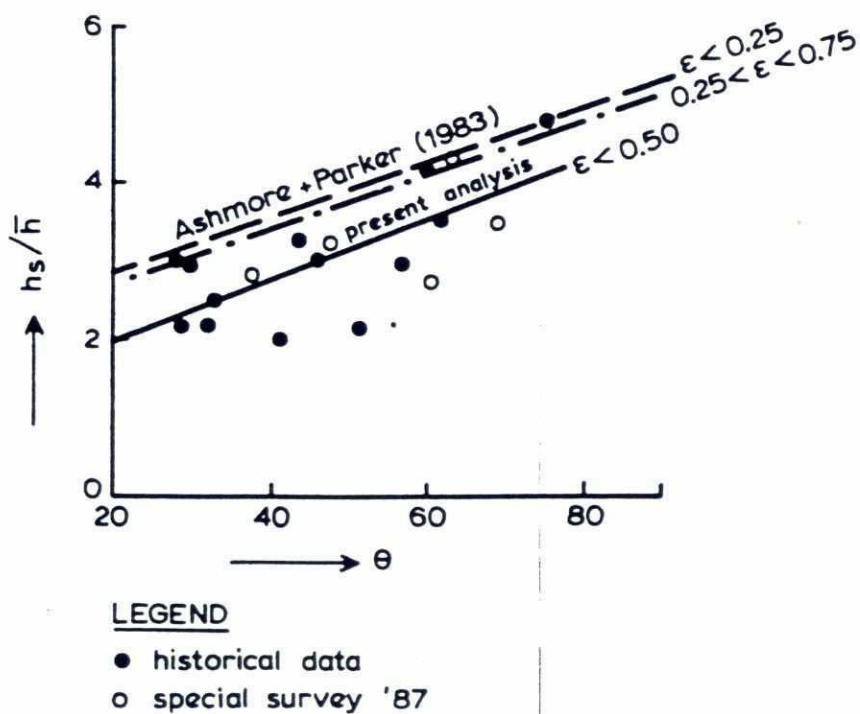


Figure 10 Tentative design curve confluence scour Jamuna River.

## 6 Morphological Modelling



## Paper 6.1

KLAASSEN, G.J. (1990), On the scaling of braided sand-bed rivers, *In: Shen, H.W. (Ed), Movable Bed Physical Models, Kluwer. Academic Publishers, pp. 59-71.*

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## ON THE SCALING OF BRAIDED SAND-BED RIVERS

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**ABSTRACT.** A scaling method is described that was developed recently to design a physical scale model of the Jamuna (= Lower Brahmaputra) River in Bangladesh. The model investigation is related to a proposed bridge across this braided river. The scaling method aims at a reproduction of the braiding pattern in order to study the effect of the proposed bridge on the channel characteristics. The scaling method itself, possible scale effects and some other aspects in designing and running such a model are discussed.

### 1. INTRODUCTION

More and more, mathematical models are taking over for problems where, in the past, scale models were usually applied. This holds also for certain types of movable bed problems, notably for rivers with fixed bank (see e.g. Struiksma, 1981). Certain movable bed river problems, however, cannot yet be tackled with mathematical models, notably problems related to channels where the banks are not fixed (regime channels) and channel patterns. In this paper a scaling procedure is discussed which was developed for a braided river, where reproduction of the channel pattern was the most important criterion. It is shown here that this results in a scaling procedure with a very limited degree of freedom; consequently considerable scale effects may be introduced.

The reference problem is a proposed bridge across the Jamuna River in Bangladesh (see Figure 1).



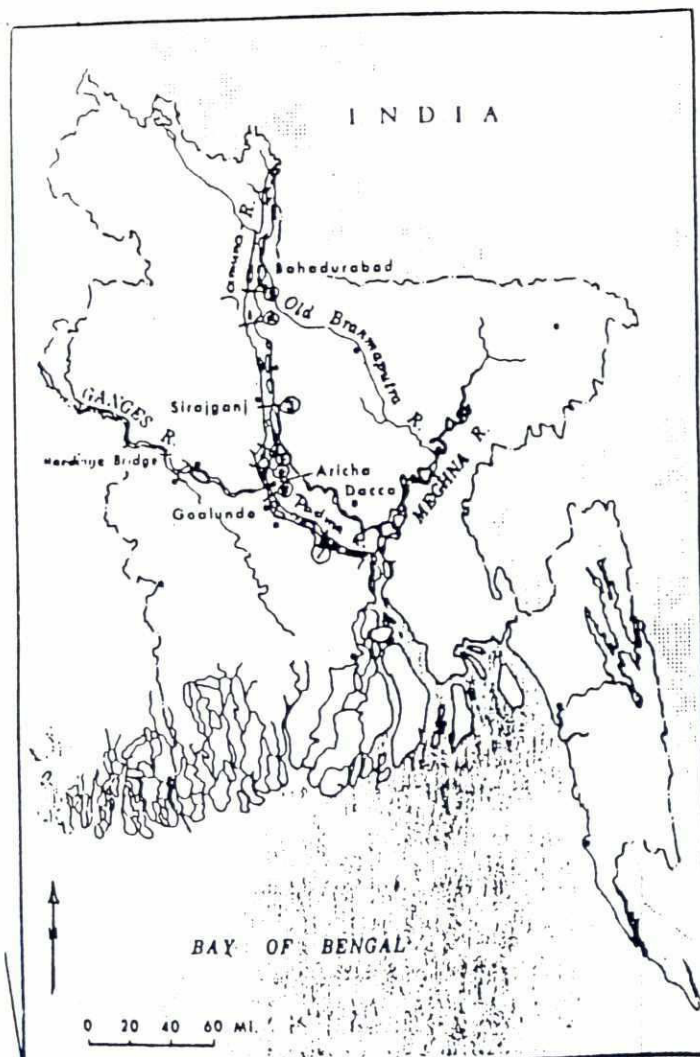


Figure 1. Map of Bangladesh with the Jamuna River.

The Jamuna River is the lowest reach of the Brahmaputra River. Figure 2 provides a 1:1,000,000 scale satellite image of the Jamuna River and of its confluence with the Ganges River. As can be observed, the Jamuna River is a braided river, with on the average three braids upstream of Sirajganj (near the proposed bridge site) and about two braids more downstream. The river bed can be characterized as a flat sand bed in which a number of deep channels are present. The shallow bars in between the channels are locally called "chars".

The main characteristics of the Jamuna Bridge are the following:

- total length in Bangladesh ~ 300 km
- maximum total width ~ 15 km
- number of main channels 2-3
- width of main channels during floods ~ 2.5 km
- average depth main channel (h) ~ 8 m
- average depth during low flow conditions 3 m
- average depth above chars during floods 1-2 m
- Chézy coefficient (average 1) ~ 70  $\text{m}^{1/2}/\text{s}$
- Chézy coefficient floods ~ 90  $\text{m}^{1/2}/\text{s}$
- $D_{50}$  ~ 200  $\mu\text{m}$
- slope (i) ~  $0.7 \cdot 10^{-4}$
- $\theta = h_i / \nu D_{50}$  (floods) ~ 1.7
- average velocity during floods ~ 2  $\text{m}/\text{s}$
- maximum total discharge ~ 100,000  $\text{m}^3/\text{s}$

For more details on the river characteristics of the Jamuna River, reference is made to RPT/Nedeco/BCL (1987) or Klaassen et al. (1988).

It was proposed to do model investigations in a movable bed scale model. The purpose of this scale model investigation would be:

- to study the effect of the river training works on channel patterns;
- to identify "worst" channel patterns (for flood and low-flow conditions) for the river training works;
- to study the effect of gradual shifts of river upstream on channel patterns near bridge and training works.



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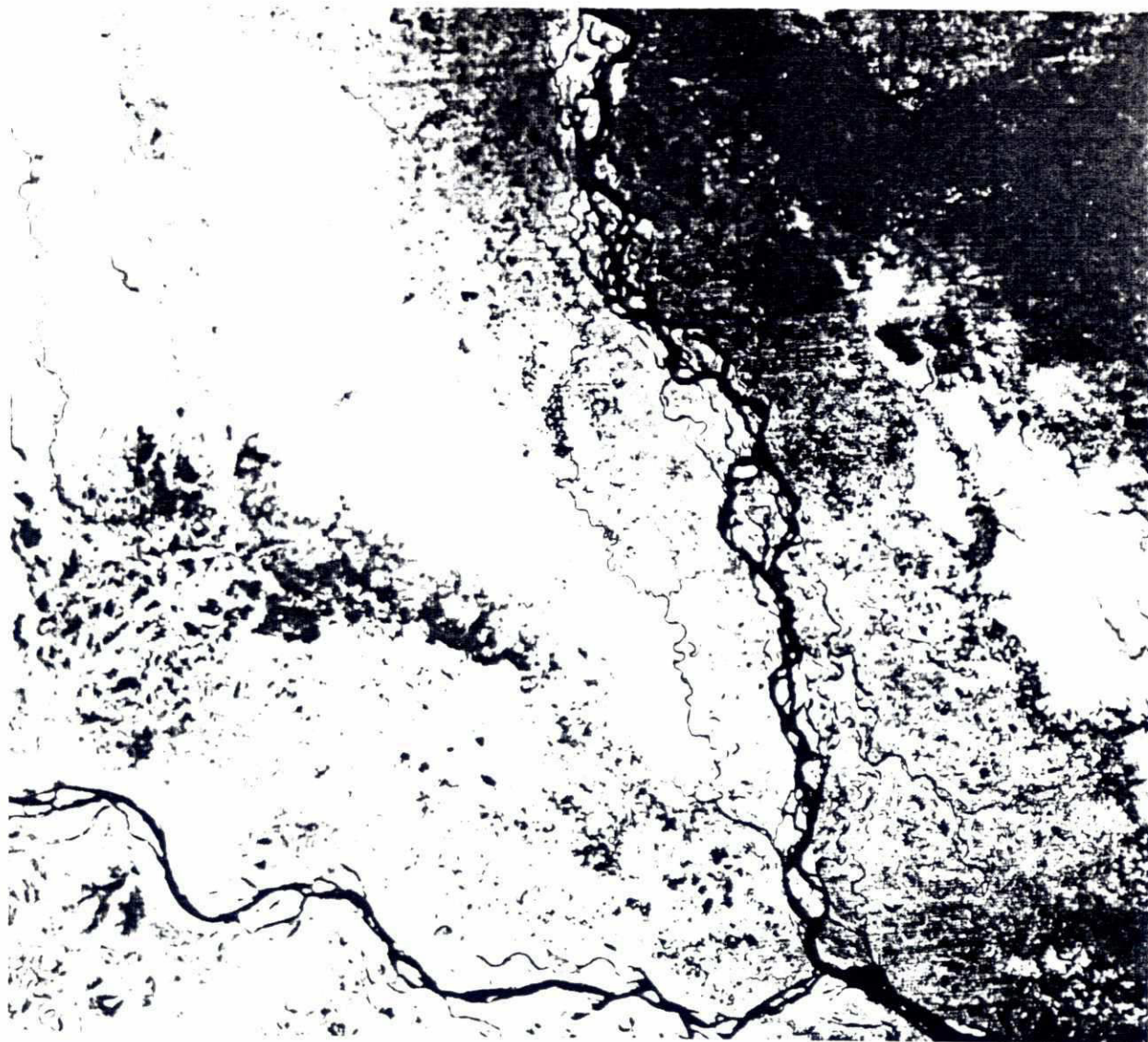


Figure 2. Satellite image of Jamuna River, February 2, 1973 (scale approximately 1:1,000,000)

Alternatively, historical channel patterns could be used to arrive at design conditions for the river training works required. From the results of an exploring model test, it was concluded, however, that the presence of a bridge constricting the river width has a significant effect on the channel patterns.

It should be stressed that the scale model discussed here is only one element of a number of research tools applied in this study. In addition the following investigations are foreseen:

- a 1-d mathematical model of the morphology of the Brahmaputra River to study long-term changes resulting from embankments, major diversion, dams, etc.;
- a 1-d mathematical model to study constriction scour, because the bridge opening will probably be only about 6 km;
- a 2-d mathematical model of the depth-averaged flow field underneath the bridge; this model will use the results of the movable-bed scale model as input and a curvilinear coordinate system will be used (Wijbenga, 1985);
- detail scale models of structures (guide bunds, groynes) that will use the results of the mathematical model of the flow field as boundary conditions;
- an extensive analysis of historical river data;
- an extensive study of satellite images to determine channel characteristics and celerities of channel pattern changes over the period 1973-1987;
- extensive hydrographic field measurements during the flood period in 1987.

## 2. SCALING PROCEDURE

### 2.1. Starting Points

The following parameters were considered to be important and less important, respectively, for the present scale model investigations:

- Important:
- channel patterns (braiding with 2-3 channels)
  - roughness condition (reproduction of eddies)
  - width of channels reproduced on length scale
  - ratio curvature/channel width

- Less important:
- Re-condition ( $Re_m > \Delta 10^3$ )
  - Fr-condition ( $Fr_m < 1$ )
  - ideal velocity-scale
  - mobility criterion (?)
  - distortion
  - horizontal celerity of channels

Other requirements:

- not too large (costs, celerity)
- high celerity of horizontal channel changes ("fast" model)
- sediment transport recirculated but measurable



## 2.2. Background of Scaling Procedure

The most important criterion which has to be used to derive scale rules for the present model is the reproduction of the channel patterns. Thus in the movable bed model anyhow a braided pattern should be present with two to three braids. No theoretical method is available in literature that allows to derive such a scale rule in a "scientific" way. However, there is some information in the literature that may be useful.

According to Leopold, Miller and Wolman (1964) a river is on the threshold between meandering and braiding (so on a transition from a single channel pattern to a multiple channel pattern) when:

$$i = \text{constant} \cdot Q^{-0.44} \quad (1)$$

where  $i$  = stream gradient (-) and  $Q$  = annual flood ( $\text{m}^3/\text{s}$ ). Channel patterns of braided streams have been studied by Howard et al. (1970). They find that the numbers of braids  $n$  is determined by:

$$n = 0.58 R_q^{-0.19} i^{0.41} \cdot Q^{0.25} \quad (2)$$

where  $R_q$  = ratio of mean annual flood to the mean annual discharge. Equation 2 can also be written as:

$$i = \text{constant} \cdot R_q^{+0.46} \cdot Q^{-0.60} \quad (3)$$

an expression which has some similarity with Equation 1. Struiksmas and Klaassen (1988) provide theoretical evidence for such a type of formula ruling channel patterns. They also indicate that from theoretical reasoning it follows that probably the ruling equation for the channel pattern reads as:

$$\frac{i}{\Delta D} = \alpha \cdot (Q)^\beta \quad (4)$$

where  $\alpha$  and  $\beta$  still have to be defined and do not need to be constant over the full range of  $Q$ . This is in line with Ferguson (1984), who shows that  $i/D$  is preferable over  $i$  alone. Equation 4 is indicated in a schematic way in Figure 3:

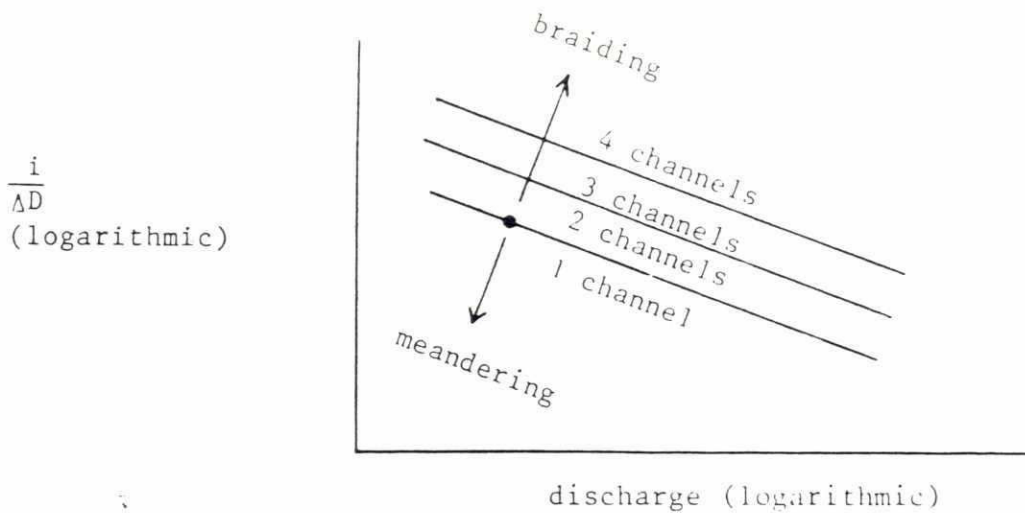


Figure 3. Channel pattern classification.

### 2.3. Resulting Scaling Procedure

Based on the observations made in Section 2.2, a scaling procedure can be derived that will result in a correct reproduction of the channel pattern. From Equation 4 it follows that:

$$n_i/\Delta D = (n_Q)^\beta \quad (5)$$

The relative density and the particle diameter  $D$  of the model are selected beforehand (see also below). Thus for a given slope  $i_m$  (and thus  $n_i$ ) the value of  $n_Q$  can be obtained from Equation 5 (of course the only problem is the value of  $\beta$ ; see for this aspect Section 4). From the prototype value of the total discharge and from  $n_Q$  the total discharge in the model  $Q_m$  is obtained.

So applying the condition that the channel pattern (number of braids) in prototype and model should be similar, results (for given  $D_m$ ,  $\Delta_m$  and  $i_m$ ) directly into  $Q_m$ . Given the total discharge in the model, all characteristics of the channels in the model are determined because these are self-forming regime channels. So the length scale, the depth scale, the distortion and all other scales cannot be influenced anymore! Most probably this will lead to scale effects. These effects are discussed in more detail in Section 3.

### 3. SCALE EFFECTS

In this Section it is attempted to estimate beforehand to what extent deviations from scale conditions will occur when a scale model is used, designed to reproduce the channel pattern correctly. It is quite obvious that the deviations are related to the degree of scaling down of the prototype. The larger the model, the smaller will the scale effects be. In designing



such a model there are, however, a number of practical limitations. For the present model investigation these limitations are listed below:

- model dimensions as a maximum at about 50 x 20 m<sup>2</sup> because of costs, but also because of the required celerity of the model;
- model material sand, because light weight material (that would be preferable) is too expensive in large quantities;
- model material size should be about 200  $\mu$ m (readily available within Delft Hydraulics), and maximum horizontal celerities according to Hickin and Nanson (1983).

### 3.1. Distortion

The channel characteristics in the model are completely determined by the model discharge. To evaluate to what extent deviations from scale conditions may be expected, it is necessary to be able to estimate these channel characteristics before-hand. In first approximation the Lacey equations for self-forming channels can be used. According to Lacey (1958) the average width  $B$  and the average depth  $\bar{h}$  of self-forming channels are related to the discharge according to:

$$B : : Q^{1/2} \quad (6)$$

$$\bar{h} : : Q^{1/3} \quad (7)$$

Accepting these equations for the time being as a basis for the evaluation intended here, the width and depth scale are related to the discharge scale via:

$$n_B - n_Q^{1/2} \quad (8)$$

$$n_{\bar{h}} - n_Q^{1/3} \quad (9)$$

An estimate of the model distortion  $r$  is obtained via

$$r - \frac{n_B}{n_{\bar{h}}} - n_Q^{1/6} \quad (10)$$

The above implies that for a width scale of  $n_B \approx 10^3$  (preferably considering the available space and other reasons), the following scales are approximately found for the other scales:

$$n_Q = 10^6 \quad (Q_m = 100 \frac{1}{s})$$

$$n_h = 10^2 \quad (h_m = 0.08 \text{ m during floods})$$

$$r = 10$$

So this scaling procedure results in a considerably distorted model. (It should be noted here that first experience from the model shows that the water depth is smaller than estimated here, resulting in a slightly less distorted model).

### 3.2. Froude Number

For a model discharge of about 100 l/s a fair reproduction of the channel pattern is probably obtained for a model slope of about  $8 \cdot 10^{-3}$  (see Section 4). Assuming the Chézy coefficient  $C_m$  in the model to be about  $25 \text{ m}^{1/2}/\text{s}$ , the average velocity in the channels is estimated to be about 0.4 m/s. This implies that the Froude number in the model is on the average in the order of 0.5 while in the prototype  $Fr = 0.2$ . Locally in the model even higher values of the Froude number may be found. Such high Froude numbers will certainly affect the flow pattern reproduction (see Struiksmā, 1981) and consequently induce scale effects. For this reason in addition to the present scale model a two-dimensional (depth-average) mathematical model will be used to determine the flow field (given a certain bed topography with channels and bars). The effect of the deviation of the Froude condition on the channel patterns cannot be evaluated beforehand, but for the time being this is assumed to be limited.

### 3.3. Roughness Condition

According to the roughness condition, that reads as:

$$n_C^2 = \frac{n_L}{n_h} \quad (-r) \tag{11}$$

the model distortion  $r$  should correspond to  $n^{2C}$ . Accepting  $C_m = 25 \text{ m}^{1/2}/\text{s}$  as a first estimate (resulting in  $n_C = 3.5$ ), it follows that the roughness condition is reasonably well fulfilled. For rivers with smaller Chézy coefficients during floods this would be less favorable.

### 3.4. Bed Topography

As shown by Struiksmā et al (1985) an important parameter for the reproduction of the bed topography in curved alluvial channels is the parameter  $\lambda_s/\lambda_w$ , where  $\lambda_s$  and  $\lambda_w$  are defined via:





$$\lambda_s = \frac{1}{\tau^2} h \left[ \frac{B}{h} \right]^2 A \sqrt{\theta} \quad (12)$$

$$\lambda_w = C^2 h / 2g \quad (13)$$

where A = empirical constant. The scale factor of  $\lambda_s/\lambda_w$  is derived as:

$$n_{\lambda_s/\lambda_w} = n_C^{-2} \cdot n_B^2 \cdot n_h^2 \cdot n_\theta^{1/2} \cdot n_A \quad (14)$$

According to the present experience of Delft Hydraulics  $n_A = 0.5$ . Introducing the other scale factors in Equation 14, results in  $n_{\lambda_s/\lambda_w} = 5$ . This deviation is mainly due to the model.

Consequently it may be expected that scale effects will be present in the reproduction of the bed topography which in turn may result in deviations in the reproduction of the channel characteristics (notably the point bar dimensions).

It should be noted that the value of  $n_{\lambda_s/\lambda_w}$  is rather high. This may indicate that the model channel dimensions are not estimated correctly here.

### 3.5. Curvature of Channels

The reproduction of the curvatures of the channels can also be evaluated by applying relationships from geomorphological sources (Leopold et al, 1964). From these the following scale rules can be derived:

$$n_{\text{meander length}} = n_B^{1.01} \quad (15)$$

$$n_{\text{radius of curvature}} = n_{\text{meander length}}^{1.02} \quad (16)$$

which would result in the following scale rule for the ratio radius of curvature/channel width:

$$n_{\text{radius of curvature}/B} = n_B^{.03} \quad (17)$$

which would result in a value of 1.23 for the present example.

Summarizing the above considerations, it should be stated that a model scaled according to the procedure outlined in Section 2.3 is subject to considerable scale effects. Therefore such a model should be used for comparative research only.

#### 4. ADDITIONAL REMARKS

The following additional remarks can be made:

##### 4.1. Selection of Model Slope and Discharge

The main problem in designing a model according to the procedure outlined in Section 2, is the fact that the Equation 4 is not completely known. This implies that the process of obtaining values for  $i_m$  and  $Q_m$  (given  $D_m$  and  $\Delta_m$ ), that result in a fair reproduction of the channel pattern in the Jamuna River, necessarily has to involve trial and error. As it is very cumbersome and time-consuming to change the slope of a model with approximate dimensions of  $50 \times 20 \text{ m}^2$ , for the time being the slope of the model has been selected to be about  $8 \cdot 10^{-3}$ . This slope was determined from an analysis of all data on channel patterns available in literature, after having selected a model discharge of about 100 l/s as desirable. At present tests are carried out with different model discharges and the resulting channel patterns are determined with aerial photography. Next braiding parameters are estimated (see Howard et al, 1970) and the model discharge (and thus  $n_Q$ ) will be selected on the basis of the comparison of these characteristics with the characteristics of the Jamuna River itself.

##### 4.2. Varying Discharge

In the preceding sections the model discharge was discussed as if it should have a constant value. From preliminary observations in a model, that has been designed according to the scaling procedure discussed before (see Figure 4), it appears that a succession of high and low flows results in a better reproduction.

##### 4.3. Lightweight Material

If, indeed, Equation 4 is preferable over Equation 1, it follows that a less serious deviation from the Froude scale would result if light weight material could be used. This stresses the need for not too expensive light-weight material!

##### 4.4. Testing of Similarity

In the end of 1987 the results of the model tests at Delft Hydraulics can be used to evaluate the possibilities of a model, scaled according to the procedure outlined above, to simulate prototype conditions.





Figure 4. Overview of movable bed model of the braiding Jamuna River during first tests.

## 5. ACKNOWLEDGEMENTS

The scale model investigation discussed here is part of the Jamuna Bridge Appraisal Study Phase II, which is funded by UNDP, the World Bank being the executive agency. The study is carried out by the combination RPT (Rendel, Palmer and Tritton, U.K.)/ NEDECO (Netherlands Engineering Consultants)/BCL (Bangladesh Consultants Ltd.); the JMBA (Jamuna Multipurpose Bridge Authority, Bangladesh) is the client. Delft Hydraulics is one of the partners of NEDECO for this study.

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## Paper 6.2

OLESEN, K.W., (1992), 2-D mathematical modelling of morphological processes in the Brahmaputra River, *Proc. Nile 2000, Cairo, Egypt, 4-5-1 - 4-5-18pp.*

## 2-D MATHEMATICAL MODELLING OF MORPHOLOGICAL PROCESSES IN THE BRAHMAPUTRA RIVER

by

Kim Wium Olesen, Danish Hydraulic Institute

### Summary

The present paper describes a two-dimensional curvilinear morphological model for river application. The model includes, among other features, a description of helical flow and space lag between the flow and suspended load transport.

The model is applied to the Brahmaputra River, which is a large braiding river. Comparison of model simulations with field data shows good agreement. The model is currently being used to investigate the influence of bank erosion on outer bend scour. Preliminary results show that this influence may be significant.

### 1. Introduction

This paper deals with two-dimensional morphological modelling of the Brahmaputra River in Bangladesh. The modelling has been undertaken as part of a feasibility study for protection of the right embankment of the river. The study is ongoing and the results presented here are therefore only preliminary.

The Brahmaputra River is one of the largest in the world. It rises in Tibet and flows eastward along the northern range of the Himalayas for approximately 1500 kilometres, before it turns south and flows into Assam in India. It enters Bangladesh about 250 kilometres north of its confluence with the Ganges River. Inside Bangladesh Brahmaputra has a catchment area of about 500,000 km<sup>2</sup>. The average recorded daily flow is 19,400 m<sup>3</sup>/s, which is equivalent to an annual runoff of 1220 mm from its entire catchment. The five months with the highest flow have an average discharge of 35,500 m<sup>3</sup>/s and the wettest, July, has an average flow of 45,300 m<sup>3</sup>/s. The averaged flow rate in the dry period is 5,600 m<sup>3</sup>/s.

Opinions about the annual sediment load in the river differ. One estimate suggests that the averaged annual bed and suspended load of the Brahmaputra River are about 387·10<sup>6</sup> tonnes and 100·10<sup>6</sup> tonnes, respectively.

The Brahmaputra is a braided river without fixed banks and with frequently shifting channels short-term channel migration can be quite drastic with annual rates of movement as high as 800 m being common. The bank erosion process is a complex mechanism and is influenced by a number of factors. In Bangladesh the total river width varies between 5 km and 15 km. The river cross-section is highly irregular and skewed, and the deep channel may reach a depth of 40 m. The Brahmaputra River within Bangladesh is shown in Figure 1.

Along the western bank of the Brahmaputra a 240 km long earth embankment has been constructed, which protects a large area from annual flooding. Bank erosion attacks frequently cause breaching of the embankment, and the embankment has been retired westward at several locations. About 100 km of retired embankment has been constructed during the past two decades.



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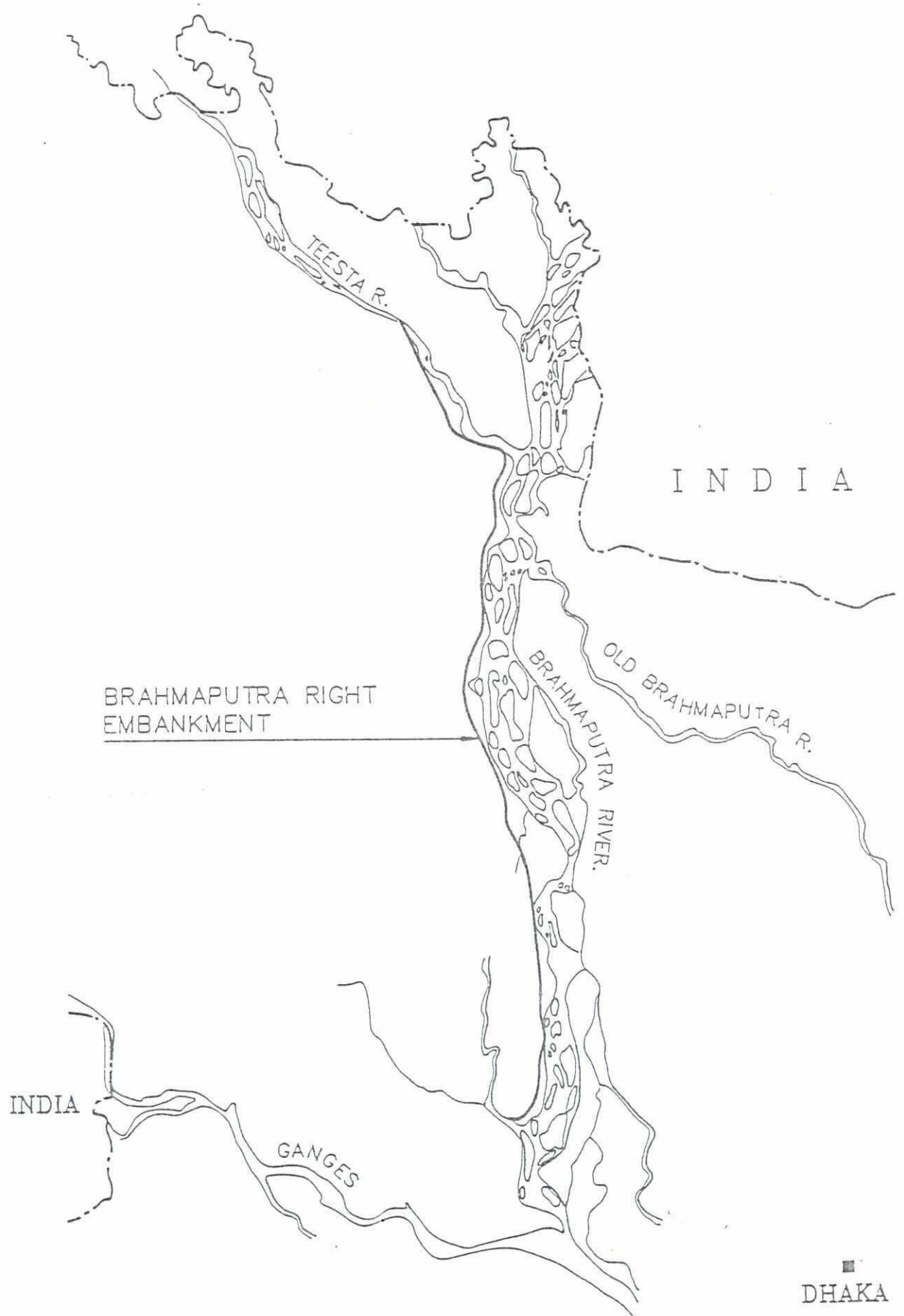


Figure 1 The Brahmaputra River System.

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Bank erosion is also causing severe problems at several townships and important infrastructures along the river.

In an increasing number of river engineering studies, mathematical modelling has replaced the role of physical modelling. The modelling of the flow in complex rivers can be carried out relatively reliably, whereas the detailed modelling of sediment transport and morphological processes are new techniques which require careful calibration and interpretation of the simulation results. Physical modelling of braided rivers with movable beds and banks unavoidably gives rise to scale effects, see Struiksmā et al., 1985, hence physical modelling of morphological processes in braiding rivers cannot stand alone.

In the present study a multidisciplinary approach has been adopted where mathematical and physical modelling studies complemented by river morphological and geotechnical studies form the basis. The present paper deals with the aspects of the study which are related to two-dimensional mathematical modelling.

In the following the applied mathematical model will be described briefly, the model application including the data collection programme will be described and finally the conclusions will be summarized.

## 2. Model Description

### 2.1 General

The modelling study has been carried out using System 21 - Curvilinear which is a generalized mathematical modelling system for the simulation of the hydrodynamics of, and sediment transport in horizontal flows. The modelling system has the capability of utilizing both a rectangular and a curvilinear computational grid.

The modelling system is composed of a number of modules relevant to sediment transport and morphological studies in rivers:

- a hydrodynamic model
- a sediment transport model
- a bed form/flow resistance model (small scale morphology)
- a large scale morphological model

These features are described in general terms below.

All these model components can run simultaneously thus the system can describe the dynamic feedback from changing hydraulic resistance and bed topography to the flow in the river.

For pre-and post-processing of input and output a number of service programs have been developed such as:

- an elliptic curvilinear grid generator
- graphical software for contour and vector plots



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System 21 requires an orthogonal curvilinear grid. This is obtained with a grid generator which solves a set of elliptic partial differential equations using an implicit alternating direction iteration solution procedure. The advantage of using an orthogonal grid is that the partial differential equations, which describe the two-dimensional flow, become substantially simpler than if a general (non-orthogonal) curvilinear grid was applied. As a consequence the truncation errors of the finite difference approximation will be smaller (and accuracy better) in an orthogonal grid.

## 2.2 Hydrodynamic Model

The hydrodynamic model simulates the water level variation and flows. These are resolved on the computational grid covering the area of interest, once this has been provided with the bathymetry, bed resistance coefficients and hydrographic boundary conditions.

The hydrodynamic model has been under continuous development at the Danish Hydraulic Institute since 1970. It has undergone many changes and improvements, so that it now constitutes one of the most reliable and accurate mathematical modelling tools available for the modelling of flows in two horizontal directions.

The hydrodynamic model solves the vertically integrated equations of continuity and conservation of momentum (the Saint Venant equations) in two directions. The following effects can be included in the equations when used for river applications

- flow acceleration
- convective and cross-momentum
- pressure gradients (water surface slopes)
- bed shear stress
- momentum dispersion
- Coriolis forces
- wind forces
- flow curvature and helical flow

The curvature of the coordinate lines gives rise to additional terms in the partial differential equations for the flow. The equations solved in S21-Curvilinear are:

$$\begin{aligned} \frac{\partial p}{\partial t} + \frac{\partial}{\partial s} \left( \frac{p^2}{h} \right) + \frac{\partial}{\partial n} \left( \frac{pq}{h} \right) + 2 \frac{pq}{hR_n} + \frac{p^2 - q^2}{hR_s} \\ + g h \frac{\partial H}{\partial s} + \frac{q}{C^2} \frac{q \sqrt{p^2 + q^2}}{h^3} = RHS \end{aligned} \quad (1)$$

$$\begin{aligned} \frac{\partial q}{\partial t} + \frac{\partial}{\partial s} \left( \frac{pq}{h} \right) + \frac{\partial}{\partial n} \left( \frac{q^2}{h} \right) + 2 \frac{pq}{hR_s} + \frac{q^2 - p^2}{hR_n} \\ + g h \frac{\partial H}{\partial n} + \frac{g}{C^2} \frac{q\sqrt{p^2 + q^2}}{h^3} = RHS \end{aligned} \quad (2)$$

$$\frac{\partial H}{\partial t} + \frac{\partial p}{\partial s} + \frac{\partial q}{\partial n} + \frac{q}{R_s} + \frac{p}{R_n} = 0 \quad (3)$$

where

s,n are curvilinear coordinates  
 p,q mass fluxes in s and n direction, respectively  
 H water level  
 h water depth  
 g gravitational acceleration  
 C Chezy roughness coefficient  
 R<sub>s</sub>,R<sub>n</sub> radius of curvature of s and n lines, respectively  
 RHS the right hand side describing Reynold stresses

The equations are solved by implicit finite difference techniques with the variables defined on a space staggered computational grid.

Helical flow is a principal secondary flow phenomenon in rivers. Whilst it does not have any strong influence on the general flow pattern in rivers with large width-depth ratios, it has a significant influence on the sediment transport direction and hence the morphological changes in the river channel (see Olesen, 1987). The helical flow is, therefore, only calculated when larger scale morphology is modelled, as distinct from the small scale bedform features. It is the cause of bend scour and plays an important role in confluence scour, and in char build up and migration.

The helical flow pattern occurs in curved flows, especially in river bends. It arises from the imbalance between the pressure gradient and the centripetal acceleration exerted on the water moving along a curved path. Near the river bed the helical flow is directed towards the centre of flow curvature. The magnitude of the helical flow (i.e. the transverse flow velocity component) rarely exceeds 5 - 10 % of the main flow velocity in natural rivers. A typical helical flow pattern is shown in Figure 2.



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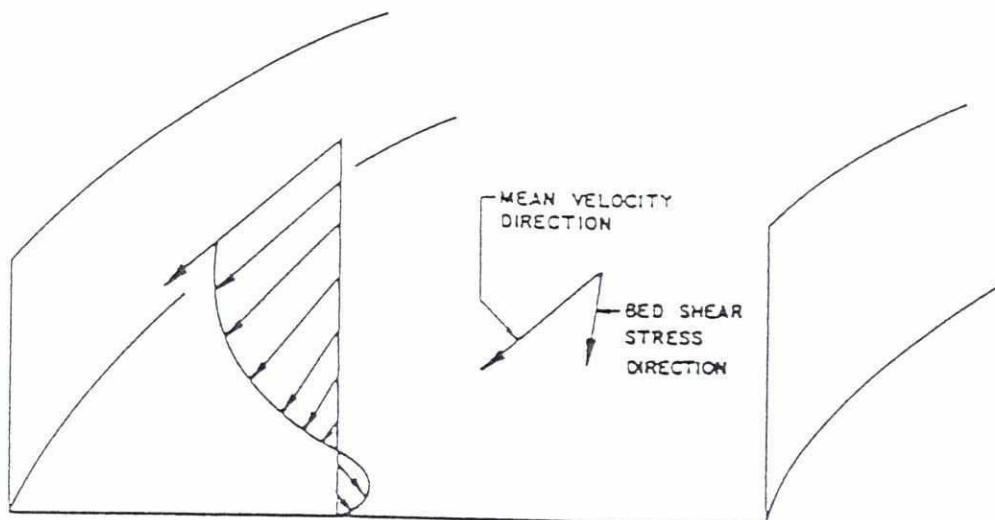


Figure 2 Helical flow in a river bend.

Assuming a logarithmic main flow velocity distribution and a parabolic eddy viscosity distribution the magnitude of the secondary flow can be shown to be proportional to the main flow velocity, the depth of flow and the curvature of the main flow streamlines. In System 21 - Curvilinear the streamline curvature is calculated explicitly from the depth integrated flow field. The gradual adaptation of secondary (helical) flow to changing curvature is accounted for by solving a first order differential equation along streamlines with the strength of the helical flow as the dependent variable (see also De Vriend, 1981). The strength of the helical flow is used to determine the direction of both bed and suspended load transport.

### 2.3 Sediment Transport

In connection with detailed (two-dimensional) mathematical modelling of sediment transport and morphology in rivers with large suspended load transports, it is necessary to distinguish between bed and suspended load in order to:

- simulate the dynamic development of bed form dimensions
- account for the effect of helical flow on the sediment transport direction

The relatively simple total load sediment transport formulas, such as Engelund & Hansen, 1967 and Ackers & White, 1973 can therefore not be used for river applications with System 21. The sediment transport models developed by Engelund & Fredsoe, 1982 and van Rijn, 1984a which distinguish between bed and suspended load, form the basis for the sediment transport description in System 21.

In System 21 both the magnitude and the direction of the sediment transport have to be calculated. The bed load transport rate is calculated directly with the Engelund & Fredsoe or the van Rijn model which relates the transport rate to the bed shear stress and the grain diameter (see Engelund & Fredsoe or van Rijn).

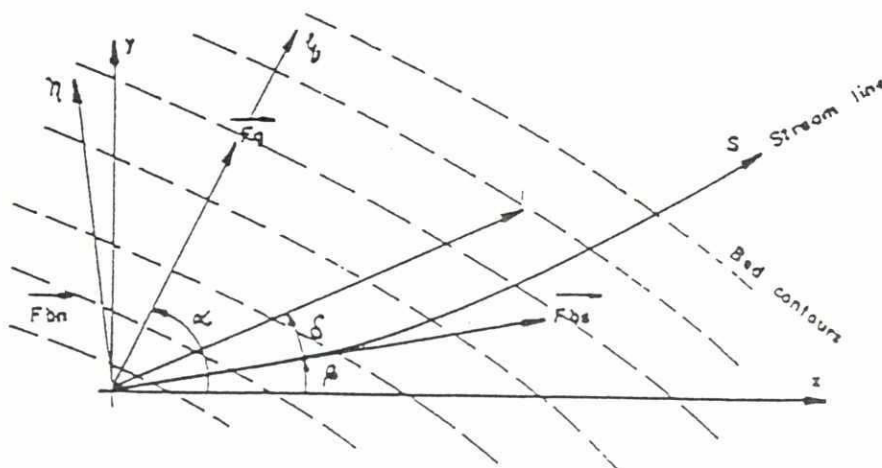
On a horizontal bed the transport direction will coincide with the direction of the bed shear

stress (which may deviate from the direction of the depth averaged flow due to helical flow as described above). On sloping beds gravity will have influence on the transport direction as outlined in Figure 3.

Several relations which describe the effect of gravity on the bed load transport direction have been implemented in System 21. Most of these relations have only been verified against data from laboratory tests and are, therefore, not necessarily applicable to natural rivers with large suspended load transports. The applicable relation is therefore often determined via calibration of the model.

In rivers where the sediment transport is mainly bed load, the transverse equilibrium bed slope at a bend can be estimated by assuming that the gravitational force is balanced by the drag force caused by the secondary (helical) flow.

The output from the bed load model is the bed load transport rate and direction at each grid cell.



#### THREE COMPONENT "DRAGS"

$\overrightarrow{F_{bs}}$  = FORCE DUE TO MAIN FLOW

$\overrightarrow{F_{bn}}$  = FORCE DUE TO SECONDARY FLOW

$\overrightarrow{F_g}$  = FORCE DUE TO GRAVITY ON A SLOPING BED

Figure 3 Force balance on bedload sediment particles.

Standard methods for calculation of suspended load are not applicable in the case of detailed (i.e. high resolution) modelling of rivers. It is necessary to include the so called time-space lag. Consider, for instance, an increase of flow velocity in a river (e.g. a constriction). As the flow velocity increases the entrainment at the river bed will increase correspondingly, but it will take some time (and hence some distance) for the sediment entrained at the bottom to disperse all through the water column. This means that the actual suspended load rate is not only a function of the local hydraulic conditions, as is normally assumed in most mathematical sediment transport models, but is also a function of what takes place upstream



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and earlier in time.

A relevant time scale for the time lag (T) is the settling time for a sediment grain at the water surface, i.e.

$$T = h / w \quad (4)$$

where

h      water depth  
w      sediment fall velocity

Correspondingly, a length scale for the space lag is

$$L = T u \quad (5)$$

where

u      flow velocity

Typical values for a large river are  $h=8\text{m}$ ,  $w=0.02\text{m/s}$  and  $u=2\text{m/s}$  the time and length scales are 400 s and 800 m, respectively. Consequently, for river applications only the space lag is important.

The space lag effect is modelled in System 21 by a depth averaged convection-dispersion model which represents the suspended sediment transport. The convection-dispersion model takes into account the vertical distribution of suspended solids and flow. It is an extension of the model first developed for one dimension by Galappatti (1983) and later in two dimensions by Wang (1989). The model by Wang, however, was primarily developed for estuarine applications and did not include the effect of helical flow. This effect is essential in the case of river applications, because the suspended load transport direction will be different from the main flow direction due to helical flow, as shown in Figure 7.

The depth averaged convection-dispersion model requires an expression for the equilibrium concentration. The models by Engelund & Fredsoe (1982) and van Rijn (1984b) are implemented for this purpose in System 21.

The output from the suspended load model is the sediment concentration and the suspended load transport and direction.

## 2.4 Bed Forms/Hydraulic Resistance

It is far more complex to determine the hydraulic resistance in alluvial rivers than in channels with a fixed bed. This is because a large part of the hydraulic resistance in alluvial rivers is caused by bed form drag. The bed forms have a configuration determined by the sediment transport and the flow. The hydraulic resistance will therefore exhibit both temporal and spatial variations.

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Generally, the hydraulic resistance is divided into two parts. One is caused by drag on the bed forms (form friction) and the other by shear forces on the bed (skin friction). The skin friction is relatively accurately determined from a logarithmic boundary layer equation based on the mean grain size of the bed. The form friction, however, can only be determined analytically if the size of the bed forms is known.

Several hydraulic resistance predictors for alluvial rivers have been proposed, including those of Engelund & Hansen, and Ackers & White. Both of these models are semi-empirical linking the hydraulic resistance to the local instantaneous flow conditions. However, there can be a significant time lag between the form friction (i.e. the bed form size) and the hydraulic conditions. For instance, in many tropical rivers, with distinct dry and wet periods, the sand dunes found at the river bed during the dry season have been formed during receding flood at the end of the flood season. In such rivers the hydraulic resistance will often be very large during the dry season. It is therefore important to account for this time lag.

In System 21 the dynamic development of the bed form size (height and length) is calculated with the model developed by Fredsoe (1979). In a subsequent step the form friction is calculated using a Carnot type formula for expansion loss. The skin friction is determined from a logarithmic boundary layer equation. In quasi-steady flow this model suggests that as the flow velocity increases the dune size and hence the hydraulic resistance increases. As the flow velocity increases further the bed form height and water depth first increase at more or less the same rate and hence the hydraulic resistance only changes slowly until the sand dunes are washed away relatively abruptly and the hydraulic resistance decreases rapidly.

## 2.5 Large Scale Morphology

Large scale in this context means changes in the general bed level within the model boundaries as distinct from bed form changes.

The bed and suspended load sediment transport models described earlier in this Section yield transport rates in two directions. The change of bed level is, therefore, easily determined by integrating the net inflow or outflow depositing or eroding within a control volume.

This integration is explicit, due to the non-linear character of the sediment transport relations applied. This implies that the large scale morphological model needs to be subjected to a rigorous stability criteria for the time step. However, a time step substantially larger than the time step in the hydrodynamic model can generally be applied.

## 2.5 Theoretical Limitations of the System

Generally flows in rivers include phenomena occurring in a very large range of length and time scales. Turbulence, for instance can have time scales of less than a second, whereas flood wave time scales easily can be of the order of magnitude of weeks or months. The mathematical model can only describe explicitly the phenomena with time and length scales greater than the time and space steps used in the model. The small scale phenomena may, however, influence the large scale phenomena; in this case they are included implicitly in the



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model via a number of simplifying assumptions. Examples are turbulence and bed form friction. Turbulence is modelled via the Reynold stress analogy as an increased viscosity of the water, and the bed form friction as an increased bed roughness coefficient. These simplifications make calibration of the model necessary. Ideally, a model calibration (and verification) should cover the range of conditions, which are to be covered in the model applications.

In addition, the model is two-dimensional, whereas flow in, for instance, river bends is strongly three-dimensional. The two-dimensional model has been obtained from the fully three-dimensional equations by depth integration. In this process various assumptions have to be made regarding the vertical distribution of flow velocity (e.g. logarithmic or secondary flow profile as shown in Figure 6). In general, these assumptions are valid, but close to the banks in river bends there may be some deviation, (see De Vriend, 1981). This, however, is not considered to be critical provided the model is properly calibrated.

A limitation of the modelling system is that it is a fixed bank model and cannot, therefore, account for the dynamic development of the bank lines and their impact on the flow and sediment transport processes. At a later stage bank erosion formulations will be incorporated, so that the computational grid itself will become time dependent.

The sediment is represented with one characteristic grain size (mean diameter) in the model, whereas the river sediment consists of a continuous spectrum of grain sizes. Segregation processes will take place during transport of the sediment and to some extent this affects the river topography. Segregation processes, however, have a larger influence during the dry season where bed shear stresses are smaller, see Ribberink (1987).

### 3. Model Application

#### 3.1 General

The river channel pattern in the Brahmaputra River changes very rapidly, and it seems to be very sensitive to the boundary conditions (i.e. to the magnitude of the floods). It is therefore impossible to make long-term predictions of the development of channel alignment within the braid belt. This has several implications. Viewed over for instance the design life time of a structure, this implies that any channel planform can develop at any locations. Design criteria for bank protection measures therefore are not site specific. In the study it was therefore decided to develop two-dimensional mathematical models that represented typical conditions in the river, rather than specific sites. The typical conditions were determined from analysis of satellite imageries of the rivers. For calibration and verification of the mathematical modelling system, however, two site specif models were developed.

Bank erosion often takes place in outer bends of the braiding channels where deep bend scour develops. Maximum bend scour in the river is therefore a key design parameter for bank protection works in the river. A conventional approach to determine this design parameters is to use direct observations of bend scour in combination with an extreme value analysis. This approach has a limitation because the bend scour determined from observations is most likely to be associated with situations where significant bank erosion is taking place. This bank erosion will tend to limit the depth of scour because the transport capacity of the



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flow is being partially used to transport the eroded material from the bank. Since the purpose of the training measures is to substantially reduce bank erosion, it can be expected that the bend scour associated with such works will be significantly greater than that adjacent to an unprotected bank. This was one of the key issues addressed with the two-dimensional mathematical model.

The two-dimensional mathematical model is a fixed bank model thus the dynamic change of alignment due to bank erosion cannot be incorporated directly. However, the eroded bank material can be introduced into the model as a sediment source and in this way have influence on the bed topography (i.e. bend scour).

### 3.2 Model Calibration and Verification

As a part of the project two areas have been surveyed in detail in order to collect data for calibration and verification of the mathematical model. The first area, Test Area 1, covers the entire width of the river and is about 11 km long. This area contains a large confluence with associated confluence scour and one large bifurcation. The second area, Test Area 2, covers only one of the branches in the braiding river and is approximately 4 by 6 km. Test Area 2 contains a sharply curved thalweg with deep bend scour and associated bank erosion rates.

Each of the areas have been surveyed at least two times in order to determine morphological changes in the form of bed level changes and horizontal movements of channels. These changes can be very substantial, as shown in Figure 4 where sample cross-sections taken at rising stage in June, during high flow in July and again during the dry season in November are depicted. The figure shows areas with erosion and deposition. The morphological model is currently being calibrated against these observed morphological changes.

In Test Area 1 detailed flow velocity and sediment transport data have been collected. Flow velocities were measured with a directional current meter at at least two points in each vertical. Derived depth averaged velocities are depicted in Figure 5. Measured and simulated velocities in a part of Test Area 1 is shown in Figure 6.

Both bed and suspended load data were collected. The suspended load data with a bottle sampler and the bed load via dune tracking. The suspended load data exhibited a large temporal and spatial variation of the sediment concentration, but the averaged concentration off all samples corresponded well with data collected on routine basis further upstream in the river. The dune tracking data revealed typical dune dimensions of  $H=3$  and  $L=150$  m migrating with a speed of 100 m/day. The data showed that the dunes are short crested (3-dimensional dune pattern).

In Test Area 2 only a few float track data and suspended load samples have been collected which are currently being processed. Bank erosion rates are also being monitored in this area. Preliminary results suggest bank erosion rates up to 500 m/year. The bank erosion problem is associated with a very deep scour hole adjacent to the bank where an impinging channel meets the bank. At this location the flow velocities are in excess of 2.5 m/s. The dry season bathymetry of Test Area 2 is depicted in Figure 8.



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Preliminary simulation results with the morphological model suggest that additional scour develops in front of the bank when the supply of bank erosion material is cut off. This result has been obtained by comparing morphological model simulation results without bank erosion material supply and simulations with the observed bank erosion rate converted to a sediment input along the bank. The simulations suggest that additional scour develops with a rate of about 1 m/month. Equilibrium scour was not established after a simulation period corresponding to one flood season.

#### 4. Conclusion

In this paper a two-dimensional mathematical modelling system for the simulation of flows, sediment transport, alluvial roughness and morphological changes in rivers is described. The model utilises a curvilinear grid that is orthogonal. The flow model is fully dynamic and includes a description of secondary flow due to flow curvature (helical flow). The sediment transport model distinguishes between bed and suspended load and takes into account the effect of helical flow. The suspended load model is based upon a depth averaged model which accounts for vertical gradients via a similarity assumption. The suspended load model also includes a phase-lag description.

The model is applied to the Brahmaputra River in Bangladesh which is a large very dynamic river. Simulation results show that the model can reproduce observed flow patterns very well and reproduce the main morphological features. A key application has been to investigate the impact of bank erosion on outer bend scour. Preliminary results show that protection of a river bank may give rise to significant additional bend scour.

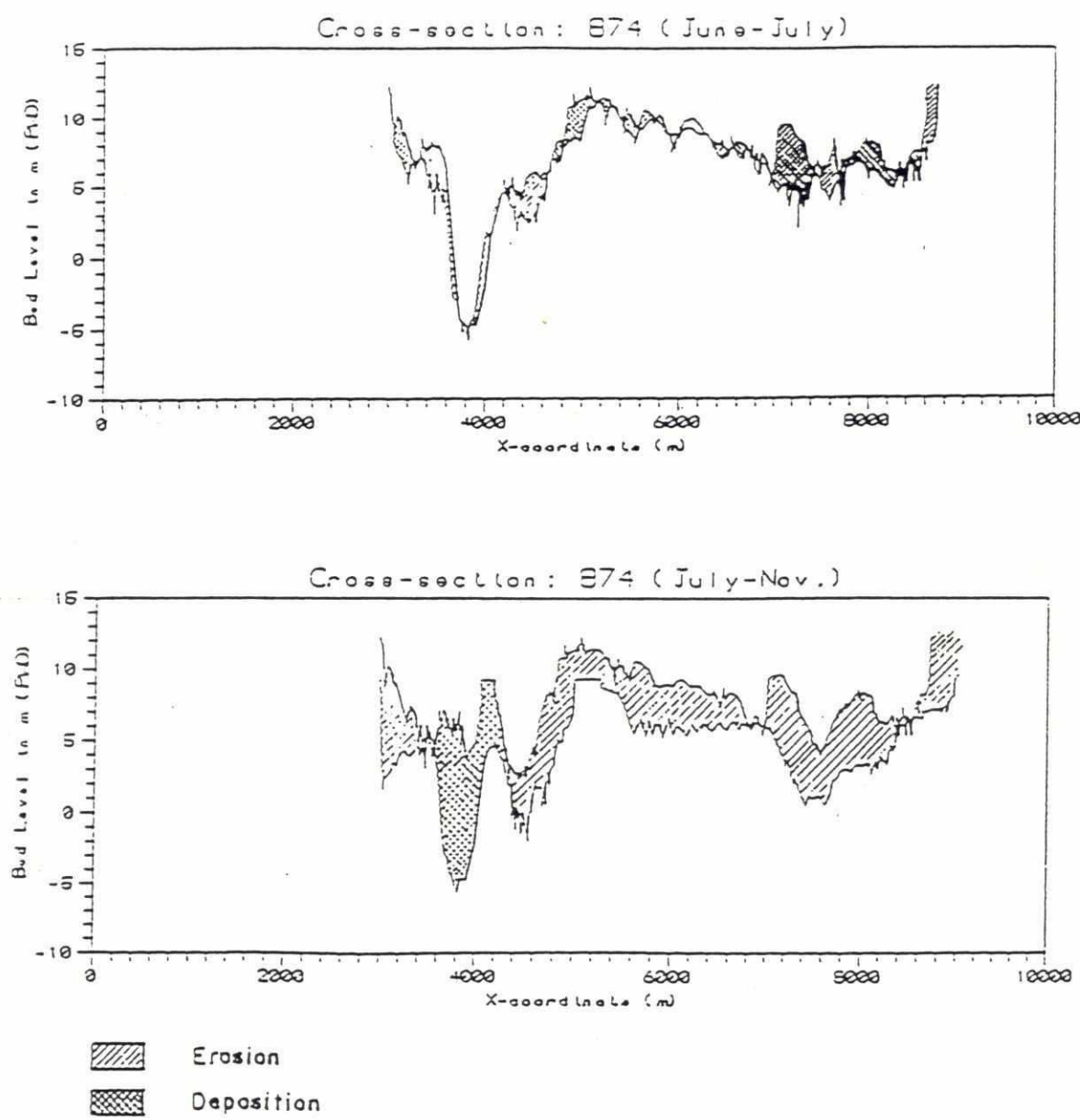


Figure 4 Cross-sections in Brahmaputra surveyed in June, July and November.



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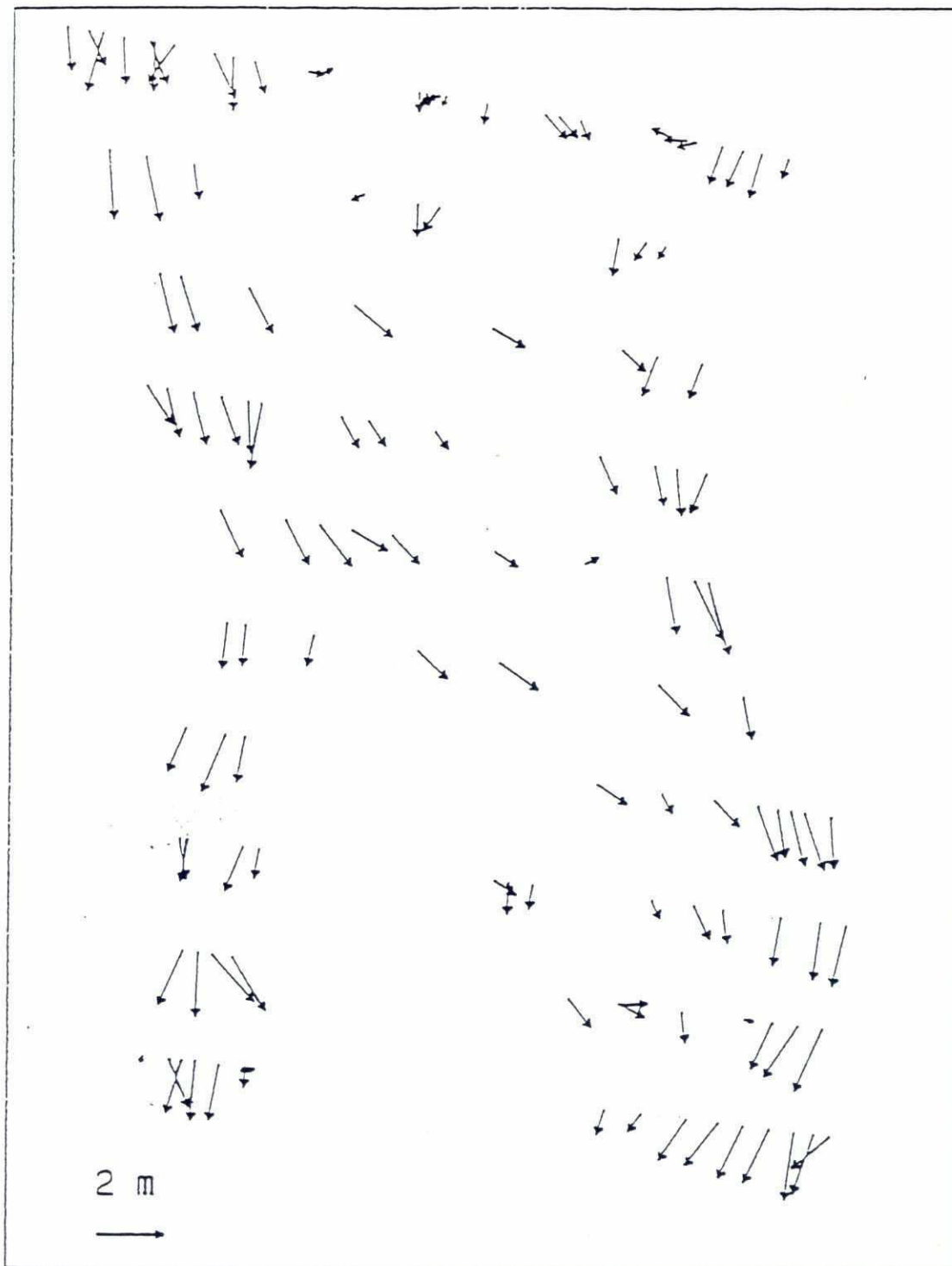
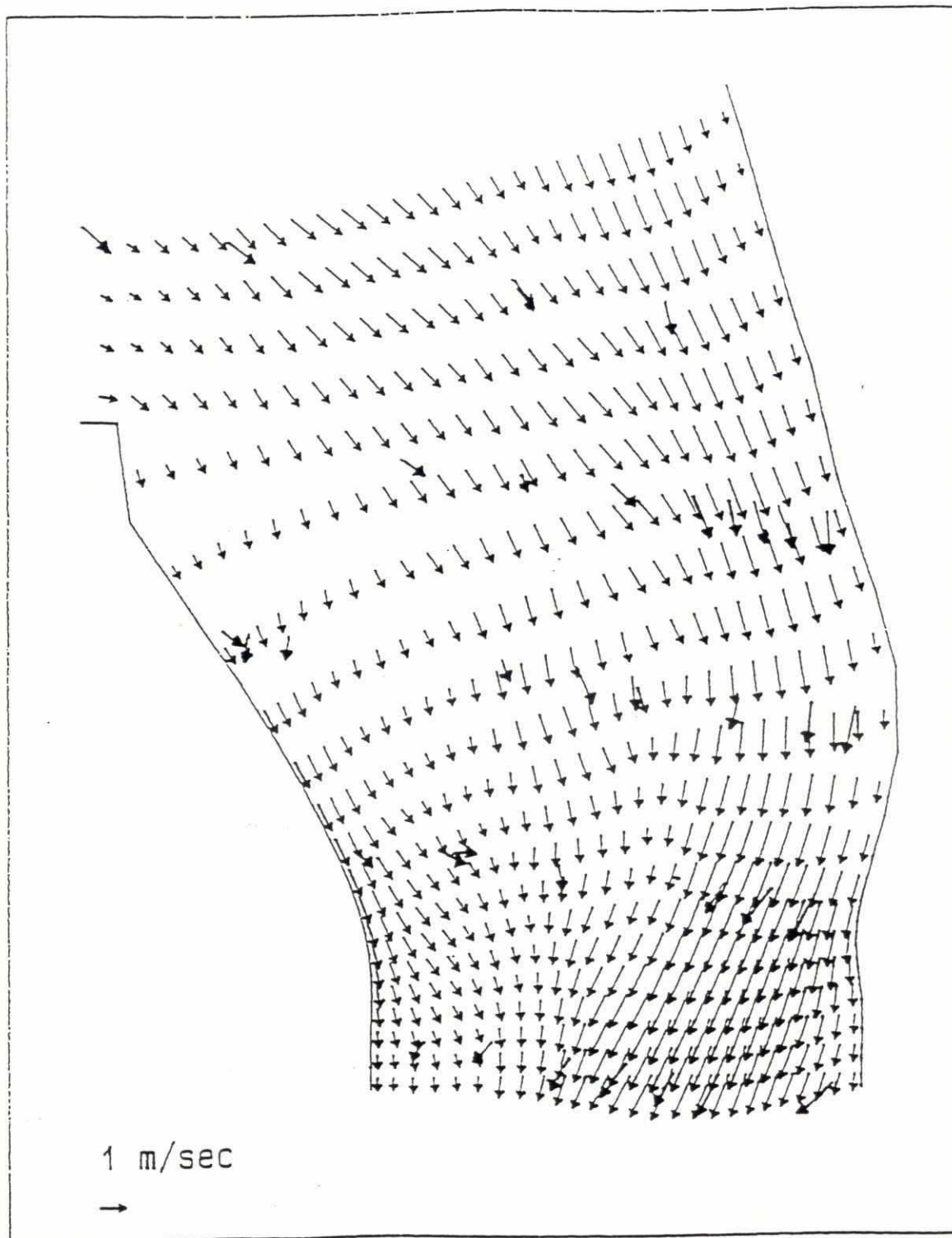


Figure 5 Observed depth averaged flow velocities.



→ Observed  
 → Simulated

Figure 6 Simulated and observed flow velocities. Lower right part of the model.



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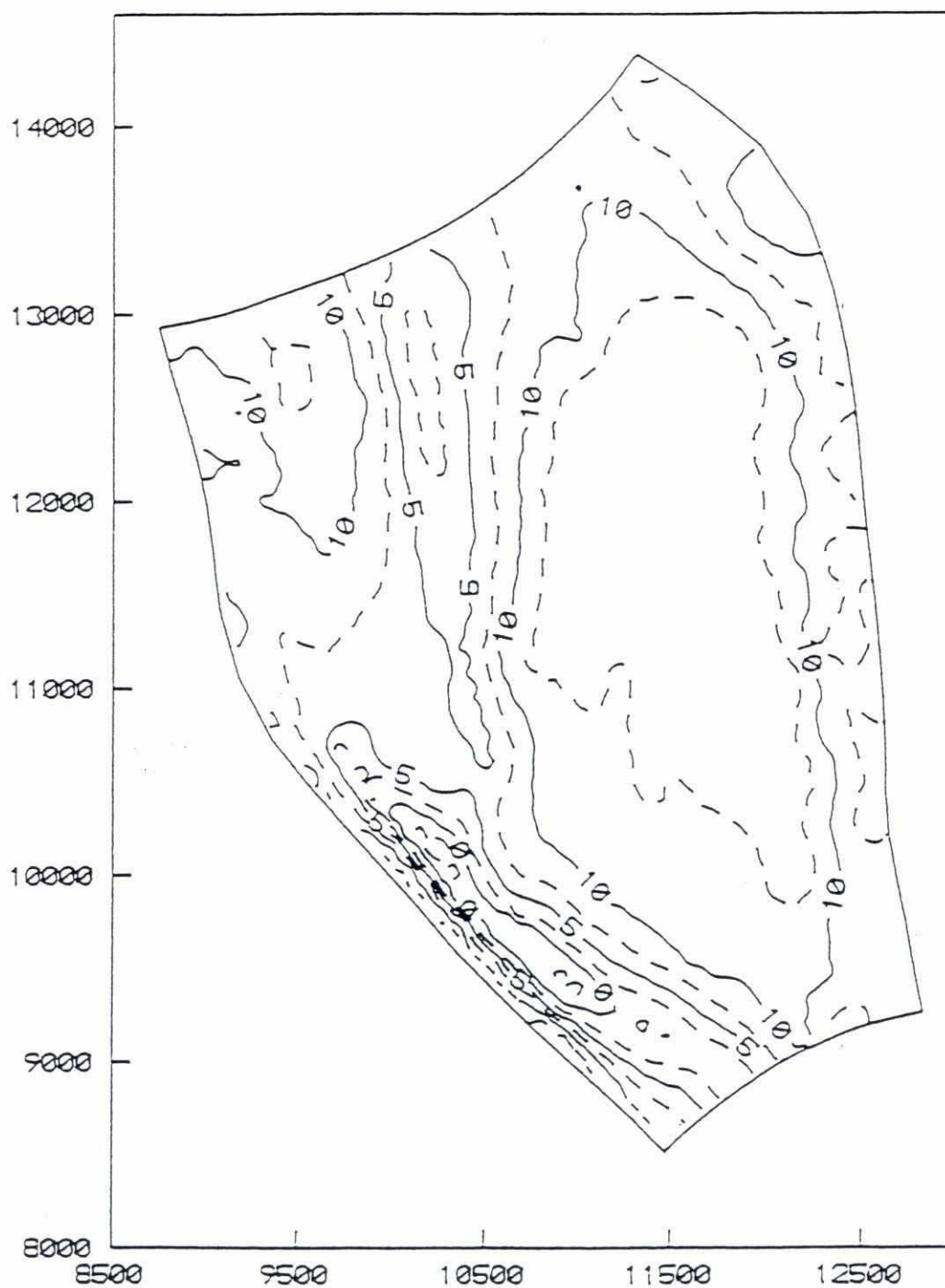


Figure 7 Model topography - Test Area 2.

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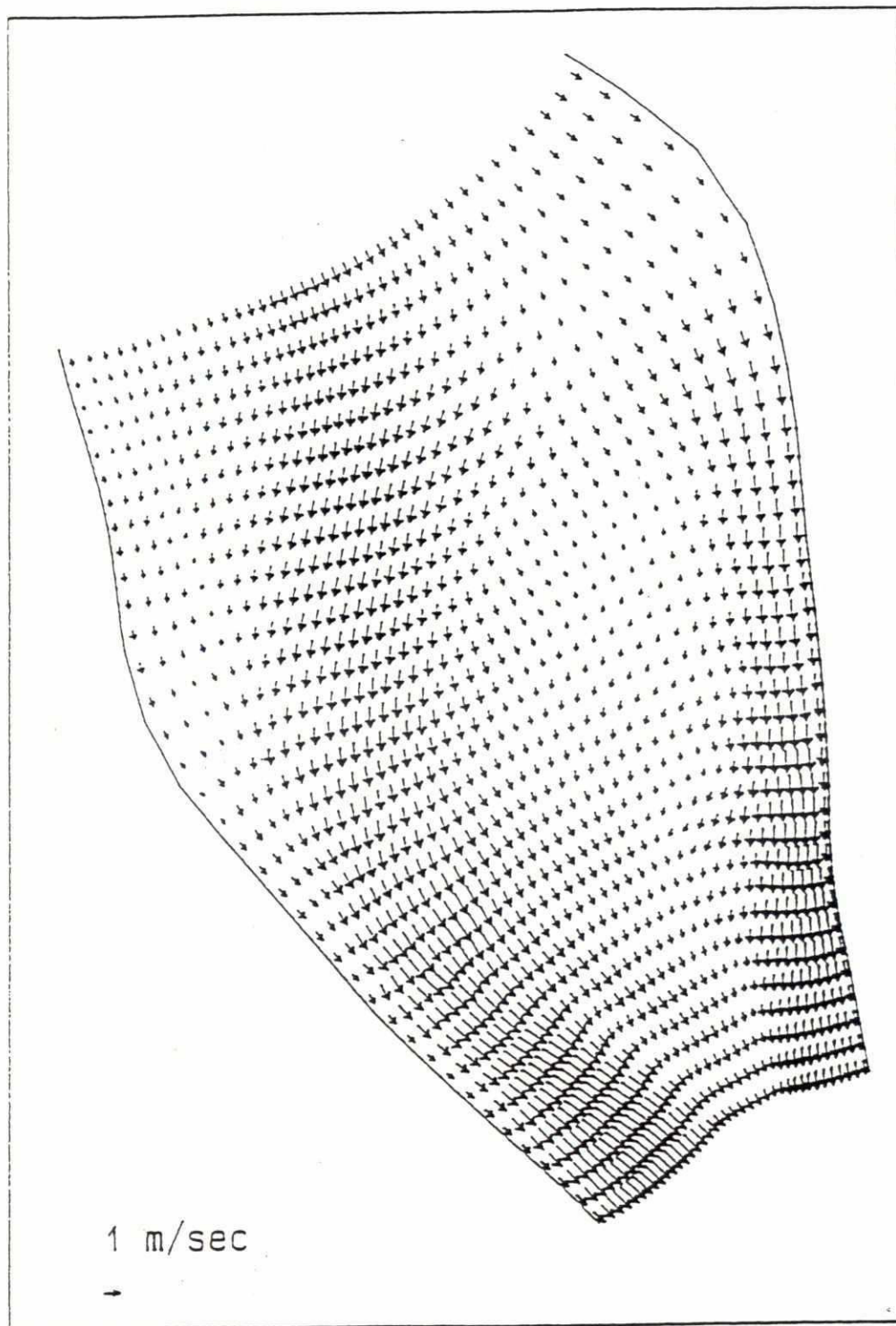


Figure 8 Simulated flow distribution in Test Area 2.



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## Paper 6.3



KLAASSEN, G.J. (1992), Experience from a physical model for a bridge across a braided river with fine sand as bed material, *Proc. 5th Intern. Symp. River Sedimentation, Karlsruhe, FR Germany*, 509-520pp.



## Experience from a Physical Model for a Bridge Across a Braided River with Fine Sand as Bed Material

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**ABSTRACT:** This paper describes the experience obtained with a physical model of the braided Jamuna (Brahmaputra River) in Bangladesh, a river with fine sand as bed material and frequent changes in channel patterns. A special scaling method was developed, aiming at the reproduction of the braiding pattern in order to study the effect of the proposed bridge on the channel characteristics. Essentially, the model had a movable bed as well as movable banks, and it had to be tilted substantially. It was found that the continuously changing braided channel pattern in the field could be reproduced in the model. The paper summarizes the scaling method, and describes the set-up of the model, the results obtained and the interpretation of these results.

### 1 INTRODUCTION

Braided river systems are notoriously difficult to model, especially rivers with fine bed material because they tend to change their channel pattern frequently (see the companion paper Klaassen & Masselink, 1992). Within the framework of a feasibility study for, and the detailed design of a bridge over the braided Jamuna River (lower reach of the Brahmaputra River in Bangladesh, see Fig. 1) a physical model was used. This model can be characterized as a model with both mobile bed and mobile banks. The model was aiming at (1) the reproduction of the braided channel pattern in this essentially non-trained river and (2) the mutual influence of this channel pattern and the proposed river training works, to determine the design criteria for both bridge piers and river training works required for the bridge.

The reproduction of the braided channel pattern could only be done by considerably tilting the model. Other methods, such as the use of light-weight material or smaller size of the bed material in the model, were found not to be feasible. Consequently, substantial scale effects were present in the model. This was the reason that in addition to this physical model other tools were used to establish of the design boundary conditions for the river training works (see Sections 2.2 and 2.3).

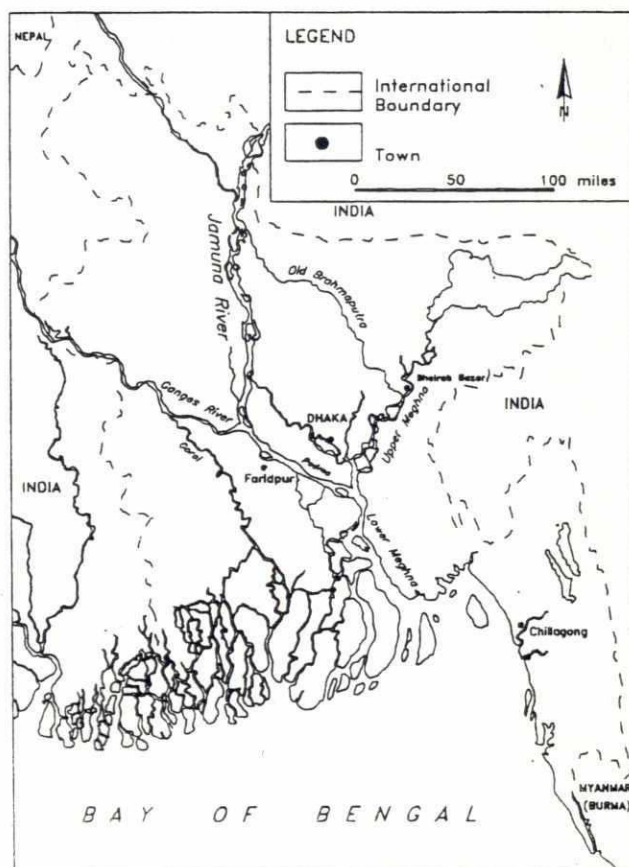


Fig. 1 Map of Bangladesh with the Jamuna River



Experience obtained from carrying out this model investigation is presented in this paper, together with a discussion of the results obtained and their implication for the design boundary conditions for the bridge and the river training works. The scaling method itself and the possible scale effects are touched upon only briefly here. More detailed information on the latter aspects is available in Klaassen (1990).

More details on the Jamuna Bridge project can be found in RPT/NEDECO/BCL (1987,1990a), and detailed information on the model investigation described in the present paper in RPT/NEDECO/BCL (1990b).

## 2 PROBLEM DESCRIPTION

### 2.1 Frame-work of modelling work

A bridge has to be built across the Jamuna River in Bangladesh. This river is the lowest reach of the Brahmaputra River, and it is a braided river with, on average, three braids upstream of Sirajganj (near the proposed bridge site) and about two braids more downstream. The river bed can be characterized as a flat sand bed in which a number of deep channels are present. Locally, the shallow bars in between the channels are called 'chars'. The location of the channels and chars changes frequently (Klaassen & Masse-link, 1992). The total width of the channel pattern is up to 17 km.

At the bridge site the river will be constricted to approximately 5 km. To accomplish this, earthen bridge approaches will be constructed. To protect these from erosion by outflanking channels, guide bunds will be built on both 'ends' of the approaches. Upstream of the guide bunds additional training river works are considered to guide the river channels towards the bridge opening and to prevent outflanking. Hence, the required river training works will comprise two guide bunds and upstream groynes or revetments.

The ultimate aim of all mathematical and scale modelling within the frame-work of the studies for the Jamuna Bridge was to establish the extent of the required river training works and the design criteria for the bridge piers and these river training works.

### 2.2 Modelling approach

The modelling approach (see Fig. 2) is characterized by the integrated use of mathematical models, physical models and field measurements (both historical ones and measurements especially done for the present project).

In addition to the physical model described here, the following tools were used during the modelling phase of the Jamuna Bridge Appraisal Study, Phase II:

- (1) a 1-D morphological mathematical model of the morphology of the Brahmaputra River to predict general scour and constriction scour;
- (2) a 2-D mathematical model of the depth-averaged flow field underneath the bridge, using the results of the present physical model as input and having a curvilinear coordinate system (Wijbenga, 1985);
- (3) an undistorted physical model to study the local scour near the guide bunds, using the results of model (2) as boundary conditions;
- (4) an extensive analysis of historical river data;
- (5) an extensive study of satellite images to determine channel characteristics and celerities of channel pattern changes over the period 1973-1987;
- (6) extensive hydrographic field measurements during the flood period of 1987.

### 2.3 Interaction between tools used

An overview of the interaction between the various tools that were used, is provided in Fig. 2. In a number of cases the results of one model were used as the input for another model. This holds especially for the 'central' series of physical model with mobile bed and banks, 2-dimensional mathematical flow field model and local scour model.

### 2.4 Specific purpose of the physical model

The specific purpose of the present physical model investigation was:

- (1) to study the effect of the river training works on channel patterns;
- (2) to identify 'worst' channel patterns (for flood and low-flow conditions) for the river training works;
- (3) to study the effect of gradual shifts of river upstream on channel patterns near bridge and training works.

The overall reason for this model investigation was to provide boundary conditions (notably 'worst' bed topographies) for the 2-dimensional depth-averaged mathematical flow field model.

Alternatively, historical channel patterns could have been used to arrive at design conditions for the river training works required. From the results of an exploring model test it was concluded, however, that the presence of a bridge constricting the river width has a significant effect on the channel patterns.



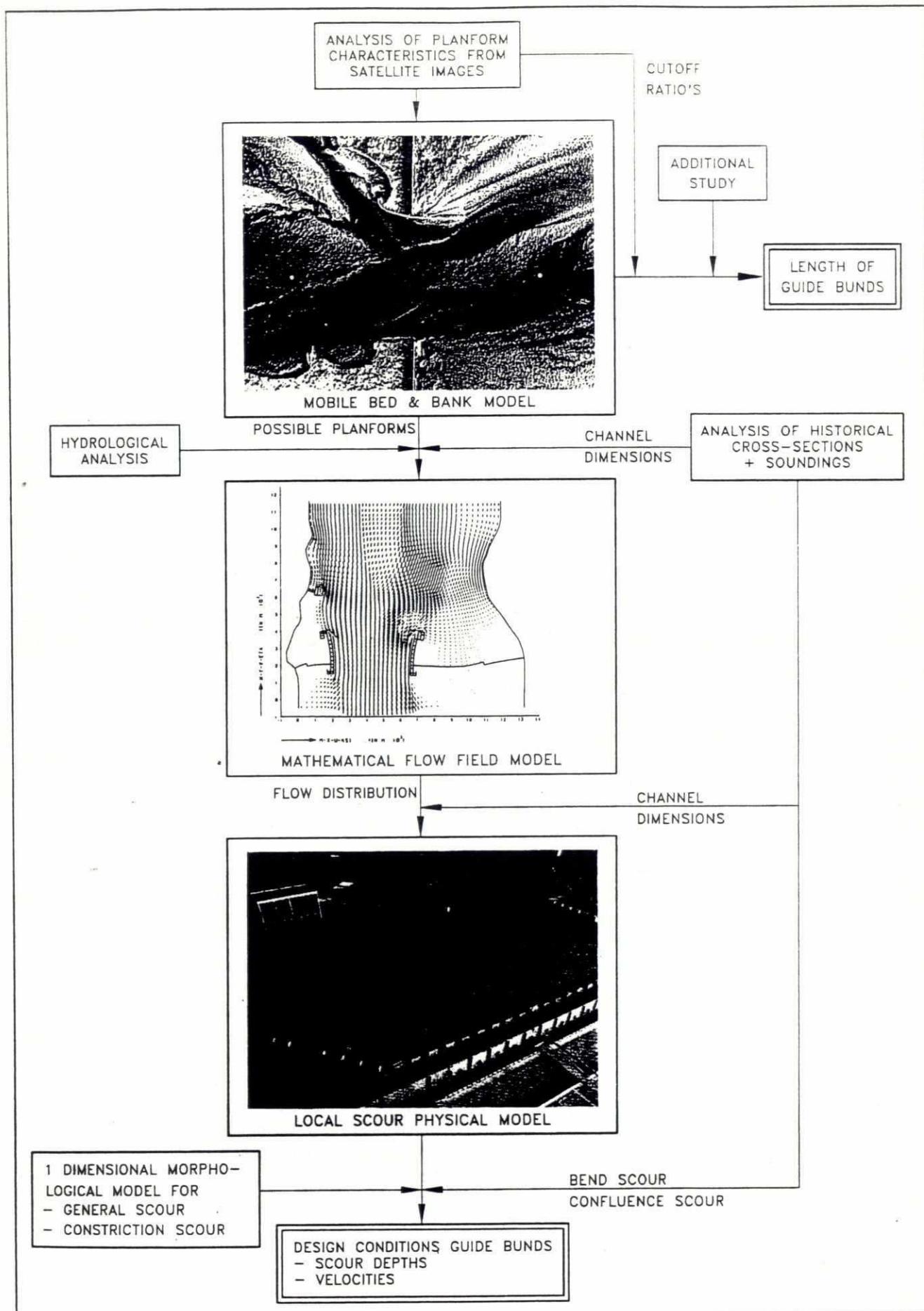


Fig. 2. Overview of tools used for the present study and their interaction

### 3 SCALING PROCEDURE

#### 3.1 Phenomena to be reproduced

The most important phenomenon to be reproduced in the present scale model investigation is the channel pattern: braiding with 2 to 3 channels per reach. Additional aspects to consider were: (1) roughness condition, required for a correct flow field reproduction, (2) the width of the channels, to be reproduced on length scale and (3) the ratio curvature/channel width. Other aspects (such as the Reynolds-condition (here  $Re_m > 10^3$  was used), the Froude-condition, the ideal velocity-scale (Jansen, 1979), the distortion, and the horizontal celerity of channels) were considered less important (Klaassen, 1990).

#### 3.2 Scale rules

As the channel pattern (braided and 2 to 3 parallel channels per reach) was the most important parameter to model, the scale rule(s) to be applied should be derived from a relationship between the number of channels and other parameters.

Based on a theoretical analysis (see Struiksma and Klaassen, 1988) it was found that probably the ruling equation for the channel pattern reads as:

$$\frac{i}{\Delta D} = \alpha \cdot Q^\beta \quad (1)$$

where  $i$  = stream gradient (-),  $\Delta$  = specific density, defined via  $\Delta = (\rho_s - \rho)/\rho$ ,  $\rho_s$  = density of sediment,  $\rho$  = density of water, and  $Q$  = 'dominant' discharge ( $m^3/s$ ). The coefficients  $\alpha$  and  $\beta$  still have to be defined and do not need to be constant over the full range of  $Q$ . Evidence for the existence of such an equation is given by Leopold, Miller and Wolman (1964) via the threshold between meandering (a single channel pattern) and braiding (a multiple channel pattern) and Howard et al (1970). See also Klaassen (1990). The left-hand term is in line with Ferguson (1984), who showed that  $i/D$  is preferable over  $i$  alone. Equation (1) is indicated in a schematic way in Figure 3.

The implication of the use of Figure 3 as scaling rule, is that the model has to be tilted substantially. For details on the applied scale rule see Klaassen (1990).

#### 3.3 Imposed and dependent variables

Using the above scale rule, the imposed and the dependent variables in the model can be identified. Assuming that in the prototype all necessary data are available, then in the model a choice has to be made as to the relative density and the particle diameter  $D$  of the model material. Once these are selected, only the model discharge and the model slope can still be

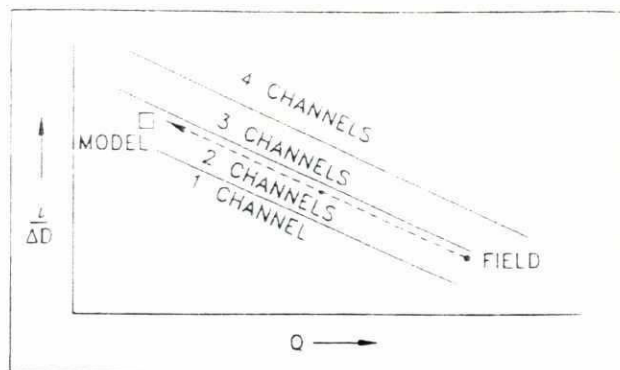


Fig. 3 Channel pattern classification diagram used for deriving scale rule

selected freely. Hence, for a given slope  $i_m$  (and hence  $n$ , where  $n$  stands for scale factor) the value of  $n_Q$  can be 'obtained' from Eq. (1). So applying the condition that the channel pattern (number of braids) in prototype and model should be similar, results (for given  $D_m$ ,  $\Delta_m$  and  $i_m$ ) directly into  $Q_m$ . Given the total discharge in the model, all characteristics of the channels in the model were determined because these were self-forming regime channels (conform Lacey (1958) and Klaassen & Vermeer (1988b)). So the length scale, the depth scale, the distortion and all other scales could not be influenced anymore! This resulted in substantial scale effects (see Chapter 6).

#### 3.4 Scale selection

In addition to the simple scale rule discussed above, other aspects influence the selection of the scales, notably:

- (a) the availability and costs of the bed material in the model,
- (b) the model should not be too large (costs, celerity),
- (c) the model should not be too small (because of the condition  $Fr < 1$ ).

In view of the costs it was decided to use sand as bed material and to build a model with an approximate length scale of 1:1,000. The model sand was almost uniform with  $D_{50} = 0.2$  mm. Based on exploring studies the slope was pre-selected at 0.007. The discharge in the model was varied until a fair reproduction of the channel pattern was obtained. See hereafter. All other scale factors were determined by comparing the value of the different parameters in model and in prototype.

### 4 DESCRIPTION OF PHYSICAL MODEL

#### 4.1 Model layout

The model was built as a special model tray at the De



Voorst station of DELFT HYDRAULICS. The main dimensions of this tray were  $55 \times 20 \text{ m}^2$ , whereas the depth of the tray was about 1 m. A plastic tissue was applied to make the model watertight. The model was filled with sand up to 0.6 m, under a longitudinal slope of 0.007 and (after some initial tests) laterally inclining slightly towards the centre line of the tray.

#### 4.2 Water and sediment circuits

The water was partly re-circulated together with the sediment, and partly coming from the central water supply system of the hall in which the model tray was built. The sediment was caught in a sand trap in the model and pumped back to the head of the model where it was re-introduced. See Fig. 4.

#### 4.3 Boundary conditions

The boundary conditions of the model are discussed separately for upstream and the downstream boundary. At the upstream boundary the discharge  $Q_m$  was imposed. In an attempt to simulate the flood hydrograph, the discharge in the model consisted of a succession of 15 minutes 65 l/s and 30 minutes 35 l/s, corresponding to approximately 26 and 52 days in the prototype (see Fig. 5).

Reproduction of the duration curves of the discharge as done here (whereby the lowest discharges in the river were discarded) provided a better channel pattern. The discharge of 35 l/s was provided by the sand pump. Hence during the period with the highest discharge another 30 l/s was added from the central water supply system. This was done automatically with a time clock. Hence the model investigation could be continued during night time and the week-ends, if and when required. The sand transport was

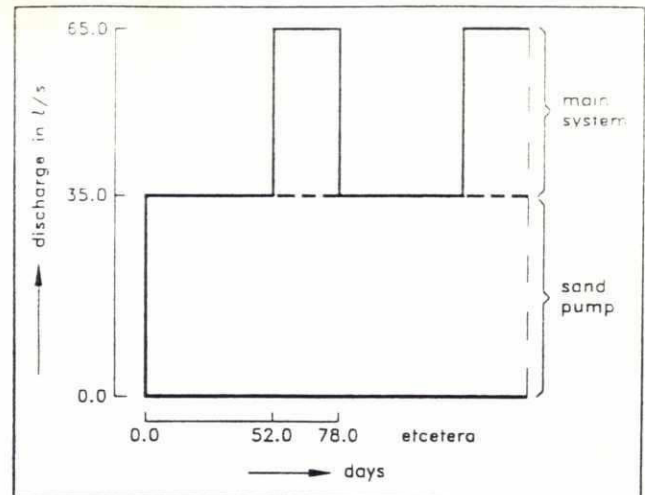


Fig. 5 Imposed discharge hydrograph

recirculated. The supply upstream was done in a flexible way. The sediment was thoroughly mixed in a container and from this using the sediment-water mixture was brought in the model using flexible tubes at up to three locations at the upstream end. Initially, the mixture was supplied only in the middle of the upstream boundary, but then the braided pattern was fully developed only after some 25 m.

The downstream boundary of the model was a schematized rating curve. This rating curve was imposed by experimentally varying the width of the downstream overflow and then selecting the best option.

#### 4.4 Verification

After some exploratory tests, a slope of 0.007 in the model was selected tentatively. For the discharge scale  $n_Q = 1,000,000$  was used (approximately corresponding to a length scale  $n_L = 1,000$  (see

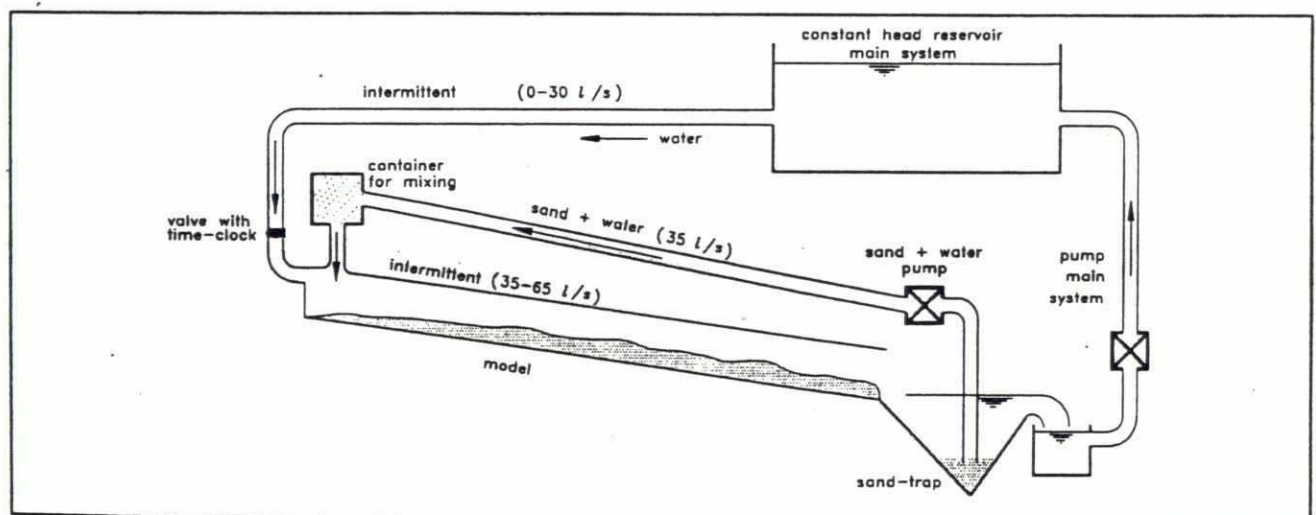


Fig. 4 Water and sediment circuits

Klaassen, 1990)) and this turned out to be a good estimate as far as the channel pattern was concerned: in the model 2 to 3 braids were observed in each cross-section.

Next the total widths of the channels in the model were measured and these were compared with prototype data. It was found that also this parameter scaled with a factor of approximately 1,000. Furthermore, the bend characteristics in model and prototype were compared, see Fig. 6. It was observed from Fig. 6 that also the bend characteristics were reproduced fairly well, although the bends with the smallest radii of curvature were not found in the model. Nevertheless it was concluded that the model provided a fair reproduction of the channel pattern in the Jamuna River.

The other scale factors were derived on the basis of observations in the model and via scale laws like the Chézy equation. Table 1 provides an overview of all scale factors, together with representative values of some parameters in model and prototype. More information on the prototype conditions is given in

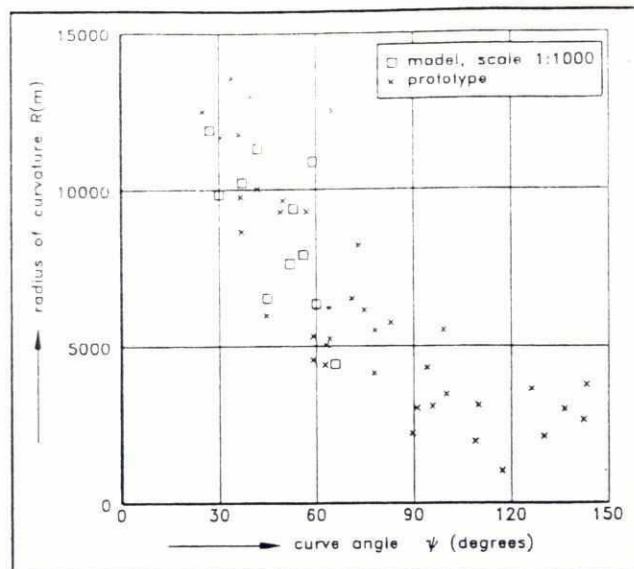


Fig. 6 Comparison of bend characteristics in model (using  $n_L = 1,000$ ) and prototype

Klaassen & Vermeer (1988b) and in Klaassen et al (1988).

| Parameter                                   | Unit                  | Value in prototype  | Scale factor n | Value in model       |
|---------------------------------------------|-----------------------|---------------------|----------------|----------------------|
| Imposed variables                           |                       |                     |                |                      |
| Particle diameter D                         | (mm)                  | 0.18                | 0.85           | 0.213                |
| Relative density $\Delta$                   | (-)                   | 1.65                | 1.00           | 1.65                 |
| Slope i                                     | (-)                   | $7 \cdot 10^{-5}$   | 0.01           | $7 \cdot 10^{-3}$    |
| Discharge Q                                 | (m <sup>3</sup> /s)   | 90,000-10,000       | 1,000,000      | 0.065-0.035          |
| Dependent variables                         |                       |                     |                |                      |
| Length                                      | (m)                   |                     | 1,000          |                      |
| - bankfull width (one channel out of three) |                       | 3,000               |                | 3.3                  |
| - total width                               |                       | 10,000-15,000       |                | 13                   |
| - largest radii of curvature                |                       | 5,000-13,000        |                | 5-12                 |
| Depth                                       | (m)                   |                     | 200            |                      |
| - flood                                     |                       | 5.80                |                | 0.032                |
| - low flow                                  |                       | 3.00                |                | 0.021                |
| Distortion $n_L/n_H$                        | (-)                   |                     |                | 5                    |
| Chézy coefficient                           | (m <sup>1/2</sup> /s) |                     | 3.5            |                      |
| - flood                                     |                       | 70                  |                | 19                   |
| - low flow                                  |                       | 55                  |                | 14                   |
| Sediment transport                          | (m <sup>3</sup> /m/s) | $1.4 \cdot 10^{-3}$ | 80             | $1.34 \cdot 10^{-5}$ |
| Duration flood T                            | (days)                | 78                  | 2,500          | 0.03                 |
| Froude number                               | (-)                   | 0.1-0.2             | 0.25           | 0.5-0.8              |

Table 1 Approximately scale factors and values of parameters in model and prototype



#### 4.5 Measurements carried out

The following measurements were carried out during this model investigation:

- (1) channel pattern via vertical photographs from the ceiling of the shed (15 m high),
- (2) position of the bank lines to monitor the bank erosion,
- (3) bed topography by manual sounding from a measuring carriage that could be driven along rails on both sides of the model,
- (4) sediment transport and composition by regularly taking samples from the water-sand mixture being supplied to the model.

### 5 RESULTS

The results of the model investigation described here are mainly possible channel patterns in the river and near the proposed bridge cum river training works. Various layouts of the river training works were investigated. Most of the results became available as vertical 'aerial' photographs. Fig. 7 presents a mosaic of these aerial photographs during one of the tests. Fig. 8 and 9 provide typical examples of the obtained results. Fig. 8 presents some observed channel patterns during a particular test. The initial bank lines of the same test are given in Fig. 9, together with the bank line location at the end of this test.

More detailed information on the tests carried out and on the obtained results are provided in RPT/NEDECO/BCL (1990b).

### 6 DISCUSSION OF RESULTS

#### 6.1 Observations

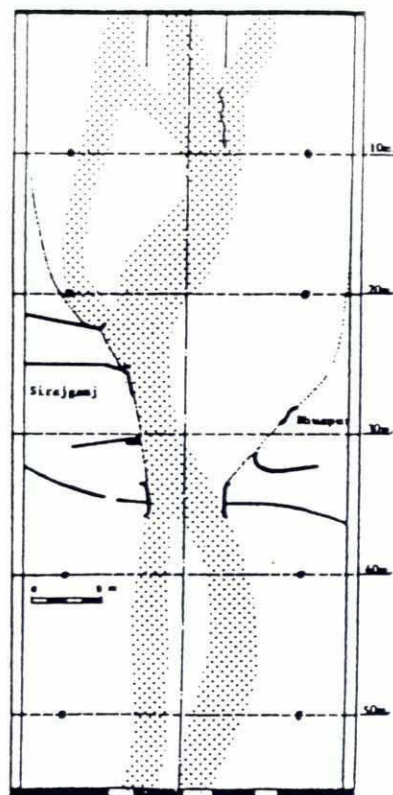
The following observations are made as to the results obtained during the present model investigation:

- (1) The model was quite successful in reproducing the continuous changing of the channel patterns, a phenomenon typical for braided rivers with fine bed material (see also Klaassen & Masselink, 1992).
- (2) The channel pattern could only marginally, and over a short distance, be influenced by the upstream boundary conditions (distribution of water and sediment over the width).
- (3) Bank erosion was intermittent and not continuous: periods of erosion at a particular location were followed by periods of no attack (in the field probably periods with accretion, but sedimentation was not correctly reproduced in this model). This intermittent erosion is also typical for Jamuna-type rivers. The bank erosion, is of course, closely linked to the channel pattern.

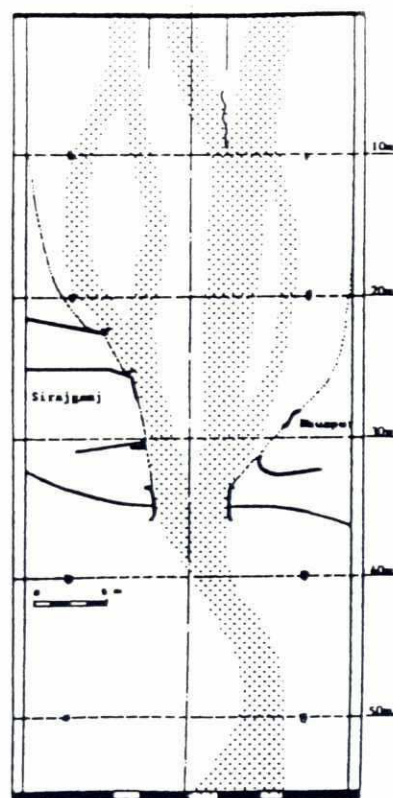


Fig. 7 Mosaic of vertical aerial photographs of the model during test T8

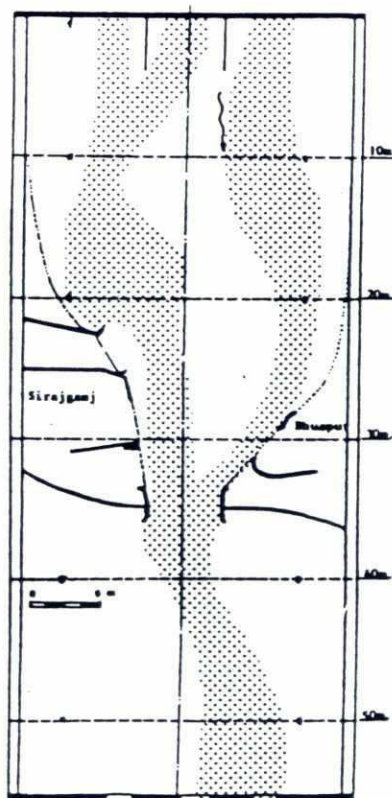




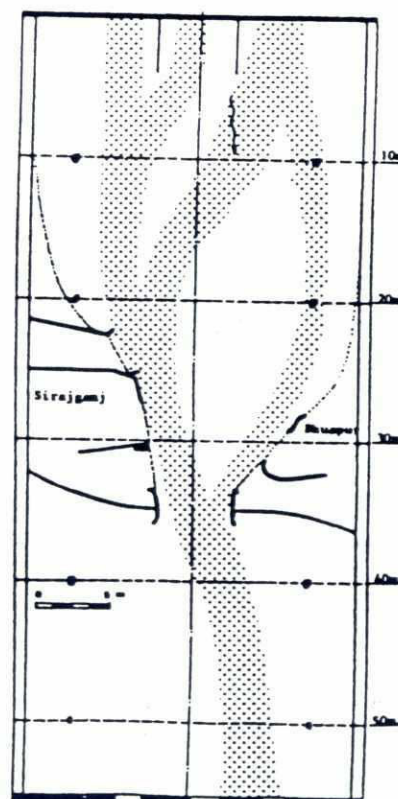
(a)



(b)



(c)



(d)

Fig. 8 Some observed channel patterns during test T8 over a period of 300 hours



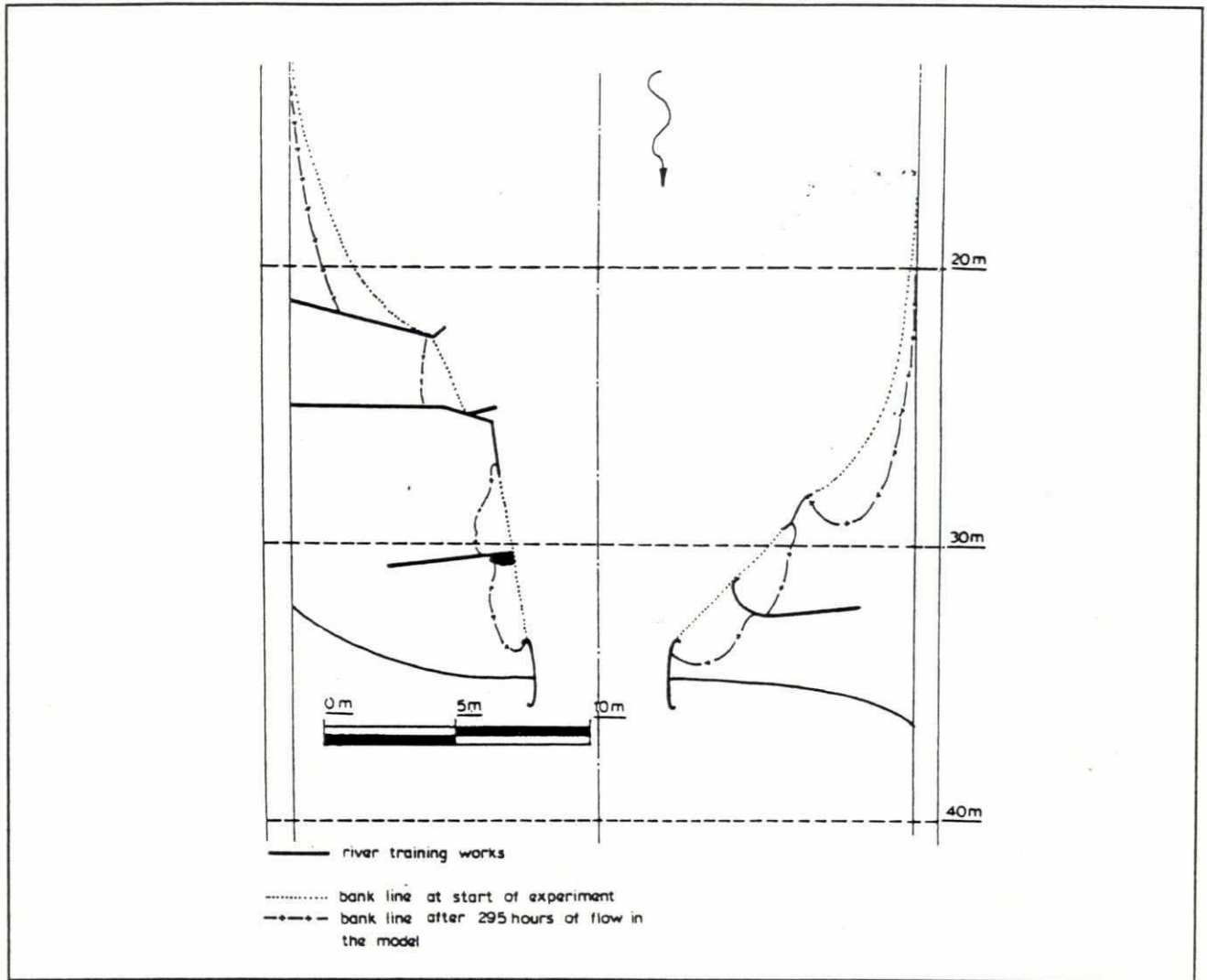
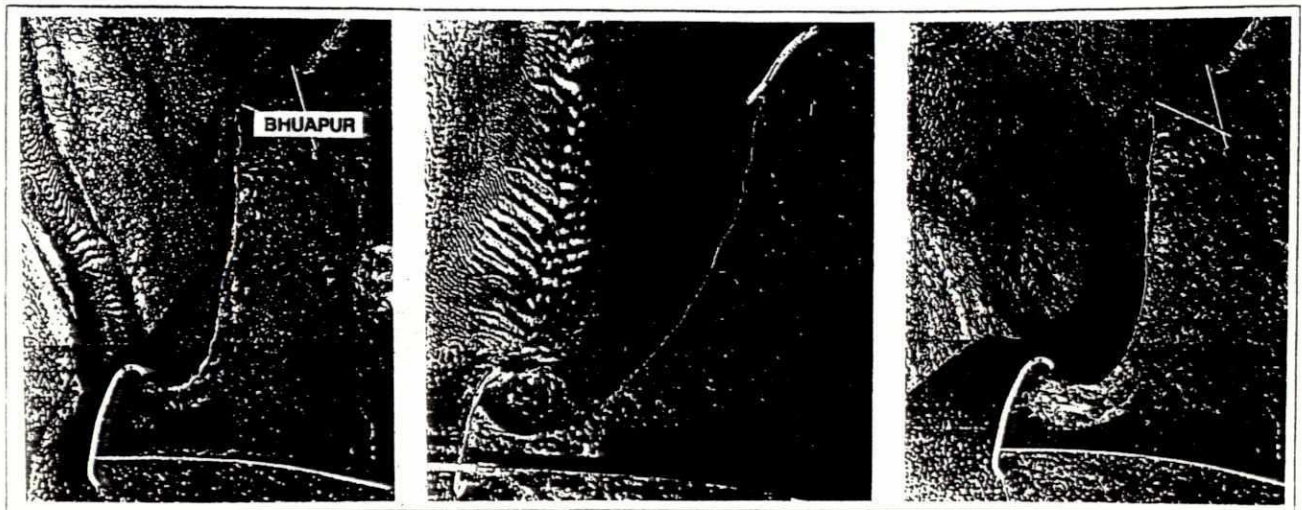


Fig. 9 Bank line locations in the beginning and at the end of test T8



(a) oblique attack

(b) frontal attack

(c) attack by outflanking channel

Fig.10 Possible attacks on the guide bunds

Regarding the possible channel patterns in relation to the river training works, the following observations were made:

(4) The river training works were not effective in training the river. Rather should they be considered as 'river defence works', as they were effective in guiding the river underneath the bridge and (via the guide bunds) in preventing the attack of the river on the bridge approaches.

(5) An attack of the river on the guide bunds is possible from three sides. These are referred to in Fig. 10 as oblique attack, frontal attack and attack by an outflanking channel.

(6) Underneath the bridge major scour holes were observed when a channel junction was present. As a matter of fact this happened quite often (see Fig. 8). The resulting confluence scour (see Klaassen & Vermeer, 1988) should be taken into account when establishing the design conditions for the bridge piers.

From a statistical point of view the model results were considered as possible attacks on the river training works, and not so much as what can be expected over the first years after the construction of the bridge.

## 6.2 Scale effects

As already argued in Klaassen (1990), the present model, while fairly reproducing the channel pattern, is subject to serious scale effects. These are discussed hereafter for some length.

For the reproduction of the flow field both the roughness condition and the Froude condition are important. However, some deviation from the Froude condition is acceptable (Struiksmā, 1990). Inspection and analysis of the scale factors in Table 1, especially as to the extent to which the various scale conditions (see Jansen, 1979) are fulfilled, yielded the following conclusions:

(1) the Froude condition which reads as:

$$n_{Fr} = 1 \quad (2)$$

was not fulfilled:  $Fr_m$  was between 0.5 and 0.8, hence much larger than  $Fr_p$ ;

(2) the roughness condition, which reads as:

$$n_C^2 = \frac{n_L}{n_h} \quad (3)$$

was fairly fulfilled.

Hence, it had to be concluded that the flow field in the model (being influenced mostly by the roughness and the Froude number, see Struiksmā (1981, 1990))

was only approximately reproduced. The possible deviation was the reason why in the modelling approach a two-dimensional mathematical flow field model was included (see Fig. 2).

Another aspect to consider is local scour. Local scour is only reproduced correctly in undistorted models that satisfy the Froude condition. Both conditions are not fulfilled, hence the local scour is not reproduced on scale. Also for the study of the local scour a separate model was used (see Fig. 2). This had to be a physical model as no mathematical models are available for predicting local scour.

Considering these scale effects (and some more, see Klaassen (1990)) it may be argued whether it has been worthwhile using a physical model so subjected to scale effects. The answer is definitively affirmative. The model has provided a good insight into critical issues in the design of the guide bunds and the river training works, which were subsequently tackled in additional studies.

## 6.3 Additional study

During working with the physical model described here it was found that outflanking channels could develop that would endanger the bridge approaches (see Figure 9), the length of the guide bunds to be provided being the most critical issue in this respect. An additional study was carried out to evaluate the influence of scale effects on this outflanking. This additional study was partly done in an experimental facility that could be tilted so as to study the effect of the Froude number. In addition satellite images were studied to determine the conditions under which cutoffs occur. These cutoffs were viewed as the mechanism that limits the extent of outflanking. Based on the obtained results a substantial reduction in guide bund length was decided upon, when compared with design rules for guide bunds in meandering rivers (see Garg 2 et al, 1970). For more details, reference is made to RPT/NEDECO/BCL (1990b).

## 7 CONCLUSIONS

In this paper a review is given of the experience obtained, when performing tests in a physical model with mobile bed and banks of the Jamuna River in Bangladesh. The following conclusions can be drawn from the results of this model investigation:

(1) Although the model had only limited degrees of freedom, a fair reproduction of the channel pattern of the Jamuna River was obtained, including the continuous changes in this channel pattern and the intermittent bank erosion typical for braided river systems with fine sand as bed material.



(2) To compensate for the scale effects additional mathematical and physical model investigations were carried out.

(3) The combination of physical models with mathematical ones, together with the extensive analysis of field data, yielded a very good basis for the determination of the design boundary conditions for the river training works for the future Jamuna Bridge.

(4) In addition, the observations in the model, later supported by an additional study, were instrumental in substantially reducing the length of the guide bunds, when compared to the traditional design rules presently in use on the Indian sub-continent.

## 8 ACKNOWLEDGEMENTS

The scale model investigation discussed here is part of the Jamuna Bridge Appraisal Study Phase II, which is funded by UNDP, the World Bank being the executive agency. The study was carried out by the combination RPT (Rendel, Palmer and Tritton, U.K.)/ NEDECO (Netherlands Engineering Consultants)/BCL (Bangladesh Consultants Ltd.); the JMBA (Jamuna Multipurpose Bridge Authority, Bangladesh) is the client. DELFT HYDRAULICS is one of the partners of NEDECO for this study. The assistance of DELFT HYDRAULICS' staff, in particular Messrs. F. de Groot, M.C.L.M. van Mierlo, H. Tukker, H. Dikken, R. Hulleman and J.P. Bouma, is gratefully acknowledged. Advice on the scaling procedure and during the model investigation was provided by Mr. N. Struiksmā.

## LIST OF SYMBOLS

C = Chezy coefficient ( $\text{m}^{1/2}/\text{s}$ )  
D = particle size (m)  
Fr = Froude number (-), defined via  $\text{Fr} = u/(gh)^{0.5}$   
g = acceleration of gravity ( $\text{m}/\text{s}^2$ )  
h = water depth (m)  
i = stream gradient (-)  
L = typical length (m)  
m = index indicating value of a parameter in the model  
n = scale factor  
 $n_x$  = scale factor of parameter X, defined as  $X_p/X_m$   
p = index indicating value of a parameter in the field  
Q = 'dominant' discharge ( $\text{m}^3/\text{s}$ )  
Re = Reynolds number, defined via  $\text{Re} = (u.h)/\nu$   
u = velocity ( $\text{m}/\text{s}$ )  
 $\alpha$  = coefficient  
 $\beta$  = coefficient (-)  
 $\Delta$  = specific density (-), defined via  $\Delta = (\rho_s - \rho)/\rho$   
 $\rho$  = density of water ( $\text{kg}/\text{m}^3$ )  
 $\rho_s$  = density of sediment ( $\text{kg}/\text{m}^3$ )  
 $\nu$  = dynamic viscosity of water ( $\text{m}^2/\text{s}$ )

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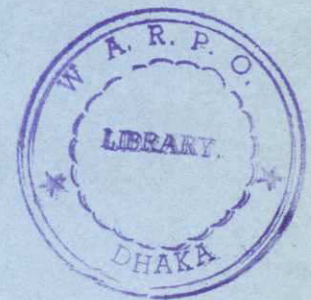


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## 7 River Training

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## Paper 7.1



VAN DER WAL, M. and H. BRÜHL (1993), Training of a braided river with surface screens, *Proc: 25th IAHR Biennial Congress Tokyo, Japan, 591-599pp.*



## TRAINING OF A BRAIDED RIVER WITH SURFACE SCREENS

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## SUMMARY

One of the components of the Flood Action Plan in Bangladesh is a study of how by using recurrent measures one can optimally train the Jamuna river. After a study of various recurrent measures for training a braiding river like the Jamuna river, bandalling and surface screens were selected as the most promising methods. The hydrodynamic and morphological effectiveness of a row of surface screens, placed at a bifurcation and upstream of the mouth of an outflanking channel, is discussed in this paper. The influence of the depth and the length of the row of screens on this effectiveness is shown. This effectiveness is also determined by the length of the outflowing channels, their length ratio and the discharges in these channels. Some aspects for testing this measure in the Jamuna river are described, which may be carried out after the confirmation of the presented study results by further scale model investigations.

## 1 INTRODUCTION

After Bangladesh had experienced its two most severe floods on record in 1987 and 1988, several initiatives were launched for finding solutions to mitigate the worst effects of such floods. In 1989 it was decided to coordinate these initiatives in order to work towards a comprehensive plan for future developments, which resulted in the Flood Action Plan (FAP). The Jamuna river is the lower reach of the Brahmaputra river in Bangladesh up to its confluence with the Ganges river. The Jamuna river is a large braiding river with two to three parallel branches in a cross-section with a width varying between 5 and 17 km. The wide valley of the river is protected partly from flooding by levees and embankments. However, these levees and embankments cannot resist a strong flow attack in an outflanking channel and as result breaches of the embankment occur regularly.

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Within the frame-work of the FAP the River Training and Active Flood Plain Management Pilot Project (FAP.22) has investigated the feasibility of recurrent river-training methods to stabilize the course of the Jamuna river and to reduce the risk of a sudden development of an outflanking channel. In this paper a summarising description is given of surface screens as a promising recurrent measure to train the Jamuna river.

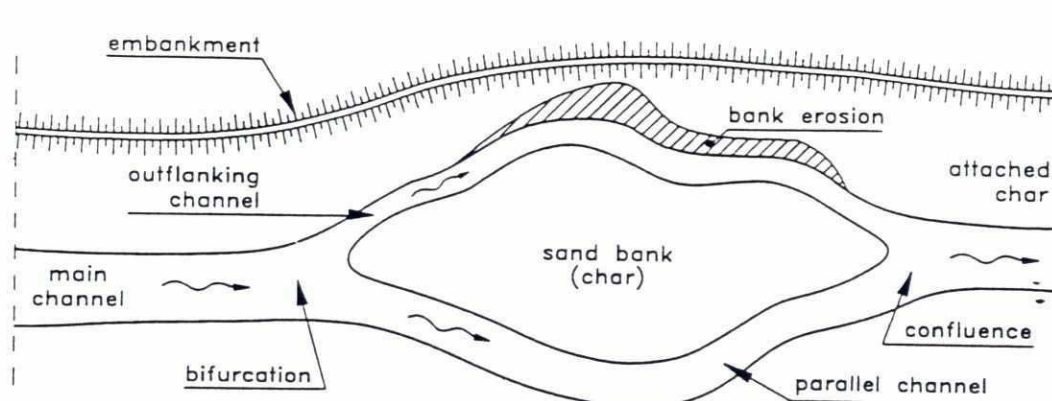


Figure 1 The schematized layout of an outflanking channel and a bifurcation in a braiding river

## 2 RIVER TRAINING WITH RECURRENT MEASURES

A prosperous development of agriculture, infrastructure and villages along the banks of the braiding Jamuna river is hampered by the threat of rapid changes in the location of channels and sand bars, sometimes causing bank erosion of up to 1 km in one flood season. A traditional approach is to narrow the river-bed with permanent river-training works such as groynes, dikes and revetments on a large scale. After decades this can result in a transformation of the now braided river into either (depending on the selected strategy) an anabranching river with fixed islands and fixed nodes, or into a river with one meandering channel. In both strategies the required permanent river-training structures would be very costly, especially in combination with the risk that these structures lay idle for quite some time.

Therefore, it was decided to study the feasibility of a more promising strategy. This strategy aims at a transformation of the river's planform by applying "flexible" recurrent measures like bandalling and surface screens at bifurcations, to defer or to close off particularly aggressive, eroding outflanking channels, see figure 1. This measure will reduce gradually the braiding index (given by the number of channels in a cross-section) and in the end result in a more stable multi-channel planform. At that stage it can be decided to continue applying recurrent measures to maintain the reduced braiding index, or to stabilize the multi-channel planform with permanent river-training works.



Surface screens are based on the same principle as the bandalling technique, but surface screens are designed for more extreme conditions than those for bandalling. In the deeper part of a bifurcation, floating barges or pontoons with swinging screens are anchored, see figure 2. The height of each screen can be adjusted so as to maintain the optimum opening under the screen. An advantage of these surface screens is their high flexibility to move them to those locations where a maximum effectiveness is expected.

One or two rows of surface screens, which are placed at a bifurcation, redistribute the discharge and the sediment transport in the outflowing channels of the bifurcation. This redistribution is often the result of a combination of one or more of the following phenomena: (1) the separation of the flow in an upperflow and underflow with a high sediment concentration, (2) the additional resistance of the surface screens for the flow in the outflanking channel, (3) the Bulle effect by a stronger curvature of the flowlines in the mouth of the outflanking channel, and (4) the enforcement of the helical flow near the flow line which separates the discharges to the outflowing channels which results in an increase of the sediment transport in the outflanking channel. In this study the first two phenomena are considered in detail. In the aggressive, eroding outflowing channel a sedimentation process is induced by this measure and after some time the bank erosion will reduce and finally stop. Consequently, the increase of the discharge and the reduction of the sediment transport in the existing parallel channel can result in some additional erosion of the banks and river-bed in that channel.

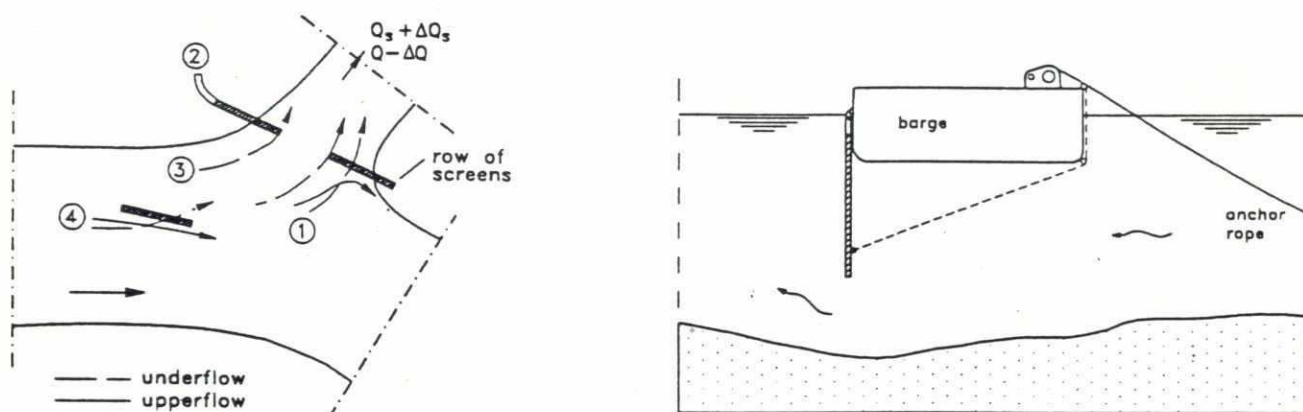


Figure 2 Lay-out and cross-section of a floating surface screen.

### 3 FLOW REDISTRIBUTION

For a first quantitative assessment of the effectivity of that method, the complicated three-dimensional flow field around a surface screen at a bifurcation is schematized to a permanent flow field, which is described with the basic equations of an one-dimensional theory. A typical bifurcation is considered with one inflowing channel, the main channel, and only two outflowing channels. The cross-sections of the channels are schematized as rectangular cross-sections. The average width and the average depth are estimated as a function of the discharge with regime equations, which are based on an analysis

of measured cross-sections in the Jamuna river (Klaassen and Vermeer, 1988). The lengths of the outflowing channels are measured from the bifurcation along the thalweg to the next confluence. The rows of surface screens are placed at the bifurcation more or less as a continuation of the bankline of outflowing parallel channel, which bottom and banks can erode because of the increased discharge.

In the initial situation before the placement of surface screens the equilibrium water depth is calculated using the Chezy equation and a selected discharge in each channel. Next the equilibrium sediment transport in the channels is calculated using a calibrated sediment transport formula (Klaassen, Vermeer and Uddin, 1988). It is checked whether the ratio of the sediment transport in the outflowing channels is almost equal to the ratio of the discharges in the outflowing channels, because of the predominantly suspended sediment transport.

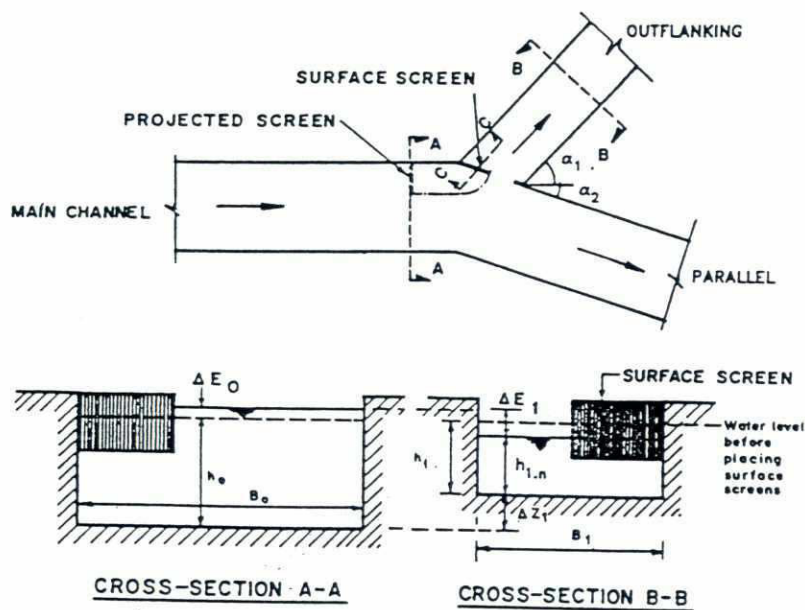


Figure 3 Schematization for the calculation of the energy head difference

is calculated with the formula of Carnot, which has been modified for this situation as follows:

$$\Delta E_0 = \frac{Q_0^2}{2 \cdot g} \left( \frac{1}{A_0 - L_B \cdot (h_0 - h_4 - \Delta z_1) \cdot \sin(\alpha_1 + \alpha_2)} - \frac{1}{A_0} \right)^2$$

in which  $A_0$  = cross-sectional area in the main channel ( $m^2$ ),  $\Delta E_0$  = rise of the energy line upstream of the surface screen (m),  $g$  = acceleration by gravity ( $m/s^2$ ),  $\Delta z_1$  = height difference between the bed level in the main channel and the outflanking channel (m),  $h_4$  = water depth in the contraction just downstream of the surface screen (m), see figure 4,  $L_B$  = length of the rows of surface screens (m),  $Q_0$  = discharge in the main channel ( $m^3/s$ ) and  $\alpha_1$  and  $\alpha_2$  are angles between the centre lines of the channels (degrees).

The surface screens at a bifurcation create a difference between the water level upstream and downstream of the screens, see figure 3.

The cross-sectional area of the surface screen below the water level is projected into the cross-section of the main channel. The partial blockage of that cross-section results in a rise of the water level  $\Delta y_0 = \Delta E_0$  upstream in the main channel. This head difference



Next, the cross-sectional area of the surface screen below the water level is projected into a cross-section of the outflanking channel, and the drop of the water level is estimated by replacing in this formula the parameters for the main channel by the parameters of the outflanking channel. The new, average water level downstream of the surface screen in the outflanking channel, is the old water level +  $\Delta E_0 - \Delta E_1$  if the drop in the water level is assumed to be equal to the drop in energy height,  $\Delta y = \Delta E_1$ . A comparison with the results of a one-dimensional flood-routing model showed that qualitative tendencies are rather well reproduced. This drop in water level results in a reduction of the discharge in the outflanking channel.

The underflow tends to pass the screens perpendicularly and therefore the underflow is schematized in a cross-section perpendicular to the direction of the screen, see Figure 4. Just downstream of the screen the underflow contracts and then decelerates. The Froude

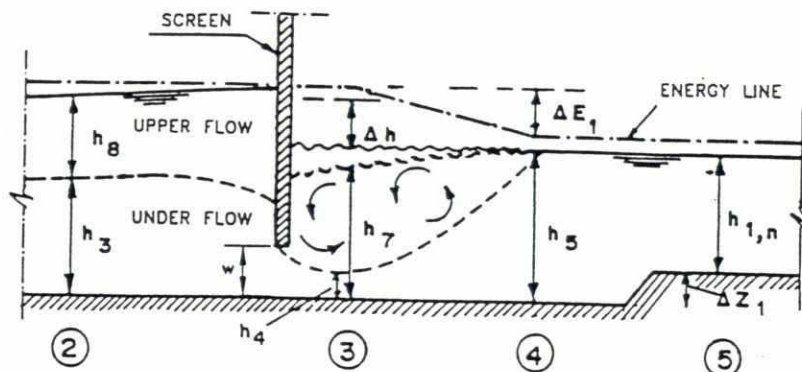


Figure 4 Schematization of the flow in cross-section C-C in figure 3.

number  $F$  of the flow in cross-section 3 with the maximum contraction of the flow determines the type of flow: if  $F < 1$  than the flow is sub-critical, and if  $F \geq 1$  than the flow is super-critical. A free hydraulic jump or a drowned hydraulic jump is determined by a criterion, depending on the height of a free hydraulic jump. The situation with a free hydraulic jump is not considered, because the forces by the flow on the screen will be extremely high.

The Bernoulli equation combined with the continuity equation for the discharge passing the surface screen, is applied to cross-sections 2 and 3 in figure 4:

$$h_3 + h_8 + \frac{q_3^2}{2 \cdot g \cdot h_3^2} = h_5 + \frac{q_3^2}{2 \cdot g \cdot h_4^2}$$

In case of a drowned hydraulic jump  $h_5$  in this formula should be replaced by  $h_7$ , according to Chow, see French, 1987, page 86:

$$h_7 = h_5 \sqrt{1 + 2 \cdot F_5^2 \cdot \left(1 - \frac{h_5}{w}\right)}$$

in which  $h_3$  = water depth of the discharge passing the surface screen (m),  $h_5$  = water depth downstream of the screens (m),  $h_8$  = water depth of the discharge deflected or diverted by the screen (m),  $q_3$  = the specific discharge passing the screen ( $\text{m}^2/\text{s}$ ), and  $F_5$  = Froude number:  $F_5^2 = q_3^2 / (g \cdot h_5^3)$ . French remarks that  $h_7$  calculated with the above formula is comparable with a more complex

formula of G. Rao, who verified his formula with data from laboratory experiments.

The elimination of  $h_3$  or  $q_3$  results in a 4th-order equation. The first approximation is the goniometric solution of a cubic equation, which is obtained if the zero-order term is neglected in that formula. In a second approximation the final solution is calculated.

#### 4 SEDIMENT REDISTRIBUTION

Under flood conditions the sediment transport in the Jamuna river is mainly suspended sediment transport with only a small part (about 10%) bed-load transport. This sediment transport can be calculated using a total-load formula, for example the straightforward Engelund-Hansen formula. An indicative verification of the formula suggests that the sediment transport in that river can be higher (on average 4 times during flood conditions) if calculated with that formula, see Klaassen, Vermeer and Uddin.

The influence of a surface screen on the sediment transport can not be described with only this Engelund-Hansen formula, but it needs the calculation of the sediment concentration vertical. This concentration vertical is calculated with the Van Rijn method (Van Rijn, 1984) and a summary of which can be found in DVWK Bulletin no 17 (1990). In this method the bed-load and the suspended sediment transport are calculated separately. The bed-load transport and the suspended sediment in the concentration vertical between the average bed level and the separation between the underflow and the upperflow at  $y = h_3$  passes under the screens. The screens function optimally as most of the sediment passes the screens in the underflow, and only a small part of the total sediment transport is deflected by the screens in the upperflow. This is a simplified method to assess the influence of a row of surface screens on the distribution of the sediment transport in a bifurcation.

#### 5 RESULTS

Some results of the developed calculation method are presented for a case, which is representative for the Jamuna river. After the selection of the discharges the characteristic parameters were determined for the initial morphological equilibrium situation in this case of a bifurcation connected with a main channel and two outflowing channels, see table 1.

The combined effectiveness  $n$  of the redistribution of the discharges and the redistribution of the sediment transport in the outflowing channels is defined as:

$$n = \frac{\Delta Q}{Q} + \frac{\Delta Q_s}{Q_s}$$

in which  $\Delta Q$  = the reduction of the discharge ( $m^3/s$ ) and  $\Delta Q_s$  = the increase of the sediment transport in the outflanking channel ( $m^3/s$ ). This effectiveness is

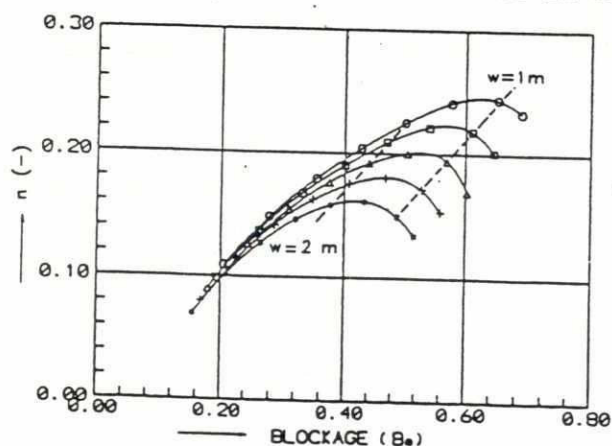


a function of the blockage of a cross-section of the outflanking channel by the area of the rows of screens under the water level. The blockage coefficient is defined as the projected area of the surface screens, divided by the area of the cross-section of the outflanking channel.

| channel             | discharge         | width | depth | length |
|---------------------|-------------------|-------|-------|--------|
| -                   | m <sup>3</sup> /s | m     | m     | km     |
| main channel        | 40,000            | 4,500 | 6.44  | -      |
| outflanking channel | 14,000            | 2,000 | 5.38  | 7      |
| parallel channel    | 26,000            | 3,000 | 5.89  | 6      |

Table 1: Characteristic parameters of a representative case

The height of the opening under the screen,  $w$ , should be more than 2.0 m, which is about 30% of the water depth in the main channel for an optimal effectiveness, see figure 5. The reduction in  $\Delta Q_s/Q_s$  by a smaller height of the opening under the screen is not compensated by the small increase in  $\Delta Q/Q$ . A study of the sedimentation process in a medium size outflanking channel showed that a minimum effectiveness of  $n = 0.2$  is required if the bank erosion should stop



- \*  $L_B = 1200$  m
- +  $L_B = 1300$  m
- $\Delta$   $L_B = 1400$  m
- $\square$   $L_B = 1500$  m
- $\circ$   $L_B = 1600$  m

almost completely after about two flood periods. This is an interesting period because the lifetime of the medium size channels in the Jamuna river is about 3 to 6 years.

Figure 5 The effectiveness as a function of the length of the row of surface screens

The total length of the two rows of surface screens which are separated by a gap in the middle of the outflowing channel should be more than 1400 m to realize a minimum effectiveness of  $n = 0.2$ . If this total length is projected in a cross-section of the outflanking channel, then 65 to 74% of the channel-width is blocked and the blockage coefficient is about 0.5, see figure 6. Calculations indicated that an increase of the sediment transport with 15% and a reduction of the discharge with about 5% in the outflanking channel results in a minimum effectiveness  $n = 0.2$ .

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The length ratio of the channels is defined as the length of the outflanking channel divided by the length of the parallel channel. Increasing the length of both channels and keeping this length ratio constant, has no influence on the effectiveness  $n$ . However, if the length of the outflanking channel is constant and the length ratio is increased from 1.17 to 1.3, then a remarkable increase of  $\Delta Q_s/Q_s$  and a small reduction in  $\Delta Q/Q$  are calculated. In the Jamuna river this length ratio varies between about 0.9 and 1.35. The conclusion is that this method can be very effective if the length ratio is between 1.2 and 1.3.

The discharge ratio of the channels is defined as the discharge of the outflanking channel divided by the discharge in the parallel channel. From the calculations it is concluded that this method can be effective if this discharge ratio is smaller than 0.6.

It has been shown that the effectiveness  $n$  increases as the blockage  $Be$  increases, but at the same time the hydrostatic forces on the screens increase as an exponential function, see Figure 6. Therefore, an optimum between a high effectiveness  $n$  and not-too-high forces on the screens, and consequently on the anchors, is expected if the surface screens are strong enough to resist a waterlevel head difference of about 0.1 m.

This effectiveness  $n$  is based on a discharge below or around the bankfull discharge. If the discharge is higher than the bankfull discharge, the flow over the chars can make the flow pattern much more complicated than the flow pattern represented in this schematized calculation method.

## 6 CONCLUSIONS

In general, a high effectiveness of surface screens at a bifurcation means a relatively fast sedimentation process in an outflanking channel. A high effectiveness by the additional resistance of the surface screens and the separation in an upperflow and an underflow is expected if the following conditions are met:

- The height of the opening under the screen is more than a minimum value.

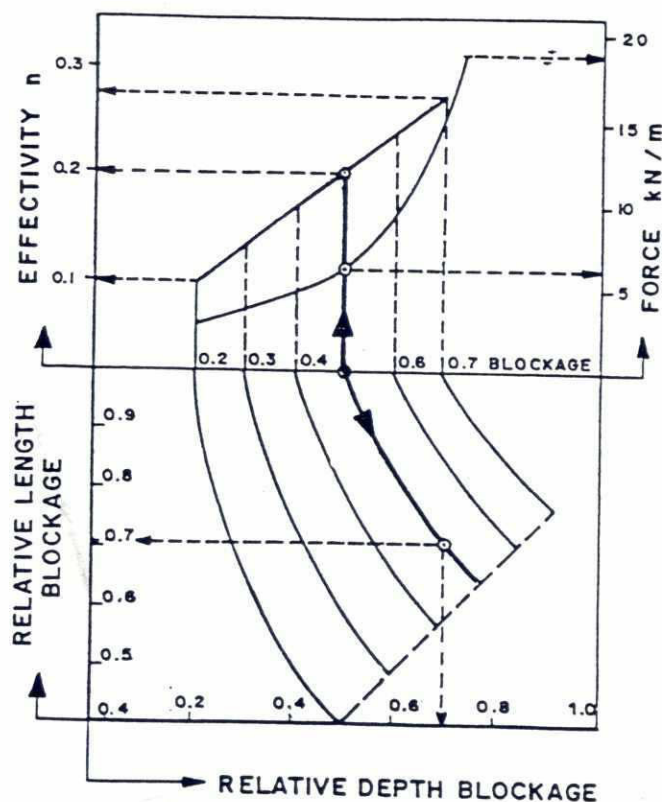


Figure 6 The effectiveness and the force as a function of the length and the depth of a row of surface screens in the presented case



- The projected length of the rows of surface screens in a cross-section of the outflanking channel is more than 50% of the width of that channel.
- The length of the parallel channels has no significant influence on the effectiveness, but the length ratio of the parallel channels should then have a high value and the discharge ratio of the parallel channels should have a low value to realize a high effectiveness.

In addition the influence of the Bulle effect and the change in the helical flow will have a positive effect on the effectiveness. But the present study did not include a quantification of these additional effects.

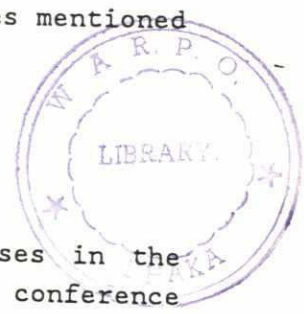
The described schematization of the flow field and the calculated effectiveness can be verified in a physical model investigation in which a few barges with surface screens are placed at a bifurcation. In this model the induced anchor forces on the screens can be measured also. After this verification of the results by means of a physical model investigation a small-scale prototype-test is planned.

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## Paper 7.2

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# RIVER BED AGGRADATION THROUGH RECURRENT MEASURES

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## Abstract

Simplified Models are presented for assessing the river bed aggradation through the application of recurrent measures in both a single channel system and a bifurcating channel system. The simplified Models have been calibrated and verified with computational results of the one dimensional model MIKE 11.

## 1. Introduction

One of the projects of the Flood Action Plan (FAP) for Bangladesh is the River Training Pilot Project (FAP 22). The scope and objectives of this project have been focused on an "active" approach for stabilizing the planform of the Jamuna River. In this respect, the technical feasibility of recurrent measures (e.g. surface screens, bottom vanes, dredging, etc.) has been studied.

The floodplain of the Jamuna River actually consists of a system of channels with islands ("chars") in between. This implies, that the outer channels along the banks of the Jamuna River are frequently headed by a bifurcation point (see Figure 1.1). In case of heavily eroding outer channels with bank erosion as result, the presence of the bifurcations is of prime interest for preventing further outflanking or to promote closing of these outer channels. The application of the recurrent measures should be such that the natural sediment and/or water discharges will be redistributed at the bifurcation. The hydraulic and morphological response to such a redistribution has been estimated with "tailor made" simplified mathematical models. These models should give a good insight in the sensitivity of the result to variations in a number of parameters (such as: Chézy's roughness co-efficient, grain size  $D_{50}$ , water level slope  $i$ , sediment transport, etcetera) on the prediction of discharge and bed level changes.

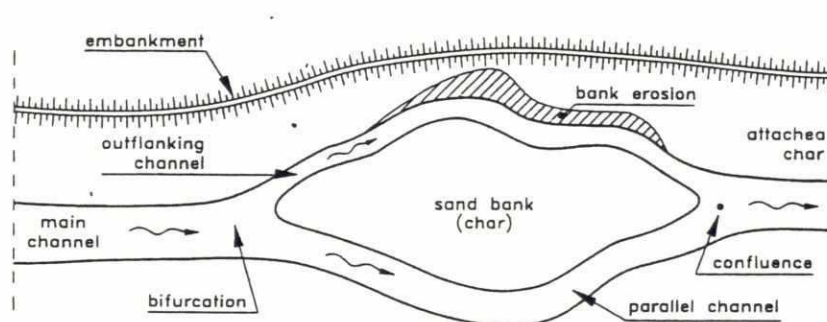


Figure 1.1 Layout of bifurcating channel system along the outer bank.

A simple mathematical description of this time dependant process should facilitate a sensitivity analysis resulting in preliminary guidelines for optimization of the effectivity (i.e. dimensions of the recurrent measure) with respect to the life time of channels in an highly dynamic river like the Jamuna. For achieving these objectives, the redistribution of flow

through the recurrent measure is schematized by a constant sediment overloading  $\Delta S$  and constant discharge withdrawal  $\Delta Q$ . The aggradation in the outer channel will last until the original discharge has completely stopped. At that time a complete closure of the outer branch has been achieved. The required simplified mathematical model has been developed by schematizing the aggradation in a single channel (Chapter 2) in order to extend this to a bifurcating channel system (Chapter 3).

## 2. Single channel system

### 2.1 Set up of Simplified Model

The hydraulic and morphological process in a single channel can generally be described with motion and continuity equations for both the water and sediment discharge (e.g. see Jansen et al., 1979). Such equations can be used to derive an equation with respect to the bottom level changes. Considering channel cross section averaged flow conditions, the resulting equation (see also List of Symbols) is as follows:

$$\frac{\partial z_b}{\partial t} + c \frac{\partial z_b}{\partial x} = R(x, t) \quad (R = \text{damping parameter}) \quad (2.1)$$

In case of a constant sediment overloading  $q_s$  to a single channel with initial flat bottom and water surface, the bottom level  $z_b(x, t)$  will rise and the sedimentation front propagates with celerity  $c$  (see Figure 2.1)

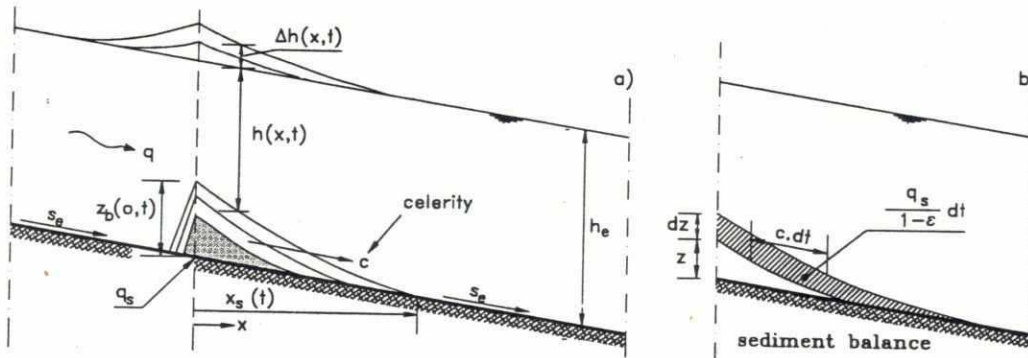


Figure 2.1 Sedimentation due to constant sediment overloading  $q_s$

The propagation celerity  $c$  will not be constant due to the sediment balance for the bottom disturbance. As it has been observed from computations with the one-dimensional model MIKE 11, the most important sedimentation features are a negative sloping front, a time dependant propagation celerity  $c(t)$  and a time dependant aggradation height  $z_b(0, t) = z(t)$ . The sediment balance leads to the following relation (see Figure 2.1b):

$$z.c.dt = \frac{q_s}{1-\epsilon} . dt \quad (\epsilon = \text{porosity of sediment}) \quad (2.2)$$

From Equation (2.2) it becomes clear that the propagation celerity  $c$  decreases while the aggradation height  $z$  increases and the sediment overloading  $q_s$  remains constant, according to:



$$c = \frac{q_s}{1-\epsilon} \cdot \frac{1}{z} \quad (2.3)$$

The validity of Equation (2.3) only holds during the phase, in which the aggradation height  $z$  is less than a value  $z_e$ . In this phase the celerity decreases to a constant value  $c_e$  which can be considered as the equilibrium celerity defined by the river characteristics. Assuming a fifth power relation between sediment transport and water velocity in equilibrium conditions, e.g.  $s_e = a_s(q/h_e)^5$  according to Engelund and Hansen (1967), the equilibrium celerity of a bottom disturbance is approximated by:

$$c_e = \frac{5s_e}{h_e} \cdot \frac{1}{(1-\epsilon)} \quad (2.4)$$

which, according to Equation (2.3), also should meet to:

$$c_e = \frac{q_s}{1-\epsilon} \cdot \frac{1}{z_e} \quad (2.5)$$

Reference is made to the List of Symbols and to Figure 2.1 with respect to the symbols used in Equations (2.4) and (2.5). As a consequence of these equations the equilibrium aggradation height  $z_e$  should meet to:

$$z_e = 0.2 n h_e \quad (n = q_s/s_e) \quad (2.6)$$

The following pragmatic assumptions (see Figure 2.2) have been made to solve the basic Equation (2.1):

- (1) the general solution is:

$$z_b(x, t) = z(t) \cdot \left(1 - \frac{x}{x_s}\right)^\alpha \quad (2.7)$$

- (2) the initial slope of the front holds to:

$$\frac{\partial z_b}{\partial x}(0, t) = a(t) = \left(\frac{z_e}{z}\right)^\beta a_e \quad (2.8)$$

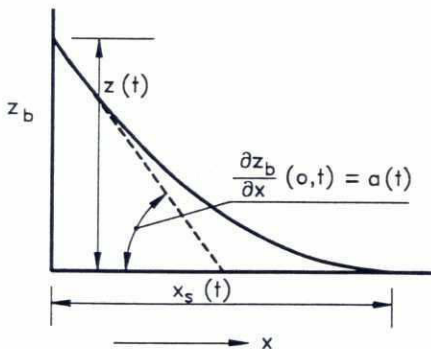


Figure 2.2 Typical front of the sedimentation

The front of the sedimentation should be linear if the power  $\alpha=1$ . However, MIKE 11 results and flume tests (see Ribberink and Van der Sande, 1984) do not show such linear fronts under the condition of a constant sediment overloading. Therefore a variable  $\alpha$  has been taken. The gradient of the front on  $x=0$  has been assumed to behave according to Equation (2.8). In addition the gradient  $a_e$  is defined as the initial slope of the front corresponding to the equilibrium aggradation height  $z_e$ .

According to Equations (2.7) and (2.8) the following should hold:

$$\frac{\partial z_b}{\partial x}(0,t) = -\alpha \frac{z}{x_s} = a(t) = \left(\frac{z_e}{z}\right)^\beta a_e \quad (2.9)$$

while using Equation (2.7) for defining the gradients  $\partial z_b/\partial t$  and  $\partial z_b/\partial x$  and substituting them into the basic Equation (2.1), the following result has been obtained:

$$\frac{dz}{dt} - \alpha \frac{z}{x_s} \cdot c = R \left(1 - \frac{x}{x_s}\right)^{1-\alpha} + \frac{x}{x_s} \left(\frac{dz}{dt} - \alpha \frac{z}{x_s} \frac{dx_s}{dt}\right) \quad (2.10)$$

Equation (2.10) shows clearly the significance of the damping parameter R. If this parameter would be neglected, the only solution could be a bottom sedimentation with constant slope ( $z/x_s$ ) of the front and consequently  $\alpha=1$ . This is not in agreement with observations from the 1-D computations as well as from model tests. Therefore the following assumption has been made in order to achieve a more general solution for the differential Equation (2.10):

$$R \left(1 - \frac{x}{x_s}\right)^{1-\alpha} + \frac{x}{x_s} \left(\frac{dz}{dt} - \alpha \frac{z}{x_s} \frac{dx_s}{dt}\right) = (\lambda - 1) \alpha \frac{z}{x_s} c \quad (2.11)$$

Substitution of this result as well as the result of Equation (2.9) into Equation (2.10) leads to:

$$\frac{dz}{dt} + \lambda \cdot c \left(\frac{z_e}{z}\right)^\beta \cdot a_e = 0 \quad (2.12)$$

Using the celerity function according to Equation (2.3) the following solution has been obtained:

$$(i) \quad z = z_e \left(\frac{t}{T_e}\right)^{\frac{1}{2+\beta}} \quad (\text{for } t < T_e) \quad (2.13)$$

$$(ii) \quad z = z_e \left\{ 1 + \frac{1+\beta}{2+\beta} \left(\frac{t}{T_e} - 1\right) \right\}^{\frac{1}{1+\beta}} \quad (\text{for } t \geq T_e) \quad (2.14)$$

$$(iii) \quad T_e = -\frac{z_e^2 \cdot (1-\epsilon)}{(2+\beta) \cdot \lambda \cdot a_e \cdot n \cdot s_e} \quad (2.15)$$

with:

$$(iv) \quad z_e = 0.2 \cdot n \cdot h_e \quad (2.16)$$

$$(v) \quad \frac{dz}{dt}(t=T_e) = \frac{z_e}{2+\beta} \cdot \frac{1}{T_e} \quad (2.17)$$

$$(vi) \quad \alpha = -a_e \cdot \frac{x_e}{z_e} \quad (\text{see Eq. 2.9}) \quad (2.18)$$



The symbols are explained in the List of Symbols. The unknown parameters are  $\alpha$ ,  $\beta$ ,  $\lambda$  and  $a_e$ . These parameters have been fitted and calibrated with the use of 1-D computational results as described in the following Section.

## 2.2 Calibration of the Simplified Model

The parameters  $\alpha$ ,  $\beta$ ,  $\lambda$  and  $a_e$  can be determined indirectly either from physical model tests or from computational results with a simulation model. The present Simplified Model has only been calibrated and verified with 1-D computational results. The most suitable output for calibration and verification consists of:

- plots giving the aggradation height  $z [= z_b(0,t)]$  as a function of time,
- plots giving longitudinal presentations of the aggradation after various time periods.

Four characteristic parameters can be collected very easily from such plots with the known boundary condition for the equilibrium aggradation height ( $z_e = 0.2 n h_e$ ). These parameters are:

|       |                                               |          |
|-------|-----------------------------------------------|----------|
| $T_e$ | = a morphologic time ( Eq. 2.15)              | (years)  |
| $b_e$ | = $dz/dt$ on $t=T_e$ (Eq. 2.17)               | (m/year) |
| $a_e$ | = slope of the sedimentation front on $t=T_e$ | (-)      |
| $x_e$ | = sedimentation length on $t=T_e$             | (km)     |

Seven calibration runs have been made. The input data and results of these runs are summarized in Table 2.1. The parameters  $\alpha$ ,  $\beta$  and  $\lambda$  have been determined with Equations (2.18), (2.17) and (2.15), while the fourth parameter  $\gamma$  has been determined on the basis of the assumed relation:

$$a_e = \gamma n i \quad (2.19)$$

Table 2.1 Calibration of Simplified Model

| INPUT: $C = 60 \text{ m}^{1/2}/\text{s}$ , $i = 7 \cdot 10^{-5}$ , $D_{50} = 2 \text{ mm}$                                                                                                                                      |                 |                          |                                     |        |                 | MIKE 11 OUTPUT   |                |                  |                    | CALIBRATION                                                                                                         |         |           |          |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------|--------------------------|-------------------------------------|--------|-----------------|------------------|----------------|------------------|--------------------|---------------------------------------------------------------------------------------------------------------------|---------|-----------|----------|
| Case                                                                                                                                                                                                                            | $h_e(\text{m})$ | $q(\text{m}^2/\text{s})$ | $s_e(\text{m}^2/\text{s})^{\gamma}$ | $n(-)$ | $z_e(\text{m})$ | $T_e(\text{yr})$ | $a_e(10^{-5})$ | $x_e(\text{km})$ | $b_e(\text{m/yr})$ | $\alpha$                                                                                                            | $\beta$ | $\lambda$ | $\gamma$ |
| 1                                                                                                                                                                                                                               | 5.1             | 5.8                      | $2.7 \cdot 10^{-4}$                 | 1.37   | 1.4             | 1.6              | -3.7           | 41               | 0.32               | 1.08                                                                                                                | 0.73    | 0.62      | -0.38    |
| 2                                                                                                                                                                                                                               | 5.1             | 5.8                      | $2.7 \cdot 10^{-4}$                 | 4.11   | 4.2             | 2.1              | -10.3          | 57               | 0.90               | 1.40                                                                                                                | 0.22    | 0.63      | -0.36    |
| 3                                                                                                                                                                                                                               | 3.0             | 2.6                      | $0.68 \cdot 10^{-4}$                | 5.44   | 3.3             | 3.4              | -13.0          | 40               | 0.45               | 1.58                                                                                                                | 0.16    | 0.58      | -0.34    |
| 4                                                                                                                                                                                                                               | 3.0             | 2.6                      | $0.68 \cdot 10^{-4}$                | 7.35   | 4.41            | 3.45             | -17.0          | 45               | 0.60               | 1.74                                                                                                                | 0.13    | 0.59      | -0.33    |
| 5                                                                                                                                                                                                                               | 4.0             | 4.0                      | $1.40 \cdot 10^{-4}$                | 2.64   | 2.1             | 2.2              | -6.3           | 42               | 0.40               | 1.26                                                                                                                | 0.39    | 0.69      | -0.34    |
| 6                                                                                                                                                                                                                               | 7.0             | 9.3                      | $5.63 \cdot 10^{-4}$                | 0.66   | 0.92            | 1.45             | -1.7°          | >50°             | 0.18°              | - °                                                                                                                 | 1.5     | 0.51°     | -0.37    |
| 7                                                                                                                                                                                                                               | 8.0             | 11.4                     | $8.0 \cdot 10^{-4}$                 | 0.46   | 0.74            | 1.25             | -1.1°          | >50°             | 0.14°              | - °                                                                                                                 | 2.2     | 0.50°     | -0.34    |
| $s_e = \frac{0.05}{\Delta^2 \sqrt{g} C^3 D_{50}} \left( \frac{q}{h_e} \right)^3$ according to Engelund and Hansen (1967)<br>°) results probably influenced by boundary conditions in MIKE11 ( $x_e >$ boundary length of 50 km) |                 |                          |                                     |        |                 |                  |                |                  |                    | <b>Results</b><br>$\alpha = 1 + 0.1 n$<br>$\beta = 1/n$<br>$\lambda = 0.6 (\approx 1-\epsilon)$<br>$\gamma = -0.35$ |         |           |          |

The calibration runs have been executed with a sediment porosity  $\epsilon$  resulting in the relations as presented in Table 2.1 for the parameters to be determined. The impression exists that the porosity  $\epsilon$  is only affecting the sedimentation shape in such a way that  $T_e$ ,  $a_e$  and  $b_e$  remain unchanged while  $x_e$  will change with varying  $\epsilon$ . The described influence of the porosity  $\epsilon$  on the sedimentation process could be included in the Simplified Model by multiplying the fitted relation for  $\alpha$  with  $0.6/(1-\epsilon)$  and assuming that  $\lambda=1-\epsilon$ . This should be investigated and verified in more detail. The present Simplified Model holds for conditions with porosity

$\epsilon=0.4$ . Any deviation from this condition should be done with care taking into account the recommended influence of  $\epsilon$  on the parameters  $\alpha$  and  $\lambda$ .

Figure 2.3 shows a calibration run for the aggradation height  $z$  regarding case 3 of Table 2.1 as well as a verification run for a Jamuna flood condition ( $q=34 \text{ m}^2/\text{s}$ ,  $h_e = 15 \text{ m}$ ,  $C = 70 \text{ m}^{1/2}/\text{s}$ ,  $s_e = 5.14 \times 10^{-3} \text{ m}^2/\text{s}$  and  $n = 2$ ).

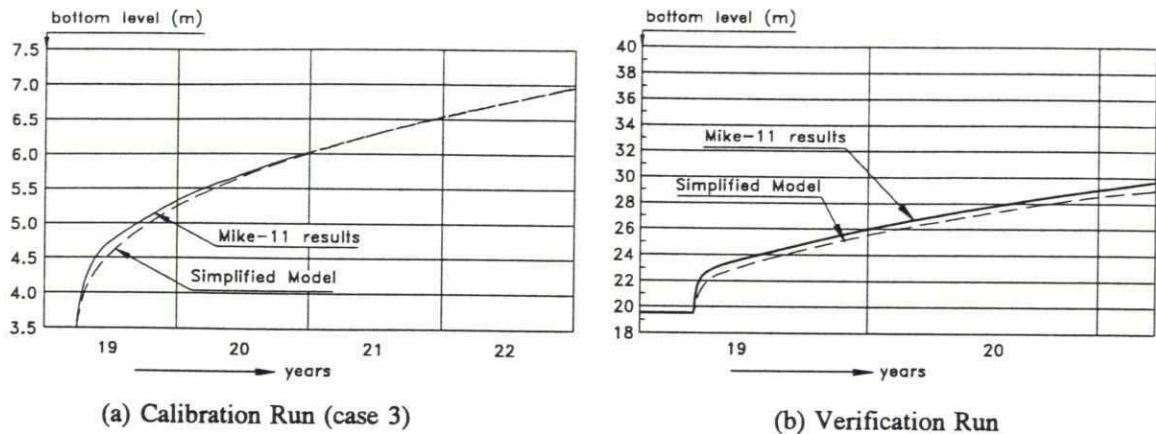


Figure 2.3 Calibration and Verification of Simplified Model

The set of Equations (2.9) and (2.13) through (2.18) including the calibration results of Table 2.1 provides the information on the aggradation to be expected, which is presented dimensionless in Figure 2.4. This figure presents design graphs for the aggradation height and length enabling a simple sensitivity analysis to a designer.

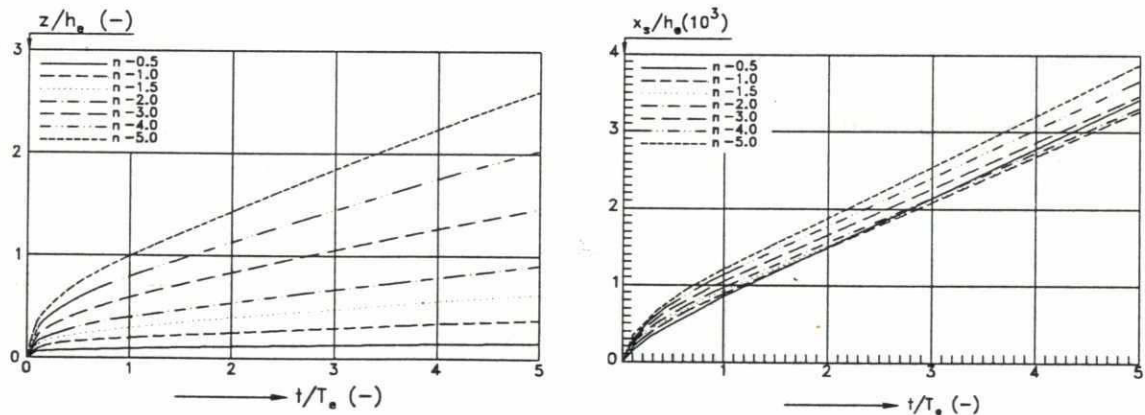


Figure 2.4 Aggradation dimensions depending on time and sediment overloading in a single river.

Once the morphological time scale  $T_e$  according to Equation (2.15) has been determined from the known river characteristics and the sediment transport ratio  $n$ , the aggradation height and length can be assessed easily from Figure 2.4.



### 3. Aggradation in outflanking channels

#### 3.1 General

The Simplified Model as presented in Section 2 has been adapted for predicting the aggradation in an outflanking channel downstream of a bifurcation as result of the sediment transport and/or water discharge redistribution. This redistribution should be considered as the effect of a recurrent measure (e.g. surface screens) at the bifurcation. Referring to Figure 3.1 the redistribution of sediment transport has been schematized as respectively a constant sediment supply  $+\Delta S$  in the outflanking channel (1) and a constant sediment withdrawal  $-\Delta S$  from the inner channel (2). Similar schematizations have been made regarding a constant water discharge withdrawal  $\Delta Q$  from the outflanking channel into the inner channel.

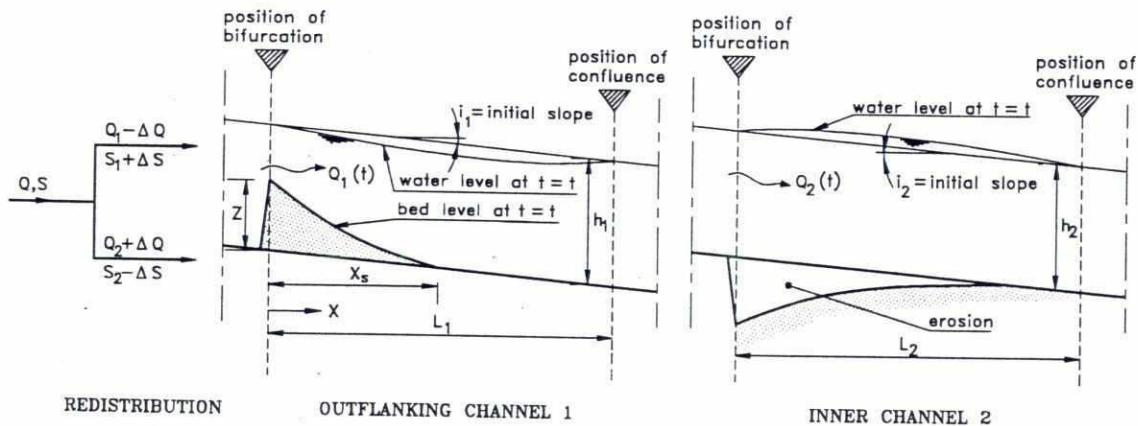


Figure 3.1 Morphological response due to flow and/or sediment transport redistribution

The induced aggradation and erosion as result of the flow and/or sediment transport redistribution is presented schematically in Figure 3.1 while the governing symbols are also indicated.

#### 3.2 Adaptation of the Simplified Model

MIKE 11 runs, with separate flow and sediment transport redistributions as well as combined redistributions, have shown that any redistribution at the bifurcation might be reduced to a fictive sediment transport redistribution. Therefore, the following aspects have been considered for adapting the Simplified Model (see Section 2) to a bifurcating channel system:

(1) The aggradation process as derived for a single channel (see Section 2) holds only during a relatively short initial period  $t < T_a$ , in which the water level is supposed not to be affected.

(2) The water depth at the crest of the aggradation decreases from its initial equilibrium water depth  $h_1$  to a water depth  $h_a$ , which enables a total sediment transport  $S_1 + \Delta S$  on  $t = T_a$ . Applying the effectivity  $n = \Delta S / S_1$  and the sediment transport equation according to Engelund and Hansen (1967), the aggradation height  $z_a$  satisfies to:

$$z_a = h_1 - h_a = \{1 - (n+1)^{-0.2}\} h_1 \quad (3.1)$$

(3) The time  $T_a$  is defined by substitution of Equation (3.1) in Equation (2.13), taking  $z_e = 0.2 n h_1$ ,  $T_e = T_1$  and  $\beta = 1/n$ . The result is as follows:

$$\frac{T_a}{T_1} = \left[ \frac{5}{n} \{1 - (n+1)^{-0.2}\} \right]^{\frac{2n+1}{n}} \quad (3.2)$$

with  $T_1$  being the morphological time scale of the outflanking branch according to Equation (2.15):

$$T_1 = \frac{\lambda}{5.25} \frac{h_1^2 \cdot B_1}{S_1 \cdot i_1} \cdot \frac{n}{2n+1} \quad (3.3)$$

(4) The discharge in the outflanking channel is only time dependant and decreases approximately linearly. The discharge gradient on the location of the aggradation crest ( $x=0$ ) should satisfy, according to the continuity requirement, to:

$$\frac{dQ}{dt} = B_1 (h_1 - z) \frac{du}{dt} - B_1 u \frac{dz}{dt} \quad (3.4)$$

Neglecting the acceleration term and assuming that  $u$  remains constant after  $t=T_a$  and that on  $t=T_a$  the local water depth above the aggradation crest is equal to  $h_a$  (see Eq. 3.1), the following result has been obtained:

$$Q = Q_1 + \frac{dQ}{dt} \cdot t = Q_1 + \dot{Q}_a \cdot t = Q_1 - \frac{Q_1}{h_1} (n+1)^{0.2} \cdot \dot{z}_a \cdot t \quad (3.5)$$

while the gradient  $\dot{z}_a$  can be computed from Equation (2.13) with the result:

$$\dot{z}_a = \frac{n^2}{5(2n+1)} \left( \frac{T_a}{T_1} \right)^{-\frac{n+1}{2n+1}} \cdot \frac{h_1}{T_1} \quad (3.6)$$

A number of 1-D calibration runs has been made for determining the unknown parameter  $\lambda$  in the morphological time scale as defined with Equation (3.3) as well as the effectivity  $n$ . The discharge gradients as computed from the 1-D calibration runs have led to the following results:

$$\lambda = 0.6 + 0.0675 \frac{\sqrt{g} h_1}{C_2 i_1^{1.5} L_1} \quad (3.7)$$

$$n = \frac{\Delta S}{S_1} + \frac{\Delta Q}{Q_1} \quad (3.8)$$



To illustrate the agreement between MIKE 11 results and the results of the adapted Simplified Model two examples are given in Figure 3.2. This Figure presents the discharge as a function of time for both examples with input data according to Table 3.1.

Table 3.1 Input data

|                                                                  |                                |                    |                                              |                    |                                           |                               |                                              |                       |                                                   |
|------------------------------------------------------------------|--------------------------------|--------------------|----------------------------------------------|--------------------|-------------------------------------------|-------------------------------|----------------------------------------------|-----------------------|---------------------------------------------------|
| MAIN CHANNEL                                                     |                                |                    | $Q = 12,000 \text{ m}^3/\text{s}$            |                    |                                           | $B = 2,000 \text{ m}$         |                                              |                       |                                                   |
| OUTER/INNER CHANNEL                                              |                                |                    | $Q_1 = Q_2 = 6,000 \text{ m}^3/\text{s}$     |                    |                                           | $B_1 = B_2 = 1,000 \text{ m}$ |                                              |                       |                                                   |
| CASES                                                            | MAIN / OUTER / INNER / CHANNEL |                    |                                              |                    |                                           |                               |                                              |                       |                                                   |
|                                                                  | $L_1 = L_2$<br>(km)            | $h_1 = h_2$<br>(m) | $C_1 = C_2$<br>( $\text{m}^{1/2}/\text{s}$ ) | $i_1 = i_2$<br>(-) | $\Delta Q^*$<br>( $\text{m}^3/\text{s}$ ) | $\Delta Q/Q_1$<br>(-)         | $\Delta S^{**}$<br>( $\text{m}^3/\text{s}$ ) | $\Delta S/S_1$<br>(-) | $n = \frac{\Delta s}{S_1} + \frac{\Delta Q}{Q_1}$ |
| 1                                                                | 24                             | 5.23               | 60                                           | $7 \cdot 10^{-5}$  | 1,200                                     | 0.2                           | 0.081                                        | 0.3                   | 0.5                                               |
| 2                                                                | 6                              | 5.23               | 60                                           | $7 \cdot 10^{-5}$  | -                                         | -                             | 0.135                                        | 0.5                   | 0.5                                               |
| *) positive $\Delta Q$ value means withdrawal from outer channel |                                |                    |                                              |                    |                                           |                               |                                              |                       |                                                   |
| **) positive $\Delta S$ value means supply to outer channel      |                                |                    |                                              |                    |                                           |                               |                                              |                       |                                                   |

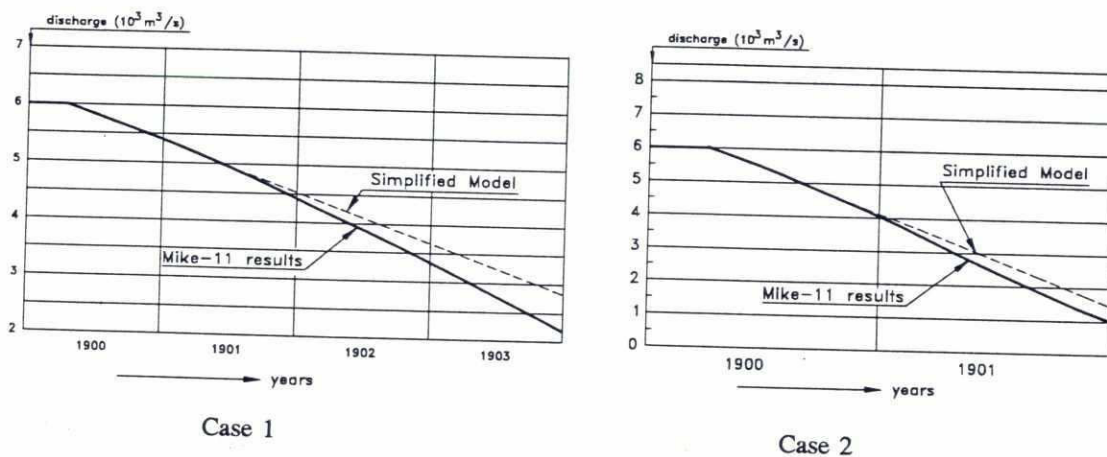


Figure 3.2 Decreasing discharges in outflanking channel

### 3.3 Closure of outflanking channels

The applicability of recurrent measures at bifurcations in the Jamuna River depends strongly on the time necessary for achieving a closure of the outflanking channel within the life time of such a channel. Some indications for the life time of the various Jamuna channels are roughly as follows:

- small channels : life time 1 to 3 years, length 8 to 10 km
- medium channels : life time 3 to 6 years, length 12 to 20 km

These indications can be considered as the boundary conditions for the type of recurrent measures to be applied. The time as required to achieve a closure of an outflanking channel can be derived from the Simplified Mathematical model as presented in Section 3.2. The substitution of the boundary condition that the discharge has completely stopped ( $Q = 0$ ) in the outflanking channel, into the discharge equation (see Eq. 3.5) leads to the closure time ( $= T_c$ ). The closure time depends on: (i) a variety of river characteristics with specific emphasis on the channel length and depth and (ii) the effectivity ( $= n$ ) of the recurrent measures. Computations have shown that surface screens with an effectivity  $n=0.2$  to  $0.3$

(see Van der Wal, 1993) seem to be a very attractive alternative for preventing bank erosion. Partial or complete closures seem possible within the life times of the various channels.

#### 4. Conclusions and Recommendations

The analysis of the river behaviour occurring as the morphological response to various measures (sediment overloading, water withdrawal and/or combinations) has led to the following conclusions and recommendations:

- (i) It has been shown that a Simplified Mathematical description of the hydraulic and morphological changes due to various (recurrent) measures is possible.
- (ii) Partial or complete closures of outflanking channels seem to be possible within time limits less than the life time of various Jamuna channels. Principles for selection of channel type and effectivity of recurrent measures are presented in Section 3.3.
- (iii) The calibration and verification of the various Simplified Models has been done with computation results of the one-dimensional model MIKE 11. Due to the schematization of the hydraulic and morphological process the validity of the developed Simplified Models should be checked and/or upgraded with physical model tests. Only such tests can produce experimental data for similar hydraulic and morphological schematizations. Once having a better insight in the validity of the Simplified Models, pilot tests in the Jamuna are recommended.

#### Acknowledgement

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## List of Symbols

|            |                                                                 |                 |
|------------|-----------------------------------------------------------------|-----------------|
| $a_e$      | = aggradation slope on origin at $t=T_e$                        | (-)             |
| $a_s$      | = sediment transport parameter                                  | ( $s^4/m^3$ )   |
| $a(t)$     | = aggradation slope on origin                                   | (-)             |
| $B$        | = width of (main) channel                                       | (m)             |
| $B_1$      | = width of outflanking channel                                  | (m)             |
| $B_2$      | = width of inner channel                                        | (m)             |
| $C$        | = Chezy's roughness parameter                                   | ( $m^{1/2}/s$ ) |
| $C_1$      | = Chezy's roughness parameter of outflanking channel            | ( $m^{1/2}/s$ ) |
| $C_2$      | = Chezy's roughness parameter of inner channel                  | ( $m^{1/2}/s$ ) |
| $c$        | = celerity                                                      | (m/s)           |
| $c_e$      | = equilibrium celerity                                          | (m/s)           |
| $D_{50}$   | = specific grain size passing 50 % of weight                    | (mm)            |
| $g$        | = acceleration due to gravity (=9.81)                           | ( $m/s^2$ )     |
| $h$        | = local water depth                                             | (m)             |
| $h_1$      | = initial water depth of outflanking channel                    | (m)             |
| $h_2$      | = initial water depth of inner channel                          | (m)             |
| $h_a$      | = water depth above aggradation crest at $t=T_a$                | (m)             |
| $h_e$      | = initial equilibrium water depth                               | (m)             |
| $i$        | = water level slope                                             | (-)             |
| $i_1$      | = water level slope of outflanking channel                      | (-)             |
| $i_2$      | = water level slope of inner channel                            | (-)             |
| $L_1$      | = length of outflanking channel                                 | (m)             |
| $L_2$      | = length of inner channel                                       | (m)             |
| $n$        | = $q_s/s_e$ , $\Delta Q/Q_1 + \Delta S/S_1$ = effectivity       | (-)             |
| $Q$        | = discharge in outflanking channel                              | ( $m^3/s$ )     |
| $Q_1$      | = initial discharge in outflanking channel                      | ( $m^3/s$ )     |
| $Q_2$      | = initial discharge in inner channel                            | ( $m^3/s$ )     |
| $Q_a$      | = discharge gradient at $t=T_a$                                 | ( $m^3/s^2$ )   |
| $q$        | = discharge per unit channel width                              | ( $m^2/s$ )     |
| $q_s$      | = sediment overloading per unit channel width                   | ( $m^2/s$ )     |
| $R$        | = damping parameter                                             | (m/s)           |
| $S_1$      | = initial sediment transport in outflanking channel             | ( $m^3/s$ )     |
| $S_2$      | = initial sediment transport in inner channel                   | ( $m^3/s$ )     |
| $s_e$      | = initial equilibrium sediment transport per unit channel width | ( $m^2/s$ )     |
| $T_a$      | = initial aggradation period                                    | (s)             |
| $T_e$      | = morphological time scale                                      | (s)             |
| $T_1$      | = morphological time scale of outflanking channel               | (s)             |
| $t$        | = time                                                          |                 |
| $u$        | = cross section averaged current velocity                       | (m/s)           |
| $x$        | = distance from origin                                          | (m)             |
| $x_e$      | = aggradation length at $t=T_e$                                 | (m)             |
| $x_s$      | = aggradation length                                            | (m)             |
| $z$        | = aggradation height                                            | (m)             |
| $z_a$      | = aggradation height at $t=T_a$                                 | (m)             |
| $z_a$      | = aggradation gradient at $t=T_a$                               | (m/s)           |
| $z_b$      | = local bottom level                                            | (m)             |
| $z_e$      | = aggradation height at $t=T_e$                                 | (m)             |
| $\alpha$   | = power of bottom level function                                | (-)             |
| $\beta$    | = power of aggradation slope function                           | (-)             |
| $\gamma$   | = a co-efficient related to $a_e$                               | (-)             |
| $\Delta$   | = specific mass (= 1.65)                                        | (-)             |
| $\epsilon$ | = porosity                                                      | (-)             |
| $\lambda$  | = aggradation co-efficient                                      | (-)             |

## **8 Unpublished Reports (not included)**



## 8. Unpublished Reports

### 8.1 Jamuna Bridge Study

- a RPT/NEDECO/BCL (1987a), Jamuna Bridge Study Phase 1: Characteristics and configuration, Annex C - Hydrology, River morphology and River Engineering.
- b RPT/NEDECO/BCL (1987b), Jamuna Bridge Study, Phase II Study, Inception Report.
- c RPT/NEDECO/BCL (1989a), Jamuna Bridge Study, Phase II Study, Feasibility Report, Annex A: Hydrology.
- d RPT/NEDECO/BCL (1989b), Jamuna Bridge Study, Phase II Study, Feasibility Report, Annex B: River Morphology.
- e RPT/NEDECO/BCL (1990), Jamuna Bridge Project, Phase II Study, River Training Works, Design Report, Volume II.

### 8.2 River Training Studies of the Brahmaputra River (FAP 1)

- a HALCROW (1992), First Interim Report, Main Report, Annex 4: Geomorphology, Annex 6: River Engineering, River Training Studies of the Brahmaputra River.
- b HALCROW (1993a), Draft final report, Annex 2, morphology, River Training Studies of the Brahmaputra River (FAP 1).
- c HALCROW (1993b), Master Plan Report, main report, River Training Studies of the Brahmaputra River (FAP 1).
- d HALCROW and RRI (1993), Draft final report, physical model studies, River Training Studies of the Brahmaputra River (FAP 1).

### 8.3 Meghna River bank Protection Short Term Study

- a HASKONING/DELFT HYDRAULICS/BETS (1990a), Meghna River Bank Protection Short Term Study, Volume II, Annex A Hydrology.
- b HASKONING/DELFT HYDRAULICS/BETS (1990b), Meghna River Bank Protection Short Term Study, Volume II, Annex B River Morphology and River Morphology.

### 8.4 China-Bangladesh Joint Expert Study

- a CBJET (1991a), Project on the Brahmaputra River in Bangladesh, Vol. 1 Analysis of hydrology and river morphology of the Brahmaputra River, China-Bangladesh Joint Expert Study.
- b CBJET (1991b), Project on the Brahmaputra River in Bangladesh, Vol. 2 General plan and design of embankment along the Brahmaputra River, China-Bangladesh Joint Expert Study.
- c CBJET (1991c), Project on the Brahmaputra River in bangladesh, Vol. 3 General plan and design for realignment project of the Brahmaputra River, China-Bangladesh Joint Expert Study.

### 8.5 River Bank protection and AFPM Pilot Study (FAP21/22)

- a RRI/L&P/CNR/DELFT HYDRAULICS (1993a), Draft final report planning study, Vol. II: Annex 1 - river training and morphological response, Annex 2 - river morphology, Bank Protection and River Training (AFPM) Pilot Project.
- b RRI/L&P/CNR/DELFT HYDRAULICS (1993b), Draft final report planning study, Vol. III: Annex 3 - surface screens, Annex 4 - simplified mathematical models for investigation of



recurrent AFPM measures, Bank Protection and River Training (AFPM) Pilot Project.

- c RRI/L&P/CNR/DELFT HYDRAULICS (1993c), Draft final report planning study, Vol. VI: Annex 11 - remote sensing study of morphological processes, Bank Protection and River Training (AFPM) Pilot Project.
- d RRI/L&P/CNR/DELFT HYDRAULICS (1993d), Draft final report planning study, Vol. VII: Annex 15 - special scour investigations, Bank Protection and River Training (AFPM) Pilot Project.
- e RRI/L&P/CNR/DELFT HYDRAULICS (1993e), Draft final report planning study Vol. IB: Main report on river training /AFPM,, Bank Protection and River Training (AFPM) Pilot Project.

## 8.6 Other Unpublished Reports

- a HARTMANN, R.A., J. RUPKE and A.C. SEYMONSBERGEN (1992), The influence of neo-tectonic movements on the planform development of the Jamuna River, Bangladesh, DELFT HYDRAULICS/University of Amsterdam, Alpine Geomorphology Research Group (limited distribution).
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**9 Published Papers (not included)**

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## 9 Published Papers (not included)

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## Index



## Index on keywords

Herafter an index is presented to the different papers and reports, either published or unpublished, based on keywords. Whenever appropriate reference is made to pages of the published papers where information can be found. The reference refers to the numbering used in this compilation, while whenever possible page numbers are given. For this the following code is used:

- "2.1/3-5" means "Pages 3 through 5 of Paper 2.1" (hence form the paper by KLAASSEN, G.J. & N. STRUIKSMA (1988a) titled "On the threshold between meandering and braiding", published in Proc. Intern. Conf. on River Regime, Wallingford, England)
- "8.1.3b" means somewhere in report 8.1.3b (hence HASKONING/DELFT HYDRAULICS/BETS (1990a), Meghna River Bank Protection Short Term Study, Volume II, Annex B River Morphology and Geomorphology

The following remarks are further made with respect of the preparation of the index:

- 1 As most of the publications are dealing with specific phenomena (like bank erosion) in specific rivers (like the Jamuna River) they publications have been entered in two places: a geographical one (in this example Jamuna River) and a keyword for the topic (in this example bank erosion).
- 2 In referencing the original page numbers have been used as applied in the respective books or conference proceedings.

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| . . . . .                | 8.5c            |

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- |                                        |           |
|----------------------------------------|-----------|
| o Jamuna River . . . . .               | 4.2/71-73 |
| . . . . .                              | 8.4a      |
| o effect of floating screens . . . . . | 7.2/1-11  |

### Dhaleswari River

## Discharges

- |                                |      |
|--------------------------------|------|
| o Ganges River . . . . .       | 9a   |
| o Jamuna River . . . . .       | 8.1c |
| o Lower Meghna River . . . . . | 8.3a |
| o Padma River . . . . .        | 8.3a |
| o Upper Meghna River . . . . . | 8.3a |

**Dune(s)**

see Bedform(s)

## Field measurements

- |                              |                     |
|------------------------------|---------------------|
| o bedforms . . . . .         | 3.6/384-386,392-393 |
| o bed load . . . . .         | 8.1d                |
| o confluence scour . . . . . | 8.1d                |
| o outer bend scour . . . . . | 8.1d                |

### Floating screens

- |                        |             |
|------------------------|-------------|
| o effect . . . . .     | 7.2/1-11    |
| o efficiency . . . . . | 7.1/591-599 |

## Flooding

- |                          |         |
|--------------------------|---------|
| o Jamuna River . . . . . | 4.8/2-5 |
|--------------------------|---------|

## Ganges River

- |                         |                     |    |
|-------------------------|---------------------|----|
| o bank erosion          | 4.1/546-547,555-556 | 9g |
| o bed material          |                     | 9a |
| o bend characteristics  |                     | 9e |
| o cross-section         | 3.2/15              |    |
| o planform              |                     | 9e |
| o sediment transport    | 3.2/13-19           |    |
|                         | 3.8/1-8             | 9e |
| o transported sediments |                     | 9a |

### General scour

- o scenario's . . . . . 8.1e
- o mathematical modelling . . . . . 8.1e

## Gorai River

### Hydraulic roughness

- o Jamuna River . . . . . 3.6/383-384,391
- o smaller rivers . . . . . 3.1/105-112

## Hydrology

. . . . . see Stages

..... see Discharges

..... see Slopes

### Instability(ies)

. . . . . see Braiding

## Jamuna River

. . . . see Brahmaputra River

## Junction

..... see Bifurcation

### Local scour

- |                             |      |
|-----------------------------|------|
| o Bhairab Bazar . . . . .   | 8.3b |
| o Chandpur . . . . .        | 8.3b |
| o Jamuna River . . . . .    | 8.1e |
| o time dependency . . . . . | 8.1e |

### Lower Meghna River

- |                                 |                 |
|---------------------------------|-----------------|
| o bank erosion . . . . .        | 4.1/547,559     |
| o bed material . . . . .        | 8.3b            |
| o braiding indices . . . . .    | 8.3b            |
| o cutoffs . . . . .             | 4.1/548,560     |
| o hydraulic roughness . . . . . | 8.3a + b        |
| o planform . . . . .            | 8.3b            |
| o sediment transport . . . . .  | 8.3b            |
| o shifting . . . . .            | 4.1/547-548,561 |
| o slopes . . . . .              | 8.3b            |

### Mathematical modelling

**delling** . . . . . see Modelling

## Meandering

. . . . . 2.1/110-111

### Meghna River

... see Upper Meghna River

... see Lower Meghna River

## Modelling

- |                                       |                  |
|---------------------------------------|------------------|
| o combined use of models . . . . .    | 8.1e             |
| o mathematical modelling . . . . .    | 6.2/4.5.1-4.5.18 |
| . . . . .                             | 7.2/1-11         |
| . . . . .                             | 8.1e             |
| o physical modelling . . . . .        | 6.1/59-71        |
| . . . . .                             | 6.3/509-520      |
| . . . . .                             | 8.1e             |
| o planform changes . . . . .          | 4.6/134-146      |
| o predictability . . . . .            | 4.6/138-139      |
| o scale effects movable bed model     |                  |
| . . . . .                             | 6.1/65-69        |
| o scaling procedure movable bed model |                  |
| . . . . .                             | 6.1/63-65        |
| o types of models . . . . .           | 4.6/139          |

## Narrowing

..... see River training



## Number of channels

- ..... see Planform characteristics
- ..... see Braiding index(ces)

## Old Brahmaputra River

### Padma River

- o bank erosion ..... 3.3/18
- o bars ..... 3.3/18
- o bed material ..... 8.3b
- o braiding indices ..... 8.3b
- o cutoffs ..... 8.3b
- o hydraulic roughness ..... 8.3a+b
- o planform ..... 3.3/17-21  
..... 8.3b
- o sediment transport ..... 8.3b
- o shifting ..... 3.3/18
- o sinuosity ..... 3.3/17-21
- o slopes ..... 8.3a+b

### Physical modelling ..... see Modelling

### Planform characteristics

- o theoretical ..... 2.1
- o Ganges River ..... 3.3/17-21
- o Jamuna River ..... 3.5/174-176
- o Lower Meghna River ..... 8.3b
- o Padma River ..... 8.3b
- o Upper Meghna River ..... 8.3b

### Planform changes

- o braided river ..... 4.6/139-141
- o Ganges River ..... 9e
- o Jamuna River ..... 4.2/63-74  
..... 4.3/5-7,14-18  
..... 4.5/459-471
- o modelling ..... 4.6/134-146

### Regime equations

- o Jamuna River ..... 3.1/7-8
- o Upper Meghna River ..... 8.3b

### Recurrent measures ..... see River training

### Remote sensing ..... see also Satellite imagery

### River training

- o effect of floating screens ..... 7.2/1-11
- o floating screens ..... 7.1/591-599
- o Jamuna River ..... 4.8/5,6  
..... 8.2c  
..... 8.4a  
..... 8.5e
- o possible scenario's ..... 8.5a  
..... 8.4a
- o recurrent measures ..... 8.5b

### Roughness ..... see Hydraulic roughness

### Sand waves ..... see Bedform(s)

### Satellite imagery

- o colour coding ..... 4.3/14,16
- o Ganges River ..... 9g
- o images used ..... 4.3/14  
..... 4.5/461
- o interpretation ..... 8.5c  
..... 8.6c  
..... 8.6d  
..... 8.6e
- o Jamuna River ..... 4.3/5-7,14-18  
..... 4.5/459-471
- o processing ..... 4.3/5-7,14-18  
..... 4.5/461
- o tectonic processes ..... 4.7/4-6  
..... 8.6a

### Scale modelling ..... see Modelling

### Scour ..... see also Combined scour

- ..... see also Bend scour
- o confluence scour ..... 5.2/395-408
- o constriction scour ..... 5.1/453-463

### Sediment transport

- o bed load ..... 3.6/385
- o Ganges River ..... 3.2/13-19  
..... 3.8/1-8  
..... 9e
- o Ganges River comparison with predictions . . .  
..... 3.2/15-18

### Sediment transport (Continued)

- o Jamuna River ..... 3.6/386-387,394
- o Jamuna River comparison with predictors . . .  
..... 3.7/17-26
- o Upper Meghna River ..... 8.3b

### Shifting

- o Ganges River ..... 4.1/546-548,555-558  
..... 9e
- o confluence Ganges-Jamuna . . . 4.1/547,556-558
- o Jamuna River ..... 4.2/67-71  
..... 4.3/5,11-13  
..... 4.5/466-467  
..... 4.8/6-18
- o Lower Meghna River ..... 8.3b

### Slopes

- o Ganges River ..... 8.6a
- o Jamuna River ..... 8.1c
- o Lower Meghna River ..... 8.3a
- o Padma River ..... 8.3a
- o Upper Meghna River ..... 8.3a



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|                                |      |
|--------------------------------|------|
| o Ganges River . . . . .       | 9a   |
| o Jamuna River . . . . .       | 8.1c |
| o Lower Meghna River . . . . . | 8.3a |
| o Padma River . . . . .        | 8.3a |
| o Upper Meghna River . . . . . | 8.3a |

|                                           |         |
|-------------------------------------------|---------|
| o influence on planform changes . . . . . | 4.6/138 |
| . . . . .                                 | 4.7/4-6 |
| o Jamuna River . . . . .                  | 4.7/4-6 |
| . . . . .                                 | 8.6a    |

## o between meandering and braiding 2.1/107-120

|                                |          |
|--------------------------------|----------|
| o bank erosion . . . . .       | 8.3b     |
| o bed material . . . . .       | 8.3b     |
| o discharges . . . . .         | 8.3a     |
| o planform . . . . .           | 8.3b     |
| o regime equations . . . . .   | 8.3b     |
| o sediment transport . . . . . | 8.3b     |
| o slopes . . . . .             | 8.3a + b |
| o stages . . . . .             | 8.3a     |

see Slopes

o Jamuna River . . . . . 4.7/4-5

